

## Management of Aquifer Recharge for Sustainability



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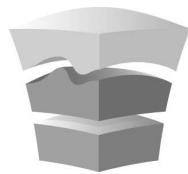


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## MANAGEMENT OF AQUIFER RECHARGE FOR SUSTAINABILITY

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## ISMAR6 Proceedings

## FOREWORD

This book contains the papers presented at the 6<sup>th</sup> International Symposium on Managed Aquifer Recharge (ISMAR6) that was held in Phoenix, Arizona, from October 28 to November 2, 2007, and symposium field trips to three diverse recharge projects in the east Salt River Valley. Previous symposia have been held in California, Florida, The Netherlands, Australia and Germany.

Groundwater augmentation can be achieved by spreading water on the soil surface and letting it infiltrate and move down to the groundwater. High quality water can also be directly injected into groundwater through wells. The source water for these systems may range from natural streams to effluent from sewage treatment plants. Much work has been done on the design of recharge systems, especially where sewage effluent or polluted surface water is used. The recharge system is then considered a tertiary treatment system and it should be designed to optimize biodegradation of organic compounds, denitrification, filtration, removal of microorganisms and adsorption. These processes are fairly well understood. However, pilot studies in the field are usually desirable. This is also true for evaluating hydraulic properties of the soils and underground formation so that infiltration rates, and groundwater mounds below the infiltration systems can be predicted. The golden rule for groundwater recharge projects is to start small, learn as you go, and expand as needed. Experiences from existing recharge systems throughout the world are very valuable for the design of new recharge systems. This requires intensive exchange and cooperation between scientists and engineers. To promote this, international symposia on groundwater recharge have been organized to stimulate information exchange and cooperation. This 6<sup>th</sup> Symposium has been organized by a committee of the Arizona Hydrological Society under the co-chairmanship of Doug Bartlett and Ivan Johnson, with the assistance of Barbara Murphy. We thank Doug and Barbara and all the other committee members for all the work they have done.

Prepared by Herman Bouwer, PhD



# INNOVATION IN GOVERNANCE OF MANAGED AQUIFER RECHARGE

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## ABSTRACT

Managed aquifer recharge (MAR) governance with some exceptions is generally in need of a major overhaul in two areas; water allocation and water quality protection. Water entitlements of water bankers and rights to transfer allocations warrant exploration due to their potential to induce private investment to secure water supplies and improve catchment water use efficiency. The difficult problems of over-allocated groundwater resources, where MAR can have beneficial effects (but sometimes claims are exaggerated) and allocation of aquifer recharge capacity among multiple water bankers, warrant further exploration. Secondly, this paper explores contradictions in health and environmental protection guidelines due to inadequate attention to the growing scientific understanding of MAR. A sound base for risk management would result in consistent applications across jurisdictions, taking into account the varying needs for efficiency in water resources and energy management, competence in managing MAR, and capacity to pay. The science and technology base necessary to support good guidelines, particularly in light of growth in water reuse, will be discussed. In each case positive examples or proposed constructs are put forward, based on current reform in Australian water governance, consistent with international trends in water policy development. If adopted, these approaches may give greater certainty to obligations and entitlements concerning water quantity and quality for those people managing aquifer recharge.

## KEYWORDS

Risk management, groundwater, water quality protection, water allocation

## INTRODUCTION

Substantial advances have been made in scientific understanding and technologies associated with Managed Aquifer Recharge (MAR) in recent years. In contrast, ad hoc and arbitrary governance arrangements have emerged as a major remaining barrier to effective MAR adoption. The reasons for this are logical from a historical perspective because of the new challenges that these technologies present to existing policy approaches in many jurisdictions. This is particularly evident where urban stormwater or reclaimed water derived from treated sewage effluent are sources of recharge, or where target storage zones comprise of brackish and saline aquifers or fractured rock and karstic aquifers. Scientific research has a role in better informing regulation intended to manage human and environmental health risk in

many jurisdictions. Advantage can also be gained by uptake of new technologies, adapting institutional arrangements, and addressing MAR within the broader context of catchment and aquifer system management. Market-based approaches are increasingly endorsed as a class of policy innovations, emphasising the careful definition of the entitlements and allocation of water resources that account for aquifer characteristics.

**TABLE 1. Resource management and environmental protection policies that are invoked (explicitly or implicitly) in managing aquifer recharge operations.**

| Attribute / Resource      | Quantity                                                                                                                                                                                                                                                                       | Quality                                                                                                                                                                                                                                                                |
|---------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Source water for recharge | <ul style="list-style-type: none"> <li>• Environmental flow requirements</li> <li>• Inter-jurisdictional agreements</li> <li>• See below if source water is groundwater</li> </ul>                                                                                             | <ul style="list-style-type: none"> <li>• Catchment pollution control plan</li> <li>• Water quality requirements for intended uses of recovered water</li> <li>• HACCP† plan for water quality assurance</li> <li>• See below if source water is groundwater</li> </ul> |
| Groundwater               | <ul style="list-style-type: none"> <li>• Groundwater allocation plan</li> <li>• Resource assessment accounting for groundwater-dependent ecosystems</li> <li>• Demand management</li> <li>• Allocatable storage capacity</li> <li>• Inter-jurisdictional agreements</li> </ul> | <ul style="list-style-type: none"> <li>• Groundwater quality protection policy</li> <li>• Water quality requirements for intended uses of groundwater (may include HACCP† above)</li> </ul>                                                                            |

† Hazard Analysis and Critical Control Points

Effective implementation of MAR is an acid test of integrated water resources management in any jurisdiction because it involves management of quantity and quality of water in surface water and groundwater (Table 1). In many cases MAR potentially creates conflict between water conservation and water quality protection and this will require regulators to have a sound basis for policies. Sectoral responsibilities for environmental and health regulation and water resource management roles will require coordination for efficient approval of new MAR projects and to avoid perverse outcomes. For example, extraction from rivers to enhance groundwater reserves may benefit local groundwater users and groundwater dependent ecosystems but have adverse effects on downstream users and river-dependent ecosystems, and in some cases impinge on inter-state or international agreements on water sharing. Unless seen in the context of groundwater management that embraces demand management, especially in over-exploited systems, MAR will not achieve an equitable distribution of benefits nor



assure continuity of supply. In the special case of trans-boundary aquifers MAR may produce benefits on both sides of the border. Water allocation issues and implications of market-based instruments on the future governance of MAR are discussed later.

Policies in Table 1 are in a varied level of maturity among countries and across jurisdictions within them including surface water catchments and groundwater basins, and institutional arrangements allow for different levels of community engagement in decision making. In some catchments surface water and groundwater allocation policies are already integrated due to awareness of interactions between resources, such as where streams are in hydraulic connection with aquifers. In many jurisdictions the different parts of the policy matrix are addressed by different organisations. Urban areas generally have the largest number of participants in decision making, including proponents of projects, urban developers, local government, water utilities, urban planners, environment protection and health regulators, water resource managers, and the communities themselves.

## ISSUES IN WATER ALLOCATION

Subject to increasingly variable rainfall patterns and growing populations, many cities have mature urban water economies, characterised by limited opportunities for new water impoundments, rising incremental water supply costs, intensified competition between diverse users and increased interdependencies amongst water uses (Randall 1981). MAR is one management strategy to augment supplies and buffer seasonal shortages and in some cases inter-annual drought by harvesting and storing urban stormwater and reclaimed water (Dillon *et al.* 1997, 2004). However entitlements to such sources of water, rights of water bankers to recover and use stored waters, and entitlements to use of recovered waters require clarification in most jurisdictions. This is currently a common impediment to investment in MAR as a low cost, low energy pathway to securing city water supplies.

An exception is in Arizona, where Title 45 (1994) grants entitlements to water bankers, give assured access to banked waters for up to 30 years into the future. This provides security for private provision of water infrastructure consistent with its requirement of developers to provide a 100 year source of water for new subdivisions. Areas with high rates of urban growth in Arizona are underlain by aquifers that are characteristically highly transmissive, have large storage capacity and contain water that is suitable as a source for drinking water supplies. These are conducive to establishment of broad markets for transfer of rights of water bankers. However where aquifers have either low transmissivity or are brackish, the benefits of MAR are only localised and the ability to develop markets in recharged waters is quite restricted.

In mature water economies, it is likely that if MAR is commercially viable for one operator, then it is also likely to be viable for a number of operators in the same aquifer or with the same water source. Each operator would impinge on the access to water for recharge of others, on the total available storage volume, and in confined aquifers, on the groundwater pressures at the sites of other proximal operators. In brackish confined aquifers the operations of one MAR site are also likely to influence the shape of the plume of fresh injectant in the aquifer at neighbouring sites, and hence influence the recovery efficiency (proportion of fresh water recovered at a salinity that meets requirements of uses) of other operations. Guidance for such interactions would be complex and could possibly be averted by the concept that a single operator be responsible for all MAR operations within an aquifer over a given region. This would not only avoid litigation, but would allow the operator to coordinate the interacting set of potentially competing MAR operations.

Finally the issue of aquifers subject to over-allocated extraction rates provides a stimulus for MAR because clearly there is capacity created within the aquifer enabling recharge to be augmented, however expectations that MAR could make existing levels of extraction sustainable in grossly-overexploited aquifers are not realistic. In fact it can be shown that unless demand management is also introduced the benefits of recharge enhancement in over-stressed aquifers will only affect a privileged minority whose wells are closest to the MAR operation or whose wells are deepest and therefore extraction prolonged. MAR may be a visible incentive in inducing compliance with groundwater allocation policies where a replenishment assessment (levy) is applied on the basis of volume extracted (eg. in Orange County California, (Mills, 2002) and in Central Arizona, Ferris *et al* 2006). Set at an appropriate rate, this would provide a means of beneficiaries covering the costs of recharge enhancement to bring the groundwater system into balance. It is likely that in areas of saline intrusion, land subsidence, and degradation of groundwater-dependent ecosystems where communities suffer a long-term loss through groundwater overuse for sectoral economic gain, that institutions such as groundwater users associations could be established to accomplish groundwater equilibrium where governments have failed.

## **WATER ALLOCATION POLICIES: A POSSIBLE MODEL FOR MAR**

The Council of Australian Governments (CoAG) has been following a national water reform agenda which has been gaining pace in recent years, ratified by the National Water Initiative and facilitated by the establishment of a National Water Commission (CoAG, 2004). Consistent with trends in international water policy development, Australian water reform has emphasised institutional and governance approaches promoting voluntary transfers of water through market exchange. The reform agenda has been primarily implemented in rural water reform, with an increasing focus on urban water reform. Key characteristics of the reform agenda are:

- separation of water entitlements from land ownership
- allowance for environmental allocations of water with the rest being shared by water users within the catchment or groundwater system
- allocations to be in the form of shares in the resource
- periodic scientific evaluation of the volumetric size of a share, based on measured water availability
- •water allocations in a common resource may be traded in a market with a view to maximising the utility of the water resource, and subject to environmental constraints
- groundwater systems and rivers are considered to be connected unless proven otherwise
- measurements are to be made to ensure that water is correctly accounted for by all entitled users

The Natural Resource Policies and Programs Committee (2007) contend there are strong possibilities that existing legislation, policy and institutions may not be readily adapted to market innovations in natural resource management (including groundwater) or subject to governance failure, with an attendant social cost. Jimenez (2007) argues a similar *ad hoc* development of stormwater governance and legislation in Victoria is pointing to limited integration between jurisdictions, institutions and managing agencies. The rapid development of regional ASR is likely to exacerbate this disjunction.

Young and McColl (2003a) and Young (2007) propose a robust, two-tiered system of instruments to distribute and allocate volumes of water efficiently over time. The first instrument establishes the rules and science based guidelines to appraise the state of a water system and subsequent to that appraisal, prescribe the rules to determine the environmental and consumptive “pools”. When more than one person has an interest in the water system or “pool” a second instrument defines the unit shares of the pool and the distribution of shares to individual interests. A robust system of water management will: i) resolve the resource allocation tension between consumptive use and the environment, amongst consumptive users, issues related to distribution and use; ii) provide secure, economically efficient and low cost trading and administration; iii) clarify the assignment of risk and circumstances of compensation; vi) and address the management of externalities. A robust system also must pass the conventional tests of efficiency and fairness in a changing world.

Young and McColl (2003a) contend that these objectives are best achieved through the robust separation of water interests and recommend:

1. unit shares as **entitlements** specified as the secure long term share of a variable periodic allocation;

2. an agreed process for **annual allocation** contingent on science and the state of the resource, managed independently of entitlements;
3. a process to assign risk defining where responsibility lies, under what circumstances compensation is due, and the processes for obtaining redress with non-controversial settlement;
4. conditions and **obligations specified in a water use licence**, cognisant of third parties;
5. the introduction of debit and credit accounting systems, water exchange rates and associated formal transaction mechanisms;
6. the guaranteed recording of all interests on a register, formal settlement procedures, and irreversibility of market transactions;
7. In an extension of Tinbergen (1950), Young and Mcoll (2003b) argue that over time, robust systems are characterised by the use of separate instruments for each distinguishable or discrete component of the water cycle or system. In the case of MAR these are capture, recharge, recovery and use (See Table 2).

**TABLE 2. Robust governance instruments for separate components of MAR.**

| <b>Component</b>                                                                              | <b>Water capture and harvesting</b>                                                                                                                                                                                                                    | <b>Recharge</b>                                                                                                                                                                                                                   | <b>Recovery</b>                                                                                                                                                                                                                                                                                     | <b>Use</b>                                                                             |
|-----------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------|
| Governance instrument<br>Entitlement, periodic allocation and use obligations and conditions. | Unit share in stormwater consumptive pool, excess to environmental flows<br>Periodic allocation rules<br>Potential for additional stormwater or effluent offsets<br>3 <sup>rd</sup> party rights of access to infrastructure for stormwater and sewage | Unit share of aquifer's finite additional storage capacity<br>Requirement not to interfere with entitlements of other water users and water bankers or land holders<br>Requirement not to over-pressurise the aquifer or aquitard | (Tradeable) extraction volume a function of managed recharge.<br>Unit share = MAR recharge volume, extraction level contingent on ambient conditions, non-interference, and depressurisation constraints<br>may need to convert existing licence to compatible entitlement to extract (unit share). | Water use licence subject to regional obligations and conditions, for use and disposal |

Young (2005) has recently applied the notions of robust separation of rights to resolve persistent tensions of over allocated water and excess irrigation drawdown from a ground water system in the south east of South Australia. We have adapted this line of reasoning as a guiding reference for the robust design for MAR governance.

Consistent with the National Water Initiative directives and the principles of robust separation of water interests, source water harvesting, aquifer recharge, recovery and use are distinguishable urban water cycle elements. Each element requires the discrete specification of a unit share entitlement of a defined pool, and independently managed rules to establish periodic allocation and the conditions of use. Final use of recovered water is subject to end use licence conditions, cognisant of third parties including the environment.

## ISSUES IN WATER QUALITY MANAGEMENT

Currently there is a diversity of approaches to managing water quality in MAR operations. While multi-barrier approaches to managing risks of groundwater contamination are common, the contrasting goals and methods in regulations applied to MAR in different jurisdictions are confusing and occasionally contradictory. Two common examples relate to establishing the environmental values (beneficial uses) to be protected, and defining the location at which water quality compliance is measured. With water reclamation via subsurface storage growing in importance in arid and semi-arid areas facing a drying climate and population growth, a blanket level of groundwater protection will become increasingly impractical. If the level of protection is tailored to the ambient conditions of the aquifer, much higher efficiency in resource management can be achieved while improving and protecting the environmental values of aquifers. For example, aquifer storage and recovery (ASR) with urban stormwater in brackish aquifers is used to reduce irrigation demand on stressed mains water supplies in South Australia. However this is unlikely to occur intentionally in the USA if this is interpreted as being in breach of the Underground Injection Control (UIC) provision of the Safe Drinking Water Act (US Congress, 1988), where injected water needs to be equivalent to drinking water quality prior to injection. A concession for the High Plains States Groundwater Recharge Demonstration Project perhaps serves as a precedent for a more liberal interpretation that protects neighbouring groundwater users.

Similarly there are discrepancies between the requirement to disinfect prior to ASR injection in USA (US EPA, 2004, Asano *et al*, 2006) and requirements in parts of Europe that injectant is not to be chlorinated. A fundamental difference appears to be that in USA aquifers are inferred by regulation to be inert with respect to pathogen attenuation (Asano *et al*, 2002) whereas in Europe and Australia aquifers are recognised as ecological units containing bacteria that react with nutrients and micro-organisms that are introduced, and often having beneficial outcomes for water

quality including pathogen inactivation (Dillon and Toze, 2005). Interestingly, US EPA (2004) recognises the pathogen attenuation capacity in the vadose zone beneath surface spreading facilities, even though much of the water may reach the water table by flow in saturated fingers.

Ironically, when chlorinated water is injected into aquifers, disinfection by-products may continue to increase in concentration due to reaction kinetics between chlorine and organic matter. Pavelic *et al* (2005, 2006a) found that under anaerobic conditions trihalomethanes degraded but under aerobic conditions at least chloroform was persistent. Recent research (Pavelic *et al*, 2007) has shown that chlorination of reclaimed injectant in a limestone aquifer at Bolivar, South Australia, made no difference to rates of clogging nor the effectiveness of its management. Hence when recharging aerobic aquifers, if the purpose of chlorination is solely to meet drinking water requirements, proponents should be aware that chlorination has potential for reversing this outcome.

With climate change now well recognised, energy conservation principles on the horizon could include mandating use of least energy intensive processes for water treatment, and all other elements of water supply, sanitation and recycling. Sustainable passive systems such as aquifer treatment may therefore be preferred to equivalent components of engineered treatment processes with transition facilitated by carbon credits or taxes. It is anticipated that preferred combinations of engineered and passive processes will evolve as science matures in the fields of passive water treatment processes and energy efficiency of engineered systems.

Arsenic mobilisation through injection of waters that can oxidise pyritic material containing arsenic, and subsequently transport this under anaerobic conditions to the recovery well, or by groundwater drift from the perimeter of the storage zone to other wells, is another issue that complicates regulation of ASR (Vanderzalm *et al*, 2007). It is possible to inject drinking quality water into an aquifer that contains potable water and recover water that has an arsenic, iron or manganese concentration in excess of the acceptable limits for drinking. This requires some understanding of the geochemistry of aquifer processes at a site in order to determine the likelihood of this problem and identify solutions, and to implement appropriate monitoring to verify the quality of recovered water and protection of other groundwater users and dependent ecosystems. This is also true for recharge of desalinated water which may be aggressive in aquifers and energy expended in removing ions contributes to dissolution of aquifer minerals.

These issues suggest that simplistic approaches to water quality management based on rules of thumb are not an adequate basis for health and environmental protection in MAR operations. Simply treating water to a standard where problems are believed to be avoided, as adopted in some guidelines (eg Dillon and Pavelic, 1996; ASCE/EWRI 2001) does not guarantee that environmental and human health protection will be met. A sound scientific basis is necessary to give adequate confidence in

sustainability of MAR operations. Development of new knowledge relies on innovation in approaches to MAR. Therefore it is logical that governance arrangements would provide for experimental MAR with intensive monitoring, and updating of approval processes based on growth in scientific knowledge of subsurface processes. Examples from Australia are given below.

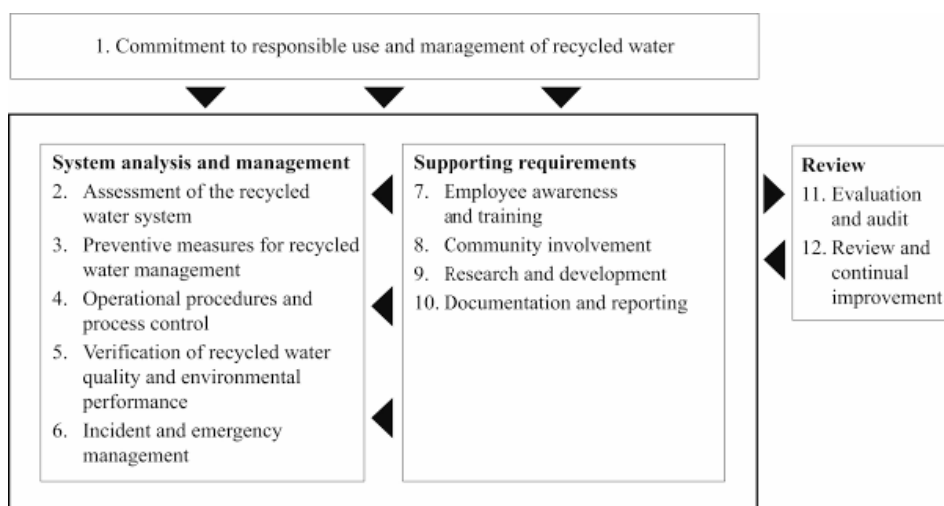
## **HEALTH AND ENVIRONMENTAL PROTECTION: AN EVOLVING MODEL**

Over almost 20 years the National Water Quality Management Strategy (NWQMS), a combined venture of Australian Federal and State Governments, has developed principles and guidelines for Australian water quality management. This has been wide ranging, covering fresh and marine waters, groundwater, urban stormwater, wastewater, and water recycling. All are based on protecting human and environmental health including all existing and potential environmental values (including beneficial uses) of affected ecosystems with community engagement in decisions concerning the environmental values to be protected. Groundwater protection guidelines (NWQMS, 1995) provide a differential protection policy whereby the level of protection depends on the localised potential beneficial uses of ambient groundwater and therefore on its current water quality. Environmental values categories include:

- drinking water,
- primary industries (including irrigation and water for livestock)
- industrial water
- recreation and aesthetics
- ecosystem support (including connected surface water bodies and groundwater dependent ecosystems)

Water Quality Guidelines for Fresh and Marine Waters (NWQMS, 2000) provide schedules of acceptable values of water quality parameters for a wide range of known indicators, inorganic and organic substances and microbiota. Hence the environmental values to be protected are determined by the ambient groundwater quality and community expectations.

The precautionary principle is adopted in all guidelines and in recent years this has become increasingly based on codified risk assessment. The Australian drinking water guidelines (NWQMS, 2004) lay out a framework for risk management from source to tap which is also adopted in water safety plans of WHO Guidelines for Drinking-water Quality (2004) and in the Australian guidelines for water recycling (NWQMS, 2006) (Figure 1).



**FIGURE 1. Elements of the framework for management of water quality and use (from NWQMS, 2006).**

The risk management frameworks were developed to replace the alternative approach of relying on water quality testing and comparison with defined targets. The latter approach has a number of flaws including practical difficulties (e.g. availability and sensitivity of methods) and costs associated with testing for ever expanding numbers of anthropogenic contaminants. Furthermore, by the time the analyses have been completed the water has already been recharged or distributed and drunk. The frameworks represent a more proactive, preventative approach that has been adapted from the Hazard Analysis and Critical Control Points (HACCP) systems that have been in use in the food industry for many years (Codex, 1996).

The frameworks apply from collection of water to extraction and final use and incorporate a systematic approach for identifying hazards, assessing risks and determining appropriate management options. Hazards include contaminants present in source waters as well as those introduced through processes such as arsenic mobilisation. The risks posed by these hazards are assessed in terms of environmental values at point of use or impact. In the case of drinking water sources, for example, the assessment should be based on the quality of water extracted from aquifers. Management options include consideration of source water protection, treatment processes and natural attenuation.

One source of information has emerged from more than 100 years of stormwater disposal into a karstic limestone aquifer at Mount Gambier, South Australia, and the relatively recent awareness that this stormwater-affected groundwater is sourcing Blue Lake, the source of the city's drinking water supplies (Vanderzalm et al, 2007b



(in these proceedings), Wolf *et al*, 2006, Gorey and King, 2005). Extensive monitoring has revealed that attenuation is occurring and drinking water guidelines are likely to continue to be met with existing stormwater recharge practices. With improved management of sources the quality of supply should be assured indefinitely. This and other work at Andrews Farm (Pavelic *et al*, 2006b) and Bolivar (Dillon *et al*, 2004) suggested that urban stormwater could be recharged to a brackish limestone aquifer and recovered at drinking water quality using a multi-well system, termed aquifer storage transfer and recovery (ASTR). A HACCP plan for that project is described by Page *et al*, (2007) in these proceedings.

MAR Guidelines are currently being developed as part of the Australian Water Recycling Guidelines. Hazards to be addressed in the MAR guidelines include: pathogens, metals and inorganics, salt, nutrients, natural organics (including algal toxins), anthropogenic organics (including disinfection by-products and endocrine disruptors), suspended solids, radio-nuclides, pressure, excessive change in groundwater levels, aquifer dissolution and well and aquitard stability, and energy and greenhouse gas considerations. Attention will be given to fractured rock, karstic and unconfined aquifers, groundwater-dependent ecosystems, including aquifers themselves (eg stygofauna, and ecosystem function responsible for sustaining biodegradation and pathogen inactivation), and to saline intrusion barriers. A section of the guidelines will be devoted to monitoring for purposes related to risk assessment and performance verification. The guidelines will recognise the stages of MAR project development and that in some cases the information necessary for final approval of a project will only become available as the project progresses through trials, for which provisional approvals will be necessary based on information at hand. Information required and monitoring requirements will depend on potential risk, which is redefined at each stage of project development. Technology advances, such as web-based real-time monitoring and reporting systems, are intended to be accommodated in the guidelines. These open new opportunities for faster, more efficient and transparent reporting tailored for proponents, regulators and communities.

## CONCLUSIONS

This paper makes a case for improved governance of MAR addressing water allocation and water quality protection. Governance in water allocation that takes more explicit account of MAR within the context of catchment and groundwater systems is warranted in many jurisdictions to allow effective conjunctive use of surface waters and groundwaters and to allocate water efficiently over time. Robust separation of water entitlements between capture, recharge, withdrawal and use is advocated to allow separate instruments to be used for economic efficiency within resource and environmental constraints, and in the face of uncertainty in the size of the available resource. Water entitlements that can be traded in markets are considered to be an important means of maximising utility of the resource, subject to

environmental requirements being met, such as sustaining groundwater-dependent ecosystems. The size and hence efficiency of markets will be defined by ambient groundwater quality and aquifer transmissivity. Few jurisdictions employ market forces in groundwater governance accounting for MAR, possibly because of the reliance of these approaches on advanced management and sound knowledge of the aquifer systems.

Irregularities in some existing approaches to water quality management have been identified that suggest a sound scientific basis is needed and consistent application of risk-based management. Risk management methodologies inform defensible and consistent decision making, allow rational and adaptable monitoring requirements, give opportunities for learning with risks constrained, and thereby help to improve the knowledge base. An Australian transition to this approach demonstrates that regulatory change is possible when scientific evidence is assimilated in regulation.

Based on current reform in Australian water governance, consistent with international trends in water policy development, constructs for governance of MAR have been proposed. These are intended to give greater certainty to obligations and entitlements of water bankers concerning water allocation and water quality, and the principles may also have merit in other jurisdictions.

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# A CONCEPT FOR MANAGED AQUIFER RECHARGE USING ASR-WELLS FOR SUSTAINABLE USE OF GROUNDWATER RESOURCES IN AN ALLUVIAL COASTAL AQUIFER IN EASTERN INDIA

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## ABSTRACT

Groundwater resources in coastal plains area are coming under increasing threat due to over extraction and unsustainable agricultural practices. Sustainable water resource management in these areas can potentially be facilitated through the use of a novel new concept in groundwater management. This concept involves matching supply and demand for irrigation water by managing excess monsoon season runoff and making it available for managed aquifer recharge (MAR), thereby allowing potentially greater extractions during the dry season. This includes the deliberate lowering of the groundwater table during the dry season, resulting in more storage capacity for MAR. In a case study the excess water was calculated and a field layout including ASR-wells and canals was developed. This groundwater management concept shows an additional potential infiltration of nearly 220 mm/a could be developed but field experiments showed problems which may be related to clogging of the injection wells. Additional enhancement of the natural recharge allows providing enough groundwater to grow a second crop during a year throughout nearly the whole area.

## KEYWORDS

ASR-wells, coastal aquifer, groundwater, managed aquifer recharge, sustainable management

## INTRODUCTION

Many fertile alluvial plains with high agricultural productivity have unsustainable groundwater management practices. In developing countries like India, China and Bangladesh there are increasing groundwater extractions being largely driven by expanding irrigation. This is expected to continue as food production is projected to double in the next 25 years (Schultz and De Wrachien, 2002) of which 80% will be produced in irrigated areas (DFG, 2003). Most experts agree that a considerable part of the increased water demand of the world in the 21<sup>st</sup> century will need to be met by an increased water storage capacity (Bouwer, 2002a).

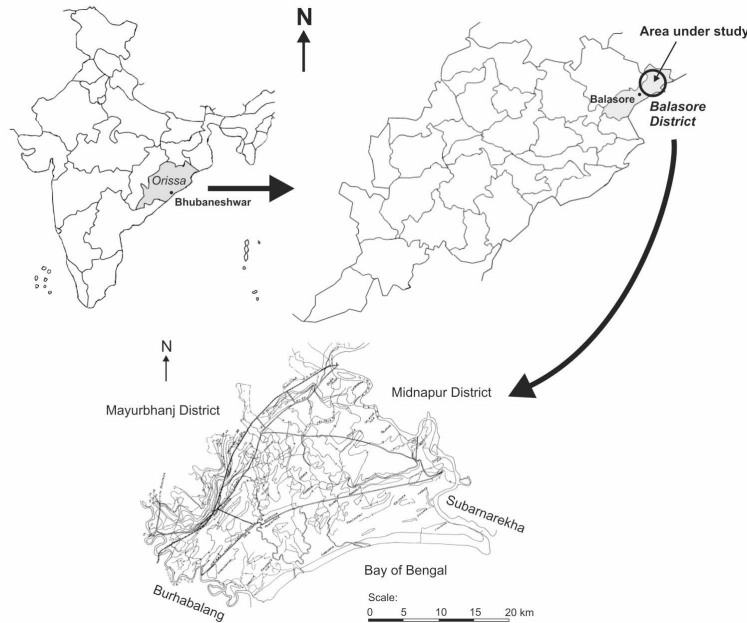
There has been a strong trend in recent years for a greater groundwater abstraction to be used for irrigation. In many areas this has resulted in a decline in the groundwater table which results in further environmental problems, e.g. increase of abstraction, ecological damage, progressive water quality deterioration, saltwater intrusion and land subsidence (Rao *et al.*, 2001; Bouwer, 2002b; Custodio, 2002; Don *et al.*, 2005). In order to avoid these environmental problems a sustainable groundwater management concept is urgently required to protect groundwater.

Good conditions for agricultural production can often be found on the coastal fertile alluvial plains in sub humid climates in Asia. However, due to the short rainy season (monsoon season) which only provides enough rainfall for a few months, most of the year there is a large water deficit for agriculture. Therefore, the future development of the water resources should include the storage of surpluses during the rainy season, so that it can be used in dry seasons for irrigation. However, the storage of large volumes of water in surface reservoirs is often not possible due to the flat relief which can be found in these coastal alluvial systems. This opens the possibility to use subsurface storage of the water (Bouwer, 2002b), e.g. by a managed aquifer recharge (MAR) (Dillon, 2005) where surplus water is infiltrated into the aquifer through infiltration wells. In coastal alluvial plains the aquifers are often overlain by aquitards which results in surface recharge techniques being unsuitable. ASR-wells (Aquifer Storage and Recovery) are able to infiltrate water into an aquifer and to later extract water according to the demand (Dillon *et al.*, 1997; Pyne, 2005). This kind of short- or long- term underground storage has been termed water banking (McClurg, 2001). Water banking still leads to quantity and quality problems like sustainability, nutrients and pollutant charges of the surface water. Many of these problems are expected to exacerbate in the future due to increasing impacts of intensive agriculture (Bouwer, 2002b).

### Study Area

The site selected for the present study is of about 840 km<sup>2</sup> and located in the Balasore district in the Indian state of Orissa (Fig. 1). The study area can be categorized into three distinct morphological units: a 3 to 5 km wide saline marshy

tract along the coast; a highly fertile gently slopping alluvial plain about 10 m above sea level; and a hilly region in the western parts with a maximum elevation of 86 m.



**FIGURE 1. Area under study.**

The climate is characterised by 3 seasons: the monsoon season with high rainfall, the winter season with relatively low temperatures and very low rainfall and the hot and dry summer season. The site receives average annual rainfall of 1,788 mm (data from 1977 to 1999, climate station Rajghat) with most (82%) occurring during the monsoon season. The daily minimum and maximum temperatures are 12 and 45 C, occurring in the months of January and May, respectively. The mean relative humidity ranges from 60 to 85%.

The agricultural area is about 63,000 ha which is 74% of the study area. 12,123 ha (19.3%) has been irrigated mainly since 1996 by groundwater (CGWB, 1999). There are two distinct cropping periods: the *kharif* in monsoon season and the *rabi* in winter. No agriculture is performed during summer. In *kharif* the whole area is predominantly used for growing rice. Whereas the cropping pattern in *rabi* are rape seed (29%), vegetables (26%), pulses (21%), grain (rice, wheat and maize) (18.5%) and spices (5.5%).

Three aquifers have been identified to a total depth of 120 m, each overlain by a semi-permeable aquitard: Each is an unconsolidated semi-confined sand and gravel aquifer with a transmissivity from  $1.8 \cdot 10^{-4} \text{ m}^2/\text{s}$  to  $3.1 \cdot 10^{-3} \text{ m}^2/\text{s}$  while the thickness of

the aquifers is 4 to 25 m, with an average of 10 m and a storage coefficient of  $1.2 \cdot 10^{-3}$  to  $1.6 \cdot 10^{-1}$  (CGWB, 1999; OLIC, 1997). All calculations are done for the upper aquifer and the upper aquitard. This part of the groundwater system behaves during short-term storage (in days) like a confined system but on a longer time frame as an unconfined system (periods of months). Most of the predictions are done on monthly based data and calculations. The aquitard has a thickness of 20 to 30 m so that all piezometric levels are situated in the aquitard. The effective porosity of the aquitard was determinate to 5-10%.

## METHODS

### Groundwater Budget

The main focus of this study is extending the concept of sustainable groundwater management in coastal alluvial plains by utilizing seasonal subsurface water storage of excess surface water through the use of MAR. To develop this concept an extended formulation for the groundwater budget is given:

$$q_{nr} = \Delta S + q_{in} - q_{out} + q_{ar} - q_w - q_e \quad (EQ 1)$$

where  $q_{nr}$  is the natural recharge in mm/a,  $\Delta S$  the changes in subsurface storage (mm/a),  $q_{in}$  the subsurface inflow (mm/a),  $q_{out}$  the subsurface outflow (mm/a),  $q_{ar}$  the MAR component (mm/a),  $q_w$  the groundwater withdrawal (mm/a) and  $q_e$  groundwater withdrawal due to evaporative discharge (mm/a).

The subsurface inflow  $q_{in}$  and the subsurface outflow  $q_{out}$  of a large scale can be calculated by Darcy's law using the groundwater gradient  $i_o$  and aquifer transmissivity data. The groundwater withdrawal  $q_w$  is due to the water demand of the population, the industry and mainly irrigation. Groundwater withdrawal for irrigation can be determined by the deficit between plant demand and rain fall. Land use (1996 and 1997), soil and climate data (1977-1999) were provided by CGWB (1999) and OLIC (1997). New scenarios were developed using different cropping patterns to calculate the deficit between crop demand and rainfall. Pan evapotranspiration was calculated using the FAO Penman-Monteith method (Allen *et al.*, 1998). The reference crop evapotranspiration  $ET_c$  [mm/d] was calculated using the method of Doorenbos and Pruitt (1977):

$$ET_c = ET_0 \cdot k_c \quad (EQ 2)$$

where  $k_c$  is the crop coefficient values for each crop which were taken from the literature (Doorenbos and Pruitt, 1977; Allen *et al.*, 1998). The groundwater recharge  $q_r$  was determinate using the WTF method (water-table fluctuation) (Meinzer, 1923;



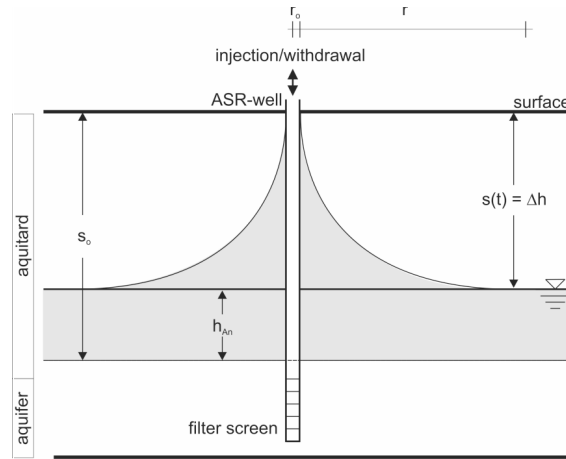
Meinzer and Stearns, 1929; Healy and Cook, 2002). The WTF-method is based on the assumption that the groundwater table rise in response to rainfall in an unconfined aquifer is caused by groundwater recharge. In order to use the WTF-method the storage coefficient and the groundwater hydrograph are required:

$$q_{nr} = n_{eff} \frac{dh}{dt} = n_{eff} \frac{\Delta h}{\Delta t} \quad (EQ 3)$$

where  $q_{nr}$  is the recharge rate [m/a],  $n_{eff}$  the effective porosity [-],  $h$  the head (here: equal to groundwater table) [m],  $t$  the time [a]. Due to the semi-confined aquifer and percolation losses from irrigation a method was developed to use eq. 3 under these conditions. The method can be found in Holländer (2005).

### Managed Aquifer Recharge

The development of a sustainable groundwater management approach presented here is based on storing surplus water in monsoon in the subsurface for extraction during the dry season. Due to the degree of confinement of the aquifers in the study area surplus water is infiltrated through ASR-wells (Pyne 2005). Optimisation of the agricultural area which can be connected to an ASR-well is required at which most of the surplus water is available for infiltration also taking economical aspects into account. The groundwater system acts like a confined aquifer during a short-term infiltration since the hydraulic conductivity of the aquitard is by comparison to the one of the aquifer very small. During longer-term infiltration the semi-confined characteristics of the aquifer system become more apparent as the overlying aquiclude hold water in storage. An upwards directed cone of water is created by the infiltration of water into an aquifer (Fig. 2).



**FIGURE 2. Principle of the groundwater fluctuation at an ASR-well during injection phase.**

All the following calculations are based on the assumption that the cone of impression/depression caused by injection/recovery for each well is not influenced by the neighbouring wells. Where the groundwater rise  $h_{An}$  is a large-scale rise in a certain time step. The infiltration  $Q_{in}$  is dependent on the transmissivity  $T$  and for gravity injection on the head difference  $\Delta h$  between the surface water ( $h$  at position  $r_o$ ) and the head in the aquifer ( $h$  at position  $r$ ). The maximal head difference and therefore also the recharge rate can be achieved if the head at position  $r_o$  is at the surface:

$$Q_{in} = (2^{-2} T \Delta h) / (\ln[r/r_o]) \quad (\text{EQ 4})$$

with  $r$  the radius where  $\Delta h$  is determined [m] and  $r_o$  the well radius [m]. Therefore the infiltration rate is linearly dependent upon the head difference  $\Delta h$ . The groundwater table will increase as infiltration progresses so that the head difference will decline resulting in a declining infiltration rate. For surplus water of an area  $A$  to be infiltrated into an ASR-well, the well needs to infiltrate a certain volume  $V$ :

$$V = A \Delta h_w(t) \quad (\text{EQ 5})$$

where  $\Delta h_w(t)$  is the average height of surplus water on the fields at a certain time step [m]. The surplus water expressed as height  $\Delta h_w(t)$  is dependent on the agricultural demand which is dominated by rice in the study area. The amount of run-off is reduced in the study by building dyke around the fields which allows storing water on the fields. The height of the water ponded on the fields may be  $S_0$ . In order to determinate the surplus water the equation of the water budget can be used:

$$h_N - h_V - h_P - h_{RO} \pm \Delta S_0 = 0 \quad (\text{EQ 6})$$

where  $h_N$  is the rainfall [mm],  $h_V$  evapotranspiration [mm],  $h_P$  natural percolation [mm],  $h_{RO}$  the run-off [mm] (Mishra, 1999) and  $\Delta S_0$  the change in surface storage [mm]. The maximal water which can be used for infiltration is the water which would run-off or lost due to evaporation. However optimal management is based on the local agricultural production technique which requires a certain standing water height on the fields  $h_s$ . Therefore, in eq. 7a the term  $\Delta S_0$  of eq. 6 is exchanged by  $h_s$  and an additional component which can be infiltrated ( $h_{inf}$ ). After the necessary water height on the fields  $h_s$  is reached,  $\Delta S_0$  is not considered as the water height on the fields is constant. This results into two equations, one for the beginning of the monsoon season:

$$h_{inf} = h_{Ao} = h_N - h_V - h_{Au} - h_s \quad (\text{EQ 7})$$

and an equation for when the water height reached a certain height:

$$h_{inf} = h_{Ao} = h_N - h_V - h_{Au} \quad (EQ 8)$$

where  $h_{inf}$  is the injection through an ASR-well [mm]. The water can be infiltrated in a certain time  $t$  into the subsurface. The area  $A$  which is connected to an ASR well is described as:

(EQ 9)

$$A = \frac{Q_{inf} \cdot t}{h_{inf}}$$

If eq. 5 and eq. 7a/eq. 7b is substituted into eq. 8 and the groundwater table rise  $\Delta h$  during MAR is given by:

(EQ 10)

$$\Delta h = \frac{h_w(t)}{n_{eff}}$$

where  $\Delta h_A$  is the groundwater table rise [m] and  $h_w(t)$  can be expressed as infiltration within a certain time  $t$  [m]. This follows for the beginning of the monsoon season:

(EQ 11)

$$A = \frac{\pi \cdot T \cdot t}{\ln\left(\frac{r}{r_o}\right) \cdot (h_N - h_V - h_{Au} - h_s)} \cdot \left(2 \cdot \Delta h - \frac{h_w(t)}{n_{eff}}\right)$$

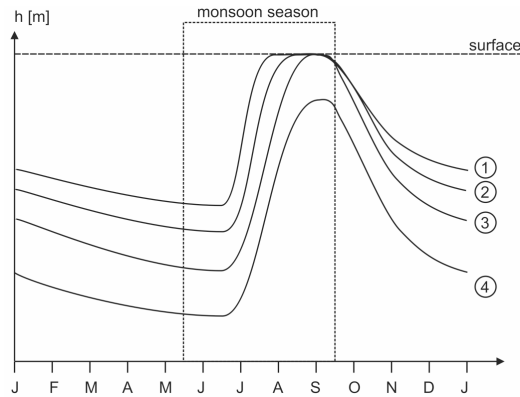
and respectively in case that the water height on the field reaching the desired certain height:

(EQ 12)

$$A = \frac{\pi \cdot T \cdot t}{\ln\left(\frac{r}{r_o}\right) \cdot (h_N - h_V - h_{Au})} \cdot \left(2 \cdot \Delta h - \frac{h_w(t)}{n_{eff}}\right)$$

## Groundwater Management

Current groundwater management practices result in groundwater extraction beyond the natural rate of replenishment during the dry season, resulting in declining groundwater fluctuation curves (Fig. 3).



**FIGURE 3. Schematic groundwater fluctuation under natural conditions (1) and under three withdrawal scenarios (2-4): (2) sustainable use, (3) maximal use under sustainable conditions and (4) non-sustainable use.**

The normal groundwater fluctuation in a year (curve 1 in Fig. 3) illustrates that the groundwater table rises at the beginning of the monsoon season to the surface, remains stable for the rest of the monsoon season before declining again during the dry season. This reduces the recharge volume which can be stored at the beginning of monsoon season. This recharge is based on natural recharge and recharge which is dependent on the field cultivation. However, due to groundwater withdrawals, the groundwater fluctuation increases (curves 2-4) and the groundwater table reaches the surface later during the monsoon season resulting a higher recharge. The natural recharge is limited and reaches the maximum when the groundwater table reaches the surface at the end of the monsoon season (curve 3). If additional groundwater is extracted in dry seasons, the groundwater table will continuously decline and MAR could be used to augment groundwater quantities available for irrigation.

The effects of infiltration and withdrawal on the groundwater budget and the head were calculated with the objective of storing a maximum volume of water in the subsurface. A long-term (15 year) simulation using MODFLOW (McDonald and Harbaugh, 1988; Harbaugh et al., 2000) was used to determine the effects of the infiltration and withdrawals and the maximum volume of water which could be stored to mitigate the negative impacts due to overexploitation of groundwater.

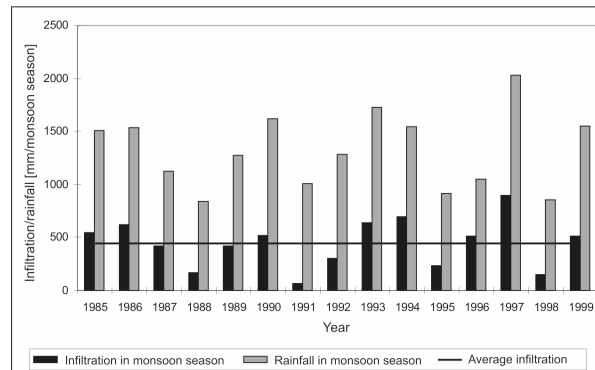
## RESULTS AND DISCUSSION

### Groundwater Budget

The simulation of the historic situation of the years 1985 to 1999 shows that the average groundwater recharge in the study area rate was 217 mm/a in the year 1985 with the total groundwater budget estimated at 368 mm. The difference of 151 mm/a was attributed to infiltration from surface water and from the subsurface inflows. Due to the increasing agricultural demand the recharge rate was increased to 250 mm/a in 1997 where 25 mm/a (10%) as a result of irrigation losses. Hence, by including the infiltration from surface water and the subsurface inflow the total groundwater recharge in the study area was estimated at 406 mm/a. A comparison of these results with the groundwater pumping rates shows that in 1999 more groundwater was withdrawn than recharged. The total renewable groundwater including the groundwater-surface water interaction and the subsurface inflow was smaller than the total withdrawal in 2003 resulting in a non-sustainable groundwater use. This was confirmed from the observed declines in the regional groundwater table.

### Managed Aquifer Recharge

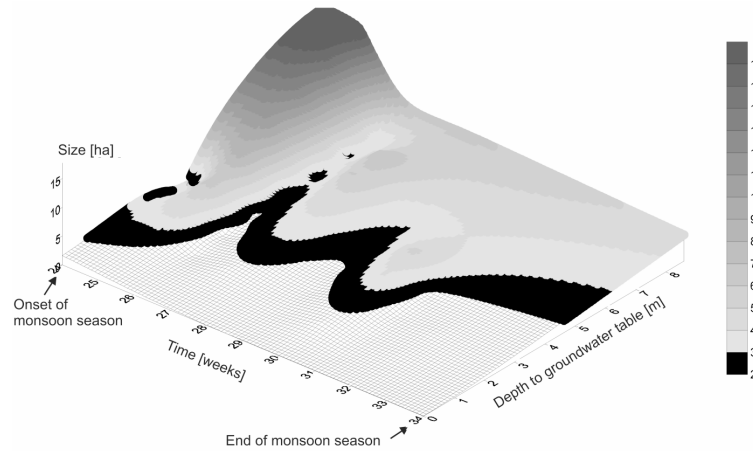
Fig. 4 shows the rainfall during the monsoon season from 1985-1999 and the calculated surplus water beyond irrigation requirements (calculated by CROWAT (FAO, 1992)). The average available surplus water potentially available for managed aquifer recharge was 445.8 mm/a but this has a high spacio-temporal variability, with water availability peaking in September (126.5 mm). The lowest available water can be found in the beginning of the monsoon season (21.9 mm in end of May).



**FIGURE 4. Rainfall and surplus water in monsoon season.**

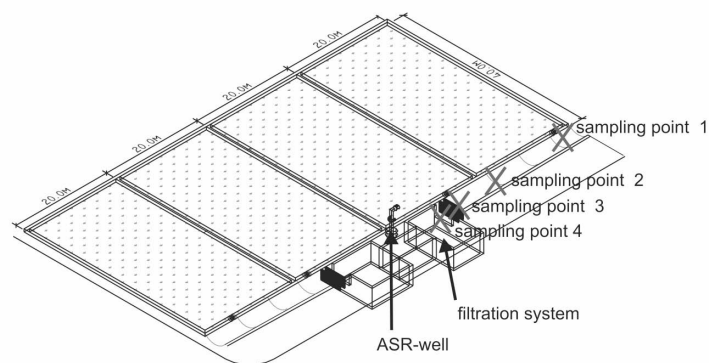
Since surplus water is only injected by gravity, the rate is linearly dependent upon the depth to the groundwater table (eq. 4). Fig. 5 shows the optimised collection area for

one ASR-well such that all surplus water is available for injection starting from onset of the monsoon season. The results are depended besides  $T$ ,  $n_{eff}$  and the depth to groundwater table (eq. 10a/b) mainly on the average available surplus water and the expected groundwater rise due to MAR and natural recharge. Fig. 5 shows that in the beginning of the monsoon season the optimal field size is up to 15 ha. This area is based on the large depth to water table and the low available surplus water at this time. At the end of the monsoon the water can be only infiltrated if the depth to the groundwater table is sufficiently large (larger than 4 m). This results in a lower available surplus water volume because under current conditions the groundwater reaches in an average year the surface 7 weeks after onset of monsoon season (Fig. 3).



**FIGURE 5. Field area per ASR-well to enable all surplus water to be infiltrated during the monsoon season.**

A major key factor for an economical successful application of this groundwater management concept is the field size which infiltrates the runoff surplus water through an ASR-well. Fig. 6 shows the layout of the experimental field at the Indian Institute of Technology Kharagpur, India (Panda, 2002). Four rice fields, 0.08 ha each are connected through drainage systems to a surface filter system and an ASR-well. The surplus water is temporarily stored in the drainage system and on the fields prior to injection.



**FIGURE 6. Layout of test fields with filter system and ASR-well.**

### Groundwater Management

This pilot plant includes a drainage canal of 80 m length, 1 m depth used for 0.64 ha field (if both field sides are connected to the canal). In order to reach an economically 2 to 3 ha field size (2 ha: black & grey area, 3 ha: grey area in Fig. 5) (Holländer, 2005), a canal length of 130 to 190 m with a maximum flow length of 65 to 95 m is required. However, the actual length of the canal needs to be related to fit the topography of the landscape. Fig. 5 shows that using field sizes of 2-3 ha (black and gray coloured area). This size is optimal for maximising surplus water injection including economical aspects.

An analysis of the incremental withdrawal of groundwater in winter revealed that during the first years the amplitudes of the groundwater mounds were increasing and that the groundwater table reached the surface during the monsoon season indicative of an increasing natural recharge and suggestive of a sustainable system. With a higher withdrawal in the winter period it was observed that the groundwater table declined faster than in the non-irrigated scenarios. Therefore, the evaporation losses from the groundwater table which depended on the depth of groundwater table were lowered by up to 105 mm/a.

The maximum natural recharge under sustainable conditions increased to 330 mm/a. Utilizing the ASR well field design criteria discussed above, 218 mm/a can be infiltrated in an average year resulting in an additional 175 mm/a available through subsurface inflow and surface-groundwater interactions. This results in a total amount of available groundwater of 723 mm/a which can be sustainably used in an average year. If more groundwater is extracted the break-even point of available groundwater to withdrawal is quickly reached and the groundwater table begins to decline. The 723 mm/a water allows the growing of a second crop in the winter throughout most of the study area.

The increasing groundwater fluctuations cause several issues. During the dry season the groundwater table falls below the suction level of centrifugal pumps which are widely used so that submersible pumps are required which are more expensive and which need a higher maintenance. Another potential issue arising from the lowering the groundwater table is a temporal negative gradient from the Bay of Bengal into the study area. This could cause saltwater intrusion into the aquifer. In order to limit saltwater intrusion, a coastal strip of about 2 km buffer zone should be excluded from the groundwater management concept. This would result in a hydraulic barrier so that the outflow of fresh water would be about 8 times higher than intrusion of saltwater thereby maintaining groundwater quality.

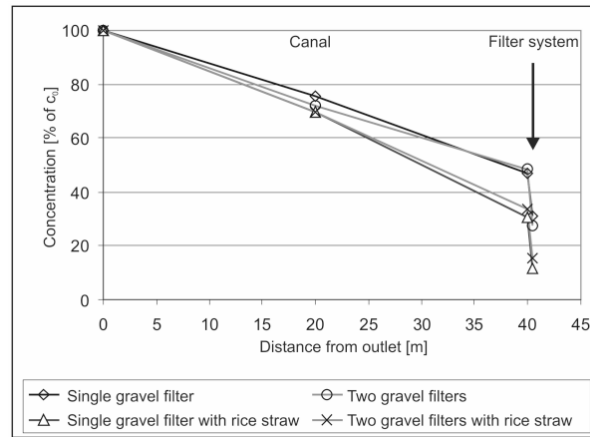
### Field Experiment

The major problem in the application of this groundwater management concept in the field (Fig. 6) is the clogging of the ASR well due to the presence of silt and clay particles in the infiltration water. At the experimental field 4 measurement station were defined (Fig. 6): (1) outflow of the most far away field (travel length 0 m), (2) in the middle between outlet and filter (travel length 20 m), (3) in front of the filter (travel length 40 m) and (4) behind the filter (travel length 40.7 m). In order to find a low-cost but effective filter system 4 filter set-ups were tested: (F1) single gravel filter, (F2) two gravel filters, (F3) single gravel filter and compressed rice straw and (F4) two gravel filters with compressed rice straw in between.

Fig. 7 shows that 50-70% of the particles were removed by sedimentation in the drainage system. The larger the total filter length, the better was the removal rate in the channel due a higher resistance in the filter and a resulting lower flow velocity and a longer travel time. All filter systems had in all cases nearly the same removal rate of 20% but they were developing different flow gradients through the filter so that the sedimentation in the channel differs. In total removal rates of 70 to 90% were achieved. The problem was that the final particle concentration (as total suspended solids) was still about 800 mg/l for the best systems (filter systems with embedded rice straw). Panda (2002) tested other materials and brought concentrations down to 180 mg/l using a filtration system made of gravel filters and embedded Cocos Matts. Still these concentrations would appear not to be suitable for a water quality to prevent clogging in these low to moderately transmissive porous aquifers (Reddi, et al., 2000; Holländer, 2005).

The high particle concentration in the water was due to an artificial rain at the field 4. The artificial rain of 100 mm within one hour produced by sprinklers may have produced a too high particle load due to the barren land, to large water drops, the high intensity and a very short retention time on the fields. Therefore, the experiment needs to be seen as a worse-case scenario.





**FIGURE 7. Average relative TSS-concentration (Total Suspended Soils) in relation to the travel length in a drainage channel and a filter system.**

Therefore, a better retention of particles needs to be secured to develop a sustainable infiltration technique. This can be based on a longer sedimentation time, better (artificial) filter systems and/or the addition of coagulation chemicals (e.g. polymers, metal salts). Additional laboratory experiments based experiments showed that biological clogging becomes another key factor in clogging of the aquifer and therefore for application of the groundwater management concept in the field (Holländer et al., 2005; 2006).

## CONCLUSIONS

Increasing population and associated agricultural water demand are important challenges for implementing more sustainable groundwater management. This study introduces a new groundwater management concept based on regionally applied MAR using ASR-wells. A modelling study of this concept in a coastal plain area in Orissa, India revealed that the groundwater resources available for irrigated agriculture could potentially be sustainably enhanced. Groundwater withdrawals during the winter season permitted growing an additional crop in the winter and resulted in a larger natural groundwater recharge (80 mm/a), facilitated by injection through ASR-wells (218 mm/a) and minor contributions from the higher infiltration of surface water into the aquifers (13 mm/a) resulting in the total of available groundwater increasing from 368 mm/a to 723 mm/a.

Field testing of the groundwater management concept identified that the performance of a surface water filtration system was inadequate to reduce particle concentrations to a level that could be used for sustainable injection. Therefore, further research and development is needed to find a suitable pre-treatment

technology which cheaply and reliably provides a sufficiently low particulate concentration under low cost conditions.

## ACKNOWLEDGEMENTS

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# EXPLORING THE POTENTIAL OF MANAGED AQUIFER RECHARGE TO MITIGATE WATER SCARCITY IN THE LOWER JORDAN RIVER BASIN WITHIN AN IWRM APPROACH

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## ABSTRACT

The Lower Jordan River Valley is a place of extreme water scarcity and constitutes an overexploited closed river basin. No surface runoff currently leaves the area and the water level of the Dead Sea, as the final sink, has already dropped by more than 20 m over the past fifty years as a result. This demonstrates, that even apart from water quality considerations, total inflows into the system do not match outflows, resulting in a continuous depletion of the available storage, i.e. all available water is already utilised. The only means of providing additional volumes of water to the area are water imports or reduction of evaporation from open water bodies or reduction of evapotranspiration from irrigated agriculture. Wastewater reuse and desalinisation would increase the amount of water fit for human demand but not affect the water balance in total. Increasing the amount of managed aquifer recharge (MAR) would be beneficial to the water availability of the region by reducing evaporation. In this background setting, the research initiative SMART (Sustainable Management of Available Resources with Innovative Technologies) has now been launched to include all available water resources of the Lower Jordan River, namely ground water, waste water, saline water, and flood water into an integrated management concept. This paper briefly explores the application of MAR technologies to the region and the barriers toward their implementation. Currently MAR technologies which aim at direct infiltration of water into the underground are not, or only at a very limited scale, implemented in the Lower Jordan River Valley. Dams and reservoirs which have been installed at the outlet of major wadis provide additional groundwater recharge but are characterised by high evaporation losses and progressive silting. Harvesting of rainwater in man-made subsurface structures (e.g. cisterns) is common in rural settings of both Palestine and Jordan. Soil aquifer treatment (SAT) concepts are successfully implied in Israel, although not in the Jordan River Basin. A significant contribution to groundwater recharge is provided through irrigation schemes with either freshwater or recycled water. Considering the prevailing aquifer characteristics (limestone and alluvial aquifers with sufficient hydraulic conductivity), MAR is considered feasible. Strong episodic rainfall runoff, especially from urban areas in the highlands, would be available to feed the MAR schemes. However, given the steep topography, often the karstic aquifers would not provide appropriate long term storage but discharge the water soon afterwards. A common concern is the quality of urban surface runoff which may require source control measures in the catchment. Due to rapid groundwater flow velocities and short residence time of contaminants in the aquifer system, karstic aquifers are particularly vulnerable to contamination. Careful site selection must take place to maximize the probability of success. At this point, major targets for managed aquifer recharge schemes could be the alluvial fan aquifers as well the carbonate aquifers in the western part of the study area. In conclusion, a series of detailed investigations on MAR applicability in the region is needed for future planning. Besides the screening for suitable aquifers, it is

recommended to use information from integrated water master planning tools, such as availability of source water and water demand for in depth feasibility studies of MAR.

## KEY WORDS

Water resources, Jordan, Palestine, Israel, artificial recharge

## INTRODUCTION

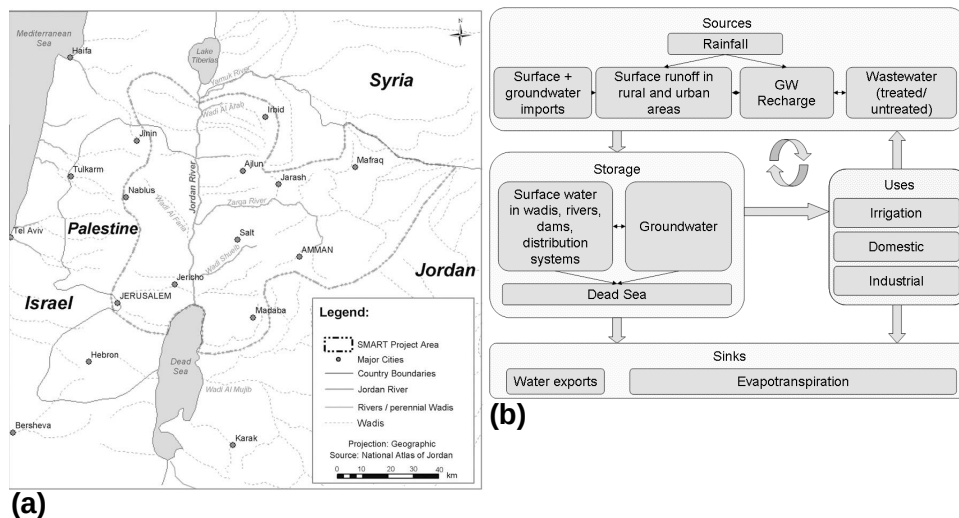
### Project background and objectives

The German Federal Ministry for Research and Technology, considering the Resolution 58/217 of the United Nations dated 20 December 2000, is supporting a research program for the Integrated Water Resources Management (IWRM) “in regions with water shortages. The SMART project, *“Sustainable Management of Available Water Resources with Innovative Technologies”* in the lower Jordan Valley is one of these projects. The consortium comprises more than 17 partners from universities, research centers, companies and NGOs from Jordan, Israel and Palestine ([www.iwr-smart.org](http://www.iwr-smart.org)). IWRM is a process that aims to promote the coordinated development and management of water, land and related resources. The purpose is to maximize the resultant economic and social welfare in an equitable manner without compromising the sustainability of vital ecosystems (GWP 2004).

The main idea of SMART is to include all water resources of the Lower Jordan River, namely ground water, waste water, saline water, and flood water into an integrated management concept. These issues are explored with a series of test sites along both sides of the Jordan Valley. Test sites are planned for infiltration of reclaimed wastewater, infiltration of water from flash floods, infiltration of urban surface runoff and irrigation of agricultural area with treated sewage. The test sites are embedded into several water balance studies and finally, a numerical groundwater flow model will be constructed for the entire Lower Jordan Valley. A separate team of socio-economists will assess the water demand in different sectors (e.g. agriculture, urban population) along with likely future trends. All information will then be integrated into a decision support framework (DSS). The DSS is based on the DPSIR approach (Drivers, Pressures, State, Impact, and Response). Together with key stakeholders from the three involved countries, scenarios of current and future water management strategies are set up. These scenarios are evaluated with special regard to their environmental, social and economical impacts on the region. Within this concept, SMART explores the role of MAR in the IWRM strategies of the Lower Jordan River.

## Geographical Location/ Study Area

The investigation area covers the Lower Jordan River Valley from the southern shores of the Sea of Galilee (Lake Tiberias) to the northern part of the Dead Sea (Figure 1a). It comprises an area of about 8000 km<sup>2</sup>. Three countries are neighbouring and sharing this basin and its margins: Jordan, the Palestinian Authority and Israel. The dominating tectonic element of the Jordan River Valley is the Dead Sea Transform (DST), a segment of the East African – Red Sea Rift System. At the northern shores of the Dead Sea the valley floor is at ca. -400 m below sea level whereas the surrounding highlands reach on average 800 m above sea level. The Jordan River flows along the valley from Lake Tiberias in the north to the Dead Sea in the south. The waters of the Jordan River are an extremely important resource to the dry lands of the area and are a politically sensitive issue between Lebanon, Syria, Jordan, Israel and Palestine.



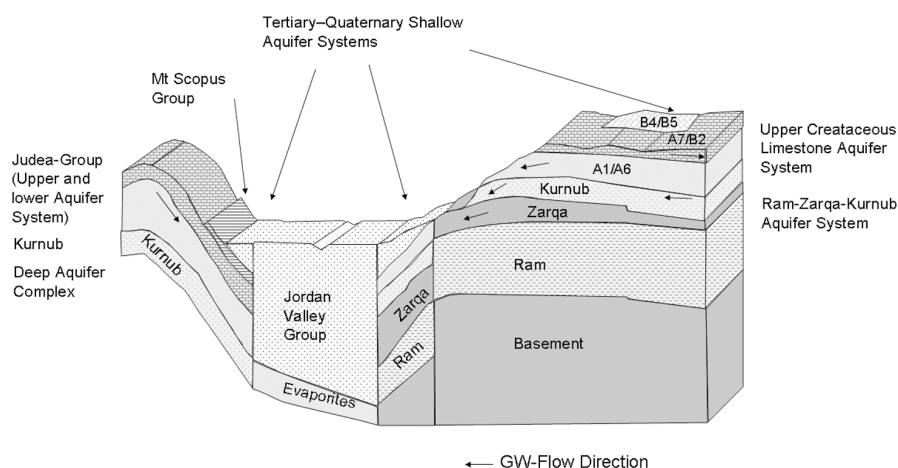
**FIGURE 1. (a) Surface watershed of the Lower Jordan River Basin excluding Syrian parts. (b) Key elements of the water cycle in a closed river basin: the case of the lower Jordan River.**

## WATER RESOURCES STATUS IN THE LOWER JORDAN RIVER BASIN (LJRB)

### General Situation in a Closed Basin

The study area constitutes a closed river basin with a pronounced water budget deficit. A progressive closure of the basin means in this case that almost no water is left to be mobilized and used while demand, notably in urban areas, keeps increasing (Venot, Molle et al. 2006). Figure 1b lists the key elements in the anthropogenic

modified water cycle of the LJRB. The final sinks of water in the LJRB are evaporation and water exports, there is no surplus water running to the open sea. The area is characterised by severe water scarcity. Aquifers are seriously overexploited and groundwater levels have been dropping in recent decades. For Jordan, the available renewable water resources decreased drastically to an annual per capita share of 160 m<sup>3</sup>/cap/y in 1997, compared to 3600 m<sup>3</sup>/cap/y in 1946 (population growth is a major driver for this). Owing to diversion of tributary waters and intensive mineral extraction, the level of the Dead Sea is dropping at a rate of up to one meter per year (Becker and Katz 2006). As a result, the surface area of the sea has already shrunk by one-third, springs around the sea are drying up and sinkholes (areas of severe land subsidence) are forming, threatening historical sites and infrastructure. (Salameh 2001a)



**FIGURE 2. Schematic hydrogeologic profile of the Lower Jordan Valley. Assembled from various sources (Salameh and Udluft 1985; USGS 1998; The Hashemite Kingdom of Jordan 2004; Sauter 2006).**

The quality of surface water is deteriorating due both to reduction in natural flow volumes but also due to the many known and unknown releases of sewage into surface water. The situation is pronounced at the Lower Jordan River, whose waters were also historically more saline than the waters north of the Lake Tiberias (Sea of Galilee) and of lower quality (Nissenbaum 1969 in Farber et al. 2005). While flows of untreated wastewater in the wadis obviously constitute a pollution hazard, they still provide an augmentation of the Lower Jordan River baseflow. If the sewage effluents are reduced as agreed in the Jordanian-Israelian Peace Treaty, the chloride concentrations in the Jordan River are expected to increase from up to 2600 mg/l at present to almost 7000 mg/l, due to the proportional increase of saline groundwater discharge into the river (Farber, Vengosh et al. 2005). Furthermore, intentional discharges of saline water into the Lower Jordan River are present. Around the Sea of Galilee, the so called saline

water is collected from several saline springs on the western shore and redirected into the Jordan in order to avoid salinisation of Lake Tiberias.

While governments of the region have expressed concern over the threats to the ecological and cultural heritage of the Dead Sea basin, they have traditionally seen the loss of the Dead Sea as an unavoidable consequence of rational economic policy (Becker and Katz 2006, Salameh 2001b). The diversion of water from the Yarmouk River and the Jordan River are the main drivers of this loss. In both cases, the majority of this water is dedicated to agriculture, which represents only 1.7 % of the gross domestic product in Israel (Central Bureau of Statistics Israel 2004), and 2.2. % of the gross domestic product in Jordan (World Bank 2005), and with only minor contribution to employment. Thanks to management efforts, freshwater demand for irrigation is now slowly decreasing in Jordan, while urban water demand is constantly on the rise. Irrigated agriculture in Jordan is the largest user of water, constituting 64% of the total water used in the country in 2002. Of the total renewable groundwater supplied for all uses estimated at 432 MCM/year, consumption of irrigated agriculture accounted for about 50%, or some 216 MCM in 2002 (The Hashemite Kingdom of Jordan 2004).

## Groundwater

As groundwater is the major source of drinking water in the Lower Jordan River basin, hydrogeological aspects exert a dominant influence on the water management. Especially on the West Bank, groundwater is the most important source of fresh water supply in the area. The tectonically and sedimentologically complex setting in the LJRB produces a large number of local and regional aquifers. While major lithostratigraphic units were mapped in the region, hydrodynamic connections between aquifers and the borders of subsurface drainage basins are still a research topic, especially on the eastern side of the Jordan. While the geology east and west of the Jordan Valley shows a similar succession of lithostratigraphic units, a displacement of more than 107 km in the north-south direction is present. The various local aquifers may be grouped into three major aquifer systems:

1. Tertiary-Quaternary Shallow Aquifer System: Alluvial aquifers are present at the floor of the Jordan Valley and the fans of the incoming wadis, where the alluvium is in contact with the aquifers of Upper Cretaceous age (Ailun series). The alluvial aquifer extends from the northern shore of the Dead Sea in the south to the downstream part of the Yarmouk River in the north. The thickness of the alluvium in the Jordan Valley varies from zero along the eastern boundary to about 750 m in the deepest part of the basin near the Jordan River. An average thickness of 400 m may be reasonable for the purpose of hydrological considerations (The Hashemite Kingdom of Jordan 2004). The deposits on the eastern side of the Jordan Valley may be further grouped into JV1 (Ghor el Khattar Formation), JV2 (Ubeidiya and Samra Formation), JV3 (Lisan formation) and quaternary alluvial fans (Salameh 2001). In parts water quality of the shallow aquifer is impacted by



salts origination from the evaporitic parts of the Lisan formation (Al Kuisi 2006). Typical hydraulic conductivities are cited as  $6.6 \text{ m s}^{-1}$  (JICA 1995). In addition to the alluvial sediments, also chalk and limestone of tertiary age locally comprise important aquifers such as the B4/B5 in the area northwest of Irbid and Azraq, Sirhan, Jafr and the Hammad basins. On the western side of the Jordan Valley, the term Shallow Aquifer Hydraulic Complex is used. It comprises Pleistocene sedimentary and alluvial deposits of the Quaternary age which receive localized annual recharge from wadi flows. The extent to which this aquifer is recharged from lower aquifers has not been determined and may be a function of faulting and fracturing. Recent work in the Arava area on the western side of the wadi showed that the thickness of the permeable sediments deposits could be up to 550 meters in depth

2. Upper Cretaceous Limestone Aquifer System: This complex consists of an alternating sequence of limestones, dolomites, marl stones and chert beds. The total thickness in central Jordan is about 700 m. It can be divided in the Lower Ajlun aquitards/aquifers and in the Upper Ajlun A7/B2 Aquifer. On the western side of the Jordan, the aquifer system is known as the Judea Group (or also as Cretaceous Hydraulic Complex) and also here it is subdivided in at least an upper and a lower aquifer system. In terms of extracted volumes, this is the most important aquifer system in the region. It receives the major part of the groundwater recharge in the area, occurring mainly in the high mountain regions on both sides.
3. Ram-Zarqa-Kurnub Aquifer System: It includes the Ram aquifer, the Khreim aquitard, the Zarqa aquifer and the Kurnub aquifer (The Hashemite Kingdom of Jordan 2004). The Ram Group Aquifer (Disi) forms a large regional aquifer system, which underlies the entire area of Jordan and parts of Saudi-Arabia. It is also present west of the Jordan river in Israel and Palestine but not a major water source there to its depth below surface. It crops out only in the southern part of Jordan and along Wadi Araba – Dead Sea Rift Valley. Currently, the total existing abstraction, including that from Saudi Arabia, exceeds the safe yield of the aquifer and this source is generally considered as unreplenishible. The Palaeozoic sedimentary sequence consists mainly of sandstone interbedded with siltstone, mudstone, limestone and dolomite. The average thickness of the formation is about 1000 m and increases to 2500 m to the east. In general the Ram Group dips gently to the east and to the north at about 5 degrees. The aquifer can be characterized as a fractured-rock aquifer. Groundwater movement is primarily through secondary openings, such as joints, fractures, and bedding-plane openings. The Khreim overlies the Ram Group and consists of about 600 – 800 m of sandstone, siltstone, and shale which is dated to be of Middle Ordovician-Upper Silurian age. It has a low permeability and forms a confining layer to the Ram Group aquifer and is therefore regarded as an aquitard. On a regional scale, water in the overlying sandstone aquifers (Zarqa and Kurnub) is interconnected with that of the Ram by leakage. As a consequence, the Ram and the upper sandstone aquifers along with

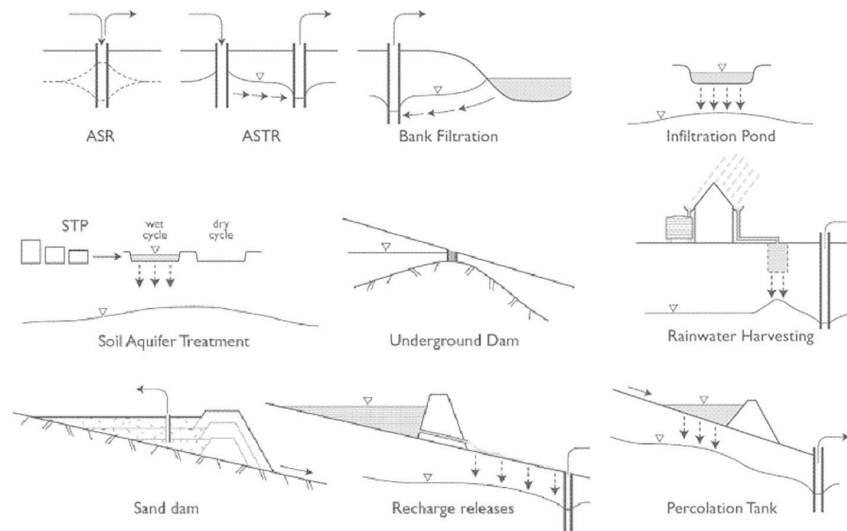
the Khreim are considered to be one hydraulic system The Zarqa Group is limited to north and northeast of Jordan. In the southern part of the country the Zarqa Group is missing. Outcrops of this group are limited to the lower Zarqa River basin and along the escarpment between the Rift and the Highlands to the east. The Mesozoic sedimentary sequence consists mainly of sandstone interbedded with siltstone, limestone and dolomite. The thickness of the formation increases from the central part of Jordan to the north and west up to 1800 m. The Kurnub Group is of Lower Cretaceous age. It consists mainly of sandstone. For the movement of groundwater the intergranular porosity of the sandstones is of minor importance, because most of the intergranular space is filled with siliceous cement.

## **AVAILABLE MAR CONCEPTS AND THEIR CURRENT IMPLEMENTATION IN THE REGION**

### **Introduction to MAR**

Managed aquifer recharge (MAR) encompasses a whole suite of methods and describes intentional storage and treatment of water in aquifers. The term 'artificial recharge' has also been used to describe this, but adverse connotations of 'artificial', in a society where community participation in water resources management is becoming more prevalent, suggested that it was time for a new name. Managed aquifer recharge is intentional as opposed to the effects of land clearing, irrigation, and installing water mains where recharge increases are incidental (Gale 2005). Figure 3 shows the basic types for MAR but the actual implementation of schemes is varying widely with different concepts in many cultures. Typical goals of managed aquifer recharge perceived in the region are to maintain and increase the natural groundwater as an economic resource. (ii) avoid further salinization and salt water intrusion (iii) decrease losses due to evaporation (iv) create long-term or short-term water storage (v) provide treatment and storage for reclaimed wastewater for subsequent reuse). Table 1 lists the applications of MAR in the three riparian states of the LJR.

While there is more than a millennium of experiences of using rainwater and surface runoff in rural areas by various forms of rainwater harvesting, the use of treated wastewater is young in comparison. Nowadays intentional replenishment of aquifers by highly treated reclaimed waters is increasingly being practised in developed countries with the full support of communities, and health and environment regulators, for aquifers that are under stress through imbalances between rates of extraction and natural recharge (Dillon, Toze et al. 2004; Dillon and Jimenez in press). With strong population growth in many urban centres and reduction of agricultural water demand by use of innovative irrigation technologies, the need to set up more sustainable urban water systems becomes obvious.



**FIGURE 3. Schematic of types of management of aquifer recharge (Dillon 2005). Abbreviations: ASR=Aquifer Storage & Recovery, ASTR=Aquifer, Storage, Transfer & Recovery, STP = Sewage Treatment Plant.**

**Table 1. Methodologies for Managed Aquifer Recharge (adapted from Gale 2005) and their applications in the Jower Jordan River Basin.**

| General methodologies for MAR             |                                                   | Applications in the LIRB  |
|-------------------------------------------|---------------------------------------------------|---------------------------|
| <b>Spreading methods</b>                  | Infiltration ponds and basins                     | Israel                    |
|                                           | Soil Aquifer treatment                            | Israel (Dan region)       |
|                                           | Controlled flooding                               |                           |
|                                           | Incidental recharge from irrigation               | Jordan, Palestine, Israel |
| <b>In-channel modifications</b>           | Percolation ponds behind check-dams, gabions, etc | Jordan, Israel            |
|                                           | Sand storage dams                                 |                           |
|                                           | Subsurface dams                                   |                           |
|                                           | Leaky dams and recharge releases                  | Jordan, Israel            |
| <b>Well, shaft, and borehole recharge</b> | Open wells and shafts                             |                           |
|                                           | Aquifer storage and recovery (ASR)                |                           |
|                                           | Aquifer storage, treatment and recovery (ASTR)    |                           |
| <b>Induced bank filtration</b>            | Bank filtration                                   |                           |
|                                           | Inter/-dune filtration                            |                           |
| <b>Rainwater harvesting</b>               | Field bunds, agricultural ponds                   | Jordan, Palestine, Israel |
|                                           | Roof-top rainwater harvesting                     | Jordan, Palestine, Israel |

Examples for successful MAR applications at international level are numerous (Dillon and Jimenez in press). Runoff from urban areas, in particular, still provides a large potential of additional groundwater recharge. Besides roof runoff, also runoff from paved areas can be utilized successfully without health threats (Nolde 2007). Scenario simulations of the urban water balance using the AISUWRS approach (Wolf, Morris et al. 2006) showed how the city of Mt Gambier, Australia is effectively reducing surface runoff by injecting stormwater into a karstic aquifer: compared with three European cities Mt Gambier recharges 45 % of its total water input and discharges only 12 % via sewers and surface runoff, whereas the European counterparts discharge between 34 % and 70% of their total water input. The stormwater infiltration is ongoing since more than a century and the sustainability has been positively evaluated within a Hazard and Critical Control Point approach (Cook, Vanderzalm et al. 2006). In addition, long term evaluations (> 5 years) are documented for the pioneer site of Andrews Farm, South Australia, where urban stormwater is injected into a brackish limestone aquifer and used to generate an additional water resource (Pavelic, Dillon et al. 2006).

## MAR EXAMPLES IN JORDAN

At this time, only a limited number of MAR applications are openly documented in Jordan. The major categories are rainwater harvesting in field bunds or at river beds, small check dams inside wadis and large reservoirs with an incomplete bottom sealing.

Rainwater collection and storage schemes are traditionally carried out in Jordan and continued especially in rural villages. One of the techniques involves the filling of excavations close to wadi beds with a clay liner at the bottom, coarse rocks in the middle and a cover at the top. This structure is then used as a reservoir with minimum evaporation in the dry months (Salameh 2004). New attempts to improve the systems by the use of capillary barrier systems are ongoing in the Karak area (A. Hamadeih, pers. comm.). For the future, some authors recommend that rain-water collection and storage schemes on large and/or small scale should be encouraged because in the long term these schemes would have an important role in securing sustainable water supplies in the Kingdom (Jaber and Mohsen 2001). In the year 1991 it was estimated that the developed quantities of water harvesting would reach 6 MCM by the end of the year 2000 (Pride Team 1992). Net additional water conservation gains for rainfall/runoff water harvesting from residential and industrial roofs was estimated to be around 4.3 MCM and 9.5 x MCM for the years 2005 and 2010, respectively (Jaber and Mohsen 2001).

A number of dams were built in Jordan, partly with the aim of groundwater recharge. A popular example for an effective recharge scheme is the Walah dam but also the Rail dam and the Siwaqa dam are described (Pavelic 2005). The first two are located

in the Mujib basin, the last one in the Azraq basin. *Sensu strictu*, none of them belongs to the catchment of the LJRB.

The Siwaqa dam is situated 70 km south of the capital, Amman and is recharging the Upper Cretaceous B2/A7 aquifer, belonging to the Aljoun Group. Groundwater levels in this area are in the range of 100m below surface. A rise of 15 m in the groundwater table in wells close to the dam has been reported. While the scheme was effective in the beginning, concerns are arising that the accumulating fine sediments in the reservoir are effectively sealing the bottom. The Wala dam is situated 45 km south of Amman in the Mujib basin. The dam was constructed in 2003 and has a capacity of 9.3 x MCM with a catchment area of 2000 km<sup>2</sup>. The dam recharges the B2/A7 aquifer and contributes to the yield of the wadi Hidan well-field located west of the dam, which supplies water for the cities of Madaba and Amman Wellfields. Rates of recharge are dependent on the water level within the dam and can be up to 1 m/day when dam capacity is high, to <0.05 m/day at low capacity. The estimated volume of water recharged over the past 3 years is 30 MCM (Pavelic 2005).

In addition to the dams, a major recharge pond is reported at a plant near Aqaba where 1.91 MCM/y are infiltrated to an aquifer (The Hashemite Kingdom of Jordan 2002, Salameh 2002).

A different MAR strategy is the construction of several small dams along the wadi course to increase infiltration. In a desert location 25km northeast of Amman, a feasibility study on this was carried out (Chehata and Dal Santo 1997), which focused on the wadi's Madoneh and Butum. Direct infiltration tests in the wadi beds delivered promising results (Abu-Taleb 1999) and detailed design criteria based on hydrological studies in the area were reported (Chehata, Livnat et al. 1997; Abu-Taleb 2003). The amount of expected recharge from the structures was estimated as 0.075 MCM/y. The project is now supported by EXACT via UNESCO-IHE together with the MWI and the construction of the dams is foreseen in 2007 (de Laat and Al-Nsour 2007). The calculation of runoff coefficients and flow volumes was recently revised based on updated hydrological time series data (Dhakal 2006). The scheme in Wadi Madoneh is recharging the Wadi-Es Sir formation as part of the B2/A7 aquifer, partly also via more recent alluvial bodies in the wadi bed.

From an institutional viewpoint it is concluded that the water scarcity and the high demand on water for irrigation as well as the location of plants do not allow for planned recharge projects at the moment although studies indicated the possibility of recharging the aquifers in some areas (The Hashemite Kingdom of Jordan 2002)

## **MAR EXAMPLES IN ISRAEL**

Managed aquifer recharge forms an integral part in the Israeli water management strategy with the main goals to (i) increase the available water resource, (ii) provide

aquifer management, (iii) utilize the storage capacity of the aquifer, (iv) reuse treated effluents (Guttman 2007). So far, only limited application exists in the LJR as the main focus areas of the aquifer recharge projects are the heavily used coastal aquifer and the surroundings of major urban centres.

One of the world largest Soil Aquifer Treatment plants is operated in the Dan region already since 1977, reclaiming some 110-130 MCM of secondary treated wastewater per annum (Idelovitch, Ickson-Tal et al. 2003). It is situated within the coastal plain aquifer, comprised mostly of Quaternary sands and calcareous sandstones with some alternating units of sandstone and clay. After 25 years of unproblematic operation high Manganese concentrations were observed in the recovered water. It is suggested that the Manganese is mobilized from the aquifer rocks as a result of the changed redox conditions (Oren, Gavrieli et al. 2007).

A successful project to use stormwater runoff in the Menashe artificial recharge plant at the northern part of the coastal aquifer about 3-4 km (Guttman 2007). With a catchment area of 189 km<sup>2</sup>, and geology composed mainly of limestone and chalk, the plant manages to provide an artificial recharge of 17 MCM per annum to a heavily used sandstone aquifer. Recharge takes place via spreading ponds. Groundwater levels have risen significantly since the start of the operation, indicating the success of the scheme. As the system is driven by gravity, only minor costs arise from maintenance. The water pumped from the recharged aquifer is used for drinking purposes (Guttman 2007). Another example is the artificial recharge scheme of the Shikma river with a catchment area of approx. 750 km<sup>2</sup>. The system is designed to cope with the strong seasonality of the rainfall. In this case a dam construction diverts the water into a sedimentation reservoir from which it is transferred to a recharge pond after settling. Around the ponds a series of wells is recovering the recharged water and ensures that water levels below the pond are low enough to allow further infiltration (Guttman 2007).

## MAR EXAMPLES IN PALESTINE

Due to the ongoing, unfavourable political situation in Palestine, there has been little scope for the construction of artificial recharge sites in the last decades. Current systems centre on rainwater harvesting, looking back upon long tradition and experience. Within the local context, managed aquifer recharge examples are grouped under the section “non conventional technologies”. Popular methods are (i) covered, underground reservoirs (locally called wells or cisterns) or (ii) pools made from earth or steel, covered with black plastic sheets to prevent algae growth (Carlo and Ghanem 2007). The cisterns supply an estimated 6.6 MCM per year within the WestBank. Cisterns serve an essential purpose, meeting water needs left unfulfilled by the devastated infrastructure. In most cases, cisterns collect water from rooftops during the rainy season, which is then stored in subsurface containers, usually

ranging in size from 60-100 cubic meters. A large percentage of water collected in cisterns is used for domestic purposes. Tankers are also used to fill cisterns, especially in the summer months when the cisterns dry up due to the lack of rainfall (Shehabadeen and Ghanem 2007).

While these systems are effective in storing rainwater and securing the water supply during dry times, they are not recharging the groundwater directly. Direct recharge, however, is achieved through the numerous retaining walls on the agricultural fields. The retaining walls hinder surface runoff and enforce downward infiltration of water.

Incidental recharge occurs from a significant number of cesspits, but is associated with high nutrient and contaminant loads.

### **SUMMARY: POTENTIAL AND BARRIERS TO FUTURE MAR APPLICATIONS IN THE LJRB**

In general, the overexploitation of all aquifers in the region calls for recharge enhancement. Even without a recovery of the injected water close to the recharge site, a major environmental benefit will be achieved. Aquifers with high transmissivities are available close to the surface and successful examples are already implemented in the region. A summary of aquifer characteristics and their influence on the potential for managed aquifer recharge was recently given (Dillon and Jimenez in press). Its application to the general aquifer types in the LJRB is listed in Table 2.

However, a range of implementation barriers is given. Beyond the hydrogeological constraints also the source of recharge water, the proximity to this source, the quality of the source and the availability of the source water must be evaluated. Within the LJRB, both surface runoff and wastewater are available as sources. While a significant amount of wastewater is produced in the urban areas, it is as far as possible used for irrigation following treatment. However, only between 50% and 80% of the population are already served by sewerage systems. The alluvial fan aquifers at the inlets of the wadis to the Jordan Valley offer a good potential for MAR due to their hydraulic conductivity, the gentle gradients and the long retention time. A major concern for the alluvial aquifers in the Jordan Valley is the mixing of the recharged water with saline groundwater. However, international studies have demonstrated that the mixing can be very low due to the slow groundwater movement and that efficient recovery of recharge water is possible (Pavelic, Dillon et al. 2006).

The Upper Cretaceous limestone aquifer system offers ample storage space but its use must be planned carefully due to the many fast discharge options via springs. Chances for successful recharge projects are given east of the water divide in the A7/B2 aquifer between Jordan Valley and the desert basins. Water recharged there will benefit the wellfields which supply the towns on the mountain ridge (e.g. Amman).

**Table 2. Aquifer characteristics relevant for MAR: status and implications for the three major aquifer systems in the LJR.**

| Characteristics                             | Status & Implications                                                                                                                                              |                                                                                                                                                         |                                                                                                                                          |
|---------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------------------------------------------------------------------------------------------------------------------------|
|                                             | Tertiary-Quaternary Shallow Aquifer System                                                                                                                         | Upper Cretaceous Limestone Aquifer System                                                                                                               | Ram-Zarqa-Kurnub Aquifer System                                                                                                          |
| <b>Confinement</b>                          | Groundwater mostly occurs unconfined.<br>>Surface infiltration methods viable, unprotected from surface contamination                                              | The aquifer system contains confined and unconfined sub-aquifers. Depth to water table is very high.<br>>Wide range of infiltration mechanisms possible | Generally confined aquifers, >Only well injection methods possible                                                                       |
| <b>Permeability</b>                         | Moderate to high permeability<br>>dispersion of recharged water, cheap recovery costs of pumped water                                                              | Moderate to high permeability<br>> dispersion of recharged water, storage potential sometimes decreased due to quick discharge                          | Moderate to low permeability<br>> recharged water more localized<br>> higher recovery costs                                              |
| <b>Thickness</b>                            | Sometimes limited thickness (e.g. B4/B5), sufficient thickness in Jordan Valley alluvium<br>> storage volume is no major constraint                                | Thick (> 100 m).<br>> high storage potential                                                                                                            | Thick (> 400 m).<br>> high storage potential                                                                                             |
| <b>Unconformity of Hydraulic Properties</b> | Homogeneous<br>> minimal mixing<br>> retention times do not vary significantly                                                                                     | Extremely heterogeneous due to karstification and fractures<br>> fast discharge, insufficient retention time if aquifer is drained by springs close by  | Moderate Heterogeneity,<br>> moderate mixing<br>> retention times do not vary significantly                                              |
| <b>Salinity of groundwater</b>              | Fresh with strongly increasing salinity toward the valley centre<br>> limited recovery efficiency at parts with high salinity<br>. less beneficial uses to protect | Dominantly fresh<br>> unlimited recovery efficiency                                                                                                     | Mostly saline<br>> limited recovery efficiency<br>> less beneficial uses to protect, so low treatment requirements of infiltration water |
| <b>Lateral hydraulic gradient</b>           | Gentle<br>> Recharged water contained closer to the point of recharge                                                                                              | Steep on the descent to the Jordan Valley, slightly more gentle in the uplands<br>> Quick dispersion of injected water                                  | Gentle<br>> Recharged water contained closer to the point of recharge                                                                    |
| <b>Consolidation</b>                        | Unconsolidated in the upper sections of the alluvial fans/ elsewhere consolidated.                                                                                 | Consolidated<br>> easier well construction<br>> less clogging problems                                                                                  | Consolidated<br>> easier well construction<br>> less clogging problems                                                                   |
| <b>Aquifer mineralogy</b>                   | Unconsolidated conglomerates, carbonates<br>> solution/dissolution should be considered                                                                            | Carbonates, Sandstone<br>> solution/dissolution should be considered                                                                                    | Sandstone, Carbonates, Shales<br>> solution/dissolution should be considered                                                             |



West of the Jordan, recharge is most sensible in the confined sections of the aquifer system. Several of the topmost aquifers are already contaminated with sewage water; the artificial recharge would enter a source which can not be used due to qualitative reasons. Regarding the Ram-Zarqa-Kurnub aquifer system the high drilling costs due to the large depth below ground might block off many attempts for recharging.

Planning for MAR must take the local circumstances into account, such as the strong seasonality of rainfall, and high intensity peak rain events which require large reservoirs to provide temporal storage for runoff during flash floods. High technology options like ASR require significant experience in set-up and maintenance, especially with regard to the prevention of clogging and may not be suitable at this point. Furthermore, surface runoff from urban areas may be strongly polluted as no effective source control measures are currently in place.

## CONCLUSIONS & FUTURE WORK

This review showed that there is considerable potential in the Lower Jordan River Basin for managed aquifer recharge. However, MAR is not yet identified as a major goal in the national water master planning. Currently the main focus is on water demand management, water supply management and institutional reforms (Taha and Magiera 2006). Considering the currently limited depth of this review of MAR activities in the Lower Jordan River basin, it is essential to increase the coverage by incorporating more of the locally available grey literature. In order to unlock the potential for MAR in the region, a series of background and feasibility studies on the following topics are recommended:

- Increase of groundwater recharge from reservoir structures by removal/disturbance of low permeability reservoir sediments.
- MAR from open reservoirs into alluvial fan aquifers
- Use of urban surface runoff for groundwater recharge
- Holistic urban water balances and construction of sustainable water systems in the fast growing urban areas
- Quantification of the net benefit from MAR in terms of evaporation prevention
- Cost-benefit analysis of MAR within the IWRM framework under the consideration that all the recharged water volumes (irrespective of the recovery possibilities on spot) are effectively adding up to the total water availability in the Jordan River basin.

The mentioned studies are considered as a prerequisite for a representation of the potential of MAR schemes in water balancing tools like WEAP, Hydroplanner or Water Strategy Man which are used as a decision support for Integrated Water Resources Management. Despite the current implementation barriers, a boost of

MAR on regional scale is possible and recommended in order to mitigate water scarcity in the Lower Jordan River Basin. The successful case studies described in this paper may serve to encourage the development of MAR in the region.

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## PROBLEMS AND SOLUTIONS WHEN RECHARGING BRACKISH AQUIFERS WITH MEMBRANE CONCENTRATE FROM A LESS BRACKISH GROUNDWATER SOURCE

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### ABSTRACT

Rapid technological improvements have made the production of fresh drinking water from brackish groundwater by reverse osmosis (BWRO) more economically viable. Although BWRO is increasingly applied in the world, there are various problems to be solved in the Netherlands (and elsewhere) before a production plant can be built. The main focus here is on the problem of membrane concentrate disposal by deep well injection. The disposal of this waste water is problematic, especially on inland sites, because its anomalous composition frequently exceeds natural backgrounds or even maximum contaminant levels, while mineral supersaturation may lead to well clogging.

Our preliminary remedy consists of the following approach: (1) select brackish source groundwater with an appropriate chemical composition (yielding a concentrate with low mineral saturation index); (2) select a target aquifer for deep well injection where the native groundwater composition is more saline and the concentration of most critical substances is higher than in the membrane concentrate; (3) keep the permeate to concentrate ratio low (ca.1.0); (4) keep the system anoxic (thus avoiding any oxidation reactions that may create flocs) and pressurized (thus preventing gas bubbles to form); and (5) maintain the salinity of the feed water at a rather constant level. The latter can be achieved by pumping several wells with deviating salinity (both higher and lower than the desired level) at a rate autocontrolled by on-line EC measurements in each well and their mixture.

The testing of this remedy, questions regarding the behavior of a.o. B, As and SiO<sub>2</sub>, and uncertainties about the predictive value of mineral supersaturation levels, called for a pilot plant study. A pilot is planned for the formerly largest well field in the Netherlands, Noord-Bergum, where 20 Mm<sup>3</sup>/a of fresh groundwater was abstracted for drinking water supply. Due to severe salinization problems the production rate decreased to 7 Mm<sup>3</sup>/a today. Pumping brackish groundwater from that well field is expected to partly restore the production level and to safeguard the productive fresh wells against further salinization.

### KEYWORDS

Brackish groundwater, reverse osmosis, salinization, fresh keeper, membrane concentrate, disposal, well clogging, water rock interaction

## 1. INTRODUCTION

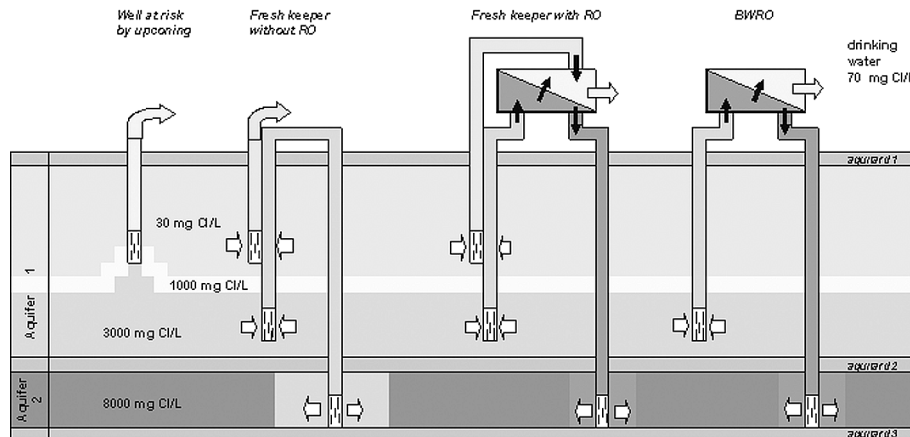
Reverse osmosis of brackish groundwater (BWRO) is increasingly applied, especially in (semi)arid countries, to prepare drinking or industrial water from brackish groundwater (chloride 300-10.000 mg/L) on a cost-effective basis. Production plants are already in operation for several decades, amongst others in the Middle East (Ahmed et al., 2001; Zubari, 2003; Afonso et al., 2004; Jaber & Ahmed, 2004) and USA (Mickley, 2006). Technological improvements, declining prices for RO systems, economical stress factors and salinization of aquifers now also force countries in temperate climates with less severe water scarcity problems to seriously consider BWRO as a 'source-treatment' option to prepare drinking water. As a matter of fact, in the Netherlands BWRO is currently under serious consideration (Kooiman et al., 2004; Nederlof & Hoogendoorn 2005; Oosterhof & Nederlof, 2006). The following advantages add to its pro's:

- Brackish groundwater is an excellent feed water for RO installations by virtue of its very low content of suspended fines (leading to a very low membrane fouling index) and lack of both organic pollutants and pathogenic micro-organisms;
- Salinized or salinizing well fields, with a separate BWRO unit for brackish wells, may continue to produce the allowed quatum or even produce more than before. This is cheaper and may take less time to realize than moving away to another or new water catchment area;
- A reduced impact on the upper aquifer system as compared to fresh groundwater pumping; and
- Taxes are imposed on fresh groundwater exploitation (Netherlands ca. 0.28 €/m<sup>3</sup>), but not on brackish and saline groundwater.

Another option is to apply the fresh-keeper (Grakist et al., 2002; Kooiman et al., 2004) without or with RO (Fig.1). In both cases a well suffering from upconing (or unilateral entry) of brackish groundwater may profit from a deeper (or upgradient) well which scavenges the salt. In the first case the brackish or salt groundwater needs to be disposed of. In case of a BWRO system or a fresh-keeper with RO the membrane concentrate will constitute the waste water. Its disposal is more problematic than that of brackish to saline groundwater, because it may contain antiscalants while several constituents of the feed water can be concentrated above natural background or even maximum contaminant levels.

In temperate climates with high population densities and severe environmental laws, many of the disposal options mentioned by Mickley (2006), like evaporation ponds and discharge to surface water, are deemed to fail or to be disapproved by regulators. In the Netherlands we believe that deep well injection into a more saline, confined aquifer may offer an excellent solution to the waste water problem (Fig.1). Yet there are various questions to be addressed, such as: (a) is the quality of the

infiltration water acceptable when compared to the native groundwater in the target aquifer? (b) which reactions occur in the mixing zone between the native groundwater and injected concentrate, and what are their consequences? (c) which reactions continue to operate between the injected bubble and the aquifer matrix, and what are their consequences? (d) how can we prevent well clogging of a supersaturated solution?, and (e) will clay minerals mobilize with a risk on mechanical aquifer plugging?



**FIGURE 1. Schematic of a fresh well salinizing by upconing, the fresh-keeper without reverse osmosis (RO), ditto with RO, and brackish water with RO (BWRO). Chlorinity of RO-concentrates ca. 6000 mg/L when quantity of permeate equals quantity of concentrate**

In this contribution we offer some preliminary answers and ways to solve or circumvent problems. With a future pilot plant near Noord-Bergum, in the Northern Netherlands, problems and solutions will be tested. On that site an appropriate quality of brackish groundwater and excellent aquifers are present around an important well field that is suffering from salinization.

Some details of the set-up will be presented.

## 2. SELECTION OF SUITABLE SOURCE WATER

Constraints on the chemical composition of brackish to saline groundwaters for a sustainable use as drinking or industrial water supply are the following:

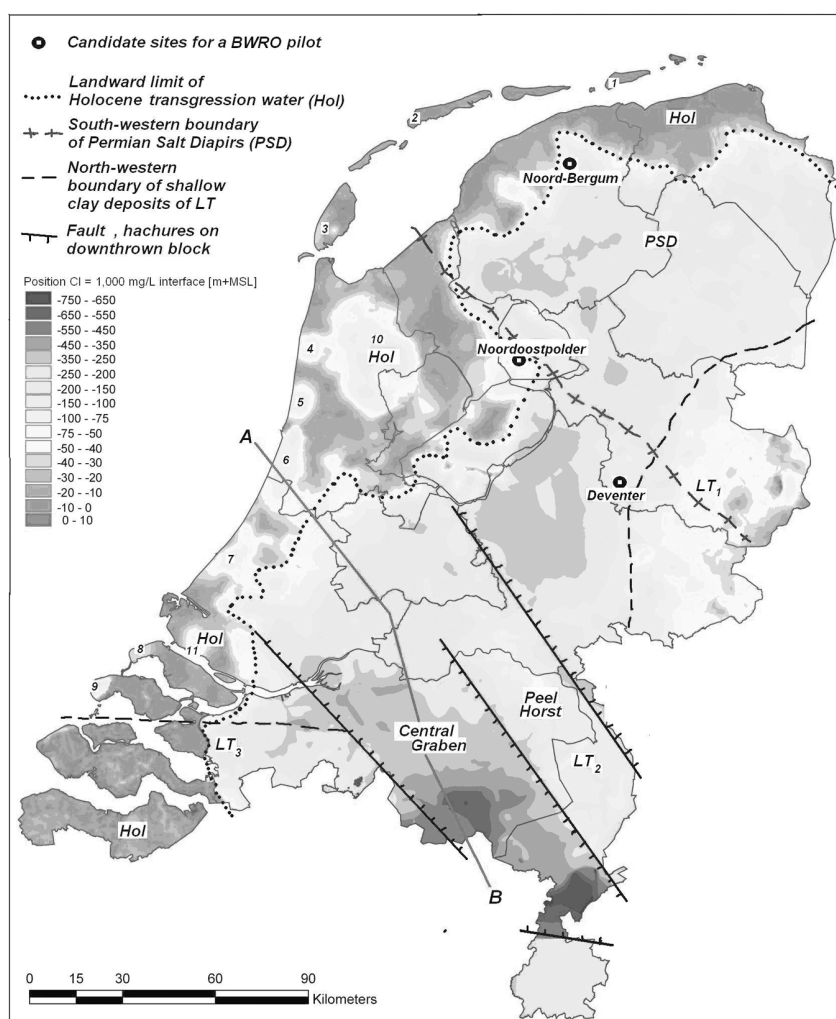
- (d) the salinity must be favorable for optimum membrane purification at reduced costs: preferably Cl < 10,000 mg/L or TDS < 20,000 mg/L;

- (e) the concentration of ions less soluble than Na, K, Mg and Cl, should be favorable concerning risks on scaling of membranes or injection wells with for instance silicate ( $\text{SiO}_2 \cdot n\text{H}_2\text{O}$ ,  $\text{MgSi}_2(\text{OH})_6$ ), sulfate ( $\text{BaSO}_4$ ,  $\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$ ), carbonate ( $\text{CaCO}_3$ ,  $\text{CaMg}(\text{CO}_3)_2$ ,  $\text{FeCO}_3$ ), or phosphate ( $\text{Ca}_5(\text{PO}_4)_3\text{OH}$ ) minerals. The use of antiscalants should be avoided in our philosophy, in order to raise the chance on getting the license for deep well injection of the waste water;
- (f) the concentration of ions in the permeate should not exceed the maximum permissible concentrations for drinking water. A simple post-treatment like aeration is adequate to get rid of gases like  $\text{CH}_4$ ,  $\text{H}_2\text{S}$  and  $\text{CO}_2$ . But uncharged dissolved species with a small size, like B ( $\text{H}_3\text{BO}_3$ ) may pose serious problems. Boron is very hard to remove, it can attain rather high concentrations in brackish groundwater, may pass membranes and then exceed the drinking water standard of 500  $\mu\text{g/L}$ ;
- (g) groundwater is preferred that does not exceed natural background levels after membrane concentration, because its deep well injection is then more likely to be permitted by regulators; and.
- (h) the abstraction should not result in salinization of the aquifer, well clogging or corrosion of well and transport mains.

Stuyfzand & Stuurman (in press) made an inventory of brackish to hypersaline groundwaters in the Netherlands down to 600 m below mean sea level, using all readily accessible hydrochemical data with  $\text{Cl} > 300 \text{ mg/L}$  (13,500 samples in total). They discerned 5 types of non-anthropogenic, brackish to (hyper) saline groundwater in the upper 600 m of the Dutch underground: (sub)recent, intruding North Sea water along the coast; Holocene transgression water in a high energy environment like open marine, tidal gully or estuary (HOH); Holocene transgression water in a low energy environment like a lagoon (HOL); relict groundwater of Late Tertiary to Early Pleistocene age (LTEP); and Permian or Devonian halite leachate.

For desalination purposes they recommended the brackish exponents of relict HOH groundwater close to former river mouths (thus reducing clay and peat contact and salinity), and relict LTEP groundwater. These waters require less energy (less saline) and reduce risks on membrane and injection well scaling (lower concentrations of  $\text{HCO}_3$ ,  $\text{SO}_4$ ,  $\text{SiO}_2$ ,  $\text{NH}_4$ ,  $\text{PO}_4$ , etc.). The stocks of both relict groundwaters are huge, and their quality is completely natural (lacking organic micropollutants). The areal extent of Holocene transgression water is indicated in Fig.2. The top of LTEP waters is observed at relatively shallow depth (<150 m) in the areas indicated in Fig.2, elsewhere they normally reside at depths in between 200 and 600 m.

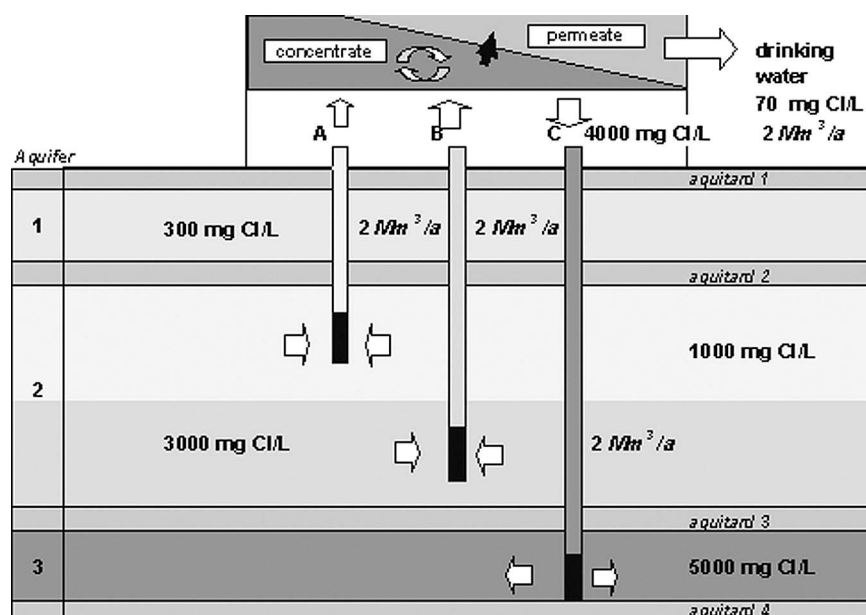




**FIGURE 2. Position of the 1,000 mg Cl/L interface (in m + Mean Sea Level) with indication of 3 candidate sites for a BWRO pilot: Deventer, Noordoostpolder and Noord-Bergum (modified after Stuyfzand & Stuurman, in press). Also shown: the approximate landward limit of the brackish to salt Holocene transgression waters (Hol), relatively shallow occurrences of marine deposits of Late Tertiary to Early Pleistocene age (LT), and relatively shallow Permian rock Salt Diapirs (PSD). 1-11 = coastal fresh water lenses down to 50-125 m-MSL.**

The feasibility of brackish groundwater abstraction depends, of course, also on the hydrogeological conditions. The hydraulic conductivity of the pumped aquifer should be sufficiently high and the brackish groundwater stock must be large enough

to supply feed water for a long time. It should be realized that inland brackish groundwaters are mostly palaeowaters lacking actual recharge. A major problem is that brackish groundwater normally is stratified with salinity increasing with depth. Pumping may induce quality variations in the feed water that adversely affect the RO process. A solution to this problem is offered by pumping several wells with deviating salinity (both higher and lower than the desired level) at a rate autocontrolled by on-line EC measurements in each well and their mixture (Fig.3).



**FIGURE 3. Schematic of a well field for BWRO in an aquifer with strong chlorinity stratification, and the solution to yield a stable feed water quality by pumping several wells with deviating salinity (both higher and lower than the desired level) at a rate autocontrolled by on-line EC measurements in each well and their mixture.**

### 3. SELECTION OF A SUITABLE DISPOSAL AQUIFER

A sustainable, deep well injection can be realized when the hydrogeological and hydrogeochemical properties of the target aquifer meet the following criteria:

- (a) hydraulic conductivity and storativity should be high enough to store a large quantity of waste water for a long time.
- (b) the aquifer must be (semi)confined so that the water injected remains contained in a limited space well below land surface;

- (c) native groundwater should preferably have a higher salinity and higher concentration of critical compounds (like nutrients, heavy metals) for license reasons mentioned earlier. This conforms to the Australian principle (Dillon et al., 1994), which states that the infiltrate should preferably have a better quality than the original groundwater on site; and
- (d) the tendency of clay minerals in the aquifer to mobilize by deflocculation should be low, because it could provoke mechanical clogging of the aquifer when the particles strand in the pore necks. This tendency is low when the sodium adsorption ratio (SAR) of the injectate and of the native groundwater is low, their salinity high, the clay mineral content very low and the dominant type of clay minerals favorable (kaolinite > illite > smectite). Further details are given by Scheuerman & Bergersen (1990).

The presence of reactive aquifer constituents like carbonate minerals, soil organic material and pyrite seems less inhibitive, because the membrane concentrate is already (super)saturated by itself and anoxic.

There is no map yet of candidate disposal aquifers in the Netherlands, especially not regarding the upper 500 m. At greater depth environmental constraints are less severe (regulated by the Mine Act), and more information exists through explorations for gas and oil as well as radioactive waste disposal. Disposal via abandoned gas or oil wells can be an option as discussed by Nicot & Chowdhury 2005.

#### 4. THE REVERSE OSMOSIS PROCESS

Unpolluted brackish groundwater can be easily treated to drinking water quality by RO, if followed by aeration to get rid of gases like  $\text{CH}_4$ ,  $\text{H}_2\text{S}$  and  $\text{CO}_2$  and by rapid sand filtration to eliminate the remaining Fe, Mn and  $\text{NH}_4$ . The permeate does not need any separate conditioning, if mixed with sufficient fresh or slightly brackish groundwater before their combined conventional treatment (aeration and rapid sand filtration). Otherwise a further post-treatment of the permeate is needed to stabilize pH at a higher level while meeting hardness requirements. When high boron concentrations in the feed water cannot be avoided, then specific high boron removal RO membrane elements should be applied.

Antiscalants like polyphosphonates are frequently applied to prevent clogging of the membranes and also of the deep well injection facilities. HCl can be applied to reduce super saturation of for instance  $\text{CaCO}_3$ . We choose for not applying them, however, because their use lowers the chance on getting a deep well disposal permit in the upper 500 m. In that case the permeate to concentrate ratio should be kept low (ca.1.0) in order to keep mineral (super)saturation levels low.

All unpolluted, brackish groundwaters in the Netherlands are composed of (deep) anoxic palaeowaters containing significant amounts of Fe, Mn and  $\text{NH}_4$  (Stuyfzand & Stuurman, in press). By keeping the feed water and the membrane concentrate anoxic, the oxidation and precipitation of Fe(III) and Mn(IV) flocs can be prevented (Nederlof & Hoogendoorn, 2005), thus facilitating the direct injection of the membrane concentrate without pretreatment.

## 5. DEEP WELL INJECTION

When the above mentioned conditions are satisfied deep well injection still may encounter difficulties due to well or aquifer clogging. The main causes probably are chemical precipitation, microbiological fouling and the formation of gas bubbles. The latter can be prevented by keeping the membrane concentrate pressurized. Microbiological fouling is frequently prevented by chlorination, but that would again reduce the chance on getting a permit in the Netherlands. If problematic then periodical mechanical well rehabilitation methods are preferred.

The mixing of the injectate with native groundwater normally does not pose serious clogging phenomena, because the water quantity involved is too small, especially in single porosity aquifers. By refraining from antiscalants and acids, mineral phases barely or not dissolve, as the solution is already supersaturated by itself. Redox reactions will be minimal as well, because the water injected and the host aquifer are both (deep) anoxic.

Water-rock interactions will therefore be limited to cation exchange and the deposition of some minerals like calcite and apatite, and possibly also dolomite, siderite, vivianite, barite and amorphous silica. As pointed out before, these precipitations can be avoided (partly) by choosing the proper source water and by keeping the permeate to concentrate ratio low.

## 6. A CRITICAL LOOK AT MINERAL SATURATION INDICES

The saturation index of water with a particular mineral M ( $\text{SI}_M$ ) is generally defined as :

$$\text{SI}_M = \log (\text{IAP} / K_S) \quad (\text{EQ 1})$$

where : IAP = Ion Activity Product of the mineral-water reaction in the sample [on a mol/kg water basis]; and  $K_S$  = the corresponding solubility product in pure water, adjusted to the temperature and pressure of the sample.

This means that water with a  $SI_M = 0$ , is in equilibrium with the mineral considered, with a  $SI_M < 0$  it is undersaturated and will tend to dissolve the mineral when it is met, and with a  $SI_M > 0$  it is oversaturated and may deposit the mineral.

An excellent computer program, that iteratively calculates for a multicomponent, natural system the speciation of dissolved inorganic substances in water and many mineral equilibria, is PHREEQC-2 (Parkhurst & Appelo, 1999). Of course, the resulting SIs must be interpreted with great care, for at least four reasons : (a) analytical errors (especially pH and  $E_H$ ) or sampling errors (introduction of  $O_2$ , escape of  $CO_2$ ,  $H_2S$  and  $CH_4$ ) may bias the calculation; (b) organic complexes are generally ignored; (c) minerals in nature are never pure whereas they are assumed so in the program; and (d) kinetics may be so slow that equilibrium will never be attained like in case of the chronic supersaturation of most groundwaters with respect to quartz.

Stuyfzand & Stuurman (in press) discuss mineral saturation indices for the 5 brackish to (hyper)saline groundwater types in the Netherlands mentioned earlier. Equilibria with sulphur containing minerals like pyrite could not be calculated by lack of (good)  $H_2S$  data. All samples appeared undersaturated with respect to halite, fluorite and opal. Some samples approached halite equilibrium ( $SI = <-1$ ), which is consistent with local halite leaching. Various samples approached opal (amorphous silica) equilibrium ( $SI = -0.1$ ), which is explained by stagnant conditions in clayey marine deposits with high diatom contents. All groundwater samples were supersaturated ( $SI = 0.5-10$ ) with respect to most silicate minerals (clay minerals, quartz, albite and K-feldspar). This probably indicates that these minerals play a minor role in dictating hydrochemistry of brackish to (hyper)saline waters at low temperatures, because of kinetic problems.

Most groundwaters were close to equilibrium with calcite due to abundant calcitic and aragonitic shell fragments in most marine formations. Minor deviations ( $-0.3$  to  $+0.3$ ) occur due to errors in pH-measurement, and the cases of clear supersaturation ( $0.5-0.9$ ) can be related to high concentrations of fulvic acids which complex Ca, and of  $PO_4$  and Mg which inhibit crystallization. The SIs for dolomite ( $CaMg(CO_3)_2$ ) often attained significant supersaturation ( $1-2.5$ ) without evidence of dolomite being actually present in the system. This is explained by an extremely sluggish crystallization, and dominant Mg sources from ocean water and cation exchange processes.

Equilibrium and supersaturation with respect to siderite ( $FeCO_3$ ) and rhodochrosite ( $MnCO_3$ ) was also observed but may likewise suffer from inaccuracies in pH measurement, unaccounted complexation of reduced  $Fe^{2+}$  and  $Mn^{2+}$  by fulvic acids, and from kinetic hindrances. Nevertheless the presence of manganous siderite has been noted in various aquifers and its dissolution or precipitation may be important.

All waters were aggressive towards gypsum ( $\text{CaSO}_4 \cdot 2\text{H}_2\text{O}$ ), which is explained by its proven absence in all cases except where halite dissolved.

Barite ( $\text{BaSO}_4$ ) has been demonstrated in geochemical surveys and shows supersaturation (0.3-1.1) in most cases. This may indicate its relevance and the role of unaccounted complexation by fulvic acids and kinetic hindrances.

The situation with respect to phosphate minerals like vivianite ( $\text{Fe}_3(\text{PO}_4)_2 \cdot 8\text{H}_2\text{O}$ ) and hydroxi-apatite ( $\text{Ca}_5(\text{PO}_4)_3\text{OH}$ ) is even more speculative because of stronger impacts of analytical inaccuracies (pH, inorganic versus organic  $\text{PO}_4$ ) and variations in mineral composition and degree of crystallization. Apatites seem to be rather abundant in marine formations, however, and may be relevant especially in high  $\text{PO}_4$  samples without strong redox reactions with organic matter.

We conclude that mineral equilibria and especially mineral supersaturation should be interpreted with great care. Mineral supersaturation of water with respect to for instance calcite, dolomite, silicate, phosphate and iron minerals does by no means indicate that equilibrium will be attained by mineral deposition. This is of paramount importance in studying the risks on chemical clogging of membranes, injection wells and host aquifers. Therefore pilots are needed to see at which saturation levels precipitates start to form and become a nuisance, as a function of contact time and contact surface conditions.

## 7. BWRO PILOTS IN THE NETHERLANDS

To date 3 candidate BWRO pilots have been defined in the Netherlands, exclusively by water supply company Vitens, respectively near Deventer, in the Noordoostpolder and close to Noord-Bergum (Fig.3).

### Deventer

The first candidate, in the valley of the Yssel River near Deventer, has been described by Nederlof & Hoogendoorn (2005). It consists of 3 nearby salinizing well fields that would benefit from the fresh-keeper principle with RO. Disposal problems of the membrane concentrate and alternative solutions of regional water scarcity problems destroyed the demonstration project. Deventer was considered as a pilot; however the pilot was never executed because of a lack of urgency

### Noordoostpolder

The polder area is part of the former Zuiderzee, a brackish bay of the North Sea into which the Yssel River (a tributary of the Rhine River) discharged. In 1932 this bay was closed off from the North Sea by a dike and automatically filled up with fresh

water from the Yssel River. In 1942 the Noordoostpolder was reclaimed from the new Lake Yssel. Permanent drainage of this new land 2-3 m below sea level results in a slow freshening especially of the shallow brackish aquifers. The increasing local population and ecological incentives to reduce the production of fresh well fields on the mainland, triggered the idea to start a BWRO pilot for large scale implementation later on.

**TABLE 1. Water quality survey of shallow and deeper brackish source water from the Noordoostpolder, and their calculated concentrate and permeate in a hypothetical BWRO-plant with 50 and 80% efficiency (=100\*permeate volume/feed water volume). Concentrate and permeate concentrations were calculated using Troi 1.5.0 (Trisep Corp.). Mineral saturation indices (SI's) were calculated with PHREEQC-2. In black: potentially problematic concentrations or mineral saturation.**

| Parameter                          | unit | Shallow brackish groundwater, Cl = 1040 mg/L |             |       |          |      | Deeper brackish groundwater, Cl = 3290 mg/L |             |       |          |      |
|------------------------------------|------|----------------------------------------------|-------------|-------|----------|------|---------------------------------------------|-------------|-------|----------|------|
|                                    |      | Raw                                          | Concentrate |       | Permeate |      | Raw                                         | Concentrate |       | Permeate |      |
|                                    |      |                                              | 50%         | 80%   | 50%      | 80%  |                                             | 50%         | 80%   | 50%      | 80%  |
| pH                                 |      | 6.6                                          | 6.84        | 7.13  | 6.04     | 5.2  | 7.35                                        | 7.52        | 7.36  | 5.67     | 5.52 |
| Cl                                 | mg/L | 1040                                         | 2038        | 4949  | 42       | 62   | 3290                                        | 6657        | 16359 | 69       | 114  |
| HCO <sub>3</sub>                   | mg/L | 577                                          | 1142        | 2803  | 11       | 17   | 854                                         | 1680        | 3797  | 11       | 17   |
| SO <sub>4</sub>                    | mg/L | 10                                           | 19.9        | 49.6  | 0.1      | 0.1  | 3.1                                         | 6.2         | 15    | 0        | 0.7  |
| PO <sub>4</sub>                    | mg/L | 1.6                                          | 3.19        | 7.93  | 0.01     | 0.02 | 2.2                                         | 4.39        | 11    | 0.01     | 0.48 |
| Na                                 | mg/L | 480                                          | 965         | 2335  | 23       | 34   | 1820                                        | 3601        | 8840  | 40       | 65   |
| K                                  | mg/L | 15                                           | 29.3        | 70.7  | 0.7      | 1.1  | 65.8                                        | 130.1       | 319.3 | 1.5      | 2.4  |
| Ca                                 | mg/L | 260                                          | 515         | 1271  | 5        | 7    | 272                                         | 541         | 1338  | 3        | 6    |
| Mg                                 | mg/L | 37                                           | 73          | 181   | 1        | 1    | 168                                         | 334         | 826   | 2        | 4    |
| NH <sub>4</sub>                    | mg/L | 20                                           | 38.9        | 93.8  | 1.1      | 1.6  | 13.8                                        | 27.3        | 66.8  | 0.3      | 0.5  |
| Fe                                 | mg/L | 55                                           | 109.6       | 272.7 | 0.4      | 0.6  | 10                                          | 19.9        | 48    | 0.1      | 2.2  |
| Mn                                 | mg/L | 1.8                                          | 3.59        | 8.93  | 0.01     | 0.02 | 1                                           | 1.99        | 4.5   | 0.01     | 0.22 |
| SiO <sub>2</sub>                   | mg/L | 16                                           | 31.6        | 77.6  | 0.4      | 0.6  | 26.8                                        | 53.3        | 132.2 | 0.3      | 0.5  |
| B                                  | ug/L | 239                                          | 239         | 239   | 239      | 239  | 757                                         | 757         | 757   | 757      | 757  |
| Ba                                 | ug/L | 600                                          | 1190        | 2930  | 10       | 20   | 300                                         | 600         | 1480  | 10       | 10   |
| F                                  | ug/L | 100                                          | 196         | 476   | 4        | 6    | 50                                          | 101         | 249   | 1        | 2    |
| Sr                                 | ug/L | 1510                                         | 2990        | 7360  | 30       | 50   | 2290                                        | 4590        | 11320 | 80       | 80   |
| <b>Mineral Saturation index SI</b> |      |                                              |             |       |          |      |                                             |             |       |          |      |
| SI - Calcite                       |      | -0.12                                        | 0.59        | 1.48  | < 0.0    |      | 0.68                                        | 1.31        | 1.67  | < 0.0    |      |
| SI - Aragonite                     |      | -0.27                                        | 0.44        | 1.32  |          |      | 0.63                                        | 1.15        | 1.52  |          |      |
| SI - Dolomite                      |      | -0.94                                        | 0.48        | 2.26  |          |      | 1.3                                         | 2.56        | 3.33  |          |      |
| SI - Siderite                      |      | 1.31                                         | 1.95        | 2.72  |          |      | 1.28                                        | 1.8         | 2.38  |          |      |
| SI - Rhodochrosite                 |      | 0.11                                         | 0.74        | 1.5   |          |      | 0.54                                        | 1.06        | 1.59  |          |      |
| SI - Strontianite                  |      | -1.82                                        | -1.11       | -0.23 |          |      | -0.86                                       | -0.23       | 0.13  |          |      |
| SI - Barite                        |      | 0.09                                         | 0.47        | 0.94  |          |      | -0.98                                       | -0.63       | 0.03  |          |      |
| SI - Hydroxypatite                 |      | -1.61                                        | 1.31        | 4.75  |          |      | 2.03                                        | 4.26        | 5.94  |          |      |
| SI - SiO <sub>2</sub> am           |      | -0.74                                        | -0.43       | -0.02 |          |      | -0.5                                        | -0.19       | 0.26  |          |      |
| SI - Al(OH) <sub>3</sub> am        |      | -1.14                                        | -0.84       | -0.61 |          |      | -1.46                                       | -1.34       | 0.46  |          |      |
| SI - Vivianite                     |      | 2.12                                         | 3.64        | 5.31  |          |      | 1.59                                        | 2.58        | 7.28  |          |      |
| SI - Alunite                       |      | -0.75                                        | 0.04        | 0.64  |          |      | -4.66                                       | -4.21       | 2.84  |          |      |
| SI - Gypsum                        |      | -2.37                                        | -1.99       | -1.51 |          |      | -3.11                                       | -2.75       | -2.05 |          |      |

Preliminary calculations on the quality of the membrane concentrate and permeate are shown, together with the quality of the local source groundwater, in Table 1. We conclude that increasing the efficiency (= 100 \* permeate volume / feed volume) from 50 to 80% will significantly raise the risk on clogging because of extreme levels of supersaturation with respect to many minerals. The less saline, brackish groundwater from shallow depth is to be preferred from a chemical point of view. It

yields a somewhat better concentrate quality during RO, and boron concentrations of the permeate would conform to drinking water standards. In Table 1 it is assumed, however, that the uncharged boron ( $\text{H}_3\text{BO}_3$ ) will pass the membranes completely whereas the uncharged silica ( $\text{H}_4\text{SiO}_4$ ) will be largely retained. In practice their behavior might be different. Similar questions can be raised regarding the behavior of the uncharged arsenite ( $\text{H}_3\text{AsO}_3$ ). The very high supersaturation levels for siderite, hydroxy-apatite and vivianite, also in the 50% concentrate, are worrying. These minerals may form and plug the membranes or well. The realization of a pilot in the Noordoostpolder depends on future water plans for the area, which probably lag behind the Noord-Bergum pilot described below.

### Noord-Bergum

Well field Noord-Bergum has a total permit for  $20 \text{ Mm}^3/\text{a}$ , but due to salinization the present-day abstraction decreased from  $20 \text{ Mm}^3/\text{a}$  in the 1990s to  $7 \text{ Mm}^3/\text{a}$  today. The cause of salinization is upconing in combination with lateral flow of brackish groundwater originating from the Holocene transgression close to the northwestern border of the well field (Fig.3).

The idea is to add a BWRO unit in the salinized parts of the well field, yielding an additional  $3 \text{ Mm}^3/\text{a}$  of drinking water. It is expected that this unit will scavenge the advancing brackish groundwater and thereby safeguard the still operating fresh wells. To date no BWRO system consisting of a brackish groundwater abstraction, treatment with RO and concentrate disposal by deep well injection has ever been operational in The Netherlands. As the Deventer and Noordoostpolder pilots finished on the design table, Vitens will start a demonstration project in cooperation with Kiwa Water Research to show how it will work. If successful, the demonstration project may be extended to a full-scale plant, which depends on other alternatives for the brackish groundwater like surface water, but the brackish groundwater is a serious alternative. Technical issues concerning the subsoil will be the clogging potential of the injection well and the long term quality changes in the target aquifer.

The demonstration project will consist of a brackish abstraction well (130-150 below ground surface), an injection well (180-220 meter below ground surface), and 2 observation wells at resp. for instance 12 and 30 m distance from the injection well.

Preliminary calculations with PHREEQC-2 indicate that an efficiency of 60% can be reached with the local brackish groundwater from 130-150 m depth, while keeping mineral saturation levels low ( $<0.8$ ). In addition, the injection of this water is calculated to result in minor cation exchange and mixing reactions that could peak mineral saturation levels.



## CONCLUDING REMARKS

BWRO with concentrate disposal via deep well injection has been applied to supply drinking water in many countries in dry climates for already about 2 decades. Yet the literature offers hardly any information on the kinetics of mineral precipitation on the membranes, in the wells and the host aquifer. Frequently antiscalants are used to deal with the problem, but they are completely unacceptable when concentrate is to be injected.

In the temperate humid Netherlands, where brackish groundwater is abundant, water scarcity problems are mild but environmental issues of big concern, BWRO is becoming an interesting 'source-treatment' option. Getting rid of the RO waste water in an environmentally acceptable way is the Achilles heel of BWRO in the Netherlands. We therefore aim at a holistic approach to reach a high quality waste water that perfectly fits into the natural chemical environment of the target aquifer for disposal. The approach consists of a severe selection of the brackish source groundwater and target aquifer, a low efficiency around 50% (permeate to concentrate ratio ca.1.0), keeping the system anoxic and pressurized, and avoiding the use of antiscalants.

A pilot study is needed to see if this approach really works. After 2 trials to realize a demonstration project, a third much more promising opportunity appeared: well field Noord-Bergum, one of the largest in the Netherlands and one with severe salinization problems. Because of delay in the procedures to get a permit for the pilot, Water Supply Company Vitens expects to start with a demonstration project in the beginning of 2008.

## ACKNOWLEDGEMENTS

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## MIXING AND CLOGGING CONSTRAINTS ON DOMESTIC-SCALE ASR AT KINGSWOOD, SOUTH AUSTRALIA

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### ABSTRACT

Harvesting of rainwater from roof surfaces can offer useful quantities of good quality water, particularly in semi-arid areas where alternative sources are scarce. A three year demonstration project to investigate the operational performance of domestic-scale aquifer storage and recovery (ASR) with rainwater was completed in September 2006. A shallow alluvial aquifer in the Adelaide metropolitan area was targeted after an earlier regional assessment indicated this area to have potential for ASR. Flow rate, volume, piezometric head, electrical conductivity and temperature were monitored continuously at the injection well and the samples of the injectant and recovered water analysed periodically. Over three years of operation 1828 mm of rainfall resulted in a net volume of 452 m<sup>3</sup> injected. Although the salinity of the groundwater was reduced during injection from an ambient groundwater concentration of 2500 mg/L, during recovery events it does not remain sufficiently fresh to be useful for irrigation supplies. High rates of mixing are attributed to regional groundwater flow, diffusive exchange between high and low permeability zones and density-effects. The relative significance of these mechanisms is the subject of further study. A gradual decline in the specific capacity of the ASR well during the trial was attributed to well clogging by particulates and biofilm growth.

### KEYWORDS

ASR; recovery efficiency; salinity; clogging; water quality

### INTRODUCTION

Successful ASR allows water to be stored below ground for recovery at a later date. Following a research project on stormwater ASR at municipal scale in a Tertiary limestone aquifer (Dillon *et al*, 1997) that led to guidelines for this practice (Dillon and Pavelic, 1996), ASR has been taken up widely in Adelaide where by 2004 there were 25 sites recharging 3 Mm<sup>3</sup>/yr of stormwater. This small but growing practice, is contributing to the security of Adelaide's water supply (Dillon *et al*/2004) and has the potential to grow to 50 Mm<sup>3</sup>/yr (Hodgkin, 2004). An evaluation of a shallow Quaternary system of alluvial aquifers of the Adelaide Plains (Pavelic *et al*, 1992) revealed that much of the metropolitan area has potential for storage of water on a smaller scale in brackish aquifers. For example, by recharging domestic roof runoff for use in garden watering in summer to further reduce demand on a stressed water supply system. This is one of the activities being explored in support of improved

stormwater management as part of plans by the Patawalonga Catchment Water Management Board (2002) and conforms to the South Australian Government (2004) strategy 'Water Proofing Adelaide'. However, the success of the project would largely rely on whether recovered water quality would meet the water quality requirements for its intended uses, namely irrigation. This is measured as the "recovery efficiency" (RE) defined as the proportion of injected water that can be pumped at a quality that is suitable for its intended use (Pyne 1995). This definition is site-specific, due to amongst other things, to the influence of end-member concentrations and maximum permissible concentration associated with the particular use of the water (Pavelic *et al*, 2006), which in this case is the irrigation threshold of TDS 1500 mg/L (approx. electrical conductivity (EC) of 2500  $\mu\text{S}/\text{cm}$ ). Prevention or minimisation of clogging of the injection well is also an important operational issue for ASR in unconsolidated aquifers. Hence the primary objective of this study was to assess the operational performance of domestic-scale ASR with roof-runoff, determine causes for departures from predicted performance, and identify solutions to any problems that arise, so that in future appropriate advice can be given when regulators license such installations. This paper summarizes some of the key results from the entire duration of the study conducted between July 2003 and September 2006, whose preliminary data was reported in Barry and Dillon (2005).

## SITE DESCRIPTION AND OPERATION

In June 2003 a domestic scale ASR demonstration trial commenced in the rear garden of a residential dwelling at Kingswood, South Australia, 6 km south-east of the city of Adelaide. Establishment of the site began in April 2002 when two bores were drilled, located 5 m apart with respective diameters of 100 mm for the northern well (#6628-20967), and 125 mm for the southern well (#6628-20968), and were installed to a depth of 24 m in the upper Quaternary alluvial aquifer. Below surficial clay the profile is a mixture of clay, sand and gravel to 21 m depth underlain by a stiff clay base. Groundwater flows in a north-westerly direction with an average hydraulic gradient of  $\sim 0.007$ . Pumping tests conducted on 12 December 2002 indicated well yields of 95  $\text{m}^3/\text{d}$  (1.1 L/s) in the 100 mm well and 34  $\text{m}^3/\text{d}$  (0.4 L/s) in the 125 mm well with the ambient groundwater salinity at 2500 mg/L TDS (approx. EC of 4100  $\mu\text{S}/\text{cm}$ ). Depth to ambient groundwater level was 12 m below ground surface. From these initial results it was decided in June 2003 to install the injection line in the 100 mm well with the higher hydraulic conductivity and hence greater capacity to receive injected water. The recovery line was connected to a pump and installed to a depth of 18 m. Run-off from a single residential dwelling of 285  $\text{m}^2$  was plumbed to the ASR well under gravity feed via a 4  $\text{m}^3$  tank (with 3  $\text{m}^3$  active storage) with a 1 mm tank strainer at the inlet and an in-line 100 mm filter connected between the outlet, and the ASR well. ASR operations then commenced on 29 June 2003. The average EC of the recharge water was 25  $\mu\text{S}/\text{cm}$ .

Between June 2003 and August 2004 two low-head flow meters monitored inflows and outflows to and from the ASR well, groundwater levels were monitored using independently logging pressure transducers in each of the wells, with occasional manual measurements and a YSI 600XLM water quality sonde was installed in the ASR well at 16 m depth and programmed to monitor the electrical conductivity (EC) and temperature. Daily rainfall was recorded using a rain gauge. Flow volumes were recorded manually on a daily basis and as well as monthly household water meter readings. Sampling of ambient water quality for each well was done in December 2002, prior to the start of any injection. Periodic samples of the roof run-off were collected before, during and after injection events as well as monthly gutter clearing (from April 2004) when the collections of leaf and gutter detritus were dried and weighed, with a view to determining potential organic carbon loads. The 100 mm filter also required regular cleaning out due to the loads introduced by the decomposing leaf litter and fine sediment in runoff. Since the beginning of recharge an average  $0.6 \text{ m}^3$  had been recovered per month to purge the ASR well of any accumulated biofilm or sediments and to assess the quality of the stored water.

Although  $142 \text{ m}^3$  of rainwater was injected during this first year, the water recovered during purging events showed the salinity of water was still close to background level and hence not suitable for garden irrigation. Following assessment of the recovered water quality data, a second set of pumping tests was carried out on 21/22 July 2004 to assess the permeability changes and it was decided to move the injection line to the less permeable well 125 mm diameter well, to determine if the reduced aquifer permeability would retain the fresh injectant closer to the well and thereby produce recovered water of lower salinity (ie. improve recovery efficiency).

In September 2004, to further automate the monitoring of the ASR site, a logger unit was connected to pressure transducers, to monitor water levels in the ASR well and the 'house tank', and a pluviometer was connected to monitor rainfall. The unit also had the capability to monitor the volumes and temperature of inflow and outflow and the electrical conductivity (EC) of recovered water. The sensors for these parameters were installed in August 2005. The logger unit recorded data in 3 modes; (i) event data collection was triggered at  $0.02 \text{ m}^3$  volume increments during injection and  $0.01 \text{ m}^3$  volume increments during recovery and readings of depth, EC, temperature and volume were recorded, (ii) two hourly readings of all sensors were collected on an ongoing basis and (iii) daily volumes were calculated from the monitored flowmeters. The rainfall catchment was increased partially between August – December 2005 when the runoff from a neighbouring roof ( $78 \text{ m}^2$ ) was connected. Figure 1 summarises the final site layout as of September 2005.

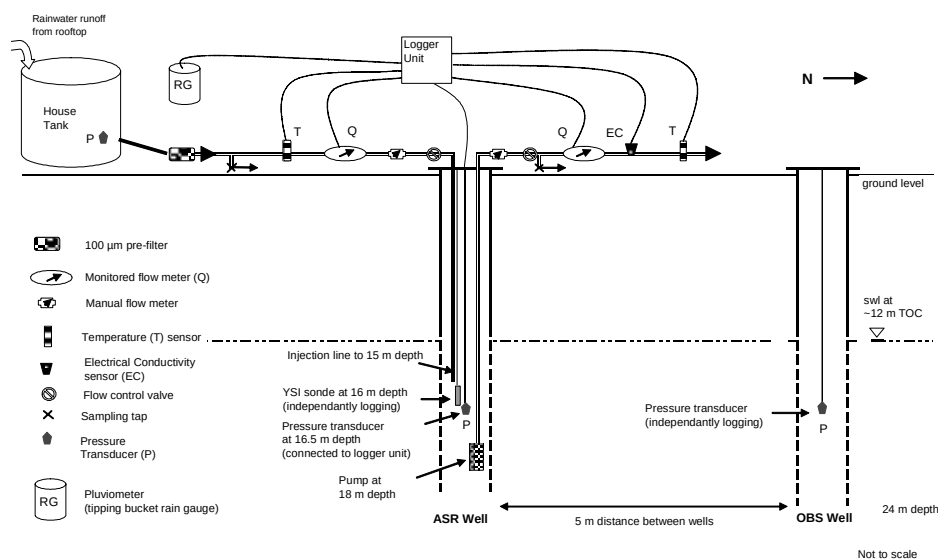


FIGURE 1. ASR site layout and monitoring equipment.

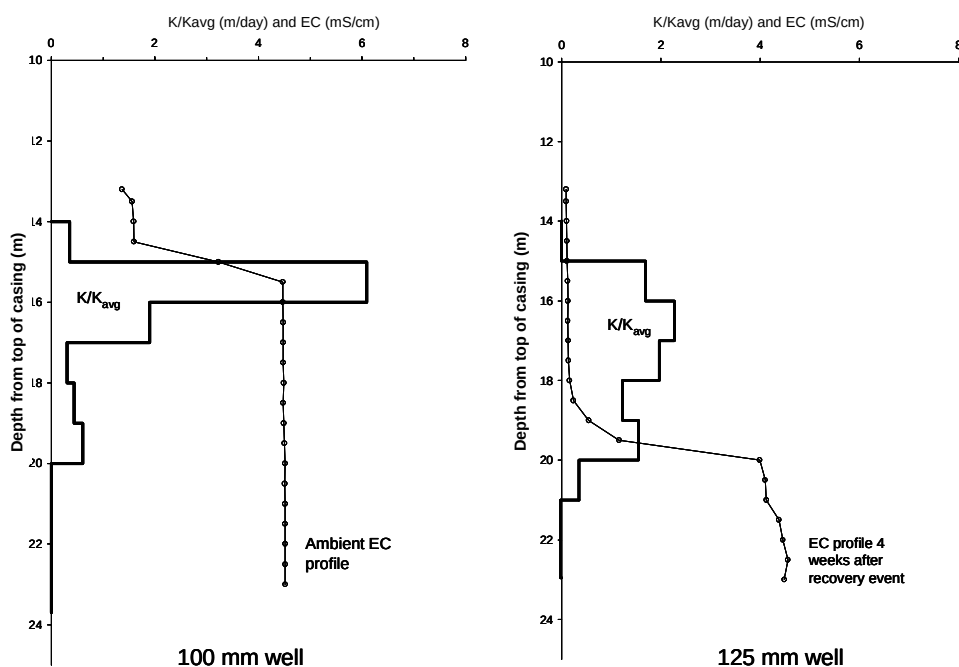


FIGURE 2. Well profiles of  $K/K_{avg}$  and Electrical Conductivity for the 100 mm (obs) and 125 mm (ASR) wells carried out on 31 May 2005.

Downhole electromagnetic flowmeter metering, downhole resistivity, gamma logging and EC profiling were carried out on 31 May 2005 to enable more detailed analysis of the aquifer hydraulic property variations with respect to depth to establish procedures in which recovery efficiency could be further improved. In the 125 mm well (the ASR well at the time) this revealed a low hydraulic conductivity section below ~ 20 m, while the EC profiling revealed a layer of more saline groundwater approximately 4 weeks after the last recovery event (Figure 2). As a consequence the ASR well was partially backfilled to 19.7 m on 2 September 2005, in an attempt to increase the recovery efficiency. Electrical conductivity and temperature profiling using a downhole sonde (YSI 600XLM) was carried out on six occasions in the 125 mm well and on five occasions in the 100 mm well, during the course of the three year study with a further series of profiles run at the end of the study in the 125 mm well (then ASR well in September 2006) before during and after a 24 hour pumping event to determine how quickly the salinity gradient returned to ambient conditions.

## RESULTS AND DISCUSSION

### Water Balance

Table 1 summarises the volumes of water injected between individual recovery events, as well as the number of days since an injection event prior to recovery and the electrical conductivity at the start and end of each recovery event. Over the 39 month period a total volume of 486 m<sup>3</sup> was injected and 34 m<sup>3</sup> recovered (7%), leaving a net increase in storage of 452 m<sup>3</sup>. Recovery consisted mostly of intermittently purging the well to remove any accumulated particulate matter.

### Ambient and Injectant Water Quality

The ambient groundwater is brackish (~4100 µS/cm) with low organic carbon (2.3 mg/L) and phosphorus (<0.025 mg/L) but has significant nitrate-N (5.8 mg/L). The rainwater/injectant is fresh (25 µS/cm) with low nutrient levels (Total N = 0.69 mg/L, Total P = 0.03 mg/L) but contained detectable zinc levels (0.14 mg/L), as a result of the galvanized steel roof. In May 2003 a stagnant sample of water considered to be a 'worst case scenario' for potential nutrient loading in the injectant was collected from a roof gutter where the down-pipe leaf guard had become choked with decaying leaves. It contained coloured water with a musty odour and had a high organic carbon and nutrient load (Total Organic Carbon = 694 mg/L, Total N = 275 mg/L and Total P = 40 mg/L). In October 2005 samples of leaf and sediment were collected from the gutters, tank strainer and 100 µm filter unit, dried, weighed and analysed for nutrients. Their respective percentage carbon contents were 39%, 36% and 32%, indicating there may be degradation and dissolution within the injectant.

**TABLE 1. Volumes of injection and recovery in each well for each recovery event of the study.**

| <b>Event No.</b>                                                          | <b>Date of Recovery</b> | <b>Recharge Volume (m<sup>3</sup>)</b> | <b>Days storage</b> | <b>Recovered Volume (m<sup>3</sup>)</b> | <b>Initial Conductivity (μS/cm)</b> | <b>Final Conductivity (μS/cm)</b> |
|---------------------------------------------------------------------------|-------------------------|----------------------------------------|---------------------|-----------------------------------------|-------------------------------------|-----------------------------------|
| <b>Injection line located in 100 mm well with open interval of 11.7 m</b> |                         |                                        |                     |                                         |                                     |                                   |
| 1                                                                         | 4/07/2003               | 3.42                                   | 3                   | 0.38                                    | 875 <sup>1</sup>                    | 1168                              |
| 2                                                                         | 17/08/2003              | 18.88                                  | 3                   | 1.20                                    | 875                                 | 1550                              |
| 3                                                                         | 7/09/2003               | 17.83                                  | 1                   | 0.29                                    | 195                                 | 1575                              |
| 4                                                                         | 21/09/2003              | 10.04                                  | 3                   | 0.30                                    | 282                                 | 1011                              |
| 5                                                                         | 5/10/2003               | 6.08                                   | 5                   | 0.28                                    | -                                   | -                                 |
| 6                                                                         | 2/11/2003               | 16.13                                  | 1                   | 0.57                                    | 180                                 | 934                               |
| 7                                                                         | 6/03/2004               | 11.80                                  | 14                  | 0.51                                    | 3380                                | 4460                              |
| 8                                                                         | 4/04/2004               | 6.31                                   | 7                   | 0.49                                    | 1835                                | 2850                              |
| 9                                                                         | 2/05/2004               | 7.30                                   | 1                   | 0.49                                    | 662                                 | 1552                              |
| 10                                                                        | 14/06/2004              | 30.75                                  | 0                   | 0.49                                    | 192                                 | 585                               |
| 11                                                                        | 4/07/2004               | 23.07                                  | 4                   | 0.51                                    | 330                                 | 1326                              |
| 12                                                                        | 21/07/2004 <sup>2</sup> | 10.23                                  | 5                   | 3.84                                    | -                                   | -                                 |
| <b>Injection line moved to 125 mm well with open interval of 10.9 m</b>   |                         |                                        |                     |                                         |                                     |                                   |
| 13                                                                        | 22/07/2004 <sup>2</sup> | 0.00                                   | 6                   | 2.04                                    | -                                   | -                                 |
| 14                                                                        | 7/08/2004               | 29.93                                  | 0                   | 1.64                                    | 73.5                                | 605                               |
| 15                                                                        | 5/09/2004               | 6.80                                   | 5                   | 0.88                                    | 90.9                                | 358                               |
| 16                                                                        | 3/10/2004               | 18.25                                  | 5                   | 0.63                                    | 113.8                               | 839                               |
| 17                                                                        | 7/11/2004               | 11.23                                  | 0                   | 0.51                                    | 108.7                               | 505                               |
| 18                                                                        | 5/12/2004               | 7.76                                   | 4                   | 0.51                                    | 108.2                               | 1255                              |
| 19                                                                        | 1/01/2005               | 4.04                                   | 22                  | 0.50                                    | 216                                 | 2280                              |
| 20                                                                        | 6/02/2005               | 7.35                                   | 5                   | 0.60                                    | 100                                 | 2000                              |
| 21                                                                        | 5/03/2005               | 4.12                                   | 24                  | 0.60                                    | 263                                 | 2470                              |
| 22                                                                        | 2/04/2005               | 2.73                                   | 28                  | 0.62                                    | 1316                                | 3200                              |
| 23                                                                        | 30/04/2005              | 0.10                                   | 15                  | 0.51                                    | 1668                                | 3650                              |
| 24                                                                        | 4/06/2005               | 0.91                                   | 21                  | 0.61                                    | 1658                                | 3720                              |
| 25                                                                        | 11/07/2005              | 29.29                                  | 2                   | 0.62                                    | 102                                 | 765                               |
| 26                                                                        | 6/08/2005               | 8.44                                   | 8                   | 0.62                                    | 58                                  | 923                               |
| 27                                                                        | 2/09/2005               | 19.01                                  | 3                   | 0.61                                    | 93                                  | 462                               |
| <b>125 mm well backfilled to approx 19.7 m - open interval 7.7 m</b>      |                         |                                        |                     |                                         |                                     |                                   |
| 26                                                                        | 3/09/2005               | 1.29                                   | 0                   | 0.60                                    | 543                                 | 884                               |
| 27                                                                        | 16/10/2005              | 29.72                                  | 9                   | 0.79                                    | 101                                 | 880                               |
| 28                                                                        | 13/11/2005              | 40.88                                  | 3                   | 0.66                                    | 75                                  | 270                               |
| 29                                                                        | 19/12/2005              | 13.65                                  | 2                   | 0.60                                    | 55.5                                | 591                               |
| 30                                                                        | 10/01/2006              | 6.46                                   | 7                   | 0.65                                    | 426                                 | 1491                              |
| 31                                                                        | 11/03/2006              | 14.31                                  | 15                  | 0.60                                    | 547                                 | 2450                              |
| 32                                                                        | 9/04/2006               | 10.08                                  | 2                   | 0.60                                    | 254                                 | 1244                              |
| 33                                                                        | 20/05/2006              | 34.10                                  | 8                   | 0.68                                    | 80                                  | 881                               |
| 24                                                                        | 18/06/2006              | 4.23                                   | 8                   | 0.59                                    | 93                                  | 1619                              |
| 35                                                                        | 26/09/2006              | 23.61                                  | 21                  | 0.58                                    | 134                                 | 2127                              |
| 36                                                                        | 27/09/2006 <sup>3</sup> | 0.00                                   | 22                  | 7.97                                    | 2270                                | 3520                              |

footnote: <sup>1</sup> EC value estimated, <sup>2</sup> 2nd set of pumptests, <sup>3</sup> final continuous pumpout

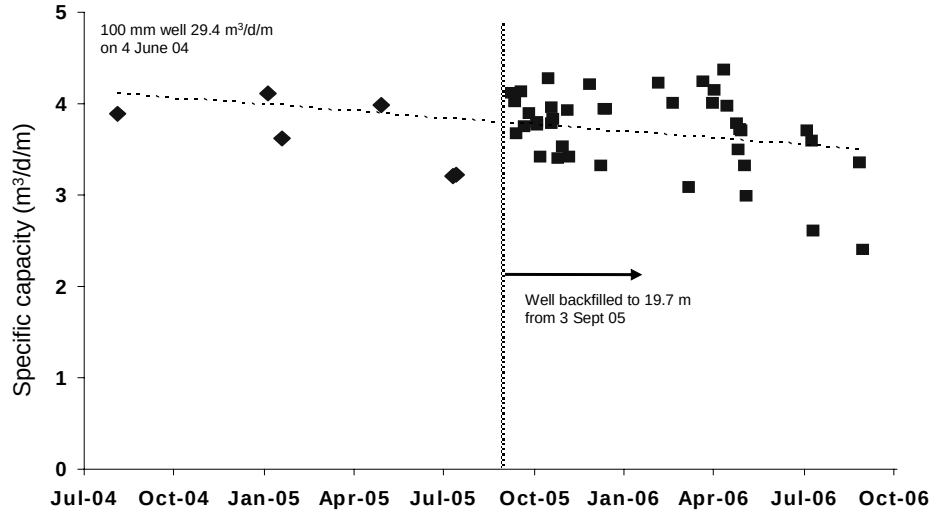


## Hydraulic Response

Standing water level (piezometric head) has been monitored since July 2002 using a combination of loggers and manual measurements. The annual groundwater level fluctuation is approximately 1 m, with the peak in December and the lowest levels in June, approximately a 3-4 month lag on seasonal rainfall fluctuations. Both wells have a similar piezometric response when there is no recharge or recovery. During the first year of operation in the 100 mm well piezometric levels only rose by about 1 metre during injection, with injection rates typically at 25 m<sup>3</sup>/d (0.3 L/s). Purging events in the 100 mm well were run at up to 86 m<sup>3</sup>/d (1 L/s) for 10 minutes, with the well never running dry. Piezometric response in the second and third years of the trial was monitored with a dedicated logger unit and monitoring reliability increased. Event triggered monitoring was introduced in September 2005, enabling detailed monitoring of piezometric response in the lower yielding well (125 mm), where head rises during injection of up to 8 m were recorded. When pumping at 1 L/s, drawdown rapidly reached the level of the pump at 18 m. Recovery events in the 2<sup>nd</sup> and 3<sup>rd</sup> years of operation were subsequently run at a lower average rate of 0.3 L/s for 35 minutes, including intermittent short periods of pumping at 1 L/s. Injection rates in the 125 mm well also began at 0.3 L/s, however rates gradually reduced and piezometric head rises increased though to the end of the study, suggesting reduced well efficiency due to very gradual clogging.

To assess the well's performance and potential clogging rate, variations in specific capacity (SC) of the well over time were determined, where specific capacity is the flow rate (m<sup>3</sup>/day) divided by the change in head (m). This was only able to be calculated for the 125 mm well on a handful of occasions prior to the event logging being triggered and once in the 100 mm well (SC = 29.4 m<sup>3</sup>/d/m on 4 June 04). Once event triggered logging commenced in September 2005, which also coincided with the period when the well had been backfilled to 19.7 m depth, more detailed analysis could be made. Figure 3 plots the specific capacity changes in the 125 mm well from August 2005 to September 2006, where it can be observed that there has been a gradual decrease in capacity of the well over time.

The turbidity of rainwater injectant ranged from 0.75 to 2.1 NTU and the total organic carbon ranged from 0.8 to 2.6 mg/L. Initial water recovered from the regular recovery/purging events was highly turbid (average 810 NTU) but after repeated 'surging' of the pump, particulate matter is removed and the turbidity drops to levels similar to the injectant, although this appears not to be enough to eliminate all the accumulated clogging agents from around the well. If this build up of clogging agents was to continue, thought to be derived from residual particulate matter and biofilm growth, then chronic clogging would eventuate, requiring more extensive treatment of the well.



**FIGURE 3. Specific capacity changes over time in the 125 mm well.**

### Mixing Processes and Recovery Efficiency

In the first year of operation the higher yielding 100mm (casing diameter) well served as the ASR well. During the second and third years of the trial, ASR was switched to the lower yielding 125mm (casing diameter) well.

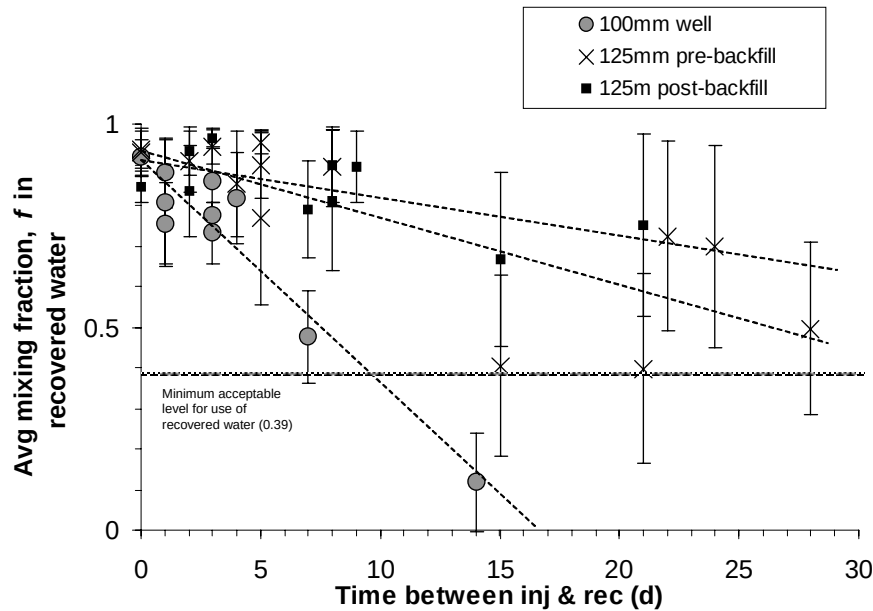
During injection events in the first year of operation the electrical conductivity (EC) of the ambient groundwater in the 100mm well ( $>4000 \mu\text{S/cm}$ ) was diluted with fresh injected roof runoff water ( $<100 \mu\text{S/cm}$ ). With regular injection events over winter the EC in the 100mm well was maintained at below  $1000 \mu\text{S/cm}$ . Over the summer months as injection events become less frequent, the EC of the groundwater in the ASR well during storage periods and regular backwash/recovery events increased, as indicated in Table 1. From the second year of operation (injection into the 125 mm well), similar trends were observed. Recovery efficiencies based upon the arbitrary  $2500 \mu\text{S/cm}$  upper limit for irrigation were typically less than 10% for each recovery event.

In order to characterize mixing, the mixing fraction ( $f$ ) which is the proportion of injectant in a given sample of groundwater or recovered water, must be defined;

$$f = (C_{mix} - C_{amb}) / (C_{inj} - C_{amb}) \quad (\text{EQ 1})$$

where  $C_{inj}$  and  $C_{amb}$  are the EC concentrations of the injected water and ambient groundwater respectively and  $C_{mix}$  is the EC value of the recovered water.

Figure 4 plots the recovered EC data expressed in terms of  $f$  in recovered water versus the time since the last injection event. Points plotted are the mean of initial volume (recorded at  $0.1 \text{ m}^3$ ) and the final value (recorded at  $0.6 \text{ m}^3$  recovery). This indicates that longer storage periods lead to lower values of  $f$ . Note that the minimum acceptable mixing fraction in the recovered water is 0.39, corresponding to the 2500 mS/cm upper limit. The average rate of decline in  $f$  during the first stage of the trial (ASR into 100mm well) of  $5.6 \times 10^{-3}$  per day was 3 times faster than the average for the second stage of the trial (ASR into 125mm well) of  $1.6 \times 10^{-3}$  per day, which in turn, was marginally faster than for the final stage of the trial (post partial-backfilling of the 125mm well) of  $0.9 \times 10^{-3}$  per day. This approach neglects the dependence of seasonality on the water balance. It also assumes that each sequence of injection/recovery can be treated as being independent for the purpose of defining the value of defining  $C_{amb}$  although we recognize that in reality there will be some dependence of the quality of recovered water on the history of previous events. Regardless of the simplifications made for the purposes of the analysis, the results suggest that the lower variation in hydraulic conductivity of the 125 mm well compared with the 100 mm well (Figure 2) does lead to longer localised retention of fresh injectant. Although the backfilling operation appeared to improve the fraction of injectant recovered after longer retention times, the degree of improvement was insufficient to improve recovery efficiencies significantly.



**FIGURE 4. Recovered EC expressed as mixing fraction ' $f$ ' compared with time between injection events (bars give the initial (upper) and final (lower) values of  $f$ ).**

Three possible mechanisms may partially or fully account for the strong mixing signatures observed:

- lateral drift arising from advective transport due to regional groundwater flow
- diffusive exchange between mobile and less-mobile parts of the aquifer due to aquifer heterogeneity
- buoyancy stratification of the injected water caused by density contrasts between the two water types

Lateral drift of the injected water down-gradient of the ASR well, the first potential factor identified above, was explored using a methodology presented by Pavelic *et al* (2002) and using equations by Bear and Jacobs (1965). These suggest that on average, the physical dimensions of the freshwater body around the ASR well as a result of the volume of injected each month, when displaced down-gradient by the amount of calculated drift due to regional flow, would still fully occupy the volume of aquifer surrounding the ASR well that is recovered during monthly pumpouts (assuming uniform radial flow and complete displacement of pore fluid). Thus it would appear that there are mechanisms in addition to drift required to account for the low recovery efficiencies observed.

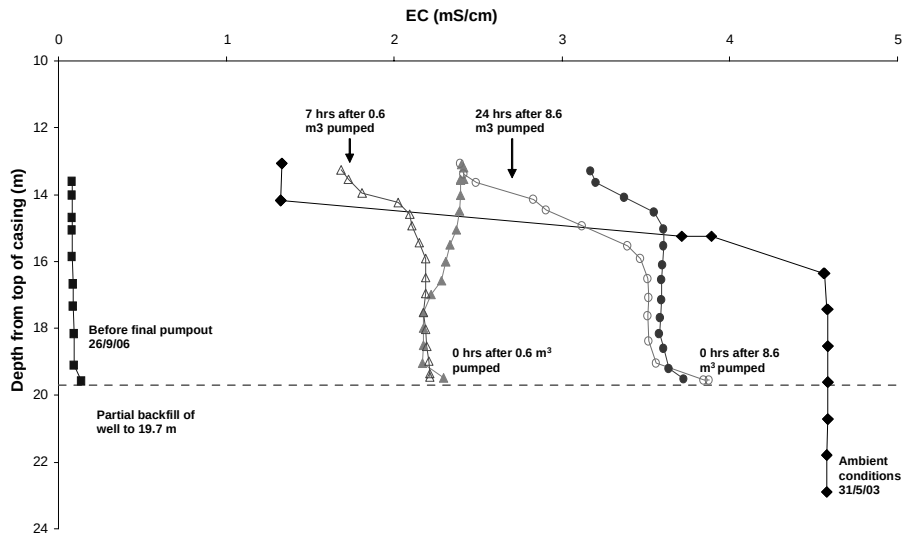
The relative hydraulic conductivity ( $K/K_{avg}$ ) profiles determined from the downhole flowmeter logging, suggests a higher degree of heterogeneity for the 100 mm well than for the 125 m well (Figure 2) and this may account for the contrasts between the 100 mm and 125 mm wells observed in Figure 4. Fresher groundwater in the top two metres of the profile overlying more brackish groundwater is a characteristic feature of both wells in ambient conditions and presumably the result of localized vertical recharge. Low permeability and high salinity in the lowest 3 m of the profile for the 125 mm well (4 weeks after a recovery event), led to the hypothesis that backfilling the bottom 3 m of the 125 mm well (to 20 m) could lead to an improvement in recovery efficiencies, which in retrospect was shown to be only slight.

It is also noted that the clay content of the profile may invalidate the earlier assumption of piston flow. Diffusive exchange between saline native groundwater occupying smaller pores incapable of flushing by advection of fresh injectant may contribute to salinity increase during storage. The slowing of the rate of mixing with ambient groundwater shown in figure 4 for the 125 mm well could also be interpreted as a gradual substitution of water in the finer grained pores.

Declining rates of EC increase with time during the course of a multi-year injection had also been observed previously within the near-well zone at the Andrews Farm site (Pavelic *et al*, 2006). Within a 25 m radius of the ASR well the aquifer clearly exhibited dual-porosity characteristics and the mechanism for mixing was thought be due to exchange between the fresh injected fluid which occupies the most permeable zones, and the brackish ambient groundwater entrained within the less

permeable parts of the aquifer. With successive flushing, all of the entrained ambient groundwater was displaced within around 3 years of flushing.

In September 2006, the final recovery event involved the regular sequence of pumping ( $\sim 0.6 \text{ m}^3$ ) followed by an extended pump-out ( $\sim 8 \text{ m}^3$ ), interspersed with monitoring of EC profiles in the ASR well. EC data, presented in Figure 5, reveal a fresh EC profile comparable to the EC of the injectant prior to pumping. With pumping of  $0.6 \text{ m}^3$ , the EC profile was partially returned to ambient conditions and after a total of  $8.6 \text{ m}^3$  had returned to 80% of the EC of ambient conditions. The rapid tendency for EC to decline in the top half of the profile in the hours following pumping appears to reflect a shift in the profile towards the ambient profile. In all cases a density gradient was observed in the EC profiles, and most notably under ambient conditions, suggesting that density-induced stratification may be an important factor affecting recovery efficiency. Further analysis using numerical modelling will be undertaken to attempt to deconvolve these effects.



**FIGURE 5. EC profiles in ASR well (125 mm diameter well) before, during after regular ( $0.6 \text{ m}^3$ ) and extended ( $8.6 \text{ m}^3$ ) pumping-out events in September 2006.**

## CONCLUSIONS

Analysis of flow rates and change in piezometric head has shown that some gradual and persistent clogging of the ASR well has occurred, even with monthly pumping out of approximately  $0.6 \text{ m}^3$  at flow rates up to three to four times higher than the recharge rate. Initially quite turbid water is recovered at the start of each event, with particulate concentrations rapidly decreasing with pumping. This suggests that purging does have real value in helping to alleviate clogging. However a gradual decline in specific capacity was observed suggesting that more vigorous well redevelopment strategies would be needed in the longer term.

Under the piston flow assumptions of Bear and Jacob (1965), preliminary assessment of the site indicated potential for a successful ASR operation using roof-top runoff, however the observed heterogeneity of the aquifer, and possibly density effects, have limited the capacity of the site, where recovery efficiency did not exceed 10% over three year study period. One problem appears to be insufficient injection volumes to counter the relatively rapid rate of groundwater drift in the brackish aquifer. In spite of shifting to a well with lower and more uniform permeability distribution, and subsequent backfilling part of that well this problem persisted. An evaluation of density-related effects will be undertaken to determine their influence on recovery efficiencies at this site.

## ACKNOWLEDGEMENTS

The monitoring reported in this paper was made possible through the support of the Patawalonga Catchment Water Management Board now incorporated within the Adelaide and Mt Lofty Ranges Natural Resources Management Board. Flow meters were provided by SA Water Corporation. Downhole flowmetering was performed by Don Freebairn and Vic Freschi from the Geophysics Section of the Department of Water, Land and Biodiversity Conservation, SA. James Ward from Flinders University of South Australia assisted with a level survey of local wells. Constructive review of the manuscript was provided by an anonymous reviewer on behalf of the ISMAR6 Organizing Committee.

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## ARTIFICIAL RECHARGE OF GROUNDWATER IN A COAL MINING AREA OF NORTHEAST MEXICO

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### ABSTRACT

Artificial recharge of groundwater has been successfully implemented to replenish extracted groundwater in a coal mining area of northeast Mexico. The aquifer is mainly composed of unconsolidated Quaternary clastic sediments and Tertiary chalks, and is overlying at least 9 coal layers, 5 of which have economic value and are exploited using surface and subsurface coal mining methods. Surface coal mining requires the complete aquifer dewatering around the open-pit (40-70 m depth) for coal production. After extracting groundwater ( $0.916 \text{ m}^3/\text{sec}$ ), water is injected downgradient and not too far from the open-pit mining area. In order to evaluate the mining effects on the aquifer, the Mexican Institute of Water Technology performed during the year 2004 a groundwater balance around the open pit, the delineation of the affected area by mining activities and a numerical model implementation. Water balance performed around the open pit showed that recharge efficiency is of 99.78. The area affected by mining activities extends 1 km around the open pit according to the water table configuration and 3 km down gradient from the open pit, according to the geochemistry data. Numerical flow simulations also showed that the preexisting conditions before mining activities will be reestablished by the year 2024.

### KEYWORDS

Groundwater recharge, coal mining, groundwater dewatering

### INTRODUCTION

Successful groundwater management requires the understanding of the different properties within a groundwater flow system. Artificial recharge is becoming an important component in groundwater management, especially in semi-arid areas where alternative water sources are limited. Although almost 65% of Mexico has a semi-arid climate, only few artificial recharge projects have been implemented within a sustainable groundwater management scheme in the country.

Coal mining is one of the main industrial activities in the Fuentes-Rio Escondido Carboniferous Basin located at Coahuila state, northeast Mexico. Surface coal mining requires overburden excavation using walking dragline procedures. The aquifer (Tertiary age) is intercepted by open cut because it is located above the coal seam (Cretaceous age). Several wells area used to pump out groundwater ( $29 \times 10^6 \text{ m}^3/$



year) and dewater the aquifer around the open-pit mining areas, producing environmental impacts on water quality (Gutierrez-Ojeda *et al.* 2005), quantity and modification of local and intermediate groundwater flow systems.

With the purpose to minimize environmental impacts related with its mining activities, Minera Carbonífera Rio Escondido (MICARE) designed, constructed and nowadays operates a successful artificial recharge project in northeast Mexico. After extracting groundwater from the aquifer above the coal seam, water is injected downgradient and not too far from the open-pit mining area. From the extraction area water is distributed by system of open unlined channels to two recharge sites, where is latter injected through several recharge wells.

The main purpose of this paper is to show the evaluation performed by the Mexican Institute of Water Technology (IMTA, 2004) to the recharge project implemented by MICARE during the year 2004.

### **The Study Area**

The study area is located at the north of Coahuila state, some 30 km south from Piedras Negras, city placed just in the international border with the United States of America (Figure 1). It covers a total area of about 3,000 km<sup>2</sup> and is situated between 100° and 101° W-longitude and between 28° and 29° N-latitude.

The area has a gently topography that slopes (0.003) towards the NE. The maximum altitude is about 400 masl (meters above sea level) at the western and 200 masl close to the Rio Bravo; some minor elevations are detected around Piedras Negras city and along principal river courses.

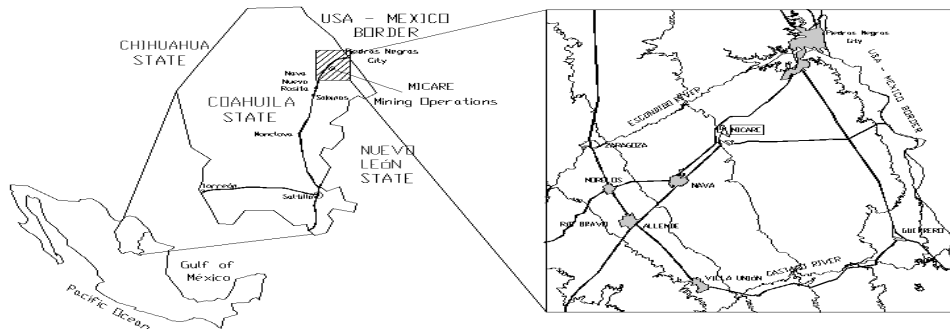
According to the Köppen classification, Piedras Negras has a dry climate, semi-warm, with an average temperature of 18 °C. The mean annual precipitation in the area is 490 mm and the rainy season is from April to October. Evaporation is high and exceeds precipitation: its mean annual value reaches 1830 mm.

The zone belongs to the Bravo-Conchos River basin and Escondido-San Antonio-Castaños rivers sub-basin. These streams are ephemeral and only the Bravo river shows base flow during the whole year.

### **Geology**

Quaternary deposits represented by unconsolidated sediments and chalk are covering older geological units in most of the plain. Cretaceous formations (Upson, San Miguel and Olmos) outcrop at the southwest region; they are mainly composed of limestone sandstone and shale. A Tertiary locally unit called Sabinas-Reynosa Conglomerate (2-6 m calcareous conglomerate lower unit and a ≈25 m sandy-clayey chalk unit locally known as “caliche” and some gypsum) discordantly overlies the

Olmos Formation. At least 9 coal layers have been identified within the Olmos Formation, 5 of them have economic value and are exploited using surface and subsurface coal mining methods.



**FIGURE 1. Location map showing the study area.**

### Hydrogeology

The Sabinas-Reynosa Conglomerate hydrogeological unit constitutes the main aquifer in the area. The groundwater flow system is mainly controlled by the gently slope towards the northeast and the calcareous conglomerate; in some parts of this unit a karst-conduit network has been identified from direct observation and core sampling. The recharge zone of the intermediate flow system is in the west part of the region while the general discharge area is adjacent to the Rio Bravo, close to the international border with the United States of America (IMTA, 2004).

### Mining Activities

As the aquifer is above the Olmos Formation, mining operations require its complete depletion around the open-pit (40-70 m depth) for coal production. A number of dewatering wells are used to capture the groundwater flow towards the open pit, modifying the natural groundwater flow direction. Total groundwater extraction for dewatering purposes accounts  $\approx 0.91$  m<sup>3</sup>/s. As soon as the extracted groundwater is on surface, a gravity driven system of open unlined channels diverts it to two recharge sites; 0.49 m<sup>3</sup>/s are distributed to the Santo Domingo site, and 0.42 m<sup>3</sup>/s towards a second facility called the Aqueduct site. Each of the sites consists of a pond with several injection wells at the bottom; gravity induced injections rates in each well are typically 0.04-0.06 m<sup>3</sup>/s.

## METHODS

The methodology followed to fulfill the objective of the work included the review of previous studies, geologic and geophysical investigations, pumping tests, well drilling, piezometry and groundwater water sampling. The information obtained from the latter activities was used to perform a hydrogeochemistry and groundwater flow analysis, including a water balance and the implementation of a numerical model. Backfilling materials were hydraulically characterized by geophysics, well drilling and pumping tests.

On 113 of 119 wells and windmills detected on the area surrounding the open pit was measured the water table elevation on July 2003. The area covered an extension bigger than that defined as affected by Lesser (2002). Each well was located with the help of a GPS. Forty of the 113 wells were surveyed by means of a differential GPS system which included a fixed station linked to a reference level. The recorded level was measured after 40 minutes with the purpose of obtaining a precision in the order of mm. Water table elevation of July 2003 was later compared with the March 1993 configuration, which was considered as previous to the mining activities operations.

About 70 groundwater samples were taken in different parts of the aquifer. Samples were obtained from active wells, springs, lagoons in abandoned open pits and water used for artificial recharge just before its injection. Temperature, specific electrical conductivity (SEC), Eh (Pt-electrode), pH and dissolved oxygen (DO) were measured in the field using an in-line flow cell. Alkalinity was also measured in the field by Gran titration. Samples for major anions were taken in 500 ml bottles, for major cations, silica and trace elements were filtered with a 0.45  $\mu\text{m}$  membrane filter and acidified with ultra-pure nitric acid (1 ml acid per 100 ml sample) and stored at 4°C until analyzed. Analytical methods used for major anions are compatible with those reported in APHA (1995), major cations measurements were done by atomic absorption; calculated ionic balance was within 5% in 100% of the samples. A complete suite of trace elements was analyzed by inductively coupled plasma mass spectrometry (ICP-MS); accuracy of ICP-MS analyses was controlled using duplicates, blanks and appropriate laboratory standards, it was also checked during each batch using reference standards including SLRS-4 and NIST 1640.

MICARE drilled three wells in the backfilling material with the purpose of determining (i) its hydraulic conductivity by pumping test; (ii) if there was a water table; and (iii) in case, characterize the groundwater quality.

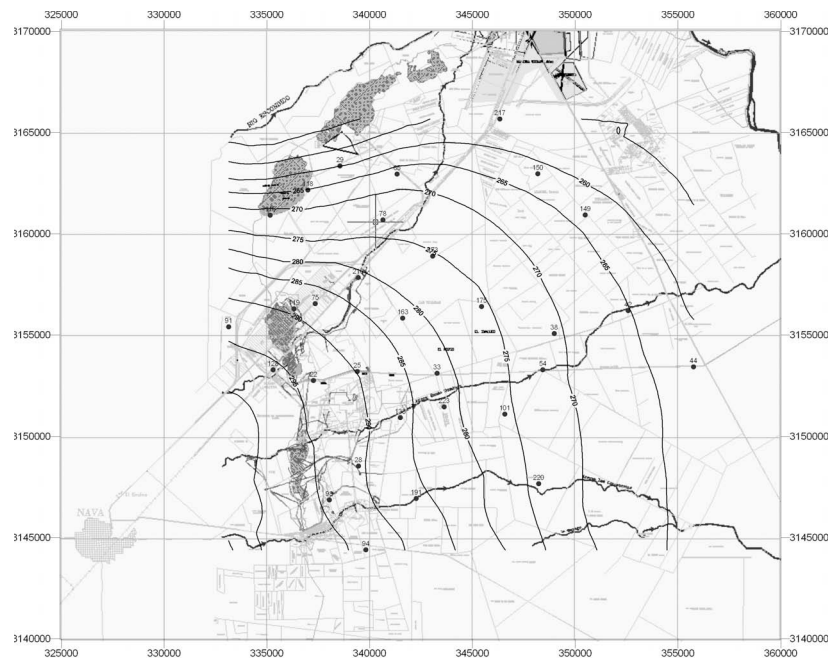
The hydraulic conductivity of the geologic units found in the area was determined by 24 pumping tests of short duration (21 to 150 minutes). The discharge was maintained constant during the test development. This parameter together with the estimated “specific yield” was used in the groundwater water model.

The vertical and horizontal distribution of the hydrogeologic units in the area was also determined with help of a geophysical survey which included 150 transitory electromagnetic time domain geophysical surveys (TEM's). With that information 9 profiles were constructed in such an array that both the open pit and unaffected areas were covered.

With the hydraulic characteristics of the backfilling material, the conceptual model was implemented with the finite difference model Modflow (WHI, 2003a). The groundwater model was set up according to the regional hydrogeologic characteristics and the MICARE exploitation plan. The model was used to evaluate the time required to reestablish the groundwater flow around the open pit mine. Two scenarios were considered in the model that (i) the groundwater flow and the abstraction rates will remain stable, and (ii) the recharge activities will continue during the whole coal exploitation plan, i.e., until the year 2012.

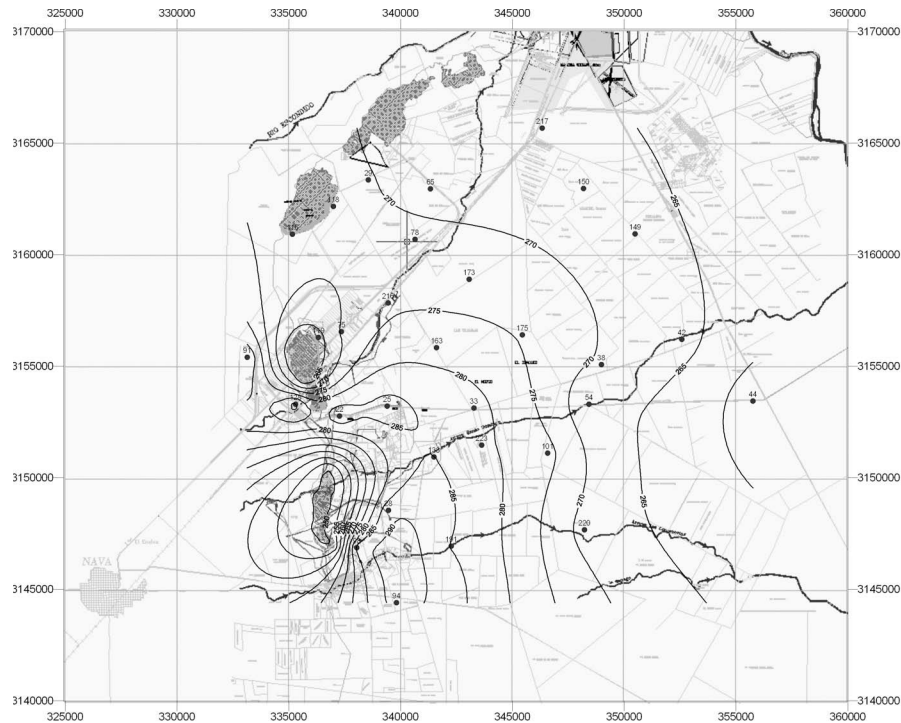
## RESULTS AND DISCUSSION

Figure 2 shows the water table configuration of March 1993, before the mining operations started. Water table elevation ranged from 295 mamsl around the open pit to 250 mamsl close to the Bravo River. It can also be observed that the groundwater flow was from SW to NE, with a hydraulic gradient of 0.0022.



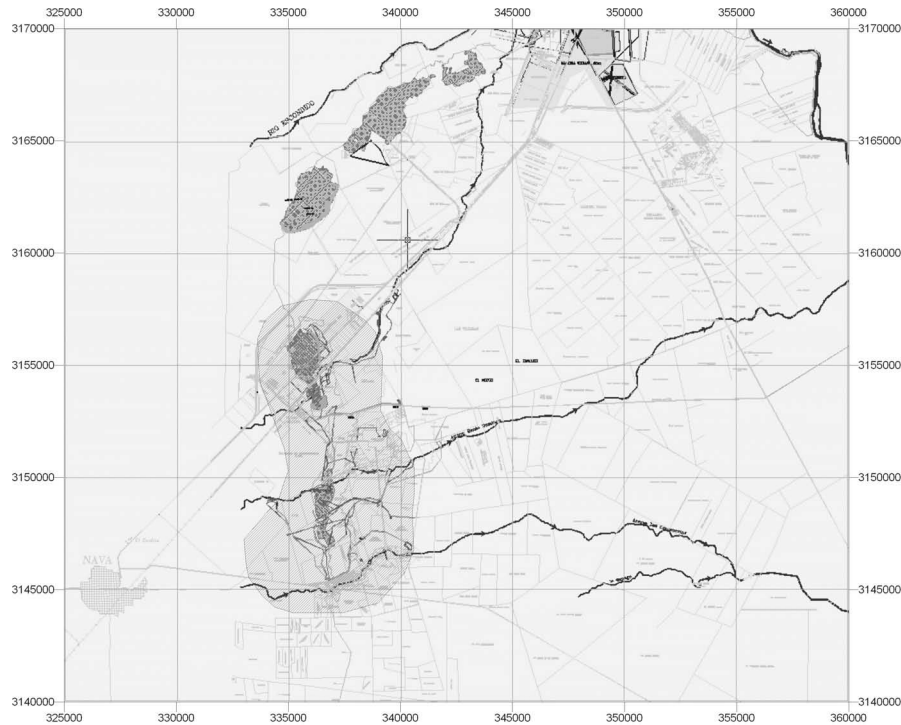
**FIGURE 2. Water table configuration by March 1993 (in mamsl).**

Water table elevation of July 2003 has the same general tendency of the groundwater flow, i.e. from SW to NE, but shows two big drawdown cones and a dome in the vicinity of the open pit mine (Figure 3). The drawdown cones show the effect of the dewatering process around the open pit. On the other hand, a dome is located around the Aqueduct pond, which together with the downgradient flow, clearly showed the positive effects of the induced recharge project.



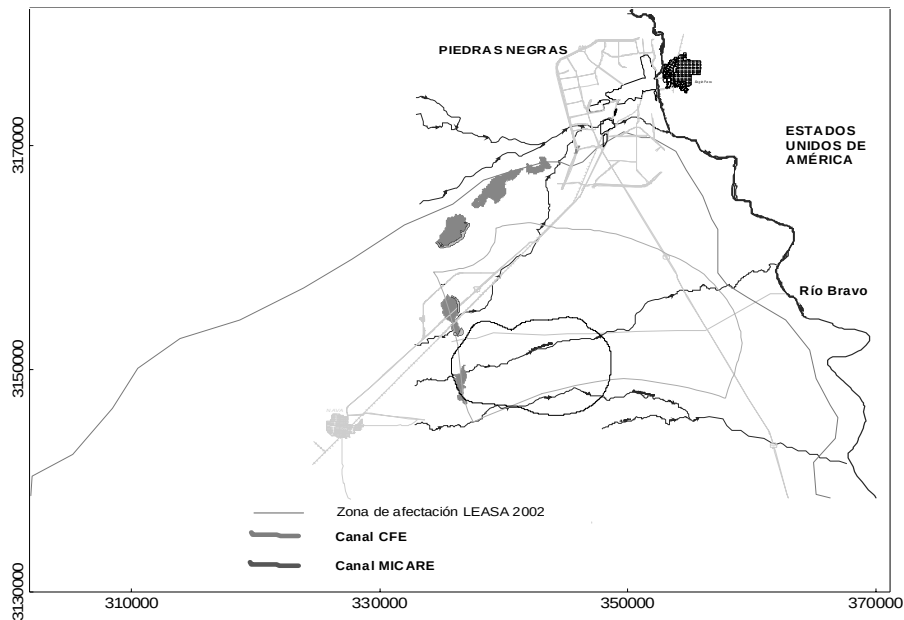
**FIGURE 3. Water table configuration by July 2003 (in mamsl).**

From the water configuration of December 2003 (Figure 4) the hydrodynamic divide around the open pit was defined as well as the affected area (7,900 ha), where the natural groundwater flow was modified. Downgradient from the hydrodynamic divide the groundwater flow is almost completely reestablished due to the recharge operations and the extraordinary rain events of 2003.



**FIGURE 4. Affected area (red dashed) by the mining activities on December 2003, defined by the groundwater divide.**

On the other hand, water injected to the aquifer has total and fecal coliform concentrations quite above the Mexican standard (SSA, 1994). Such contamination is associated with the presence of cattle on the channels used to divert the dewatered water. The coliform spatial distribution (Figure 5) is therefore indicative of the affected area which in fact was much smaller than originally reported in 2002 (Lesser, 2002). It is important to note that the affected area was defined with base on the reference value of 400 NMP/100 ml, which is higher to 200 NMP/100 ml considered as a reference background value of the aquifer. The affected area defined in this work corresponds to the hydrogeologic conditions of December 2003 which can change due to the dynamic nature of the aquifer system both in space and in time. The area defined by Lesser (2002) has a surface of 38,850 ha, while in this work was defined as 9,000 ha. On the other hand, samples taken at the drilling sites of the backfilling materials had manganese concentrations above the Mexican standard of 0.05 mg/l, which was attributed to the influence of the well casing material.



**FIGURE 5. Affected area by the mining activities on December 2003, defined by the geochemistry (total coliform concentration).**

Analysis of the pumping test was done with Aquifer Test 3.5 software (WHI, 2003a). Although the duration of the pumping tests was very short, some wells experienced drawdown stabilization. As the recorded drawdowns were small (less than 10% of the saturated thickness) pumping tests data were adjusted to Theis (1935) and recovery Theis models. The estimated hydraulic conductivity values range from  $1.16\text{E-}03$  to  $9.35\text{E-}07$  m/sec, which corresponds to silty-sandy to clayey-sandy materials (Freeze & Cherry, 1979). The backfilling material has an average hydraulic conductivity of  $6.09\text{E-}05$  m/sec. Considering a mean hydraulic conductivity of  $4.04\text{E-}04$  for the aquifer natural materials, the results indicates that the mining activities reduce the hydraulic conductivity by an order of magnitude.

TEM's profiles show the presence of 4 geologic units: (i) a surface partially saturated and permeable unit (resistivity  $> 8$  ohm-m) of 3-5 m thick composed of vegetal soil, chalk and conglomerates; (ii) a saturated permeable unit (resistivity 8-25 ohm-m) of 20-40 m thick which includes the shallow aquifer located at 12-18 m depth composed of compact chalk, and the deep aquifer located at 35 m deep composed of karstic and granular material, both of the Sabinas Formation; (iii) a low permeable unit (resistivity 1-8 ohm-m) composed of shale, sandstone lenses of 20-100 m thick of the Olmos Formation; and (iv) a very low permeable unit (resistivity 8-100 ohm-m) composed compact sandstone of the San Miguel Formation. The shallow aquifer

shows a limited lateral extension while the deep one has a more regional presence. The backfilling material shows resistivities in the range of 10-40 ohm-m, which correspond to units with some degree of permeability, which was corroborated by the presence of water table on the wells drilled by MICARE.

Surface water balance performed around the open pit is as follows (in Mm<sup>3</sup>/year):

Input

- Dewatering pumping 28.915

Output

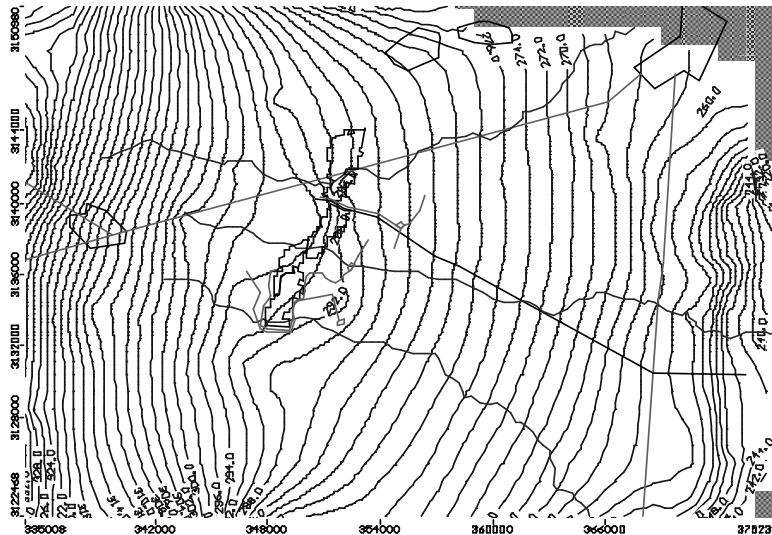
- Channel infiltration 8.221

- Pond infiltration 20.581

- Evaporation in channels and ponds 0.113

Therefore, water consumption due to the mining activities around the open pit is 0.113 Mm<sup>3</sup>/year, which represents 0.40 % of total extraction, while the volume of water infiltrated represents 99.6%.

The numerical model results indicate that until the year 2024, the water table will be completely reestablished around the open pit mine area and its configuration will be similar as the one observed in the year 1993 (Figure 6), i.e., before the coal mining activities.



**FIGURE 6. Water table configuration by December 2024.**



## CONCLUSIONS

The backfilling materials are not impermeable and allow a smaller groundwater flow than the unaffected and natural materials that composed the aquifer. This fact was important especially for the modeling exercise.

To preserve the groundwater quality and reestablished the groundwater flow is necessary that MICARE refill the open pit mine as well as to continue with the recharge activities during the whole coal exploitation plan and beyond, that is the year 2024.

To solve the coliform water contamination it is necessary to protect channels and pond with fences.

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## ON RECOVERY EFFICIENCY OF MANAGED UNDERGROUND STORAGE OF RECOVERABLE WATER SYSTEMS

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### ABSTRACT

One of the key questions raised in feasibility assessment of a managed underground storage (MUS) of recoverable water system is: “how much recharge water can be recovered?” When fresh water is stored in the brackish or saline aquifers the answer to this question must consider degradation of water quality that occurs when part of the recharge water mixes with the ambient brackish groundwater to form a transition zone. In this paper the authors evaluate existing definitions of recovery ratio and recovery efficiency, clarify their similarities and differences, and introduce operational recovery efficiency (ORE) and system recovery efficiency (SRE). The ORE defines recoverability of the stored water for an individual recharge-recovery cycle of a MUS system, such as one injection-pumping cycle for an Aquifer Storage and Recovery well/system, while the SRE is defined as a percentage of total recoverable water over total stored volume over the life of the system. Understanding transition zone is the key to better defining recovery efficiency. The transition zone varies as a function of hydraulic gradient, dispersivity, permeability and porosity heterogeneity, relative density of the native and recharge water and hydrodynamic dispersion. The authors also discuss other factors that affect operational and system recovery efficiency.

## KEYWORDS

Managed underground storage of recoverable water; aquifer storage recovery (ASR); operational recovery efficiency; system recovery efficiency; transition zone.

## INTRODUCTION

Aquifers are increasingly used as an underground storage reservoir of fresh water in short-term and long-term water resources management. One of the key questions raised in feasibility assessment of a managed underground storage of recoverable water system is “how much water can be recovered?” When fresh water is stored in the brackish or saline aquifers the answer to this question must consider degradation of water quality that occurs when part of the recharge water mixes with the ambient brackish groundwater to form a transition zone. Although physically recoverable, water from the transition zone is frequently termed nonrecoverable because it does not meet a water quality criterion. Approximately one third of all aquifer storage and recovery (ASR) wells are storing potable water in a brackish or saline aquifer (Pyne, 2005). Therefore, the characterization of the transition zone between recharge water and ambient groundwater is of great importance in assessment of recoverability of the stored water. The transition zone varies as a function of hydraulic gradient, dispersivity, permeability and porosity heterogeneity including presence of dual porosity, relative density of the ambient brackish groundwater and recharge water and hydrodynamic dispersion (Reese, 2002; Pyne, 2005; Maliva, Guo and Missimer, 2005 and 2006; Lowry and Anderson, 2006; Vacher, Hutchings and Budd, 2006).

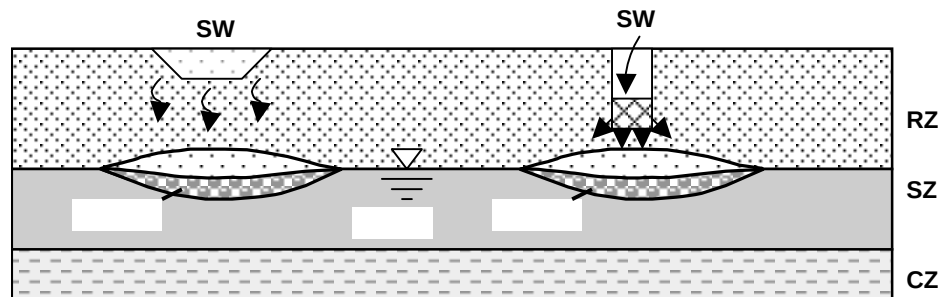
The two terms widely used to describe the volume proportion of recoverable fresh water - recovery ratio and recovery efficiency have led to confusion among decision-makers trying to understand and evaluate results from storage test programs in that the results from a single recharge-pumping cycle were used to represent overall performance of the ASR system. The recovery ratio is defined as volume of pumped recharge water over volume of recharge water (Bear and Jacobs, 1965; Bear, 1979). It was also called molecule for molecule recovery (Reese, 2002), which usually result in a lower estimate of recoverability of recharge water due to ignoring mixing of stored and native waters (Pyne, 2005). The conventional recovery efficiency is defined as the percentage of the water volume stored in an operating cycle that is subsequently recovered in the same cycle while meeting a target water quality criterion, for example potable water quality, in the recovered water (Reese, 2002, Pyne, 2005; Lowry and Anderson, 2006). In this definition, the mixing between recharge water and native groundwater is allowed and accounted for depending on how the targeted water quality criterion is set. It may result in a higher estimate of recoverability for the stored water by replacing non-recoverable stored water with native water or stored water from previous cycle during the recovery.

In this paper, the authors evaluate existing definitions of recovery ratio and conventional recovery efficiency, clarify their similarities and differences, and introduce two new terms: operational recovery efficiency (ORE) and system recovery efficiency (SRE). The ORE is used to define recoverability of the stored water for an individual recharge-recovery cycle of a managed underground storage (MUS) of recoverable water system (National Research Council, 2007), such as one single recharge-pumping cycle for an Aquifer Storage and Recovery system, while the SRE characterizes overall performance over the operational or design life of the MUS system.

## METHODS

### Hydrological Zones and Processes

To facilitate description of the managed underground storage of recoverable water system dynamics, Figure 1 includes three major components: hydrologic zones in the aquifer, types of water in these zones and hydraulic/hydrogeologic processes (National Research Council, 2007). In broad terms, recharge of a MUS system occurs via infiltration, gravity flow, or injection through a well. During recharge, the source water (SW) displaces the native groundwater (NGW) in the aquifer storage zone (SZ). Mixing of the two waters occurs in the transition zone (TZ) located along the outer margin of the recharge water (RW) (Merritt, 1985). The stored water (RW + part of TW) will be recovered by ASR wells or neighboring production wells as needed.



(a) Spreading Basin (b) vadose zone well/dry well  
(c) Deep well in a confined aquifer (d) Deep well in an unconfined aquifer

**FIGURE 1. Hydrological processes for three different recharge methods: surface spreading basin; vadose recharge well; and deep recharge wells (National Research Council, 2007).**

In the recharge basin schematic (Figure 1a), the source water (SW) infiltrates into an unconfined storage zone (SZ) through a recharge zone (RZ). Recharge water (RW) is

depicted as a “mound” on native groundwater (NGW). Water comprising the base of the lens is transition and corresponds to a transition zone (TZ) containing a mixture of recharge water and native groundwater or transition water (TW). The entire lens is the storage zone (SZ), whereas the vadose zone located above the water surface is the recharge zone (RZ). The groundwater level will still fluctuate in response to changes in natural recharge, which is affected by changes in land use, seasons and climates. Other factors include pumping of neighboring production wells and the influence of nearby recharge projects.

In the vadose zone recharge well schematic (Figure 1b), the source water (SW) was injected using a vadose dry well into an unconfined storage zone (SZ) through a recharge zone (RZ). Recharge water (RW) shown as a mound on native groundwater (NGW) with underlying water as a transition (mixture) water (TW) between RW and NGW. The entire lens is the storage zone (SZ); the part of the saturated zone containing TW is the transition zone (TZ). For above schematics, production wells need to be constructed to recover the recharge water in the vicinity of recharge zone. To achieve higher recovery efficiency, a collector well system can be considered if hydrological condition permits. This system is very similar to the aquifer-storage-transfer-recovery (ASTR) system developed in Australia (Rinck-Pfeiffer, Pitman and Dillon, 2005) except deep wells were used to recharge water.

The ASR schematic includes recharge of source water (SW) into a storage zone (SZ) located either above (Figure 1c) or beneath a zone of relative confinement (CZ) through a well (Figure 1d). In the context of ASR, the recharge zone (RZ) is the same as a storage zone (SZ). Recharge water (RW) has an irregular shape to reflect aquifer heterogeneity. Vacher, Hutchings and Budd (2006) showed that “bottle brush” better represents the geometric complexity predicted from the known and inferred heterogeneity. Only in non-buoyant, isotropic, homogenous aquifer conditions would the term “bubble” appropriately describe the shape taken on by the recharge water. Proximal to the RW is the transition water (TW), which is located in a transition zone (TZ). Recharge water can be recovered by dual function wells themselves or neighboring production wells.

## Recovery Efficiency

In this paper we will focus on the recovery of recharge water through deep wells. First, let's take a look of recovery ratio  $\lambda$  defined by Bear and Jacobs (1965). It depends on the ratio of pumping and injection rates, time of pumping, and time of injection.

$$\lambda = \frac{\text{Vol. of Pumped Recharge Water}}{\text{Vol. of Recharge Water}} = \frac{\int_0^{t_p} Q_p^{(1)} dt}{\int_0^{t_i} Q_i dt} \quad (1)$$

where  $Q_i$  and  $Q_p^{(1)}$  are instantaneous rates of injection and pumping of previously recharged water, and  $t_i$  and  $t_p$  are times for recharge and pumping periods.

It only accounts for the recharge water in the pumped water or molecule for molecule recovery (Reese, 2002). The recharge water can be fully recovered only if the ultimate injected front does not penetrate the water divide of pumping stage (Bear and Jacobs, 1965). Otherwise part of recharge water can never be recovered by the injection well itself once it travels beyond the water divide of pumping stage.

Bear and Jacobs (1965) used the instantaneous relative concentration  $\varepsilon$  to denote changes in recovered water relative to recharge water,

$$\varepsilon = \frac{C - C_0}{C_i - C_0} = \frac{Q_p^{(1)}}{Q_p} \quad (2)$$

where  $C$  is the salt concentration in the pumped water,  $C_i$  is the concentration in the recharge water and  $C_0$  is the concentration of the native water assuming larger than  $C_i$ , and  $Q_p$  is the total pumping rate.

If the  $C$  in equation (2) is equal to  $C_i$ , or  $\varepsilon$  is equal to 1, it means that pumping solely recovers the recharge water. Once  $C$  exceeds  $C_i$  or  $\varepsilon$  is no longer equal to 1, pumping started to extract transition water (TW) as shown in Figure 1c and 1d where native groundwater is mixed with the remaining recharge water. When  $C$  equals to  $C_0$ , pumping solely extract native groundwater beyond transition zone.

Second, conventional recovery efficiency is defined as percentage of volume of recovered water over volume of recharge water for a recharge-pumping cycle as long as the recovered water meet target water quality criterion in terms of Total Dissolved Solids (TDS) or chloride concentration (Reese, 2002, Pyne 2005). Here we will call it Operational Recovery Efficiency (ORE).

$$ORE = \frac{\text{Vol. of Pumped Water}}{\text{Vol. of Recharge Water}} = \frac{\int_0^{t_p} (Q_p^{(1)} + Q_p^{(2)}) dt}{\int_0^{t_i} Q_i dt} = \lambda + \frac{\int_0^{t_p} (Q_p^{(2)}) dt}{\int_0^{t_i} Q_i dt}$$

(3)

where  $Q_p^{(2)}$  is instantaneous rate of pumping of native groundwater, and the total pumping rate  $Q_p = Q_p^{(1)} + Q_p^{(2)}$ .

Let's assume  $\varepsilon$  is equal to 1, which means no transition water (TW) is pumped, i.e.  $Q_p^{(2)}$  equals to 0, then equation (3) becomes equation (1). However, for most ASR projects the salt concentration of pumped water (C) will be greater than that of the recharge water ( $C_i$ ). As desired, TW will be pumped as much as the targeted water quality criterion permits during the recovery. As a result, ORE is usually greater than  $\lambda$ . It should be noted that ORE only reflects performance of a single ASR well or ASR well system for one recharge-pumping cycle, which does not necessarily represent long-term performance of the ASR system.

To assess long-term overall performance of the ASR system, system recovery efficiency (SRE) over the operational or design life of the ASR system is introduced.

$$SRE = \frac{\text{Total Vol. of Pumped Water}}{\text{Total Vol. of Recharge Water}} = \frac{\sum_{j=1}^n V_{p,j}}{\sum_{k=1}^n V_{i,k}} = \frac{\sum_{j=1}^n \frac{V_{p,j}}{V_{i,j}} V_{i,j}}{\sum_{k=1}^n V_{i,k}}$$

$$= \frac{\sum_{j=1}^n ORE_j V_{i,j}}{\sum_{k=1}^n V_{i,k}} = \sum_{j=1}^n ORE_j \frac{V_{i,j}}{\sum_{k=1}^n V_{i,k}} = \sum_{j=1}^n ORE_j P_j$$

(4)

where  $V_{i,j}$  and  $V_{p,j}$  are volume of recharge and pumped water for the cycle  $j$ .  $ORE_j$  is the operational recovery efficiency for the cycle  $j$ .  $P_j$  is percentage of recharge water volume for the cycle  $j$  out of the total recharge volume over the operational or design life of the system.

SRE not only accounts for performance within each individual recharge-pumping cycle, but also represents for interaction between cycles. For example, in one recharge-pumping cycle water was recharged, but not fully recovered due to operational requirements or lack of demand, which will result in a lower ORE for this

cycle. The subsequent cycle continues to recharge the aquifer as usual, however may recover the full amount of recharged water during the current cycle or even withdraw water stored there from previous cycle as long as the recovered water meets water quality criterion. In this case a higher ORE is obtained for this cycle. Therefore, it is misleading to use either of those OREs to represent the overall performance of the system.

## RESULTS AND DISCUSSION

### Operational vs. System Recovery Efficiency

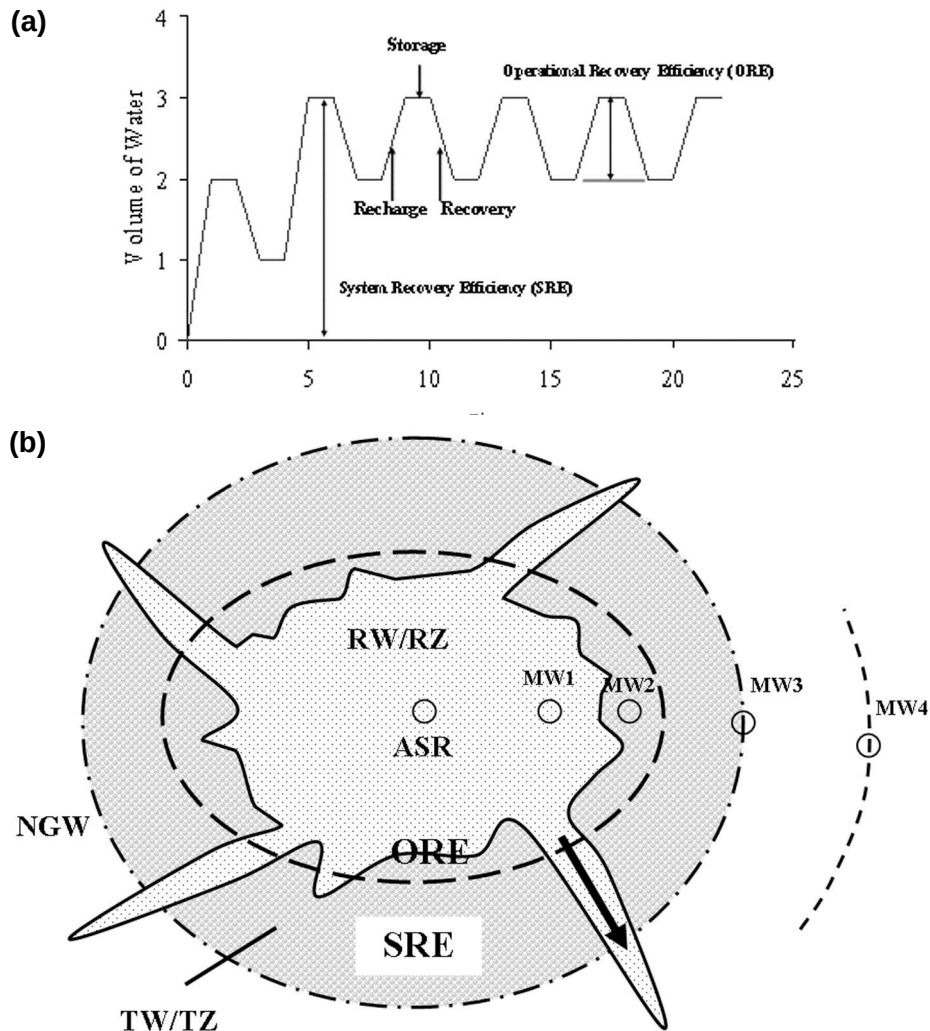
Even though SRE is related to ORE for each cycle, SRE is different from ORE and is more appropriate to describe overall performance of the system. Figure 2 illustrates differences between ORE vs. SRE in terms of volume of recovered vs. recharged water as well as hydrologic zones where different types of water occur.

A common practice exists to define recoverability of recharge water based on an individual cycle. As an ASR well field is developed through cycle testing, or perhaps development of a “total storage volume” (TSV) (Pyne, 2005), “recovery efficiencies” are generally reported to increase substantially, often reaching 100%. In fact, these “efficiencies” reflect an ORE, not a SRE. In the context of the overall system performance, this is misleading.

Lets look at an example at Delray Beach, Florida, where the storage zone is brackish, confined limestone artesian aquifer between 310 and 366 m (1,020 and 1,200 ft) with a TDS concentration of 4,200 mg/l. Recharge and recovery rates are about 9,465 and 10,600 cubic meters per day ( $\text{m}^3/\text{d}$ ) or 2.5 and 2.8 million gallons per day (MGD), respectively. The desired volume of recovery is about  $189,270 \text{ m}^3$  (50 MG) per cycle. The TSV of  $946,350 \text{ m}^3$  (250 MG) was initially formed in order to achieve a recovery TDS concentration not to exceeding 550 mg/L, which was increased to  $1.21 \text{ million m}^3$  (320 MG) to meet the 450 mg/l TDS goal (Pyne, 2005).

It means that a total of  $1.21 \text{ million m}^3$  of fresh water was recharged before the next recharge-pumping cycle. Assuming ORE is 100% or  $189,270 \text{ m}^3$  of recharged water is fully recovered for each of 20 cycles without adding additional water to maintain the TSV; theoretically the SRE will be up to 76% for 20 complete cycles. In this calculation, previously stored  $946,350 \text{ m}^3$  of TSV was not taken into consideration. Should the water quality in transition zone deteriorate with time, it could result in a lower ORE in later cycles and/or require additional water be recharged to restore the TSV. It will further





**FIGURE 2. Definition of operational recovery efficiency and system recovery efficiency: (a) volume of water recharged and recovered; (b) Recharge zone and transition zone (lobes reflect dual porosity-conducts or fractures) (National Research Council, 2007).**

Pyne (2005) points out a perception issue with recovery efficiency. Scientists prefer to be analytical in defining the term in order to truly reflect the amount of water recharged versus recovered at a given ASR site. Several attempts have been made in assessing conventional recovery efficiency or ORE using numerical models (Brown, 2005, Lowry and Anderson, 2006). Additional research is needed to develop methods and approaches to evaluate SRE using hydrological information of the aquifer, water

chemistry data needed for characterizing physical and chemical process and microbial interaction during whole phases of recharge, storage and recovery, cycle test results, and well design parameters.

### **Factors Affecting Recovery Efficiency**

Will the recharge water stay at storage zone in the aquifer until the recovery occurs? In many cases, the recharge water will migrate or mix with native groundwater due to hydrological and boundary conditions of the aquifer. If water is stored in shallow aquifers, stored water may discharge into a neighboring stream as baseflow or flow downward to recharge deep aquifers as the hydraulic head increases. As water levels increase in the shallow aquifer, evapotranspiration from the shallow aquifer may also occur.

System recovery efficiency varies with recharge methods, hydrological properties of the geological formation, and recovery methods. In general, surface recharge may have less system recovery efficiency comparing deep injection in the same aquifer due to evapotranspiration and other losses for surface spreading. Principal factors that influence the sustainability and recoverability of stored recharge water involve the quality, quantity and movement of water. Recovery of freshwater stored in brackish-to-saline-water aquifers is controlled by a wide variety of factors that pertain to hydrogeologic conditions, well or well field design, and operational management (Merrit, 1985; Reese, 2002; Maliva, Guo and Missimer, 2005 and 2006; Lowry and Anderson, 2006). The hydrogeologic factors of a storage zone that are important to recoverability include (1) ambient salinity, (2) aquifer permeability and distribution, (3) aquifer thickness, (4) confinement, (5) ambient hydraulic gradient, and (6) structural setting. Important design and management factors to consider are (1) thickness and location of the storage zone within the aquifer, (2) volume of injected water, (3) duration and frequency of cycles and cycle storage periods, (4) well performance problems such as wellbore plugging, and (5) multiple well configurations. Most of these factors and their control on recoverability of recharge water have been numerically simulated (Merritt and others, 1983; Merritt, 1985; Brown, 2005; Maliva, Guo and Missimer, 2005; Lowry and Anderson, 2006). For example, simulation of the SEAWAT code, a three-dimensional variable-density groundwater flow program indicated that heterogeneity within storage zones decreases recovery efficiency in that it results in an interfingering of layers containing stored freshwater and native formation waters (Maliva, Guo and Missimer, 2005). Zones with high hydraulic conductivities also experience rapid migration of stored water under natural and pumping-induced hydraulic gradients. Lowry and Anderson (2006) have also demonstrated interactions between hydrogeologic and operational parameters on predictions of recovery efficiency using coupled MDFLOW and MT3D model.

In either scenario as shown in Figure 1, the transition zone varies as a function of hydraulic gradient, dispersivity, permeability and porosity heterogeneity including presence of dual porosity, relative density of the native and recharge water, and hydrodynamic dispersion. Recovery efficiency decreases as regional hydraulic gradient was increased. Hydraulic gradient becomes even more sensitive parameter when smaller volumes of water are injected and in combination of high value of hydraulic conductivity and lower effective porosity (Lowry, 2004). As effective porosity is inversely related to groundwater velocity, effective porosity directly affects recoverability of recharge water (Lowry and Anderson, 2006). Mechanical dispersion is a scale-dependant parameter that pertains to fluid mixing due to flow through heterogeneous media. Diffusion reflects the movement of dissolved species from higher to lower areas of concentration and does not require flow. In a dual porosity storage zone, the role of diffusion is significant, whereas in a more homogenous aquifer, the effects of diffusion are masked by dispersion (Figure 2b). Increased mixing of recharge water and ambient native groundwater causes a decrease in both ORE and SRE. Transmissivity is also a significant factor with regard to recovery of recharge water. For example, exceedingly high transmissivity not only increases the potential for water to travel beyond the zone of capture or recovery, but it also promotes mixing with native groundwater, which may be undesirable due to native water-quality. On the other hand, transmissivity that is too low may require extreme wellhead injection pressures during recharge and may cause large drawdown during recovery.

## CONCLUSIONS

The two terms, namely recovery ratio and recovery efficiency, widely used to describe recoverability of stored fresh water, have led to confusion among decision-makers in that a single cycle test result was misused to represent the system performance. To avoid such confusion, the authors introduced two new terms, namely operational recovery efficiency (ORE) for a single recharge-pumping cycle and system recovery efficiency (SRE) for the overall performance of the MUS system over its operational or design life.

The ORE is defined as percentage of volume of recovered water over volume of recharge water for a recharge-pumping cycle as long as the recovered water meets target water quality criterion in terms of Total Dissolved Solids (TDS), chloride concentration or other chemical constituents. It is identical to conventional recovery efficiency. It is noteworthy that if we set up recovered water quality criterion using the recharge water quality, ORE becomes recovery ratio  $\lambda$ , an indicator for the sole recovery of recharge water. SRE is defined as percentage of total volume of recovered water over total volume of recharge water for the operational or design life of the MUS system. It not only accounts for ORE for each recharge-pumping cycle, but also represents for interaction between two cycles.

Multiple hydrogeologic and operational factors control both ORE and SRE. Their interactions further complex their impacts on the recoverability of recharge water. Numerical models can be used to assess effects of key hydraulic factors both physical and operational on recoverability of recharge water. A better understanding of the effects of those key hydraulic factors will not only provide a deep insight into science of the managed underground storage of recoverable water system, but also provide guidelines for water managers to make right decisions in development and operation of such systems.

## ACKNOWLEDGMENTS

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## AQUIFER STORAGE AND RECOVERY OF RECLAIMED WATER IN FLORIDA: HYDROGEOLOGICAL ISSUES

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### ABSTRACT

Reuse of reclaimed water for irrigation is part of the State of Florida goal to encourage and promote conservation of freshwater resources. Aquifer storage and recovery (ASR) is being looked upon by utilities as a means of increasing the supply of reclaimed water during the dry season and reducing demands on freshwater resources. Migration of reclaimed water during storage is a critical hydrogeologic issue controlling both ASR system performance and regulatory compliance. Heterogeneity in hydraulic conductivity profoundly controls the distribution of the stored water within an ASR storage zone aquifer and the rate and direction of the stored water migration. The failure of some ASR systems in Florida has been due to the lack of recognition of the existence or potential impacts of highly transmissive flow zones or conduits within the chosen ASR storage zones. The potential for adverse fluid-rock interaction, such as arsenic leaching, was also not fully appreciated. The Florida ASR experience illustrates the importance of performing a detailed aquifer characterization before making the commitment to construct a full-scale ASR system at a site. Modeling that incorporates site-specific data on aquifer heterogeneity and geochemistry can allow for more accurate assessment of ASR feasibility and prediction of system performance.

### KEY WORDS

ASR, reclaimed water, reuse, hydrogeology, Florida

### INTRODUCTION

Aquifer storage and recovery (ASR) is the underground storage of water during periods of excess supply for later use during periods of peak demand or water shortages. No other area of the world has more ASR systems in various stages of development than the State of Florida. The hydrologic regime of Florida makes the state an ideal candidate for ASR. A pronounced seasonality of rainfall and a rapidly growing population is resulting in freshwater shortages during the winter and spring dry season. Most of Florida's average annual rainfall of over 50 inches (127 cm) occurs during the summer wet season, which results in a large volume of excess water that is potentially available for storage. Water stored in existing or planned ASR systems in Florida include treated potable water, treated surface water, raw groundwater, and reclaimed water.

Reuse of reclaimed water for irrigation is being pursued in Florida because it frees higher quality fresh groundwater resources for greater value uses, particularly potable water supply. In some communities, such as Collier County and the City of Cape Coral, nearly all of the dry season reclaimed water flow is reused. Excess reclaimed water is available during wetter periods, but potential customers are generally unwilling to commit to irrigate with reuse water unless a year-round supply can be provided. ASR provides a means of storing excess reclaimed water available during wet periods to increase dry season supply, which would allow for an increase in the overall average reuse of reclaimed water. The ultimate goal is to put nearly all reclaimed water to a beneficial use.

Successful operation of an ASR system requires site-specific hydrogeologic conditions that are favorable for the recovery of most of the stored water. In the case of reclaimed water ASR, the storage of reclaimed water should not adversely impact aquifers that are currently, or may in the future, be used for potable water supply. There is growing recognition that ASR systems are hydrogeologically more complex than originally perceived. This paper summarizes the hydrogeologic issues facing ASR implementation in Florida, particularly with respect to the storage of reclaimed water.

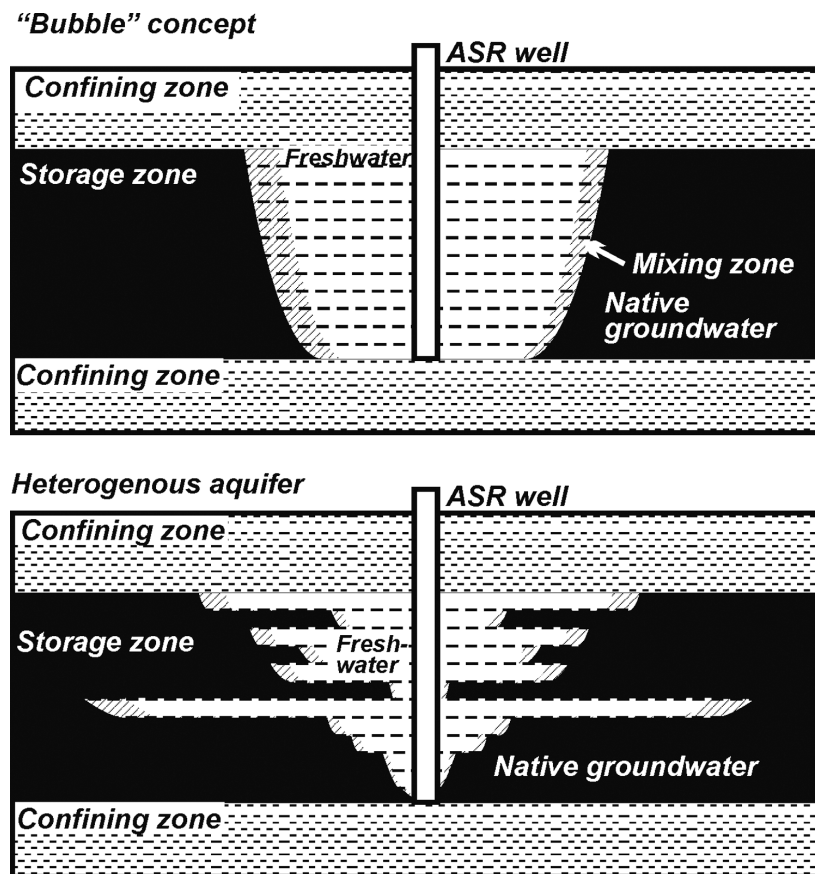
## USEFUL STORAGE

A successful ASR system provides hydrologic benefits. First and foremost, it must result in the useful storage of water. The injection of water into an aquifer must create a new resource that can be exploited in the future. No aquifer in Florida is currently being mined in the sense that the aquifers are not being depleted by the progressive long-term removal of water from storage. Instead, aquifer drawdowns that are believed to cause adverse impacts, such as the dehydration of wetlands and saline-water intrusion, are dynamic responses to aquifer withdrawals. A key point is that if withdrawals stop, aquifer heads and water levels quickly recover. Conversely, once injection to an ASR system is terminated, aquifer heads quickly recover to background levels, as if no injection had occurred. Injection does not result in residual head build-up in the aquifers of Florida.

ASR projects provide a hydrologic benefit in Florida to the degree that they shift groundwater withdrawals during high-demand periods from heavily exploited freshwater aquifers to under-utilized aquifers that contain poorer quality water. Aquifers containing brackish water are mainly used for storage zones in Florida. Injection of freshwater creates useful storage by displacing native brackish water. Drawdowns during the recovery phase of ASR systems do not significantly contribute to the drawdowns, and associated environmental impacts, in shallower freshwater aquifers. ASR in Florida using freshwater aquifers as storage zones, in most instances, would have little hydrologic value because the drawdowns during recovery would cause increased cumulative drawdowns and associated impacts.

## AQUIFER HETEROGENEITY

The prevailing paradigm for ASR systems has been that the stored water forms a compact “bubble” that does not migrate far from ASR wells (Figure 1). Implicit in the “bubble” concept is that aquifers are homogenous (Vacher et al., 2006). The vast majority of existing and proposed ASR systems in Florida use carbonate aquifers as storage zones. Carbonate aquifers tend to have high degrees of heterogeneity with respect to hydraulic conductivity. The heterogeneity present in Florida aquifers is due to primary (depositional) compositional and textural differences, diagenesis, and differential response to structural deformation, particularly fracturing (Maliva et al., 2002, 2005; Budd and Vacher, 2004). Hydraulic conductivity of strata within an aquifer may have a range of several orders of magnitude.



**FIGURE 1: Schematic diagrams of the metaphoric ASR bubble and heterogeneous aquifer. In a heterogeneous ASR system, stored water is confined largely by low permeability strata within the storage zone.**



Aquifer heterogeneity impacts ASR systems in several manners. Injected freshwater will preferentially enter beds roughly in proportion to their hydraulic conductivity. The bulk of the water will enter the most conductive beds, whereas little water will enter the tightest beds. Rather than having the shape of a bubble, stored bodies of freshwater are more accurately described as having the shape of an inverted Christmas tree or bottlebrush (Missimer et al., 2002, Hutchings et al., 2004, Vacher et al., 2006; Maliva et al., 2006).

The morphology of the freshwater bodies has a profound effect on ASR system performance. The historic model for ASR systems in brackish-water aquifers is that the stored freshwater is bounded above and below by confining strata with low vertical hydraulic conductivities (Figure 1). The confining strata minimize both the vertical migration and mixing of stored freshwater, which are critical for maintaining water quality. In heterogeneous systems, aquifer layers containing freshwater are interdigitated with layers containing native formation water in the storage zone. The freshwater is confined largely by the less permeable strata within the storage zone (internal confinement), not by the confining strata that adjoin the storage zone (external confinement; Figure 1). Internal confinement may be much less effective than external confinement in preventing migration and mixing of stored and native waters. The results of ASR system performance simulations using density-dependent solute-transport programs, such as SEAWAT (Guo and Langevin, 2002), indicate that aquifer heterogeneities can result in large decreases in the recovery efficiency of the system (Maliva et al., 2005, 2006).

Aquifer heterogeneity also impacts the horizontal extent and migration rate of stored freshwater. Basic geometric considerations indicate that if most of the injected freshwater enters only a small fraction of the total thickness of the storage zone aquifer, then the horizontal extent of the freshwater body will be much greater than would be the case in a homogenous aquifer. As the horizontal extent of the stored freshwater body increases, there is a greater possibility that it will intercept a vertical flow conduit, such as a fault, fracture zone, karst feature, or improperly constructed or abandoned well (Missimer et al., 2002). Vertical flow conduits can result in a large part of the stored water becoming irretrievable.

According to Darcy's Law, groundwater flow velocity is directly proportional to hydraulic conductivity. Horizontal flow rates may therefore vary between beds by several orders of magnitude in heterogeneous aquifers used for ASR storage zones. The greatest flow rates will occur in the most transmissive beds, which receive a disproportionate amount of the injected freshwater. Actual horizontal migration rates will be much greater than values calculated using an average hydraulic conductivity values obtained by dividing the aquifer transmissivity by its thickness.

The implications of aquifer heterogeneity and flow rates on system performance is that the injected stored water may migrate rapidly away from the ASR well under prevailing hydraulic gradients. As the freshwater migrates away from the ASR, native

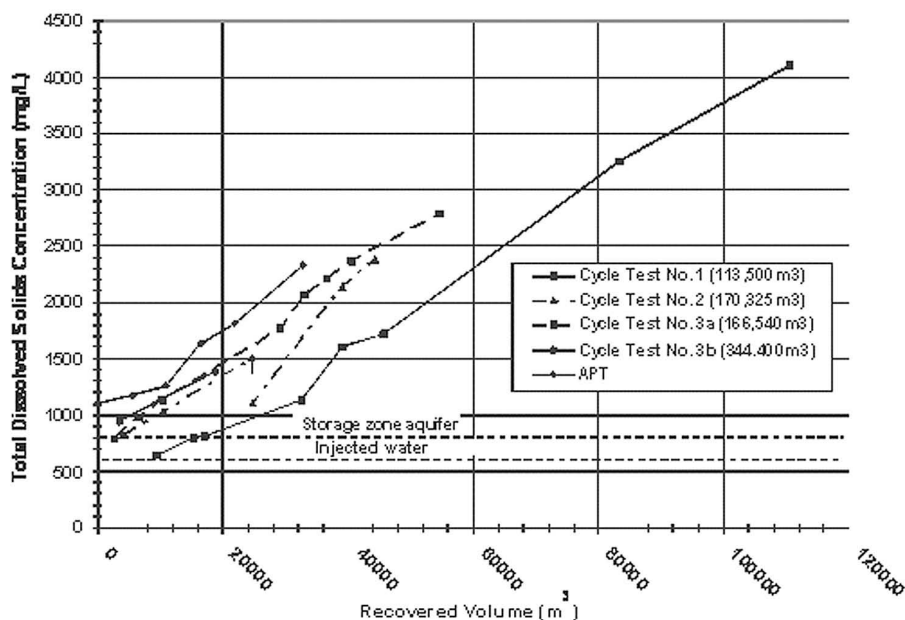
formation water will correspondingly migrate towards the ASR well. Rapid horizontal migration of injected water can thus greatly reduce ASR system performance. For example, an ASR system in the San Carlos Estates area of Lee County, Southwest Florida, was abandoned because of poor recovery (= 3%; Reese, 2002). A flowmeter log of the 51 ft (15.5 m) thick injection zone indicates that most of the flow into the storage zone occurs within a 4 ft (1.2 m) thick zone.

Groundwater flow rate is also directly proportional to hydraulic gradient. The effects of aquifer heterogeneity on the migration rate of stored freshwater will increase with hydraulic gradient. If an ASR system is located in an area with a low regional hydraulic gradient, differential migration rates within an ASR storage zone may be a minor issue as the migration rates of all of the beds will be low.

A more extreme type of a heterogeneous aquifer is a dual-porosity system in which a large part of the groundwater flow is focused in fractures, karst conduits, or other secondary features. Dual-porosity systems can experience rapid migration of stored water and a high degree of mixing (Maliva et al, 2006). Dual-porosity systems can also allow for the rapid migration of poor quality water into ASR systems. An abandoned reclaimed water ASR system in northwestern Hillsborough County, Florida, provides a good example of the effects of saline water migration on ASR system performance. The salinity of the recovered water rapidly increased with recovered volume irrespective of the amount of water injected in an operational testing cycle (Figure 2). The salinity versus recovered volume trends differed little from the trend for an aquifer performance test (APT) conducted before the start of injection. Evaluation of the water quality data, using three component mixing equations with chloride and fluoride concentrations as conservative tracers, indicate that the deterioration in recovered water quality was due to the introduction of a small amount of relatively high salinity water. The recovered water consisted mostly of reclaimed water, as evidenced by its retention of the elevated fluoride concentration of the reclaimed water. Fluoride concentration is a useful indicator of reclaimed water because of the fluoridation of drinking water. The most likely source of the saline water is migration through an unidentified fault, fracture zone, or karst feature.

Evaluation of aquifer heterogeneity may involve a combination of geophysical logging, coring of storage zone strata, and performance of multiple off-bottom pumping or packer tests. Geophysical logging is particularly valuable because it can provide fine-scale, in situ measurements of aquifer and confining zone properties. The water industry has lagged way behind the oil industry in the application of geophysical logging for the characterization of subsurface strata, which is starting to change. For example, Schlumberger has performed advanced geophysical logging at test wells for the Comprehensive Everglades Restoration Plan (CERP) ASR test wells. Geophysical logs such as nuclear magnetic resonance (NMR) can provide fine-scale measurements of hydraulic conductivity for an entire borehole. Log-derived measurements of hydraulic conductivity can be calibrated against aquifer testing

data. Geophysical logging can also identify secondary flow zone (e.g., fractures and karst conduits), whose presence can have a great adverse impact of ASR system performance.



**FIGURE 2: Operational (cycle) testing results from an ASR system in northwestern Hillsborough County, Florida. The salinity versus recovered volume trends for the cycles tests are similar to that of the pre-injection aquifer performance test (APT).**

## FLUID-ROCK INTERACTION

The ASR program in Florida was blindsided with the discovery that the arsenic concentration of freshwater in many ASR systems was increasing during underground storage to levels above the United States and Florida maximum contaminant level (MCL). The leaching of arsenic into stored water had become a more serious problem for ASR system owners and operators with the lowering of the arsenic MCL in the United States from 50 µg/L to 10 µg/L in January 2006. Systems that were in compliance with the old arsenic MCL became out of compliance with the new MCL. Leaching of uranium has also occurred in ASR systems in Florida, but concentrations have remained below the MCL.

The arsenic concentration injected water is well below the new arsenic MCL. However, the increase in arsenic concentration in ASR systems is still a violation of Federal and Florida Underground Injection Control (UIC) rules in that underground

injection is causing a violation of an applicable groundwater standard in an underground source of drinking water (USDW). A USDW is defined as a non-exempt aquifer with a total dissolved solids (TDS) concentration of less than 10,000 mg/L. Elevated arsenic concentrations may restrict the use and disposal of recovered water and may necessitate post-treatment, which would be a significant additional capital and operational cost for an ASR system.

The increased arsenic concentrations in recovered water from ASR sites in Florida appears to be caused by the oxidation and dissolution of trace amounts of finely disseminated arsenic-bearing pyrite and possibly organic matter (Arthur et al., 2000; 2001, 2002 2005; Price and Pichler, 2006; Jones and Pichler, 2007). Arsenic leaching has occurred in both ASR systems storing potable water and reclaimed water. The extent of arsenic leaching at levels of concern ( $> 10 \mu\text{g/L}$ ) in Florida ASR systems is not fully known. Until recently, analysis of recovered water for arsenic was not required by the Florida Department of Environmental Protection (FDEP) because arsenic leaching was not recognized to be a problem.

The release of arsenic and radionuclides in concentrations of regulatory concern has not been reported in the majority of ASR systems in Florida. Water quality changes in ASR systems in South Florida were recently reviewed by Mirecki (2004). The Mirecki study included 11 ASR systems, in which arsenic data obtained using low detection limits methodologies (graphite furnace atomic absorption) were available from 7 systems. Arsenic concentrations exceeding the new  $10 \mu\text{g/L}$  MCL were detected in recovered water from only 2 of the 7 systems; Olga (Lee County) and Marco Lakes (Collier County). The available data suggest that arsenic leaching might be more of a problem in the Charlotte Harbor to Tampa Bay region. Nevertheless, the possibility of significant arsenic leaching cannot be excluded at any ASR system. The current state of knowledge is such that it is not possible to predict with certainty the potential for significant arsenic leaching in advance of operational testing.

The geochemical process of arsenic leaching in response to the introduction of oxygenated water into an aquifer is not a newly discovered phenomenon. The leaching of arsenic, uranium, and other metals in ASR systems is analogous, for example, to processes that naturally occur in the formation of uranium roll front deposits (e.g., Drever, 1982). Arsenic and other metals in mineral phases present in trace amounts in aquifers may be the source of the bulk of arsenic released in ASR systems. It is therefore important to perform a mineralogical/chemical investigation of the aquifer matrix to identify and characterize minor mineral components at an early stage in ASR systems (Price and Pichler, 2006). Historically, very few ASR programs in Florida included even minimal mineralogical or geochemical analyses, such as basic thin section petrography. The science involved in ASR projects in Florida was largely restricted to the requirements of the permitting agency (FDEP). Anything beyond meeting permitting requirements was often viewed as wasting money by creating a “science project”.

The tide is starting to turn and more scientifically rigorous approaches are being taken in some ASR projects in Florida. For example, continuous wireline cores of the storage zone and confining zone strata were obtained for two ASR systems in Seminole County Florida, that were funded by the St. Johns River Water Management District. The Florida Geological Survey was contracted to perform bench-scale leachability testing on core samples obtained from the two sites as a first step in a developing tool to predict metals leaching in ASR systems. Once the leaching of metals in ASR systems can be satisfactorily simulated in the laboratory, a tool would exist to evaluate various pre-treatment options to minimize or prevent arsenic leaching in ASR systems.

In systems where arsenic leaching has occurred, concentrations have decreased over time. The amount of arsenic present in aquifer strata in a labile, leachable form thus appears to be limited. There are number of potential strategies to mitigate the arsenic leaching problem including pre-treatment of injected water, post-treatment of recovered water, and changing the regulatory compliance point to peripheral storage zone monitor wells (Pyne, 2005). However the uncertainty over whether or not arsenic exceeding the 10 µg/L MCL will occur in a new ASR systems and over how the Florida Department of Environmental Protection (FDEP) and U.S. Environmental Protection Agency (USEPA) will ultimately address the arsenic leaching issue is currently slowing implementation of ASR in Florida.

## **MICROORGANISMS AND EMERGING POLLUTANTS**

The overall objective of the USEPA and FDEP UIC programs is to prevent endangerment of Underground Sources of Drinking Water (USDWs). Endangerment is considered an activity that results in the presence of any contaminant, such that it results in non-compliance with any national or state primary drinking water regulation. Class V injection wells, including ASR wells, are required to meet applicable groundwater standards, which in Florida are the primary and secondary drinking water standards, or the natural background concentration of a parameter if the concentration is greater than the promulgated maximum contaminant level (MCL). Rule 62-520.400 of the Florida Administrative Code requires that all groundwater shall at all places and at all times shall be free of contaminants, which among a number of criteria, are harmful to plant or animal life, pose a serious danger to public health, safety of welfare, or create or constitute a nuisance. A variety of pathogenic microorganism and emerging pollutants may potentially be present in wastewater that could present a danger to public safety and the environment, but for which there is currently no MCL.

From a technical perspective, pathogenic surface water and wastewater microorganisms do not appear to be a significant concern for ASR systems in Florida because they undergo rapid inactivation in groundwater systems. Microbial

inactivation rates, which are usually expressed in terms of  $\log_{10}$  declines per day, vary between organisms and environment. A  $0.1 \log_{10} \text{ day}^{-1}$  rate means a 1 log (90%) decline in 10 days. A review of published studies of microbial survival in groundwater gave median viral and bacterial inactivation rates of 0.02 to  $0.2 \log_{10} \text{ day}^{-1}$  (John and Rose 2005). In bench-scale experimental studies that simulated storage of surface water in ASR systems in Florida, protozoans appear to be the most robust organisms. *Giardia lamblia* had the slowest first order inactivation rates of 0.0042 to  $0.081 \log_{10} \text{ day}^{-1}$  (John et al. 2004). Decay rates of microorganisms associated with wastewater are greatest in the presence of indigenous groundwater microorganisms (Gordon et al. 2002; Toze and Hanna 2002). Increased pressure at depth may also increase decay rates. Although many factors affect the survivability of microorganisms in groundwater environments, the available data suggest that pathogenic microorganisms in injected surface water and wastewater will not be detectable after 2-3 years in a Florida groundwater environment.

Emerging pollutants, which include endocrine disrupting compounds (EDCs) and pharmaceuticals and personal care products (PPCPs), are potentially more problematic because there are differences in attenuation rates between compounds. Some compounds rapidly biodegrade or adsorb onto aquifer rock and organic matter, while others are much more resistant to biodegradation in soil and groundwater environments (Khan and Rorije 2002; Drewes et al. 2003; Heberer et al. 2004; Snyder et al. 2004; Ying et al. 2004). For many compounds, few data are available on attenuation rates in general, or in specific geochemical environments in groundwater. Much less is known about the actual risks a given compound poses at various concentrations when present in water stored underground. The major human and environmental health concern with EDCs and PPCPs is not the acute effects of a one time exposure from wastewater, but rather serious impacts from long-term chronic exposure. Risk assessments of underground injection assume that withdrawals would stop, and thus human exposure quickly terminated, if wastewater were detected in an ASR storage zone (Bloetscher et al. 2003, 2005).

The regulatory approach taken in Florida with respect to public health issues specifically associated with reclaimed water ASR has been to implement an additional layer of requirements (beyond the Class V injection well requirement to meet groundwater standards) based whether or not indirect potable reuse might occur. Chapter 62-610 of the Florida Administrative Code (Reuse of Reclaimed Water and Land Application) makes a major distinction in the treatment and disinfection requirements based on the salinity of the receiving water (i.e., storage zone aquifer). For systems injecting into groundwater with a TDS concentration of 3,000 mg/L or greater, only the principal treatment and disinfection requirements must be met, which at a minimum, include secondary treatment, high-level disinfection and filtration for total suspended solids control. The additional treatment requirements are minor because there is a very low probability that groundwater with a TDS

concentration of 3,000 mg/L or greater would be consumed without advanced treatment, such as reverse-osmosis desalination.

For systems with receiving groundwater with TDS concentrations of less than 3,000 mg/L, the much more rigorous full treatment and disinfection requirements must be met, which essentially assume that indirect potable reuse will occur. Principal treatment and disinfection requirements may apply for a storage zone aquifer containing between 1,000 and 3,000 mg/L TDS, if it can be demonstrated that the aquifer is not reasonably expected to be used for public water supply. Among the many requirements for full treatment and disinfection are that the reclaimed water meet drinking water standards (not groundwater standards) and that a rigorous 12 month pilot testing program be implemented to provide an affirmative demonstration that the treatment and disinfection processes proposed for inclusion in the wastewater treatment facility are capable of meeting the full treatment and disinfection requirements and that the reclaimed water will be of sufficient quality to protect public health and environmental quality. The reclaimed water is required to be demonstrated to be of a quality the same or better than other sources of drinking water currently used in the area. In practice, having to meet the full treatment and disinfection requirements would effectively make a reclaimed water ASR system economically unviable.

## **DISINFECTION BYPRODUCTS**

The MCLs for disinfectant byproducts (DBPs) of 0.080 mg/L for total trihalomethanes (THM) and 0.060 mg/L for haloacetic acids are primary drinking water standards and must therefore be met at all times for reclaimed water ASR systems. Meeting the DBP MCLs is a problem for some Florida utilities that use chlorination for disinfection. DBPs are also a problem for surface water ASR in Florida because surface waters often have relatively high concentrations of dissolved organic compounds. The expense of changing disinfection systems at a wastewater treatment plant so as to meet the DBP MCLs for some utilities would make ASR unfeasible. One option is to install a dedicated disinfection system to treat only the wastewater flow to the ASR well. For example, a dedicated ultraviolet disinfection system could be installed near the ASR well head. THM concentrations tend to naturally decrease under the reducing conditions found in most aquifers used as ASR storage zones.

## **ECONOMIC ISSUES**

Reclaimed water has a low economic value compared to potable water. The use of reclaimed water for irrigation in Florida by potential customers is still to a large degree optional. For wastewater utilities, the costs to construct and operate reclaimed water ASR systems often cannot be fully recouped from reuse customers. Furthermore, a wastewater utility is not required to provide reuse water to meet the

needs of all potential customers. Reclaimed water ASR systems are thus often voluntary, unprofitable ventures for utilities. Utilities have the option of just providing or selling their normal reuse flows, as available and disposing of any excess.

Although expanded reuse systems may decrease the use of fresh groundwater resources, the benefits may not go directly to the utility because there is not a direct regulatory linkage between the implementation of reclaimed water ASR and an increase in groundwater allocation. Nevertheless, there is a societal benefit in optimizing the management of water resources in the state. Everyone in the state benefits from a sustainable long-term supply of freshwater. Funding from the Florida water management districts therefore has been and will continue to be critical for the implementation of reclaimed water ASR.

## CONCLUSIONS

Rather than being a waste product requiring disposal, reclaimed water is a valuable resource whose reuse is an important component of integrated water resource management. ASR provide a means for the reuse of nearly all of the reclaimed water flow of a utility by allowing for the storage of excess flows for later use in high demand periods. ASR systems inherently have a high degree of hydrogeology complexity, much more so than other water and wastewater projects. There is a narrow envelope of hydrogeologic conditions that must be met for a system to operate at a high recovery efficiency, avoid adverse water quality, and to protect the public health and the environment. ASR systems have failed in Florida because hydrogeologically unfavorable conditions were not recognized or understood early in the project development. The Florida ASR experience illustrates the importance of performing a sophisticated aquifer characterization before making the commitment to constructing a full-scale ASR system at a site. An exploratory well program should collect sufficient data to characterize aquifer heterogeneity and evaluate the potential for fluid-rock and mixing interactions. Incorporation of site-specific data on aquifer heterogeneity obtained from a test well program into a solute-transport program that includes density-driven flow and fluid mixing (e.g., SEAWAT) can allow for the better prediction of the likelihood of success of ASR systems and the optimization of ASR system performance.

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# CALIFORNIA'S QUANDARY: MANAGED AQUIFER RECHARGE FOR INCREASED WATER SUPPLY RELIABILITY UNDER A VERY COMPLEX REGULATORY ENVIRONMENT – WILL IT WORK?

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## ABSTRACT

California is the tenth largest economy in the world with a 2005 gross product of \$1.54 trillion, produces half the fruits, nuts and vegetables consumed in the US, one of the three largest states and the most populous state in the US, and has one of the most complex regulatory systems and water infrastructure systems in the world. Managed aquifer recharge has been implemented successfully in California for decades. However, with increased impetus to exercise more basins with managed aquifer recharge and aquifer storage and recovery to provide added water supply reliability has come increasing scrutiny and regulation. Many programs under pilot stage or construction stage are facing high stakes and risk in a complex and uncertain regulatory environment. Issues such as the presence of disinfection-by-products in source water, level of degradation of DBPs in receiving water, the state antidegradation policy, basin plan water quality objectives, and state agency incentives to increase water supply reliability juxtaposed state agency regulation of water quality. The paper presents an overview of the complex California regulatory framework, the technical and political issues, past and current proposed legislative solutions, and the current status of managed aquifer recharge in California through narrative description and case studies.

## KEYWORDS

Groundwater management, regulatory, policy, legislation

## INTRODUCTION

California has a long history of managed aquifer recharge, starting as early as the beginning of the last century. California's local managed aquifer recharge projects have been supplemented by imported water provided by the statewide complex of federal, state, and local reservoirs, aqueducts and canals, including the Central Valley Project and the State Water Project. Fueled by a large amount of water bond funding over the last seven years, today a large number of different projects are being developed and implemented in a variety hydrogeologic environments. Technological advances in chemical analyses have made it possible to detect much lower concentrations of potential constituents of concern which may be contained in recharge source waters. Coupled with the complex California regulatory framework

which imparts different standards and requirements on projects in different regions of the state, it currently appears to be a fairly uncertain environment for managed aquifer recharge project proponents. The state faces a number of serious challenges ahead, including a recent court order curtailing Delta pumping and deliveries of surface water to the south state due to environmental issues, concerns about potential failure of the Delta levees due to earthquakes flooding, and future consequences of climate change and rising sea level. All these challenging issues make it very clear that groundwater storage and managed aquifer recharge will have an expanding role of to meet future water supply needs in the state.

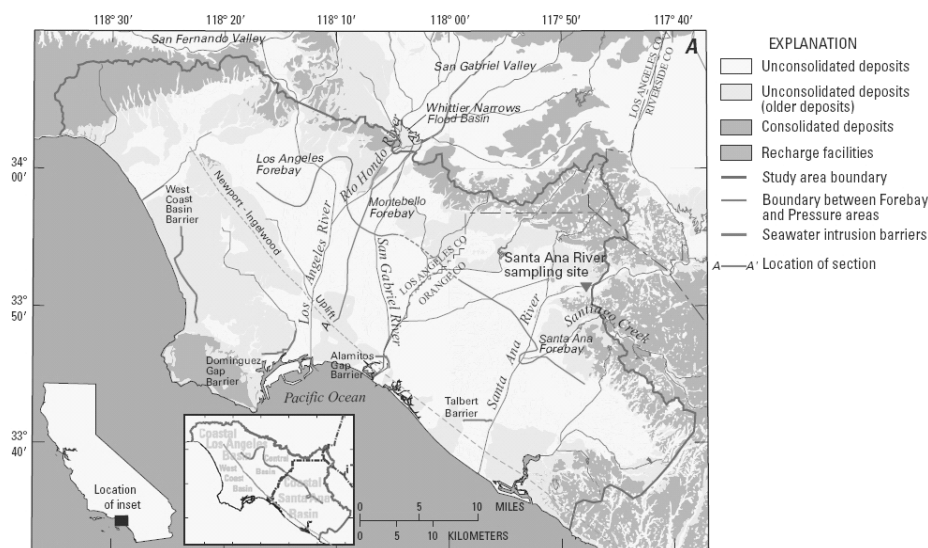
## **HISTORIC MANAGED AQUIFER RECHARGE IN CALIFORNIA**

### **Los Angeles Area**

Since the early part of the last century, managed aquifer recharge has been practiced in the Los Angeles area, California, in order to increase the amount of water recharged (often focused in small areas) to meet the water demand from groundwater pumpage in public supply wells to provide for a growing population and agriculture. People have been drinking this "recharged" groundwater for over 40 years. The Los Angeles physiographic basin is subdivided into two basins determined by the sources of recharge water—the Coastal Los Angeles Basin and the Coastal Santa Ana Basin. Each of these coastal basins has distinct radial flow paths originating from the managed aquifer recharge facilities, although the hydrogeologic setting is similar between the Coastal Los Angeles Basin and the Coastal Santa Ana Basin. These two exercised basins, which are perhaps two of the most studied basins in the state if not the U.S., can be further subdivided into two areas—forebay and pressure areas—based on hydrogeologic characteristics.

With concentrated urbanization and lining of the Los Angeles River, much of the natural groundwater recharge in the Coastal Los Angeles Basin was eliminated in the first half of the 20<sup>th</sup> century, and increased pumping caused severe overdraft with associated chronic declining groundwater levels and seawater intrusion at the coast. Adjudications in the 1960's in the Coastal Los Angeles Basin resulted in the courts setting a ceiling on pumping amounts, but at quantities above a sustainable level for the basin. An intensive program of monitoring and managed aquifer recharge necessary to offset historic overdraft and pumping began during the early 1960s with formation the Water Replenishment District of Southern California (WRD), a unique institution formed by a majority vote of the people of California to recharge aquifers and manage groundwater in the Coastal Los Angeles basin. WRD continues today with the majority of the replenishment of the aquifers occurring through managed aquifer recharge facilities, supplying 43 cities and with a population of more than 4 million. Groundwater flows primarily laterally and radially from the managed aquifer recharge facilities in the Central Basin, which consist of spreading basins, and

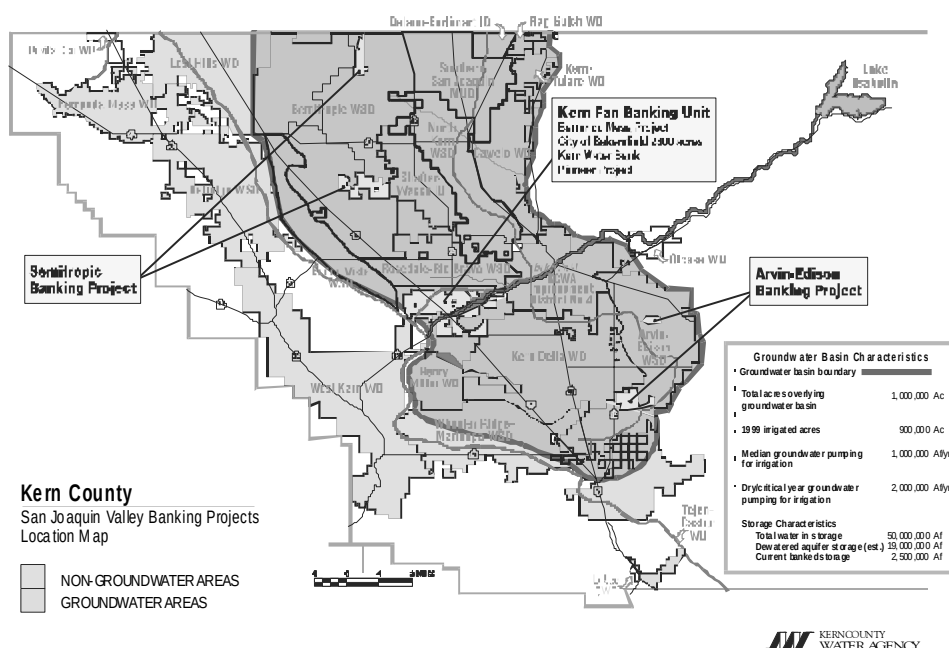
eastward from the Pacific Ocean in the West Coast Basin, where recharge facilities consist of several series of injection wells which form the seawater intrusion barriers. The source water for recharge in the forebay area is a mixture of stormwater runoff, imported water, and treated recycled water, and the source water in the injection wells is a blend of imported water and treated recycled water. In an average year, WRD purchases approximately 143 cubic meters for recharge operations including 73 million cubic meters imported water from the Colorado River and State Water project, and 70 million cubic meters of recycled water.



With a largely agriculture economy, including substantial orange groves siphoning more than 247 million cubic meters of the groundwater out of the Coastal Santa Ana basin annually, in the early of the 20<sup>th</sup> century groundwater levels dropped more than 20 meters. The Orange County Water District (OCWD) was formed in 1933 by a special act of the California Legislature, and was empowered to protect the water supply and the rights of those who depended upon it, which at that time was 60,000 people with 86 percent agricultural water use. This type of institution is unique in California, with significant powers to manage, regulate, control, purchase, acquire, transport, exchange water and groundwater within the basin. Over time, dams were constructed on the Santa Ana River, which limited the flow into the basin, and a growing practice of importation of large quantities of water to recharge the basin began to occur. Groundwater extraction continued to outpace recharge and by the mid 1950's seawater intrusion was evidenced as far as 5.6 kilometers inland. Today's OCWD covers well over 809 square kilometers, serves 20 cities and water agencies and a population in excess of 2 million. In the forebay area, managed aquifer recharge consisting of spreading basins along the Santa Ana River facilities receive a combination of treated wastewater, Santa Ana River water, and imported water, and

these recharge facilities provide the majority of recharge to the groundwater basin, recharging approximately 308 to 339 million cubic meters per year. Seawater intrusion is mitigated by pumping a blend of recycled water and deep well water into a series of injection wells near the coast. The recycled water treatment train includes chemical clarification, re-carbonation, multimedia filtration, granular activated carbon, reverse osmosis, chlorination and blending.

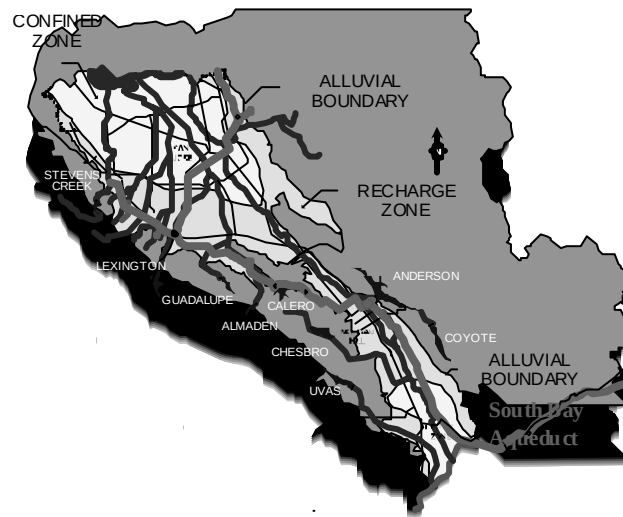
## Kern County



Intensive agricultural beginning in the latter half of the 19th century and continuing throughout the 20th century with related ground-water development resulted in ground-water-level declines of more than 60 meters and land subsidence of as much as 9 feet in the early- to mid-1900s in the Kern County groundwater basin. The Kern County groundwater basin is situated in the southern end of the Central Valley of California, a 800 kilometer long valley which provides have the fruits, nuts of vegetables in the US, or about one-quarter of the nation's table food on only 1 percent of the country's farmland. Water banking was initiated in the subbasin in 1978, and seven projects with facilities including over 40 square kilometers of recharge basins and more than 120 recovery wells, now contain over 4.9 billion cubic meters of banked water in dewatered aquifer storage space. Approximately two thirds of this storage is in the Kern River Fan area west of Bakersfield; the remainder is in the Arvin-Edison Water Storage District in the southeastern subbasin or in the Semitropic Water Storage District in the northwestern subbasin.

## Santa Clara Valley Water District

Much like Kern County, intensive agricultural and urban development, beginning in the latter half of the 19th century post-Gold Rush era, and continuing throughout the 20th century, and related ground-water development resulted in ground-water-level declines of more than 60 meters, land subsidence of as much as 3.7 meters and related flooding between the early 1900s and the mid-1960s. In the late 1920's, the Santa Clara Valley Water conservation District was formed and thus began a long stretch of facilities construction for surface storage and managed aquifer recharge via percolation in small ponds. Since the 1960s, Santa Clara Valley Water District has imported surface water to meet growing demands and reduce dependence on ground-water supplies. Currently, the district operates and maintains 18 major recharge systems, which consist of both in-stream and off-stream facilities. Local reservoir water and imported water are released in over 30 local creeks for artificial in-stream recharge to the groundwater basin. In addition, the district releases locally conserved and imported water to 71 off-stream facilities (percolation ponds) which range in size from less than 4,000 square meters to more than 80,000 square meters. Through these streams and percolation ponds, the district recharges the groundwater basin with about 193 million cubic meters of water each year. (SCVWD 2005).



## STATE LEGISLATION AND GROUNDWATER FUNDING PROGRAMS

Legislation and Bond funding have created many opportunities to improve groundwater management, construct new groundwater projects, and increase the role of groundwater storage in California.

### **AB3030 and SB1938**

In 1992, part of the State Water Code was repealed and a section was added, State Water Code Section 10750 et seq. (Assembly Bill 3030), to allow certain local public agencies that have authority to deal with water to form a groundwater management plan. The local agency has to lie within a groundwater basin as defined in California Department of Water Resources (DWR) Bulletin 118 or subsequent editions of that bulletin. Such a public agency, subject to notice, hearing and protest requirements, may adopt and implement a groundwater management plan and may exercise the powers of a water replenishment district to raise revenue. Although a groundwater management plan has no legally required components, twelve components were recommended in AB3030 including monitoring, groundwater replenishment, facilitating conjunctive use, control of contamination and saline water intrusion, mitigation of overdraft, wellhead protection, and land use planning coordination. To implement and enforce the groundwater management plan, the local agency is required to adopt rules and regulations.

The DWR was required by legislation in 1999 to develop components that should be included in any groundwater management plan. In 2002, with the passage of Senate Bill 1938, the State Water Code was amended to require that, if the agency formulating the plan intends to seek state funding for groundwater projects, the following components must be included in the plan: (1) documentation regarding public participation in plan development, (2) plan to involve other local agencies in the plan areas, (3) a map of the plan area, (4) management objectives for the groundwater basin, (5) include components for monitoring and management of groundwater levels, groundwater quality, groundwater related subsidence, and effects of surface water-groundwater interaction, and (6) adopt monitoring protocols. The result of SB1938 was to tie eligible for state funds for groundwater projects to developing a groundwater management plan that meets certain criteria.

### **Proposition 13**

In March 2007, Proposition 13, the Safe Drinking Water, Clean Water, Watershed Protection and Flood Protection Act, was passed by voters authorizing a total of \$1.97 billion in bonds and included \$200 million for groundwater storage projects and \$30 million for groundwater recharge projects. DWR was required to develop a solicitation and selection process for the award of funding to public agencies on a competitive application basis. DWR gave preference to projects located in groundwater basins in overdraft, projects that met critical supply needs, and projects subject to a formal groundwater management plan or program. On 190 applications received during three application cycles, 62 projects for a total of \$205 million was funded on over \$1 billion in total costs of projects. The projects included facilitation, planning, studies and construction, new infrastructure conveyance and wells, recharge basins, treatment plant upgrades, all related to increasing



groundwater storage in the state. Base on the estimates provided by project proponents and DWR, Proposition 13 has funded projects that should provide an average annual additional yield of 300,000 acre feet.

### **AB303 Local Groundwater Assistance Programs**

In 2000, with the passage of the AB303, the Local Groundwater Management Act was enacted to provide grants to public agencies to conduct groundwater studies or to carry out groundwater monitoring and management activities. DWR was required to form a technical advisory panel to develop a solicitation and selection process for the award of funding to public agencies on a competitive application basis. Over the course of five years between 2000 and 2005, for 330 applications received, 128 projects were awarded a total of nearly \$28 million in grant funding. The program was not funded in 2006 or 2007, but \$6.4 million in funding is in place for 2008.

### **Prop 50 and 84**

Proposition 50, *Water Quality, Supply and Safe Drinking Water Projects Coastal Wetlands Purchase and Protection*, was passed in the fall of 2002 by voters. The \$3.4 billion bond includes \$500 million for integrated water management, water quality, conservation, wetlands and groundwater recharge projects. Many of the integrated water management projects incorporate projects or portions of projects related to managed aquifer recharge. Proposition 84, *Water Quality, Safety and Supply, Flood Control, Natural Resource Protection, Park Improvements*, was passed in the fall of 2006 by voters. The \$5.4 billion bond includes \$1 billion for integrated regional water management and implementation legislation carves out \$200 million of the \$1 billion for groundwater storage.

### **2008 Water Bond**

There has been a flurry of activity at the Capitol this year. Currently, there are two different proposals floating, one by the Republicans and one by the Democrats, one with \$4 billion for two surface water reservoirs and one without funding for surface water reservoirs; both have \$500 million for groundwater storage. The Governor has just called for a special legislative session focused on water to help resolve the differences and hopefully come to consensus on a water bond that will provide a Delta fix and comprehensive improvements to the state water system

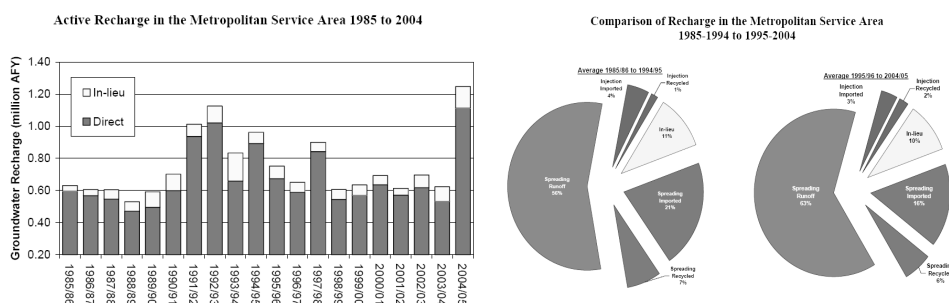
## **METROPOLITAN WATER DISTRICT OF SOUTHERN CALIFORNIA PARTNERSHIPS AND PLANS**

The Metropolitan Water District of Southern California (Metropolitan) is a consortium of 26 cities and water districts that provides drinking water to nearly 18

million people in parts of Los Angeles, Orange, San Diego, Riverside, San Bernardino and Ventura counties. Metropolitan currently delivers an average of 6.4 million cubic meters of water per day to a 13,500 square kilometer area. Metropolitan has historically supported groundwater storage programs within its service area. Extensive groundwater facility improvements have been made over the past 20 years, which have supported an 8 percent increase in groundwater production. Improvements include construction of groundwater treatment facilities, a portion of which are funded under Metropolitan's Local Resources Program.

At this time, about 740 million cubic meters of groundwater storage is currently allocated for storage programs within the service area, including dry-year conjunctive use, supplemental storage and cyclic storage programs. As of June 2006, about 327 million cubic meters was in storage in these programs. The agreement for Metropolitan's initial contractual storage program was executed in 1995, and since then, an additional nine programs have been developed utilizing State Proposition 13 funds and Metropolitan capital funds. Facilities to implement these programs are currently under design and construction, and are anticipated to be fully operational prior to 2010. A tenth program is currently in preliminary design and environmental review.

Managed aquifer recharge is an important component of groundwater management within the Metropolitan service area. For the last 20 years, an average of approximately 831million cubic meters or about 90 percent of the total managed aquifer recharge was directly recharged to the groundwater basins by injection or spreading. For this time period, about 95 million cubic meters was recharged to the groundwater basins via in-lieu methods. Methods of managed aquifer recharge have generally remained unchanged over the past 20 years as the relative proportion of each method is essentially the same over the past 20 years. Likewise, the total recharge remained nearly constant with a decrease of about 2 percent between the two timeframes. With respect to the water supply portfolio for recharge of the groundwater basins, the proportion of runoff used increased 7 percent while the use of imported water supplies decreased 5 percent (Metropolitan 2007).



Reference: Metropolitan 2007. Note 1 million acre-feet = 1.2 billion cubic meters

## **CALIFORNIA GROUNDWATER LEGAL AND REGULATORY FRAMEWORK**

California is still the Wild West when it comes to groundwater: there is no groundwater regulation or law. A landowner may install a well and start pumping groundwater from the well, without the need for a permit for extraction. All waters belong to the people of the state, all overlying landowners have a correlative right to groundwater, and all use of water resources is subject to the constitutional requirement of reasonable and beneficial use. Only in a few cases, such as adjudication, or where a local agency has zoned an area, or if a subterranean stream is considered to be present, is groundwater regulated or access to groundwater controlled in the state. In an adjudicated basin, litigation has occurred and a judge has handed down a decision on how much pumpers can extract over what period of time, and has assigned a watermaster to monitor and maintain controls. Some cities have zoned areas prohibiting the installation of wells and controlling access to groundwater in return for water supply provided by that local agency. (Bachman, et.al. 2005).

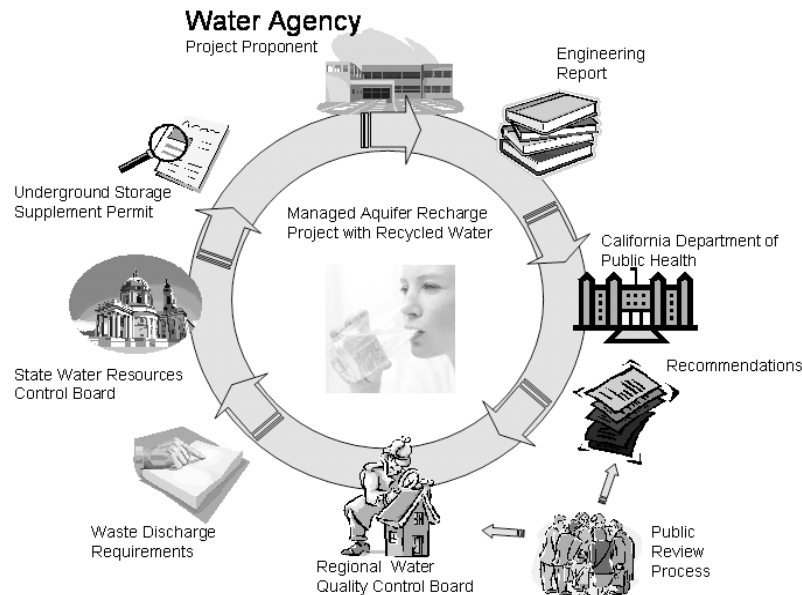
For purposes of jurisdiction, groundwater in California is generally either classified as “percolating”, or “a subterranean stream flowing through known and defined channels.” Percolating groundwater in California is considered to include a vast mass of subsurface water in a basin, always moving toward some stream or outlet, and is not subject to state jurisdiction or permitting. A subterranean stream is most frequently characterized as moving through permeable material, typically alluvial in character, which underlies or comprises the bed of a stream and is essential to the existence of the stream. Subterranean streams are subject to SWRCB jurisdiction and permitting, and are often a disputed issue between competing parties to groundwater resources and the state. (Bachman, et.al. 2005).

### **State Agencies and Structure**

The California Water Code specifically states, “The Legislature further finds and declares that the health, safety and welfare of the people of the state requires that there be a statewide program for the control of the quality of all waters of the state; that the state must be prepared to exercise its full power and jurisdiction to protect the quality of the waters in the state from degradation originating inside or outside the boundaries of the state; ”. The California Water Code designates the State Water Resources Control Board and the nine regional boards as the principal state agencies with primary responsibility for the coordination and control of water quality. (Bachman, et.al. 2005).

A variety of agencies have some responsibility over managed aquifer recharge projects, including the California State Water Resources Control Board, the Regional Water Quality Control Boards, and the Department of Public Health. The steps

needed to permit a managed aquifer recharge facility with recycled water are outlined in the figure below. The State Water Resources Control Board has a joint authority of water allocation and water quality protection to provide comprehensive protection for California's waters.



### State Jurisdiction for Managed Aquifer Recharge with Recycled Water

California legislative policy mandates that whenever possible, wastewater should be reclaimed (or "recycled") and used for beneficial purposes, including recharging aquifers. When recycled water is used for aquifer recharge, special regulations apply to protect aquifers from potential impacts. The regulations are promulgated by the California Department of Public Health (DPH), which have been in draft form for more than a decade, although these regulations are being used by the DPH to evaluate recharge projects. The DPH requires an approved plan which addresses the sampling and analysis of nitrogen, regulated contaminants and physical characteristics, diluent water, recycled water, total organic carbon, and emerging contaminants. DPH also requires operations optimization, monitoring between recharge area and downgradient supply points, annual and 5 year reporting. Required separation between downgradient supply wells and recharge points are:

- 1) Surface spreading - minimum 150 meters AND six months travel time.
- 2) Well injection - minimum 610 meters AND twelve months travel time.

Projects involving groundwater recharge with recycled water require a public review and approval process.

### **State Jurisdiction for Groundwater Quality Protection**

The Porter-Cologne Water Quality Act of 1969 sets the goals of regulating activities to achieve the highest reasonable water quality in California. Reasonableness is determined on the basis of demands on waters and all the values involved. Porter-Cologne establishes that the State Water Resources Control Board (SWRCB) has ultimate authority over state water quality policy and also establishes nine regional water quality boards covering nine separate hydrologic regions in the state. The nine regional boards operate autonomously, each with nine board members, and engage separately within the state in tasks related to water quality. Regional boards prepare water quality plans, commonly referred to as "basin plans", for all areas within their respective regions. The basin plans contain three components:

- 1) Beneficial uses to be protected,
- 2) Water quality objectives, and
- 3) Implementation plan to meet water quality objectives.

Beneficial uses that may be protected against water quality degradation include, but are not limited to, domestic, municipal, agricultural, and industrial supply, power generation, recreation, aesthetic enjoyment, navigation, and preservation and enhancement of fish, wildlife, and other aquatic resources. Water quality objectives are established by each regional board and set limits or levels of water quality constituents and characteristics that provide reasonable protection of beneficial uses or the prevention of nuisance, and it is at each board's discretion to establish reasonable standards consistent with the overall statewide interest. Generally, boards have established objectives that are a combination of numeric values comparable to secondary and maximum contaminant levels, and narrative, toxicity based levels, for example, for fish which are being applied to groundwater (e.g., individual trihalomethane compounds at low part per billion levels versus total trihalomethanes for drinking water). Any person discharging wastes or proposing to discharge wastes that could affect the quality of the waters of the State is required to file a Report of Waste Discharge with the applicable regional board.

Part of the RWQCB process to review the report of waste discharge and develop an order of waste discharge requirements involves a review of the project consistency with the State antidegradation policy. The state adopted Resolution No. 68-16 in 1968, a policy to preclude water quality degradation in the state unless conditions are met, known as the antidegradation policy: Statement of Policy with Respect to Maintaining High Quality of Waters in California: Whenever the existing quality of water is better than the quality established in policies as of the date on

which such policies became effective, such existing high quality will be maintained until it has been demonstrated to the state that any change will be consistent with the maximum benefit to the people of the state, will not unreasonably affect present and anticipated beneficial use of such water and will not result in water quality less than that prescribed in the policies. Under the antidegradation policy, the lowering of water quality can be allowed, provided that a comprehensive analysis demonstrates that it (1) will not unreasonably affect present and anticipated beneficial uses; (2) will not result in water quality less than that prescribed in policies (Id.); and, (3) is consistent with maximum benefit to the people of the state. Implementation of best practicable treatment and control of all discharges is a prerequisite to consideration of allowing lower water quality where the water is considered to be of high quality. (SWRCB 68-16.)

### **Underground Storage Supplement**

As indicated previously, while groundwater is not regulated in the state, surface water is regulated and a water rights permit is generally required. The underground storage of surface water is not part of most surface water right permits. If so, an underground storage supplement is required to be filed with SWRCB. The USS must include points of diversion from the stream channel, description of physical diversion and conveyance, maps, and be a beneficial use of the water.

### **Jurisdiction for Injecting Water**

Finally, the US EPA Underground Injection Control (UIC) Program, which was established under the Safe Drinking Water Act to protect underground sources of drinking water from unsafe injection practices. The UIC Program works with state and local governments to oversee underground injection of waste in order to prevent contamination of drinking water resources. Under the UIC, all ASR wells should provide inventory information to US EPA.

## **CURRENT STATUS OF REGULATORY FOCUS, PROJECT EXAMPLES, AND ISSUES**

In California, many water industry managed aquifer recharge project proponents have a number of concerns related to regulatory inconsistency in application of policy, and the uncertainty in the development of costly facilities without adequate assurances for regulatory approval. some of these issues and examples are provided below.

### **SWRCB Staff and Management Focus**

Current SWRCB staff and management regulatory concerns and focus:

- Technical challenges including recharge water variability, recovered water beneficial uses, and adequate information on site subsurface conditions,
- Water quality issues including disinfection by-products, mobilization of salts and inorganics,
- Water rights considerations including surface water diversion permitting, and groundwater space ownership and rights, and
- Regulatory and policy issues include regulating based on recharge water type, type of project and point of compliance.

The SWRCB is considering whether there should be a statewide policy for groundwater storage, consistency in dealing with issues, and coordination with recycled water policy under development.

### **California RWQCB Focus**

Individual California RWQCBs have slightly different focuses on groundwater storage permitting approaches in their respective regions. For example, in the Central Valley, the RWQCB is currently concerned with ASR projects which involve source and receiving waters of different qualities, especially in cases where they feel MCLs may not be stringent enough. The ASR applicant is required to characterize groundwater and injection water quality, survey groundwater users, delineate the “bubble,” (zone of injected water around the well), demonstrate control over the bubble, meet water quality objectives at the edge of the bubble, monitor to verify compliance, and develop and implement a contingency plan if not meeting compliance conditions. The RWQCB then adopts a conditional waiver of waste discharge requirements (WDR) with a monitoring and reporting program.

In the Santa Ana area, RWQCB is currently concerned with the regulation of salt from water imported into basins, especially those closed basins with an existing very high adverse salt balance, and ASR projects. For salt management, the Santa Ana Watershed was divided into management zones, and volume-weighted zone analyses of TDS and nitrate concentrations were completed for the entire watershed, taking into account local hydrogeologic conditions. This approach was used to look at historical and current salinity values to determine if basin management objectives were being met. A similar approach is being considered for the regulation of recharge water from the State Water Project, Colorado River, and inter-basin transfers. State contractors have proposed doing this under a cooperative agreement with the RWQCB (versus a waste discharge requirement) where recharging agencies agree to model effects of groundwater basin water quality from all proposed recharge activities, and prevent or mitigate violations of objectives as appropriate. This proposal is currently under consideration by water agencies’ boards and counsel. For ASR projects in the Santa Ana Region, the regulatory approach is dictated by quality of recharge water, basin objective, and assimilative capacity. If recharge

water quality is better than the basin objectives and ambient quality, then a waiver is very likely. If recharge water quality is better than the objectives but poorer than ambient quality, then the level of assurance of capture and extraction is important; if there is a high assurance of re-capture, then a waiver is likely; if not, WDR is likely. If recharge water quality is poorer than the basin objectives, then assurance of capture is critical, regulation is likely, and may include approaches using: assimilative capacity of the basin; cooperative agreement with anti-degradation analysis; or individual WDRs.

### **San Francisco Bay RWQCB ASR Project Example**

The East Bay Municipal Utility District (EBMUD) has proposed conducting phase 1 of a project involving the injection and storage of treated drinking water from EBMUD's distribution system into the South East Bay Plain Groundwater Basin during wet years for later recovery during dry years as source of supplemental drinking water supply. Since the proposed project is a supplemental drought year project, the sequence and duration of injection, storage and recovery will be dictated by a combination of climatic conditions, drought frequency and duration, and demand within the EBMUD service area. Phase 1 is designed to use only one well, the Bayside Well, for injection and recovery with a limited annual capacity of 3,785 cubic meters per day. The Bayside well is a 45 centimeter diameter well screened in the deep aquifer at depths between 158 and 198 meters below ground surface. The source water for the project is from the Mokelumne River which flows from the Sierra Nevada Mountains into reservoirs in the Sierra Nevada foothills, and is of better quality than the aquifer receiving water for a number of parameters including total organic carbon, total dissolved solids, chloride, manganese, radon, hardness and sulfate. Source water is treated with a tertiary treatment process prior to injection, with the addition of chloramine for disinfection. Raw untreated source water was not considered for injection since it would require a separate piping system and would potentially create a high level of biological fouling of the injection well. Initial pilot studies were conducted between 1997 and 1999, with three complete injection, storage and recovery cycles with a total of 375,000 cubic meters injected and 439,000 cubic meters recovered. Based on the pilot studies, the project proponent concluded that the existing well will provide long term extraction and recovery rates of 4164 cubic meters per day and 6,057 cubic meters per day, respectively, and that recovered water will meet all drinking water standards with treatment. (SFBRWQCB 2007).

The San Francisco Bay Area RWQCB issued a waste discharge requirements order to EBMUD in May 2007, which includes self monitoring and reporting requirements, and a contingency plan. Monitoring consists of water level measurements of 14 wells with frequencies from 30 minutes to an hour, and annual water quality sampling of eight wells for residual chlorine, total dissolved solids, ammonia, nitrate, chloride, manganese, iron, total trihalomethanes, haloacetic acids, general minerals and regulated constituents for drinking water. Trihalomethanes are identified as a key



consideration and concern in the RWQCB order, and the applicable basin plan water quality objective for trihalomethanes is the maximum contaminant level (MCL) of 80 micrograms per liter (ug/L). The RWQCB acknowledges that the storage of treated drinking water may lead to the lowering of groundwater quality with respect to trihalomethanes. However, the RWQCB also acknowledges that the trihalomethanes are below the applicable MCLs for drinking water, and are also expected to undergo biological attenuation with their concentrations decreasing with distance from the injection point. There is further acknowledgement that a number of water quality parameters would be improved and that the maximum benefit provided by this project to the people of the State would significantly outweigh any minor degradation that may occur. (SFBRWQCB 2007).

### **Central Valley RWQCB ASR Project Example**

The City of Roseville (Roseville) has proposed conducting phase II of a demonstration project involving the injection and storage of treated drinking water from Roseville's distribution system into the Mehrten formation aquifer during wet years for later recovery during dry years as source of supplemental drinking water supply. Since the proposed project is a supplemental drought year project, the sequence and duration of injection, storage and recovery will be dictated by a combination of climatic conditions, drought frequency and duration, and demand within the Roseville service area. Phase II is designed to use only one well, the Diamond Creek Well, for injection and recovery with a limited six month capacity of 1,094 acre feet of water. The Diamond Creek well is a 51 centimeter diameter well screened in a water bearing zone which consists primarily of gravelly sand and sandy gravel of the Mehrten formation at depths between 95 and 137 meters below ground surface. The source water for the project is from Folsom Lake which emanates from American River flows from the Sierra Nevada Mountains, and is of better quality than the aquifer receiving water for a number of parameters including total dissolved solids, chloride, magnesium, sodium, hardness and sulfate. Source water is treated with a tertiary treatment process prior to injection, with the addition of chlorine for disinfection. Raw untreated source water was not considered for injection since it would require a separate piping system and would potentially create a high level of biological fouling of the injection well. The 96-day Phase I demonstration project was completed in December 2003, with one complete injection and three recovery cycles totaling 29 days with a total 195,000 cubic meters injected and 541,000 cubic meters recovered. Based on the pilot studies, the project proponent concluded that the existing well will provide long term extraction and recovery rates and that recovered water will meet all drinking water standards with treatment. Roseville has implemented institutional controls in the city limits to prohibit any additional well installation which effectively eliminates possible access to the water stored in the aquifer. (CVRWQCB 2005).

The Central Valley RWQCB issued an order waiving waste discharge requirements to Roseville in August 2005, which includes mandatory monitoring and reporting requirements, and a contingency plan. Monitoring consists of water level measurements of 4 wells with frequencies from weekly, bi-weekly and monthly, and weekly, bi-weekly and monthly water quality sampling of the same 4 wells for residual chlorine, dissolved organic carbon total dissolved solids, ammonia, nitrate, chloride, fluoride, manganese, molybdenum, iron, trace metals, total trihalomethanes, haloacetic acids, general minerals and regulated constituents for drinking water. Injected water quality, including trihalomethanes are identified as a key consideration and concern in the CV RWQCB order, and the applicable basin plan water quality objective for trihalomethanes is the MCL of 80 ug/L, AND also includes the narrative toxicity values for the individual compounds including chloroform (1.1 ug/L), bromoform (4 ug/L), bromodichloromethane (0.27 ug/L), and dibromochloromethane (0.37 ug/L). The CV RWQCB has determined that the injection of drinking water for future extraction is not a discharge of waste that warrants issuance of waste discharge requirements due to the limited nature, extent and duration of the of the above-described discharge, the preclusion of other uses and users by elimination of access to the aquifer, and the fact that all injected water will be recovered. However, an injection of drinking water that exceeds MCLs within the aquifer storage zone or results in an exceedance of water quality objectives outside the aquifer storage zone may be subject to waste discharge requirements that require compliance with the basin plan and also possibly remediation of water quality impacts. The groundwater outside the limits of the aquifer storage zone (a) may have other users/uses now or in the future, and (b) must meet all applicable water quality objectives. Monitoring of the demonstration study is necessary to obtain technical data sufficient to assess potential long-term water quality impacts and the need for mitigation measures associated with more extensive testing or a full-scale project, including an analysis of whether Roseville must implement Best Practicable Treatment Control (BPTC) measures to reduce the concentrations of disinfection byproducts or other constituents of concern in the water injected into the groundwater to eliminate any exceedances of water quality objectives for individual trihalomethane compounds. Roseville must conduct sufficient monitoring during the demonstration study to assess whether a full-scale ASR project would comply with the Antidegradation Policy and demonstrate that dilution and/or degradation will reduce the concentrations of any constituents of concern to below applicable water quality objectives outside the bubble. (CVRWQCB 2005).

## CONCLUSIONS

California has a long and colorful history of successful managed aquifer recharge, and currently the water supply reliability of the state seems to be riding on managed aquifer recharge, along with other options. Certainly there will be future funding for additional groundwater storage projects in the state. The question remains as to

whether regulations will be constructive or obstructive, and how the uncertainty for project proponents will effect future water supply decisions. The legal and regulatory framework in California currently does not provide certainty in terms of being able to predict what will be required to successfully implement a managed aquifer recharge project in the state. Requirements appear to differ by regional board, and there is currently no statewide policy for these types of projects. Added that since groundwater is not regulated in the state, the process of managing groundwater storage projects tends to have a certain level of uncertainty associated with it, especially with regards to potential future battles over ownership of the storage capacity and the stored water. The SWRCB is currently working with the RWQCBs to develop a policy on groundwater storage, which also makes many project proponents uneasy. Whether or not a policy is developed remains to be seen, and until such time managed aquifer recharge projects in California will continue to be considered by the RWQCBs on a case-by-case basis, and will include some level uncertainty as to whether the project will be a success.

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## THE FEDERAL PERSPECTIVE ON AQUIFER STORAGE AND RECOVERY WELLS, REGION 5 AND 7

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### ABSTRACT

In the United States, there are over 175 aquifer storage and recovery (ASR) projects in the lower 48 states. This process falls under the regulatory purview of the United States Environmental Protection Agency's (USEPA) Underground Injection Control (UIC) Program.

The USEPA's Region 5 and 7 offices have developed conservative approaches to the issuance of ASR permits. In Regions 5, the permitting process will be a two permit process. The first permit will be a pilot permit, which will include at least three cycling tests. Once the cycles have been completed and if everything appears satisfactory, an operational permit will be issued. In Region 7, there will be only one permit issuance. This permit will cover both the testing and operational phases of the project.

ASR technology has not been widely utilized in the upper Midwest, due to uncertainties regarding cost and regulation. Information gathered through the first few permits issued will help to clarify the viability of the proposed regulatory approach. Once an approach has been fully adopted, it will minimize much of the uncertainty, which in turn may allow the development of more projects, thus helping drinking water systems better meet water resource needs without excessive new infrastructure.

## KEYWORDS

Aquifer Storage and Recovery, United States Environmental Protection Agency, Region 5, Region 7

## INTRODUCTION

As the world's population continues to increase from 6.5 billion in 2005 to a projected 8.2 billion in 2030 (UNESA, 2005) our demand for water will continue to increase. The affects of this increased demand for water will also be felt in the United States, although to a lesser degree then other areas of the world. Brown (1999) projected that by 2040 the United States would be withdrawing approximately 62.2 billion m<sup>3</sup> per year of water for public and domestic use. This amount is a 42 % increase in public and domestic water usage from 1995 (44.2 billion m<sup>3</sup>). However, in 2000 the United States of America consumed approximately 59.4 billion m<sup>3</sup> of water for public and domestic use (Hutson et al., 2004), which is approximately 23 % more than what Brown predicted (45.6 billion m<sup>3</sup>).

Underground storage of water appears to be one way that may help deal with the rising need for water. Currently, in the United States there are 1,050 ASR wells in operation. The largest numbers of these wells are found mostly in Florida, South Carolina, Arizona, California and Guam. Aquifer storage and recovery wells are increasingly being utilized in areas with rapidly expanding populations and limited fresh water supplies. Most prominent are the states of Florida, Arizona and California. The practice is beginning to spread in other parts of the country as well, including the Pacific Northwest, and the Midwest., as the perceived/purported benefits of these projects become more widely known.

The United States Environmental Protection Agency (USEPA) classifies ASR wells as a subcategory of aquifer recharge wells. The USEPA defines aquifer recharge wells as wells that are primarily intended to replenish the water in an aquifer. The difference between aquifer recharge and ASR is that ASR has two objectives (1) storing water in the subsurface and (2) typically recovering the stored water from the same well for a beneficial use (USEPA, OGWDW, 1999). Aquifer storage and recovery wells are not specifically defined in the regulations, but are considered Class V injection wells under 40 CFR Parts 144 and 146. In 1999, the USEPA conducted a study to determine the potential risk associated with various types of Class V wells (USEPA, OGWDW, 1999). At that time, available ASR data was very limited and did not address major issues, such as metal leaching and drinking water treatment residuals.

Aquifer storage and recovery has a wide appeal and is versatile in application. The appeal of ASR is due to the ability to store water long term without increasing the size of the surface water treatment facility. Storing water underground also can

provide an efficient method of providing for peak demands in areas with limited water supplies, particularly during the summer season. It can also reduce capital costs and result in long-term savings by avoiding the need to build additional costly transmission and treatment facilities. However, to date there are no national standards on what is required to properly maintain and operate ASR wells, and it has become increasingly apparent that there is a lack of data on the impacts on the aquifer when treated drinking water is mixed with the native formation water. The State of Florida has required perhaps the most extensive monitoring in the country, and experiences there have led to the realization that viability of the practice is very dependent on site-specific characteristics.

## **CASE STUDIES**

### **Wisconsin**

Wisconsin has tested two ASR pilot systems, one in Oak Creek, a Milwaukee suburb, and a second system in Green Bay. Both ASR system pilot tests used Lake Michigan as the source water, although their intake locations and treatment processes make for substantial differences in the characteristics of the injected water. Both systems proposed ASR as a way to postpone or avoid surface water treatment plant expansion, due to growth in demand. In Green Bay, the use of ASR was also tied to a regional plan to aid the surrounding communities' compliance with the radium standard. To facilitate rule development, the Wisc. Department of Natural Resources (DNR) formed a Technical Advisory Group (TAG). The Group was made up of three University professors, a USGS Hydrologist and a state toxicologist. Their charge was to review ASR operations at other locations and advise the DNR on issues and concerns to guide the rule development.

The Wisconsin ASR TAG produced a report that identified four main concerns.

1. Introduction of contaminants into the aquifer
2. Fate of water treatment disinfection by products
3. Geochemical reactions in the aquifer
4. Hydraulic control of injected ASR water

The Wisconsin ASR system operating rules needed to comply with existing applicable regulations, which prohibit injection of water that does not meet primary drinking water standards, the federal Safe Drinking Water Act (SDWA), which requires that water recovered from an ASR well must meet primary drinking water standards before being put into the water distribution system, and the Wisconsin Groundwater Law, which prohibits practices that will lead to exceedances of state

groundwater quality standards at specified “points of compliance”. Wisconsin State groundwater quality standards (ch. NR 140, Wis. Adm. Code) are usually based on SDWA maximum contaminant levels (MCLs), if they exist for a substance, although there are exceptions. Wisconsin State groundwater quality standards for trihalomethanes (THMs), chemicals that form as by-products of disinfection processes designed to remove microbial pathogens from drinking water, are more stringent than federal MCLs established for these compounds. The utilities pilot testing ASR worked with State legislators to draft an exemption to Wisconsin's groundwater law (s. 160.257, Stats.) covering regulation of THMs at ASR system sites.

### **ASR SYSTEM PERFORMANCE STANDARDS IN SUBCHAPTER XIV OF NR 811, WISCONSIN ADMINISTRATION CODE REQUIRES THAT:**

1. Injected ASR water must, at a minimum, meet state primary drinking water standards. The injected ASR water must also not exceed State groundwater quality preventive action limits, unless compliance is determined to be technically or economically infeasible.
2. Recovered water must meet State primary drinking water standards and be treated to provide a disinfectant residual prior to recovery into a water distribution system.
3. Stored ASR water shall at all times comply with State primary drinking water standards and the ASR system must be operated to ensure compliance with state groundwater quality standards at all applicable compliance points.
4. At the completion of each ASR cycle, water remaining in the ASR storage zone may not exceed applicable state groundwater quality standards.

### **Oak Creek Pilot Test**

The Oak Creek Water Utility operated a groundwater based water supply system utilizing four deep sandstone aquifer wells until it converted to surface water in 1977. The Oak Creek Utility surface water plant uses traditional treatment of water drawn from Lake Michigan. The Oak Creek Pilot test was designed to utilize two of the utility's existing water supply wells. The wells are approximately 549 meters deep and 55 meters apart. These wells are open to formations of Ordovician and Cambrian age that are confined by a shale aquitard unit, the Maquoketa Shale. The Oak Creek Water Utility ASR pilot test used Oak Creek Well #3 as the test ASR well and Oak Creek Well #1 as a monitoring well. The monitoring point in Well #1 was a 76 liters per minute (lpm) pump set at a high flow zone at 405.3 meters below ground surface (bgs).

Pilot testing of ASR at Oak Creek raised a number of concerns including:

**Increasing THMs.** During the Oak Creek ASR pilot THMs were generated within the aquifer, increasing from injected levels of around 20 ppb to over 40 ppb during ASR storage. Also, THM levels in the aquifer at the end of ASR pumpout appeared to be increasing.

**Increasing Mn.** Manganese oxides appear to have been dissolved from the aquifer matrix due to operation of the Oak Creek ASR system. Manganese levels at the end of ASR pilot test cycle pumpout were increasing and exceeded the state groundwater quality standard for manganese of 50 ppb. One manganese measurement during the Oak Creek ASR pilot test was at 140 ppb (between cycles 5&6).

**Increasing ORP.** The ASR injection water has a much higher oxidation reduction potential (ORP) than the receiving aquifer. The aquifer ORP increased with each pilot test injection.

Oak Creek has been granted a conditional permit to operate their ASR system with requirements for additional sampling to monitor for geochemical stability. The reason for the conditional permit is due to the concerns listed above.

### **Green Bay Pilot Test**

The City of Green Bay Water Utility ran two ASR pilot test cycles before abandoning the project. The first Green Bay ASR pilot test cycle injected into three Cambrian - Ordovician aquifer units. For their second ASR pilot test the ASR injection well was lined to allow injection into only the deepest aquifer unit. For the pilot test, a monitoring well was installed and packered off so that all three aquifer units could be independently monitored.

Some of the problems encountered during the Green Bay ASR pilot test were foreshadowed by early activities at the site. Historically the pilot test ASR well had produced water with a maximum level of arsenic at 3 ppb and nickel at 11 ppb. Two months after completion of the site coring and monitoring well construction, during a pump test of the ASR well, arsenic and nickel concentrations increased to 73 and 285 ppb, respectively. Tri-halomethanes were present in this water suggesting that municipal water was associated with the release.

The initial ASR pilot test injection of 36.7 million liters (MI) of treated Lake Michigan surface water caused an immediate release of arsenic and other metals from the ASR storage aquifer. After spiking at 280 ppb, from a background level of 3 ppb, arsenic stabilized in the middle monitored zone to around 100 ppb and remained at about that level throughout the remainder of the test. This immediate arsenic release was caused by rapid oxidation of sulfide minerals or by arsenic desorption from metal oxides. In the lower monitored aquifer zone, arsenic did not increase until 12 weeks into the test. This secondary delayed oxidative release from sulfide minerals was caused by excessive dissolved oxygen (DO) in the ASR injection water (up to 18.4



mg/l). High DO levels in the injected pilot test ASR water were the result of ozonation water treatment. Other metals and contaminants (e.g. Nickel, Cobalt, Manganese, Uranium, Radium and Gross Alpha) were also mobilized during the first pilot test cycle. It was necessary for the utility to recover 1200% of the volume initially injected for the ASR pilot test cycle before background concentrations for arsenic and other parameters were reestablished.

An additional problem with ASR pilot test cycle 1 at Green Bay was lack of hydraulic control, as evidenced by breakthrough of ASR injection water at the ASR site monitoring well, located 15.24 m away from the test ASR well, in less than three days after injection. Tri-halomethanes in the injected ASR water also did not degrade as quickly as initially predicted.

A second pilot test injection cycle was authorized at Green Bay after the ASR well casing was extended to seal off all formations other than a lower stratigraphic zone that was believed to be free of the arsenic bearing deposits. For the second Green Bay ASR pilot test cycle, approximately 9.1 Ml of treated Lake Michigan surface water was injected over five days and stored for eight weeks. The Green Bay Water Utility elected to terminate ASR pilot testing when elevated arsenic levels were once again detected in the ASR storage aquifer. During the second pilot test, all three monitored stratigraphic zones showed a rapid arsenic concentration increase and then stabilized to levels over drinking water standards. Eight weeks later, when ASR pumpout began, water samples indicated that arsenic levels had risen significantly, indicating that a delayed oxidative release had again taken place. The levels of arsenic and metals measured in the aquifer were not as high as in pilot test cycle 1 primarily due to the smaller volume of water injected, a lower DO (due to a higher water temperature) and the fact that, due to pilot cycle 1 ASR pumpout, almost 15.9 kilograms of arsenic had been removed from the system. For the second Green Bay ASR pilot test cycle, the ASR well was pumped to waste for 19 days. About 1150% of the volume of ASR water injected had to be removed from the ASR storage aquifer in order to return ambient groundwater to near background conditions.

### **Areas of Concern**

The Oak Creek and Green Bay ASR pilot tests confirmed that it is possible to place water into Wisconsin's deep sandstone aquifer system at a desirable injection rate; however, pilot testing has yet to demonstrate that ASR operations can be sustainable over a long period time without exceeding State groundwater quality or drinking water standards. Other concerns are:

1. The degradation of by-products of chemical disinfection has not been demonstrated with sufficient certainty to allow state regulators to confirm that long-term operation of an ASR well will not violate state groundwater quality standards for these substances.

2. Inefficient recovery of all injected ASR water may create an expanding zone of groundwater impact in the ASR storage aquifer that would result in system perimeter monitoring wells no longer being positioned to effectively monitor ASR system impacts.
3. Long-term sustainability of an ASR well or ASR-based system requires a level of engineering, and hydrogeologic, and geochemical competency that many water utilities do not have at this time. The cost of acquiring this level of competency may offset or reduce any savings projected from the use of ASR technology.

Pilot testing of ASR technology in Wisconsin to date has not been encouraging. Aquifer storage and recovery may yet prove to be a useful water management tool; however, the results observed in the Wisconsin pilot tests and supported by documented studies elsewhere suggest that the long-term viability of the technology may require additional, up front geological and engineering study and may incur additional costs for adequate system monitoring.

## **IOWA**

The Des Moines Water Works (DMWW) experienced a catastrophic emergency when its only water treatment plant was flooded in 1993 and it was unable to provide water to a quarter million customers for 11 days. To provide a reliable supply of water to meet residential and commercial demands during emergency situations, DMWW constructed a second water treatment plant at the Maffitt reservoir and began the process of evaluating the feasibility of ASR as a way of helping to offset costs of building additional drinking water treatment plants in order to meet peak demands during the summer months. The DMWW ASR demonstration test was conducted in two phases between 1995 and 1998.

The objectives of the project were to determine the technical feasibility of ASR, establish permitting procedures, and develop design criteria, operating strategies, and cost information for the technology. The Jordan Aquifer in Des Moines was selected as the test aquifer for this project and at about 804.7 m bgs, it is the deepest aquifer ever used for this purpose.

### **Ankeny Pilot Project**

Two former production wells penetrating the Jordan Aquifer in the northern part of the DMWW's service area in Ankeny were available for the pilot project. The wells were originally part of the Ankeny water supply and had been abandoned in 1992 when Ankeny became a wholesale water customer of the DMWW. Each of these wells was similar in age and construction and their condition was evaluated using geophysical logging techniques to determine which well would serve as the ASR test well.

The well that was selected is roughly 823 m deep total, cased to about 640 m, and is open to the Jordan Aquifer for the remainder of its depth. The open hole part of the well penetrates three permeable bedrock formations: the Root Valley Sandstone, the Oneota Dolomite, and the Jordan Sandstone. The permeable formations are about 101 m thick total. Fractures and solution features in the Oneota Dolomite appear to be the primary water-transmitting features. The Jordan Aquifer is confined above and below by less permeable aquitards. Regional literature suggests a low to moderate transmissivity for the aquifer but ASR demonstration testing indicates that the well is more transmissive than the literature suggests. The depth to water, about 113 m bgs, has historically been declining due to regional over pumping of the aquifer system. Forty-one Jordan Aquifer wells were identified within 64.8 km of Ankeny.

The Jordan Aquifer (Cambrian-Ordovician in age) extends from Missouri to Minnesota and Nebraska to Indiana, and numerous regional utilities obtain water from it. In the Des Moines area, the aquifer is very deep and contains slightly brackish water. The native groundwater exceeds the secondary drinking water standards for total dissolved solids (TDS), iron, and fluoride, and the primary drinking water standard for combined radium activity.

### **ASR Facility Cycle Testing**

Originally, four cycles of testing were planned. The first cycle would be a small volume cycle (about 11.4 MI) designed to evaluate geochemical reactions near the well bore. The second and third cycles would also be relatively small volume cycles (37.9 MI) to establish the improvement in recovery efficiency expected from successive cycles at the same volume. The fourth and final cycle would be a full-scale seasonal operational scenario.

The first cycle entailed recharge of 10.6 MI of water, storage for five days, and recovery for three days. About 140 % of the volume of water recharged was recovered (15.1 MI). Water was recharged for 10 days (35.6 MI) for the second cycle, and immediately recovered for almost six days until reaching a drinking water standard. The recovery efficiency was almost 85 % for the first two cycles. After analysis of the data from the first two cycles and consultation with USEPA and Iowa Department of Natural Resources (IDNR), it was decided that the third small volume cycle was unnecessary, and the third and final cycle was the full-scale seasonal operational scenario. As run, cycle three included 128 days of recharge (445.9 MI), 30 days of storage, and 67 days of recovery. The full-scale recovery efficiency for cycle three was about 70 % (314.6 MI) before reaching the secondary standard for TDS and the primary standard for combined radium activity. Based on the contractor's experience the anticipated recovery percentage was 100 %, however the well was only able to yield an 85 % recovery rate before reaching secondary drinking water standards for TDS.

## **Well and ASR Hydraulic Performance**

As noted earlier, the well's performance improved over the course of the demonstration test. There was no evidence of particulate or geochemical plugging observed during recharge/recovery cycles. Test results suggest that the aquifer has a transmissivity of about 1282 m<sup>2</sup>/day, which is about three times higher than regional aquifer values reported in the literature. Analysis of the hydraulic test data was complicated by headloss in the 15.24 cm casing liner and temperature differences between the recharge water and the Jordan Aquifer.

During the pilot tests, the water level rose about 31 m in the well during recharge at 2460 liters per minute (lpm) and fell by about 73 m during recovery pumping at 3974 lpm. No long-term regional effects on water level were observed. Several meters of water level response were measured in the other Jordan well about 716 m away during recharge and recovery operations. Water levels quickly returned to static water levels in both wells after recharge or recovery operations ended.

The calculated radius of the recharge water bubble beneath Ankeny during cycle three was between 82 to 116 m, depending on the assumed permeable thickness and porosity of the storage zone. Lateral migration of the recharged water was estimated to be 1 to 4 m per seasonal cycle.

## **Water Quality Evaluation**

Water samples were collected from the native groundwater, system water used for recharge, and water recovered from the ASR well throughout the demonstration test. The samples were analyzed for SDWA parameters and inorganic constituents needed to evaluate geochemical changes in the water chemistry during ASR cycle testing.

The data shows that the native groundwater is non-potable without treatment. It exceeds secondary drinking water standards for TDS, sulfate, fluoride, and iron, and it exceeds the primary drinking water standard for combined radioactivity. The system water used for recharge meets all drinking water standards and is a well-buffered calcium-magnesium-bicarbonate-sulfate type water with a TDS of about 200 mg/L and an elevated pH of 9.4.

While geochemical changes were seen, they resulted in no violations of monitored parameters at the ASR well. Dissolved oxygen and chlorine were consumed and reduced, lowering the pH and oxidation-reduction potential in the recovered water. Despite the reduction in pH, the recovered water maintains a positive calcium carbonate precipitation potential, and pH adjustment of the recovered water is unnecessary. The water recovered from the first two small scale cycles met drinking water standards until reaching about 85 % recovery, when the secondary TDS limit was exceeded. The full-scale cycle achieved 70 % recovery efficiency before exceeding the TDS and combined radium standards. Recovery efficiency did improve

after more full-scale ASR cycles but the typical recovery rate during normal operation has been at 85 %.

Most analytical parameters showed a linear mixing response in the recovered water samples as the poor quality native groundwater is diluted and displaced by the high quality recharge water. This would appear to indicate that a buffer at the periphery of the storage zone was being formed, consisting of a mixture of the two waters. Over the life of the project, as the buffer was further displaced by subsequent full-scale cycles, the recovered water chemistry has become more similar in composition to the initial recharge water chemistry with the exception of radionuclides, which show an increase during storage.

### **Iowa's Experience**

From both Region 7 and DMWW's perspective, the pilot test was technically successful. The well received permission from IDNR to operate as part of the water system and is being used seasonally to store/recharge about 567.8 Ml per cycle, and recovering the water until reaching the secondary drinking water standard for TDS. Field parameters continue to be monitored, and water samples are collected and analyzed in accordance with the permit. This information will be used to build the understanding of ASR operations in the Jordan Aquifer in and around the greater Des Moines, Iowa area.

Several conclusions were reached through the pilot test:

1. Seasonal peaking and emergency operations using ASR are feasible using the Jordan Aquifer. Recharge rates of 3.8 MID or more and recovery rates of 5.7 MID or more are possible.
2. No adverse geochemical reactions were observed. The recovered water is a mixture of the recharge water and the native groundwater. The recovered water meets all drinking water standards up to 85 percent volume recovery.
3. Hydraulic characteristics of adjacent wells and the Jordan Aquifer were unchanged by operation of the ASR well.
4. No lasting impact to water levels in Ankeny or in the Jordan Aquifer in the Des Moines area, were observed over the course of the demonstration test and subsequent full scale operation of the well.

Des Moines Water Works is now in the process of expanding the system and has already installed one new Jordan Aquifer ASR well which was permitted by EPA and is in the process of applying for another AST well which would be installed during the summer of 2007. The ultimate goal of DMWW is the installation of approximately 10 Jordan Aquifer ASR wells over the next five to ten years.

## UIC REQUIREMENTS

The major role of the UIC program is to insure the protection of underground sources of drinking water (USDW). As a result, the USEPA's Region 5 and 7 offices (figure 1) have developed conservative approaches to the issuance of ASR permits in the Midwest. This approach was developed in part because of the issues identified at ASR projects in Wisconsin (see case study above) and other areas, such as Florida. At several of these sites, there has been a notable increase in trace metal concentrations (e.g. arsenic and manganese) within the aquifer because of the recharge process. The release of such trace metals can be directly linked to the changes in the water chemistry (e.g. pH and redox potential) of the aquifer surrounding the injection well after treated drinking water was injected. Some of these sites have also found increases in tri-halomethane concentrations within the aquifer. The increase in concentration of chemicals like tri-halomethanes is likely due to residual disinfectants from the treatment process and increased concentrations of dissolved organic carbon in the recharge water.



**FIGURE 1. The USEPA Region 5 includes the states of Minnesota (MN), Wisconsin (WI), Illinois (IL), Indiana (IN), and Ohio (OH). The states of Iowa (IA), Nebraska (NE), Missouri (MO), and Kansas (KS) are administrated by the USEPA Region 7 office.**

## Approaches

In USEPA Region 5, the permitting process will be a two-permit process. The first permit will be a pilot permit, which will include at least three cycling tests. These tests will be designed to determine what effect, if any, the project will have on the aquifer. The pilot permit will also be concerned with the construction of the well, well testing, and sampling of the water (effluent and in situ). This permit will only be in effect until the end of the cycling phase. Once the cycles have been completed and if everything appears satisfactory, an operational permit will be issued. Some of the major aspects of the operational permit will be well testing and the reduced sampling needs, and plugging and abandonment of the well.

In USPEA Region 7, the process is slightly different from Region 5's in that there is only one permit. This permit covers both the pilot and operational activities at the ASR site. Another difference is that in Region 7 requires submission of a permit application that follows Class I injection well guidelines for a number of geological

parameters, as there are no specific requirements for this information in the application for Class V wells.

## Details

One of the major concerns from a regulatory perspective is the well construction. This helps ensure that the well will not endanger the environment, more specifically USDWs. In this regard, the UIC program is concerned with the material used in the construction and the amount of cement used to secure the well. As noted by David Pyne (2005) there are three types of casing used in the design ASR wells. These are steel, PVC and Fiberglass. In accordance with 40 CFR Part 146, the UIC program needs to consider all casing size and grade. The operator needs to submit to the Region 5 office the specifications of the material that will be used to case the hole. In part, this is to ensure that the material will have sufficient strength to withstand the pressures encountered during the lifetime of the well.

When looking at the well construction both Region 5 and 7 are also concerned with the cementing of the casings. Cementing of ASR wells will likely be more stringent than for traditional water supply wells. This is because while wells are producing water, there is negative pressure in the well so that there is less force to move fluid around the outside of the casing. However, injection wells increase the pressure inside the formation of interest and doing so increases the possibility of fluid being forced along the outside of the casing. Regions 5 and 7 policy for the cementing of casing strings is that each string needs to be cemented from setting depth to surface. This is typically proven through cementing records and/or cement bond logs.

From a regulatory perspective, well testing is an aspect of the permit that is of interest during both pilot and operational phases. Well testing usually takes the form of mechanical integrity tests. Determining the mechanical integrity of a well comes in two parts. These tests are performed to determine whether the well may pose a problem to USDW. The first part of mechanical integrity determines the soundness of the inside of the well. In both Region 5 and 7, this is typically done through standard annulus pressure testing. This type of test becomes a challenge when there is no packer in the well, as in the case of most ASR wells. In There are two solutions to remedy this issue 1) inserting a temporary packer and then pressuring up the annulus or 2) removing the tubing and inserting a temporary bridge plug and pressuring up the well. The second aspect of mechanical integrity is concerned with the movement of fluid outside of the casing. This is typically tested by looking at one of a variety of tests, such as temperature logs and cement bond logs. The periodicity mechanical integrity tests for ASR wells will depend on the depth of the well. Aquifer storage and recovery wells that are in the uppermost USDW will only need to perform part one of mechanical integrity and will likely only need to do this a few times. Wells that penetrate the deeper USDW will be required to perform both aspects of

mechanical integrity. This is to ensure that the fluid that is being injected is not moving from the intended aquifer to a USDW that lies above the zone of interest.

Another aspect that is of interest in the pilot and operational phases is the sampling of the effluent and in situ fluid. Due to the possibility of chemical reaction that could occur in the zone of interest when the injectate mixes with the connate water and rock matrix an aggressive sampling procedure will be required, unless there is sufficient hydrogeological information to demonstrate otherwise. If this is not the case, the UIC program in Region 5 encourages the applicant to work with the permit writer to develop sampling procedures that will address concerns without being too onerous. Typical sampling procedures will look at each aspect of the cycling process (injection, storage, and recovery). The frequency of the sampling will also change with the length of each cycle. Once the facility passes the requirements of the pilot permit and a degree of confidence is established in the viability of the site, the amount of sampling required can be reduced in frequency and sometimes in terms of parameters sampled.

According to 40 CFR (40 CFR Sections 144.52(a)(6)), in order for a well to be issued a permit under the UIC program there needs to be a plugging and abandonment (P&A) plan. A proper P&A plan must discuss the steps that will be taken when the well is abandoned. This entails a description of the location and number of plugs, the amount of cement that will be needed and a cost estimate for the plugging. A third party must do the cost estimate to assure that the cost estimate is sufficient for the plugging. Financial assurance for P&A is another permit requirement, according to 40 CFR Section 144.52(a)(7). The amount set aside in the financial assurance mechanism must be enough to cover the cost of plugging the well. This monetary guarantee is set aside in case the operator defaults and the USEPA has to plug the well. The cost estimate stated in the P&A plan is used to determine whether the financial assurance mechanism is adequate.

## CONCLUSIONS

To date, ASR technology has not been widely utilized in the upper Midwest due to uncertainties regarding cost, viability, and regulation. Information gathered through the first few permits issued will help to clarify the appropriateness of the proposed regulatory approach. Once an approach has been finalized, it will minimize the uncertainty, which in turn may allow the development of more projects, thus helping drinking water systems better meet water resource needs without excessive new infrastructure. At the same time, USEPA has formed a National ASR Workgroup to assess current practices, with the aim of identifying common criteria that should be considered at each site. This internal workgroup is gathering data that are more definitive on the status of all actual and proposed projects, along with researching current technical data from a variety of sources to identify recommended safeguards and procedures. The group will also consider legal and/or regulatory approaches



that can be taken to implement any recommendations. It is anticipated that final recommendations will be made in early 2008. These efforts are intended to provide a consistent approach that ensures that ASR projects are protective of all USDWs.

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## ARTIFICIAL RECHARGE STRATEGY FOR SOUTH AFRICA: WHAT IS IT ALL ABOUT AND WILL IT GIVE IMPETUS TO ARTIFICIAL RECHARGE IN SOUTH AFRICA?

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### ABSTRACT

South Africa has recently published its first National Artificial Recharge (AR) Strategy – a strategy that falls within the context of Integrated Water Resource Management (IWRM). The purpose of the document is to provide a national strategy on how to create an enabling environment for implementing artificial recharge. The strategy document consists of two main parts: 1) South Africa's AR strategy and 2) Supporting information. The purpose of the national artificial recharge strategy is captured in the vision statement: To use natural sub-surface storage as part of Integrated Water Resource Management wherever technologically, economically, environmentally and socially feasible. In order to realise this vision, seven themes were identified: 1. Knowledge Theme; 2. Legislation and Regulation Theme; 3. Planning Theme; 4. Implementation Theme; 5. Management Theme; 6. Research Theme; and 7. Strategy Implementation Theme. Associated with each theme is an objective of the theme, a description of the current status of each theme, the approach to addressing the objective of the theme, and specific actions or tasks that are required to reach the objectives. The supporting information to the strategy covers topics such as: A description of the uses and value and AR as well as possible constraints, risks and disadvantages of this technology; selected international and local case studies; international and local guideline documents; the broad criteria for success in South Africa; the delineation at a national scale of possible priority AR areas; and a recommended implementation and authorization process.

### KEYWORDS

Artificial recharge strategy, South Africa, integrated water resource management

### INTRODUCTION

Increasing water demands within a growing economy are forcing the South African government to investigate alternative water resource and supply options. Although AR forms one such option, the general awareness and knowledge of potential benefits is limited amongst water resource planners. The challenge is to introduce

AR as a “real” and easily executable bulk water resource augmentation option, to make the AR concept accessible to water resource engineers and planners, and to bring it within the grasp of non-technical decision makers. This paper describes the legal and water resource management framework within which the AR Strategy sits, and it presents an outline of the AR Strategy.

The term artificial recharge has been used in the strategy document. Since 2000, the term AR has, internationally, become less used, often being replaced by managed aquifer recharge (MAR) or managed aquifer recharge and storage (MARS). In the South African AR Strategy, the term MAR has not been adopted because it is also an established abbreviation for Mean Annual Runoff.

## **AIMS**

The overall aim of developing a national AR strategy is to create an enabling environment for implementing artificial recharge. The purpose of the AR Strategy document is to inform the water sector of artificial recharge and to provide a “road map” on how to go about making AR accessible to water resource planners and managers.

## **METHOD**

The process for developing the AR Strategy was first to present a Draft Strategy for comment. This was completed in June 2005, and was upgraded into Version 1.1 in February 2006. This version was upgraded in April 2007 to Version 1.2. During the course of upgrading the strategy pilot studies were carried out in order to “ground” the strategy within the country’s legal and institutional systems, and to ensure that it takes into account available skills levels within the municipal sector. Prior to this, local studies focussed primarily on the technical aspects of artificial recharge (Murray, 2002; Murray, 2004; Murray and Tredoux, 1998; Murray and Tredoux, 2002). The involvement of municipalities ensured that the AR strategy was also based on the non-technical aspects that often inhibit progress in implementing AR schemes.

## **RESULTS AND DISCUSSIONS**

In the Republic of South Africa (RSA) the National Water Act (DWA, 1998) and Water Services Act (DWA, 1997) govern water resources management and planning, and the Minister of Water Affairs and Forestry is appointed by National Government as custodian of all water resources in the RSA. The Minister uses the Department of Water Affairs and Forestry (DWA) to fulfil this responsibility.

## **Artificial recharge in the context of water legislation and Integrated Water Resource Management**

### **The National Water Act**

The National Water Act (NWA) (DWAF, 1998), is the principal legal instrument relating to water resources management in South Africa and contains provisions for the protection, use, development, conservation, management and control of South Africa's water resources. It sets the legal framework for implementing IWRM principles on a national scale. The NWA is implemented through the National Water Resource Strategy (NWRS), Catchment Management Strategies and a water use licensing process.

### **National Water Resource Strategy**

The NWRS gives a broad description of the available water resources and current management thereof, and it introduces future water resource management strategies for national water resource managers and decision makers (DWAF 2004a).

### **Catchment Management Strategies**

South Africa is divided into 19 Water Management Areas, and the process is currently underway for developing Catchment Management Strategies for each Water Management Area... The aims of the Catchment Management Strategies are to introduce catchment-specific strategies that are in line with the NWRS. The Catchment Management Strategies will draw on the existing catchment-based Internal Strategic Perspectives, which capture current water resource management practices, identify strategic IWRM approaches and develop specific goals that will ensure IWRM (DWAF, 2004b).

#### *Water Use Licences*

The NWA identifies 11 water uses, and according to the Act all water uses must be registered or licensed, except water used for domestic needs. The following water uses could relate to artificial recharge:

- Section 21(a). Taking of water from a water resource, for example equipping and pumping from a borehole.
- Section 21(c). Impeding or diverting the flow of water in a water resource, for example if a borehole is drilled in or near a stream and it changes the flow of water towards the borehole as a result of bank filtration.
- Section 21(d). Engaging in a stream flow reduction activity (SFRA) for example bank-filtration systems where boreholes are drilled close to rivers and thereby reduce flow.

- Section 21(e).Engaging in a controlled activity, where, as a result of the activity, the groundwater may be polluted.
- Section 21(g).Disposing of waste in a manner which may impact on a water resource.
- Section 21(j).Removing, discharging or disposing of water found underground if it is necessary for the efficient continuation of an activity, or for the safety of people, for example in mining activities where groundwater inflow needs to be controlled.
- Section 21(k).Using water for recreational activities, such as holiday resorts that use warm, deep-circulating groundwater for therapeutic spas.

### **The Reserve**

The NWA also makes provision for the Reserve, which is a reflection of the Present Ecological Status of a resource. In terms of the NWA, a licence cannot be issued before the Reserve has been determined. The Reserve is built into the NWA to determine the optimum water use without degrading the environment, and it takes people's basic needs into account.

### **IWRM and the Water Services Act (WSA)**

The WSA sets the foundation for providing access to basic water supply and it provides the regulatory framework for water services institutions (DWAF, 1997). The WSA provides the mechanisms to implement IWRM at the water services or municipal level. The WSA describes how water service authorities, providers, intermediaries, boards and committees are to be established and their functions. It describes the institutional arrangements of water services and the link with IWRM. The district municipalities are regarded as water services authorities and they develop WSDPs as part of their integrated development plans (Municipal Systems Act 32 of 2000). The concepts and principles captured in the NWRS and the CMSs need to be reflected in the WSDP. Aspects considered as part of the WSDP are social-economic issues, services levels, technical options, water resources, water conservation and demand management, water services infrastructure, water balance; water services institutional arrangements, customer services, financial requirements and current and future projects (Bohlabela District Municipality, 2005).

### **Discussion**

After considering the existing IWRM framework, the Department of Water Affairs and Forestry decided to develop a supporting sub-strategy, the *Artificial Recharge Strategy*, and make it part of the NWRS. This will ensure that AR will be considered at national level. It will also be considered at water management levels through the

CMSs, and at local levels through the WSDPs where AR can form part of local water service delivery plans. The Artificial Recharge Strategy is thus a national strategy that will give impetus to AR at all key levels of planning and governance in South Africa.

### **Artificial Recharge Awareness**

The first step in getting AR to be considered a viable water conservation or management option is to elevate the AR concept amongst water resource planners and managers. This is listed as the first theme in the AR strategy. In the interim, the AR Strategy document itself includes a considerable amount of information on AR and it is written to accommodate a wide audience – from practitioners in the field of AR to decision-makers that as yet have little knowledge of AR. Supporting information to the AR strategy include, amongst others:

- Types of AR
- International experience
- Southern African experience
- Criteria for successful implementation
- The authorisation process
- Existing international guideline documents
- South Africa's AR potential.

## **DISCUSSION**

The AR Strategy document provides a thorough introduction to AR and any water resource planner or manager who studies the document will be well placed to consider its applicability in their area of jurisdiction. The challenge, however, lies in creating a broad-based awareness of its applicability. This challenge is identified as the first task that needs to be undertaken when implementing the AR strategy.

### **Artificial Recharge Strategy**

The Artificial Recharge Strategy has been written in the same format as the NWRS and the catchment-based Internal Strategic Perspectives. It is presented in the following manner:

|                            |                                                                |
|----------------------------|----------------------------------------------------------------|
| The Vision:                | The long-term goal                                             |
| The Themes:                | Areas that need to be addressed in order to realise the Vision |
| The Management Objectives: | The objective of each Theme                                    |

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|                            |                                                                                       |
|----------------------------|---------------------------------------------------------------------------------------|
| The Situation Assessments: | The current status (“where we are now”) in relation to the Management Objectives      |
| The Strategic Approach:    | Key focus areas that need to be addressed in order to reach the Management Objectives |
| Actions:                   | The tasks required to fulfil the Strategic Approach                                   |
| Responsibility:            | The organisation responsible for implementing the Actions                             |
| Priority:                  | The order of implementing the Actions                                                 |

The purpose of the national artificial recharge strategy is captured in the vision statement:

To use natural sub-surface storage as part of Integrated Water Resource Management wherever technologically, economically, environmentally and socially feasible.

In order to realise this vision, seven themes were identified. Table 1 lists the themes and the management objectives.

**TABLE 1. Artificial recharge themes and management objectives.**

| <b>Artificial recharge themes</b>   | <b>Management objectives</b>                                                                                                           |
|-------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------|
| 1. Knowledge Theme                  | To create awareness and provide education on artificial recharge.                                                                      |
| 2. Legislation and Regulation Theme | To enable water management and water services institutions to adopt and regulate artificial recharge as part of IWRM.                  |
| 3. Planning Theme                   | To facilitate the use of artificial recharge in achieving sustainable, efficient and cost effective water resource use and management. |
| 4. Implementation Theme             | To support water management and water services institutions in implementing artificial recharge.                                       |
| 5. Management Theme                 | To optimise the management of artificial recharge schemes.                                                                             |
| 6. Research Theme                   | To develop a body of knowledge that supports efficient and effective implementation and operation of artificial recharge schemes.      |
| 7. Strategy Implementation Theme    | To implement and update the artificial recharge strategy.                                                                              |

Theme 2, Legislation and Regulation, has been used in this paper as an example of the Situation Assessment and Strategic Approach.

Theme 2 Situation Assessment: Artificial recharge with fresh water is not mentioned specifically in the National Water Act but is referred to indirectly in the regulations promulgated in terms of the Act. The regulations effectively state that the storage of any volume of water underground requires licensing. However current application forms are not suited to artificial recharge, as they are geared towards storage in dams.

Artificial recharge with waste water is mentioned in the NWA. The discharge of wastewater into an aquifer is regulated and requires registration and licensing in all cases.

The only other reference to artificial recharge is in a Government Gazette (No 26187a), in the context of stream flow reduction activities and altering a watercourse. It lists artificial recharge structures as one of the types of infrastructure that is covered by those regulations.

The National Environmental Management Act (NEMA) and associated regulations describe a number of activities that would “trigger” the need for either a basic assessment or a full environmental study, comprising a scoping study and an environmental impact assessment. Many of these trigger activities may be part of an artificial recharge scheme, like building roads, a dam etc., but only listed activity 13 is groundwater specific. This stipulates that a basic assessment is required if the volume of groundwater abstracted is greater than the amount authorised under a general authorisation. In summary, artificial recharge is not listed but groundwater abstraction above the general authorisation is a listed activity for a basic assessment and most artificial recharge schemes will trigger a basic assessment on that basis. Once triggered, the environmental study would typically include the assessment of the impact of the project on natural, cultural and socio-economic environments.

Theme 2 Strategic Approach: Under the current legislation artificial recharge projects can be implemented and authorised. However, authorisation will be problematic because the terms and definitions used in current legislation are open to various interpretations when applied to artificial recharge projects. This should be addressed by undertaking the following actions:

5. Clarify the current legal requirements for authorising artificial recharge projects.
6. Clarify the authorisation process and provide guidance and training on this.
7. Review the current legislation and establish whether amendments or new regulations are needed.

## **Discussion**

The AR strategy provides the “road map” on how to go about making AR accessible to water resource planners and managers. It unpacks the existing legal framework and provides a recommended process for authorising AR projects. It identifies all the challenges that need to be addressed in order to create an enabling environment for AR to be implemented in a timely manner. These are articulated in seven themes, each of which contains a number of “actions” in order to reach the objective of each theme.



## CONCLUSIONS

To the author's knowledge, this is the first National Artificial Recharge Strategy. One of the objectives in the strategy is to create awareness of AR within the water sector in South Africa. The Department of Water Affairs and Forestry initiated the development of the strategy in late 2004 and between then and April 2007 when Version 1.2 was completed, numerous meetings, workshops, presentations and discussions have taken place on AR. These events together with direct involvement in case studies have substantially increased local awareness of AR. At the time of writing this paper the AR strategy document had not been published, but one of the first steps in implementing the strategy, will be to roll out, throughout the country, the *Artificial Recharge Strategy*. Up-scaling the status of AR is gathering momentum in South Africa, and the strategy document should provide a huge impetus to incorporate AR within the realm of Integrated Water Resource Management in the country.

## ACKNOWLEDGEMENTS

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## MANAGEMENT OF CLOGGING FOR RECLAIMED WATER ASR IN A CARBONATE AQUIFER

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### ABSTRACT

At a reclaimed water ASR trial site in a limestone aquifer at Bolivar in South Australia, rates of clogging and unclogging were investigated in relation to measures tested to maintain and improve well efficiency and to general improvements in the level of source water pretreatment over a four year study period. Rates of clogging within a 4 m radius of the ASR well, defined by changes in the relative intrinsic permeability of the aquifer, were found to be enhanced at higher levels of turbidity, total nitrogen and pH due to their resultant effects on particle filtration, microbial growth and calcite dissolution respectively. At this site acceptably low rates of short- and long- term clogging were achieved for turbidity < 3 NTU, total nitrogen < 10 mg/L and pH < 7.2 in the source water. These criteria may provide useful water quality targets for sites in similar non-karstic carbonate environments, but should be used cautiously, particularly in cases of lower transmissivity or calcite content than reported here.

A single acidisation, combined with daily backwashing was found to be effective in stabilising injection rates which have been maintained over the study period. Tests of a by-pass filter system to assess whether sloughing was a major cause of clogging show a failure of the filters to capture a significant proportion of the particles present in the injectant that was due to the tendency for the organic flocs to break-up under shear stress.

The general consensus prior to undertaking this research was that injection of high quality reclaimed water into aquifers was acceptable but large uncertainty existed with regard to the nutrient-rich, particulate-laden injectant used at Bolivar. This study has clearly shown how source water quality along with well remediation and redevelopment affect the hydraulic performance of ASR wells and can be used to define the water quality needed for sustainable injection.

### KEYWORDS

Clogging, redevelopment, reclaimed water

### INTRODUCTION

Despite more than half a century of experience, clogging still remains a major operational issue for aquifer storage and recovery (ASR) wells that can limit storage volumes, necessitate regular well redevelopment, or in extreme cases cause project abandonment (Ehrlich *et al.* 1979; Oberdorfer and Peterson 1985; Hijnen and van der Kooij 1992; Pérez-Paricio and Carrera 1999; Pavelic *et al.* 2007).

The viability of injecting reclaimed water is dependent upon the physico-chemical and microbiological composition of the source water and the physical and mineralogical characteristics of the target aquifer that determine the nature of the interactions between the solid and liquid phases. For example, it is well-known that lower permeability aquifers clog more easily than high permeability aquifers (Hutchinson and Randall 1995) and therefore place much tighter constraints on the quality of water that can be injected. Alternatively, moderately transmissive limestone aquifers allow much poorer quality water to be injected and are also more tolerant of short-term fluctuations in source water quality (Rinck-Pfeiffer *et al.* 2002; Pavelic *et al.* 2006a).

Use of treated sewage effluent (reclaimed water) as a source water for ASR can be particularly problematic due to the elevated nutrient status of the water creating conditions conducive to biofilm growth, which tends to be most prevalent around the well. Whilst it is technically feasible to treat reclaimed water to high standards that avoid physical and biological forms of clogging entirely, this is costly. Attempts to develop guidelines on the quality of water suitable for injection have been based on sparse data, and have yet to be reliably validated (e.g. Pérez-Paricio and Carrera 1999).

In this study, we investigate the rates of clogging of an ASR well due to injection of reclaimed water in relation to measures used to maintain well efficiency during the earlier years when water quality was poorest and by general improvements in the level of pretreatment over the four year study period.

Related facets of this study including greater attention to the type, magnitude and interrelationship of individual clogging mechanisms, and on a mass balance model of permeability changes developed from laboratory and field data are reported by Pavelic *et al.* (2007) and Rinck-Pfeiffer *et al.* (2000; 2002).

## **SITE DESCRIPTION AND EXPERIMENTAL DESIGN**

This study was conducted at the Bolivar ASR trial site in South Australia, a site that has been the focus of considerable investigation over the past decade (eg. Dillon and Martin 2005; Greskowiak *et al.* 2005; Pavelic *et al.* 2006b; Vanderzalm *et al.* 2006).

The target aquifer is a confined, variably cemented (non-karstic), sandy-limestone aquifer encountered between depths of 100–160 m below ground surface and characterized by moderate transmissivity, ( $150 \text{ m}^2/\text{day}$ ) and a layer-cake distribution of hydraulic properties (Wright *et al.* 2002; Pavelic *et al.* 2006b). The injection/recovery well, referred to as the ‘ASR well’ penetrates almost the entire aquifer thickness and is completed without a screen (ie. as ‘open-hole’).

The source water for injection undergoes conventional secondary treatment followed by detention in wastewater stabilisation ponds, dissolved air flotation/filtration (DAFF) and chlorine disinfection in order to meet the South Australian standard for unrestricted use for irrigation. Between the first and second cycle, a biological nutrient removal (BNR) treatment process became effective, replacing the former trickling filters. Additional chlorine dosing was given immediately upstream of the ASR wellhead for parts of the trial. The chlorinator was operational for only part of cycle 1 due to persistent mechanical failures; fully operational in cycle 2; and deactivated in cycle 3.

Three cycles of ASR were performed between October 1999 and July 2005, whereby a total of  $483 \times 10^3 \text{ m}^3$  of reclaimed water was injected at average rates of 8 to 15 L/s and  $366 \times 10^3 \text{ m}^3$  recovered at a rate of 15 L/s. The ASR well was backwashed by pumping at 2-3 times the rate of injection for 60 minutes of continuous pumping initially and later adjusted to 24 minutes of surging at average recurrence intervals of 1.5 days in cycle 1, 1.0 days in cycle 2 and 19.6 days in cycle 3.

Greater detail on the site description and experimental design can be found in Pavelic *et al.* (2007) and the related studies listed above.

## METHODS TO EVALUATE CLOGGING AND UNCLOGGING

The following field and laboratory measurements and techniques were employed to identify the rates of clogging and unclogging; the main processes that cause and exacerbate clogging; and the effectiveness of various remediation and redevelopment strategies to maintain or improve well efficiency:

- 1) measurement of flow rates and pressures in the delivery line and in the ASR and observation wells to determine spatial and temporal variation in hydraulic conductivity and intrinsic permeability (\*)
- 2) analyses of the quality of injectant (\*), groundwater and backwash waters to define near-well biogeochemical processes
- 3) evaluation of the performance of three approaches to well redevelopment (\*)
- 4) evaluation of the effectiveness of well acidisation (\*)
- 5) by-pass filter testing of coarse particulate loadings in the injectant (\*)
- 6) analysis of water and particles in backwashed water during redevelopments and calculation of mass balances (\*)
- 7) MFI testing of the injectant

- 8) lab batch and in-situ sand chamber experiments of microbial growth kinetics
- 9) periodic step-testing of the ASR well
- 10) periodic down-hole video camera surveys and caliper logging of the ASR and observation wells (\*)

The elements that are discussed within this paper are indicated by an asterisk above. Greater detail on each of the facets can be found in Martin and Dillon (2005) and Pavelic *et al.* (2007).

## RESULTS AND DISCUSSION

### Acid treatment of ASR well

Cycle 1 injection commenced with two small injection tests. Because the average injection rate of 8 L/s was unacceptably low and rates of short-term clogging were intolerably high during the first test, several rehabilitation strategies were tested before and after the second test. During the first intermission, the well was dosed with concentrated sodium hypochlorite solution and then pumped by vacuum airlifting. Modifications were also made to the injection delivery system to eliminate the possibility of air entrainment due to cascading. Improvements in the second test were negligible and confirmed the need for a more rigorous well restoration program.

A total of 10 m<sup>3</sup> of 16% w/w hydrochloric acid solution was fed into the open interval of the ASR well over a two-day period (6-7 June 2000). This dosage was approximately four times the volume of the water in the open interval to ensure adequate penetration of acid into the formation. The acid was retained for two days to neutralize reactions before approximately 800 m<sup>3</sup> of water was airlifted and later pumped to recover the reaction by-products.

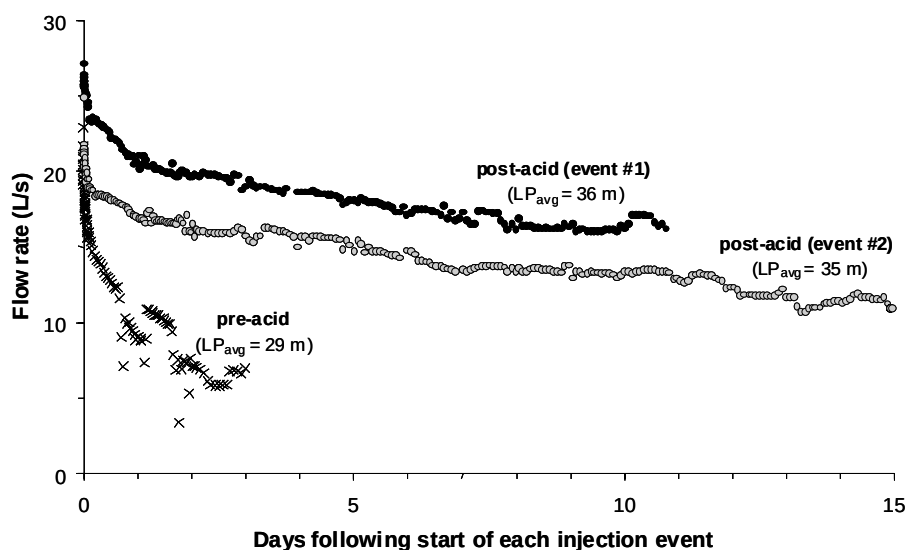
Reaction between the hydrochloric acid and calcite, the principle reactive material that composes 50-90% of the aquifer matrix, would be expected to result in elevated concentrations of Ca and Cl in solution; CO<sub>2</sub> being formed leading to secondary acidity and elevated HCO<sub>3</sub>, and increased TSS concentrations from siliceous material mobilised through dissolution of the carbonaceous matrix (Wojcik and Wieczysty 1986).

Time-series sampling over the course of the pumping revealed concentrations of Ca, Cl, HCO<sub>3</sub> and TSS that were elevated by over an order of magnitude above background. The recovered solute masses were dominated by Ca (660 kg) and Cl (1,110 kg) as expected, along with some excess of Mg (25 kg) and Fe (6 kg) indicative

of the dissolution of Mg/Fe bearing minerals such as ankerite, the next-most common carbonate mineral present after calcite. The measured amount of chloride and calcium recovered was within 10% of that stoichiometrically estimated to have been dissolved.

Approximately 80 kg of sediment, largely composed of quartz minerals, was also recovered from the well. The quantity of calcite that reacted with the acid was estimated to be 2,400 kg and the total amount of aquifer material lost from the ASR well when combined with the sediment loss during recovery totalled 2,500 kg. The average diameter of the open interval of the well was determined by caliper logging to have increased from 230mm to 330mm and the corresponding increase in surface area of the well-face was about 50%. Consequently, a reduction in the clogging rate would be anticipated even if the rate of clogging per unit surface area of well-face were unchanged.

Some heat was generated through the reaction, and the maximum measured temperature increase above background was  $\sim 2^{\circ}\text{C}$ . Monitoring of observation wells as close as 4 m from the ASR well indicated that the effect of the acid was very localised and had not impacted beyond the immediate vicinity of the ASR well.



**FIGURE 1. Injection rates pre- and post- acid treatment (redevelopment by backwashing occurred prior to the commencement of each injection event).**

## Evaluation of alternative redevelopment approaches

In the weeks following acidization, injection rates were found to be acceptable but not as high as anticipated and the volumetric losses due to backwashing were high. In order to improve the efficiency and effectiveness of redevelopment, an experiment was conducted to establish the efficacy of alternate redevelopment methods that were viable at the site within existing constraints. Three types of approach were tested that involved two different redevelopment methods, the first of which had been the standard prior to the experiment and the second was carried out at two different frequencies:

Type-A: daily pumping at constant rate of 32 L/s for 60 minutes (115 m<sup>3</sup>/day).

Type-B: daily surging over 24 minutes: 3 mins pumping at 32 L/s then 3 minutes rest repeated 3 times finishing with 6 minutes of pumping (30 m<sup>3</sup>/day).

Type-C: as for B and implemented on alternate days (15 m<sup>3</sup>/day).

An experimental program was conducted involving 18 redevelopment events, six of each type, undertaken as three pairs, with pairs undertaken in randomized order to minimise systematic errors that could be caused by residual effects of prior redevelopments. The experiment was undertaken between 4 October and 3 November 2000 during a period in injection cycle 1 when relatively stable hydraulic conditions had prevailed (Pavelic *et al.* 2007). Continuous monitoring of injection line pressure, rate of injection and backwash, and turbidity of the injectant and backwash water were used to calculate the mean line pressure, mean gross injection rate, mean net injection rate (accounting for the backwash volume), mass of suspended solids injected and the proportion of injected solids recovered.

These results are summarised in Table 1, which presents figures for the second of each of the paired events, which are least influenced by the previous type of redevelopment. The gross injection rates for types A, B and C are similar at 9.8 Ls<sup>-1</sup>, neglecting the final A-type case where line pressure was anomalously low. However when recovered volumes are taken into account, B appears to outperform C and A for net injection rate. The maximum difference between the means (8%) are comparable to the standard deviations, and an analysis of variance (ANOVA) confirms that differences in net injection rates are not significant ( $p > 0.95$ ).

The distinction in suspended solid mass balances was much stronger than for water balances. Type-A redevelopments remove an average of 59% of injected particles, substantially greater than B at 21% and C at 8%. Although there are variations among replicates for each redevelopment type, the differences are clearly significant.

**TABLE 1. Summary of results from an experiment on performance of three different redevelopment methods.**

| Type of redevelopment                                                                                                                                                                                                                                                     | Avg. Line Pressure (m) | Gross Inj Rate <sup>A</sup> (L/s) | Net Inj Rate <sup>B</sup> (L/s) | Initial <sup>C</sup> $K_{0-4\text{ m}}$ (m/day) | Mass recovery (%) | Water recovery (%) |
|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------------------------|-----------------------------------|---------------------------------|-------------------------------------------------|-------------------|--------------------|
| A                                                                                                                                                                                                                                                                         | 38.57                  | 10.18                             | 9.51                            | 0.33                                            | 55                | 12.5               |
| A                                                                                                                                                                                                                                                                         | 36.69                  | 11.06                             | 9.73                            | 0.30                                            | 86                | 25.6 <sup>D</sup>  |
| A                                                                                                                                                                                                                                                                         | 29.60                  | 8.18                              | 7.51                            | 0.29                                            | 38                | 12.1               |
| Mean±stdev:                                                                                                                                                                                                                                                               | 35.0±4.7               | 9.81±1.5                          | 8.92±1.2                        | 0.31±0.02                                       | 59.2±24.7         | 16.7±7.7           |
| B                                                                                                                                                                                                                                                                         | 37.48                  | 9.17                              | 9.00                            | 0.24                                            | 22                | 4.3                |
| B                                                                                                                                                                                                                                                                         | 37.52                  | 10.20                             | 9.88                            | 0.27                                            | 10                | 3.2                |
| B                                                                                                                                                                                                                                                                         | 37.16                  | 10.14                             | 9.95                            | 0.28                                            | 30                | 3.0                |
| Mean±stdev:                                                                                                                                                                                                                                                               | 37.4±0.2               | 9.83±0.6                          | 9.61±0.5                        | 0.26±0.02                                       | 20.9±10.0         | 3.5±0.7            |
| C                                                                                                                                                                                                                                                                         | 37.83                  | 9.89                              | 9.55                            | 0.27                                            | 13                | 2.2                |
| C                                                                                                                                                                                                                                                                         | 37.93                  | 10.35                             | 10.00                           | 0.24                                            | 8                 | 1.4                |
| C                                                                                                                                                                                                                                                                         | 38.00                  | 8.31                              | 8.18                            | 0.30                                            | 2                 | 1.4                |
| Mean±stdev:                                                                                                                                                                                                                                                               | 37.9±0.1               | 9.52±1.1                          | 9.24±1.0                        | 0.27±0.03                                       | 7.8±5.4           | 1.7±0.5            |
| <sup>A</sup> does not account for recovered volume; <sup>B</sup> accounts for recovered volume; <sup>C</sup> values one hour from start of an injection event after redevelopment; <sup>D</sup> injection not continuous, thus leading to a higher proportion of recovery |                        |                                   |                                 |                                                 |                   |                    |

In general, type A recoveries resulted in more sustained but not necessarily higher concentrations and that the enhanced period of pumping associated with type-A clearly recovers much more of the injected particles than B or C, although at the detriment of volumetric efficiency, since losses vary from 13-26% for A, 3-4% for B and 1-2% for C.

Hydraulic conductivity was derived from injection rates and head differences between the ASR well and the observation well situated 4 m away. Table 2 shows that A-type redevelopments result in 15% higher hydraulic conductivities than types-B or C, testifying to the benefits of the enhanced mass recovery due to the extended period of pumping ( $K_{0-4\text{ m}}$  was determined one hour after the commencement of injection). The effect of unrecovered suspended solids appears not to have a chronic effect on clogging because much of the colloidal material is degraded (as shown by Vanderzalm *et al.* 2006), producing carbon dioxide, which in turn may also play a role in calcite dissolution. Within an hour of the recommencement of injection the small net benefit in hydraulic conductivity observable for type-A redevelopment is not sufficient to offset its additional backwash volume.

On the basis of these results, the redevelopment strategy for all future redevelopments was changed from type-A to type-B.



### By-pass filter bag

The potential for sloughing of coarse debris from the sides of the pipeline or the mobilisation of aquatic organisms from an upstream balancing storage pond was recognised as a potential cause of clogging that may go undetected by periodic manual sampling. To determine its significance, a simple by-pass filter bag was installed along the delivery line to the ASR well to provide data on the integrated particulate load entering the ASR well.

The capture efficiencies of particles for the three filters sizes tested of 1, 10 and 100  $\mu\text{m}$  varied from only 3 to 8%, which was considerably less than that anticipated from particle size distribution data (Table 2). Thus, the by-pass filter system offered a poor measure of the integrated particulate load entering the aquifer. The failure of the filters to capture a significant proportion of the particles present in the injectant, irrespective of the effective aperture size, is due to the tendency for the organic flocs to break-up under shear stress. Massmann *et al.*, (1999) showed that subjecting the injectant to greater shear stress led to a reduction of the median size and increased the spread of particle sizes.

**TABLE 2. Results from ten by-pass filter tests**

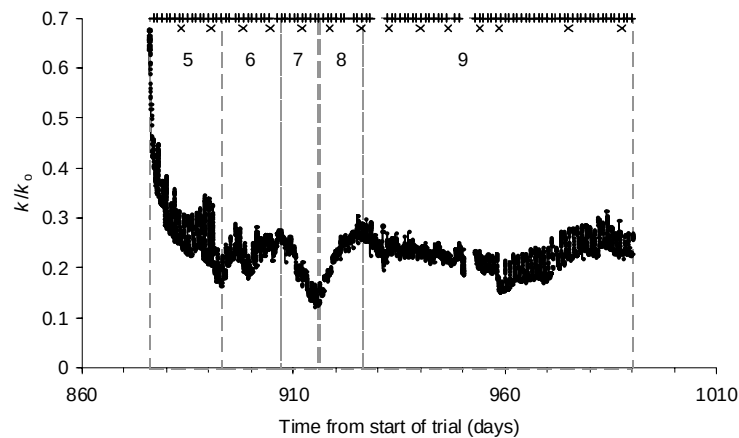
|                                                    | 6-11<br>Apr<br>2000 | 4-15<br>Aug<br>2000 | 15<br>Aug -<br>14 Sep<br>2000 | 14 Sep<br>- 5 Oct<br>2000 | 5-31<br>Oct<br>2000 | 31<br>Oct -<br>4 Dec<br>2000 | 4 Dec<br>- 30<br>Jan<br>2001 | 30<br>Jan -<br>29<br>Mar<br>2001 | 5-20<br>Mar<br>2002 | 27<br>May<br>- 25<br>Jun<br>2002 |
|----------------------------------------------------|---------------------|---------------------|-------------------------------|---------------------------|---------------------|------------------------------|------------------------------|----------------------------------|---------------------|----------------------------------|
| Duration of test (days)                            | 5                   | 11                  | 30                            | 30                        | 21                  | 26                           | 34                           | 57                               | 15                  | 29                               |
| Effective size of filter ( $\mu\text{m}$ )         | 100                 |                     |                               |                           | 10                  |                              |                              |                                  | 1                   | 10                               |
| Mean concentration captured by filter (mg/L)       | 0.33                | 0.35                | 0.18                          | 0.66                      | 0.69                | 0.29                         | 0.26                         | 0.15                             | 0.43                | 0.12                             |
| Mean TSS of water passing through filter (mg/L)    | 7.8                 | 10.8                | 8.8                           | 12.2                      | 8.4                 | 8.0                          | 6.8                          | 4.0                              | 11.2                | 3.7                              |
| Capture efficiency (%)                             | 4                   | 3                   | 2                             | 5                         | 8                   | 4                            | 4                            | 4                                | 4                   | 3                                |
| Particles in water greater than size of filter (%) | 5                   | 63                  | 53                            | 90                        | 92                  | 94                           | 86                           | 80                               | -                   | -                                |

If sloughing of particles into the source water stream and hence the ASR well occurs in an episodic fashion, it would be expected that the probability of intercepting the fragments of these materials would increase as the frequency of sampling also

increased. Thus, the measurement of particulate concentrations from bulked samples from an auto-sampler programmed to collect and bulk injectant on an hourly basis would be higher than grab samples collected during fortnightly site visits. Comparisons between the integrated sampler with grab samples showed that the turbidity values from the bulked samples were no higher than for the grab samples. The weight of evidence suggests that episodic sloughing into the well is unlikely to be significant.

### Water quality criteria for sustained operations

The rate of well clogging was deduced from changes in relative intrinsic permeability ( $k/k_o$ ) (Pavelic *et al.* 2007) within a 4 m radius of the ASR well over the course of the trial (post-acidization). Intrinsic permeability was used since it normalizes for the effects of line pressure and water temperature fluctuations. Over the four year study period,  $k/k_o$  values were in decline for 25% of the time, stable for 55% and rising for the remaining 20% of the time. We refer to these trends as *clogging*, *no clogging* and *unclogging* respectively and they are defined by a minimum duration of 10 days and an average rate of absolute change in  $k/k_o$  of  $>0.005$  /day for *clogging* and *unclogging* and  $<0.002$  /day for *no clogging* (stable) (this classification accounts for the entire period of injection). There are 13 such periods in total: 5 with *clogging*, 5 with *no clogging* and 3 with *unclogging*. To illustrate these concepts, data from cycle 2 is presented in Figure 2. No long-term net deterioration in well efficiency was observed after a rapid decline at the start of the first cycle. Clogging also occurs over the short-term ( $<1$  day) at rates that are considerably higher than long-term clogging but are reversible by backwashing.



**FIGURE 2. Normalised intrinsic permeability ( $k/k_o$ ) data in the 0-4 m interval for the second injection cycle. Backwash events are indicated by 'x' and samplings for injectant water quality by 'x' at the top of each plot. 5 time periods were identified in this cycle, with *clogging* in periods 5 and 7, *unclogging* in period 8 and *no clogging* in periods 6 and 9.**

Reclaimed water injection into a calcite-rich aquifer was shown by Rinck-Pfeiffer *et al.* (2000), Wood *et al.* (2005) and Pavelic *et al.* (2007) to cause clogging by particulate filtration and microbial growth, and unclogging by calcite dissolution. These processes conform with those identified in related laboratory column studies with similar qualities of water (Ehrlich *et al.* 1979; Oberdorfer and Peterson 1985). Other clogging mechanisms (e.g. air entrainment, iron precipitation), were evaluated and discounted as absent or of limited consequence (Pavelic *et al.* 2007).

Pronounced improvements in the physico-chemical characteristics of the injectant occurred due to improved pre-treatment processes within the water reclamation plant over time (Table 3). For example, the inclusion of the activated sludge process-step from cycle 2 decreased the levels of total nitrogen from 20 mg/L to 4 mg/L on average through more effective nitrification and denitrification. TOC levels were also reduced from 18 mg/L to 13 mg/L.

**TABLE 3 Characteristics of the injected water in each of the three cycles and local ambient groundwater (means and standard deviations reported).**

| Parameter                        | Cycle 1<br>Injectant<br>(n=18) | Cycle 2<br>Injectant<br>(n=14) | Cycle 3<br>Injectant<br>(n=17) | Ambient<br>Groundwater<br>(n=17) |
|----------------------------------|--------------------------------|--------------------------------|--------------------------------|----------------------------------|
| Temperature (°C)                 | 20.3 ±                         |                                |                                |                                  |
| pH                               | 7.1 ± 0.4                      | 7.0 ± 0.4                      | 7.6 ± 0.4                      | 7.3 ± 0.1                        |
| Dissolved Oxygen (mg/L)          | 4.4 ± 3.4                      | 6.0 ± 1.4                      | 10.4 ± 2.2                     | 0.1 ± 0.1                        |
| Turbidity (NTU)                  | 3.8 ± 3.4                      | 2.6 ± 1.8                      | 1.7 ± 0.9                      | 13 ± 9                           |
| Total Nitrogen (mg/L)            | 19.7 ± 11.1                    | 7.8 ± 2.5                      | 4.4 ± 3.3                      | 0.08 ± 0.03                      |
| Total Phosphorus (mg/L)          | 0.57 ± 0.55                    | 2.3 ± 0.9                      | 0.86 ± 0.99                    | 0.08 ± 0.06                      |
| Total Organic Carbon (mg/L)      | 17.6 ± 1.9                     | 20.2 ± 2.1                     | 13.0 ± 1.1                     | 0.3                              |
| Chlorine residual – total (mg/L) | 0.7 ± 0.4                      | 2.6 ± 2.4                      | -                              | 0                                |

Marked differences in the  $k/k_0$  trends appear to be linked to temporal changes in the quality of the injectant. Three sets of cluster analysis were undertaken to establish the role that specific, routinely monitored water quality constituents (as listed in Table 3) have in affecting long- and short- term clogging behaviour, each based upon the following datasets and conditions:

- 1) for all three injection cycles with respect to the three classes of long-term clogging previously defined
- 2) for periods of *no clogging* only with  $k/k_0$  values partitioned into three classes to differentiate between relatively low ( $k/k_0 < 0.18$ ), medium ( $0.18 < k/k_0 < 0.24$ ) and high ( $k/k_0 > 0.24$ ) permeability

3) for the 30 discreet injection events that followed backwashing with coincident injectant water quality data. Short-term clogging is defined as the net change in  $k/k_o$  between 1 hour and 18 hours after the commencement of each injection event and partitioned into three classes to differentiate between *high clogging*, *clogging* and *unclogging*.

The analysis revealed that the level of turbidity in the source water was strongly associated with short-term clogging due to particulate retention, but over the long-term its significance was reduced as the predominantly organic particles in the presence of oxygen and nitrate were degraded, thereby providing additional substrate for biomass production. The level of total nitrogen in the source water appeared to be the limiting factor for microbial growth in the long- and possibly the short-term. Other primary nutrients (i.e. P, OC) were examined and found to be non-limiting. pH was a consistent and significant indicator of unclogging by calcite dissolution in the short- and long-term, offsetting the clogging effects associated with filtration and biogrowth. Chlorine levels had a marginal but non-significant effect on long-term clogging. Water temperature had no significant direct effect on clogging, although it indirectly affected rates of biogrowth and source water quality. The inter-relatedness of the parameters was generally small. Detailed results are presented in Pavelic *et al.* (2007).

The goal was to define the appropriate quality of source water needed for sustainable operations, ie. that which produced acceptable levels of clogging over both the short- and long-term. Achievement of an intermediate level of short-term clogging data or better would require turbidity < 3 NTU, total nitrogen < 12 mg/L and pH < 7.2. Further, values of turbidity < 3 NTU, total nitrogen < 10 mg/L and pH < 7.3 lead to *no clogging* in the long-term or short-term (data presented in Pavelic *et al.* 2007).

## CONCLUSIONS

Over the four year study period,  $k/k_o$  values were in decline for 25% of the time, stable for 55% and rising for 20% of the time. No long-term net deterioration in well efficiency was observed after a rapid decline at the start of the first cycle prior to a one-off acidization of the ASR well, refinement in the well redevelopment method, and a gradual improvement in the quality of the injected water.

Reclaimed water injection into a calcite-rich aquifer leads to clogging by particulate filtration and microbial growth, and unclogging by calcite dissolution. These processes conform with those identified in related laboratory column studies with similar qualities of water.

Clogging also occurs over the short-term (<1day) at rates that are considerably higher than long-term clogging but are entirely reversible by backwashing. Testing of three possible redevelopment methods revealed that daily pumping for one hour

was more effective than surging for 24 minutes in recovering injected particulate matter but resulted in larger volumetric losses. Delaying redevelopment intervals from one day to two days reduced particulate recoveries but appeared to have little impact on injection rates or hydraulic conductivities. Surging of the well on a daily basis had the highest volumetric efficiency and was the preferred redevelopment technique. There is latitude for reducing the frequency of redevelopment and mass recovery of particulates with only a minor reduction in volumetric efficiency of injection. In fact, reducing the backwashing frequency by 13- to 20- fold in cycle 3 as compared to cycles 1 and 2 did not appear to have a substantial impact on flow rates or permeabilities.

Acidization was performed during the first cycle following two small injection tests when initial teething problems were experienced and a modified injection line failed to improve performance. The acidization resulted in over two tonnes of calcite being dissolved and 100 kg of sand mobilised resulting in a doubling of the volume of the open hole. The initial acidization, combined with regular backwashing was found to be effective in stabilising injection rates which were maintained over the study period.

The by-pass filter system offered a poor measure of the integrated particulate load entering the aquifer, irrespective of the filter size tested. Failure of the filters to capture a significant proportion of the particles present in the injectant was due to the tendency for the organic flocs to break-up due to shear stress.

Achievement of an acceptably low level of short-term clogging and no clogging or unclogging in the long-term was found to require a turbidity < 3 NTU, total nitrogen < 10 mg/L and pH < 7.2. Very limited data also suggests that bacterial regrowth potential < 1 mg/L may also be a criterion. These criteria may provide useful water quality targets for sites in similar non-karstic carbonate environments, but should be used cautiously, particularly in cases of lower transmissivity or calcite content than reported here.

## ACKNOWLEDGEMENTS

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## THE ANTELOPE VALLEY WATER BANK PILOT BASIN PROJECT: FROM ONE ACRE TO 1600 ACRES

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### ABSTRACT

An innovative hydrogeologic investigation and small-scale pilot basin test were conducted to predict the performance of a 123,350,000 m<sup>3</sup> per year groundwater recharge project. The water bank will use approximately 659 hectares of agricultural fields to store surface water in the 46 to 61 meter thick portion of aquifer that was dewatered by historic groundwater overdraft. Surface trenches and exploration boreholes were constructed at various project area locations to determine sub-surface soil characteristics and deeper vadose zone lithologies. A one-acre 0.4 hectare location representative of the larger Project area was then selected and a pilot basin test implemented to provide a suite of highly detailed subsurface hydraulic property data as well as in-situ flow related data during the test to determine wetting front advancement, subsurface mounding and perching, and vadose zone water and groundwater quality.

Based on several monitoring methods, the recharge was observed to advance 332 feet (101 meters) to the water table within 78 days. Water perching within 0 to 40 meters bgs was intermittent and not sustained during recharge whereas greater mounding was observed at deeper depths. Total dissolved solids and nitrate from historic agricultural operations were observed over California MCLs in suction lysimeter samples; groundwater has remained below all MCLs. Mass balance estimates from field and laboratory data indicate that these antecedent concentrations will be diluted by recharge water to below primary or secondary standard levels within 2 to 5 recharge seasons, respectively.

### KEYWORDS

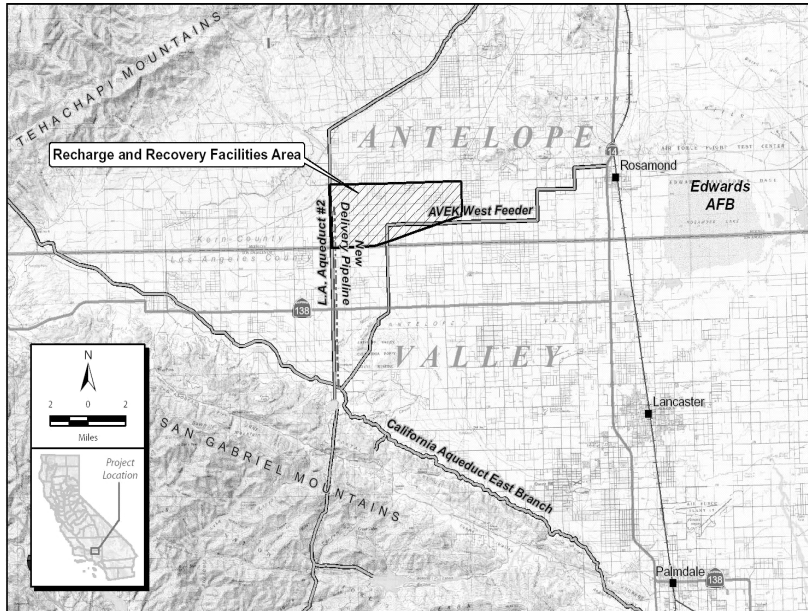
Perching recharge, suction lysimeter, water quality, vadose zone monitoring

### INTRODUCTION

The Antelope Valley Water Bank (AVWB, the Project) is located in Kern County near the boundary with Los Angeles County at the western end of the Antelope Valley (Figure 1) and encompasses an area of 1,629 acres (659 hectares). The Project will recharge up to 100,000 acre-feet (123,350,000 m<sup>3</sup>) per year (afa) of imported surface



water into existing agricultural fields during the off-growing season, and have a total storage capacity of at least 500,000 acre-ft (616,740,000 m<sup>3</sup>) of water. This stored water will be recovered when needed using a combination of existing and new wells.



**FIGURE 1. Antelope Valley Water Bank Project Area.**

Antelope Valley is arid, averaging less than 10-inches (250 mm) rain per year and natural aquifer recharge is considered insignificant. The Valley is a graben, an area that has dropped downward due to movement on the San Andreas and Gerlock faults that bound it. Over time the basin has filled with several thousand feet of alluvial materials eroded from the bounding mountain ranges. The uppermost-unconfined aquifer within these alluvial materials has been partially dewatered up to 200 ft (61 m) by historic over-pumpage for agricultural irrigation. The current water table is located at approximately 330 ft below ground surface.

The project area aquifer is highly transmissive; wells consistently yield more than 1,000 gallons per minute (227,000 liter/sec) and the water quality is excellent. Numerous investigators have evaluated the west end of the Antelope Valley for aquifer recharge projects since the 1940's. The pilot test described herein was designed to conclusively demonstrate that the project area could recharge up to annual and maximum storage capacity design rates.

Project area soils were classified into different textural types via soils mapping, test pit logging and surface sampling. In order to ensure that the pilot test would provide results reasonably representative of the larger Project area, a location was selected with a particle size distribution that is finer textured than 80 percent of the Project

recharge area. The pilot test location has also been farmed since at least the 1960s, allowing potential impacts from deep percolation of irrigation water salts to be evaluated.

The pilot test was performed in an approximately 1-acre (04 hectare) bermed corner of a recently farmed field. California State Water Project (SWP) water was used to supply the pilot recharge basin for an initial 5.5 month test period beginning December 19, 2005 and ending June 1, 2006. A second recharge period began on July 24, 2006 and ended in October 2006.

The resulting pilot basin along with local investigation points is shown in plan view in Figure 2. These investigation points were designed to resolve the following important recharge related issues/questions related to the Project goals:



**FIGURE 2. Plan view aerial of Pilot Basin.**

- 1) The average achievable recharge rate (acre-ft/day per acre or ft/day).
- 2) The rate (velocity) at which percolating water moves downward towards the uppermost aquifer.
- 3) The degree to which percolating water can migrate through and around lower-permeability intervals in the vadose zone and the degree to which perched water conditions may evolve above these intervals.
- 4) The specific yield (effective porosity) throughout the vadose zone, as a measure of available storage space.
- 5) Water quality impacts, if any, on the uppermost aquifer, taking into account the initial flush of soluble salts present in the vadose zone naturally and as a result of long-term agricultural irrigation practices.
- 6) The degree to which pilot test results are representative of the larger Project area.

## METHODS

Previous studies included:

- Excavation, sampling, geologic logging, and infiltration testing of 17 soil trenches, 11 to 15 ft (3.3 to 4.6 m) deep at various locations in the Project area.
- Drilling, sampling, geologic logging, and borehole geophysical logging of three exploratory test holes to depths from 398 to 478 ft (121 to 146 m) bgs in the northern portion of the Project area.
- Groundwater sampling and analysis of 2 irrigation wells located in the northern portion of the Project area.

Supplemental work performed to support the pilot test included:

- Particle size distribution testing of 56 near surface soil samples collected from areas outside the pilot test basin, and 2 additional soil samples from within the pilot test basin.
- Cone penetrometer testing to a depth of 60 ft (18 m) at 10 locations around the pilot basin prior to and after 5.5 months of water application.
- Drilling, sampling, geologic logging, instrumenting, and completing the following boreholes within the pilot basin:
  - MW-1, a groundwater monitor well with nested air and water piezometers in the vadose zone.
  - VW-1, a nested vadose zone well with suction lysimeters, and advanced tensiometers at various depths
  - N-1, a neutron logging access tube
- Conducting laboratory physical and hydraulic property tests on borehole drill cuttings and core samples.
- Conducting leaching tests on core samples with leachate water quality analyses.
- Conducting field in-situ saturated hydraulic conductivity testing using borehole permeameter methods in MW-1 water piezometers prior to the pilot basin test.
- Measuring saturated hydraulic conductivity of subsurface materials using atmospheric pressure testing methods with air piezometers in MW-1 and suction lysimeters in VW-1 prior to the pilot basin test.
- Monitoring groundwater levels, temperatures, and electrical conductivity in MW-1 and in vadose zone water piezometers prior to and during the pilot basin test.
- Groundwater quality sampling and analysis of inorganic constituents in MW-1 prior to, during, and following the pilot basin test.

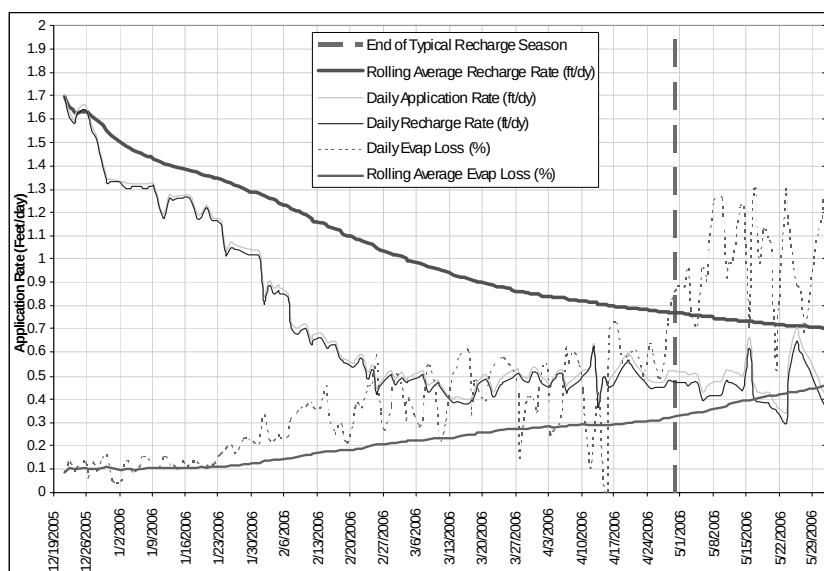
- Monitoring vadose zone matric potentials in VW-1 via prior to, and during the pilot basin test.
- Collecting and analyzing vadose zone water samples from VW-1 during the pilot basin tests.
- Conducting neutron logging in neutron access tube N-1 prior to and during the recharge test.
- Monitoring pilot basin water inflow and calculating outflow as the difference between inflow and evaporation.

## RESULTS AND DISCUSSION

Interpretation of data from this work has led to the following conclusions:

### Achievable Volumetric Recharge Rates

Daily recharge rates as well as cumulative (rolling average) recharge rates for first 5.5 month period are presented in Figure 3. Evaporative losses ranged from insignificant during certain winter days up to 13% in late May, averaging 3% over the period of the typical recharge season (December through April). The cumulative average recharge rate for the typical recharge season (December through April) was 0.8 ft (0.24 m) per day.



**FIGURE 3. Average recharge rates during first 5.5 month recharge period.**

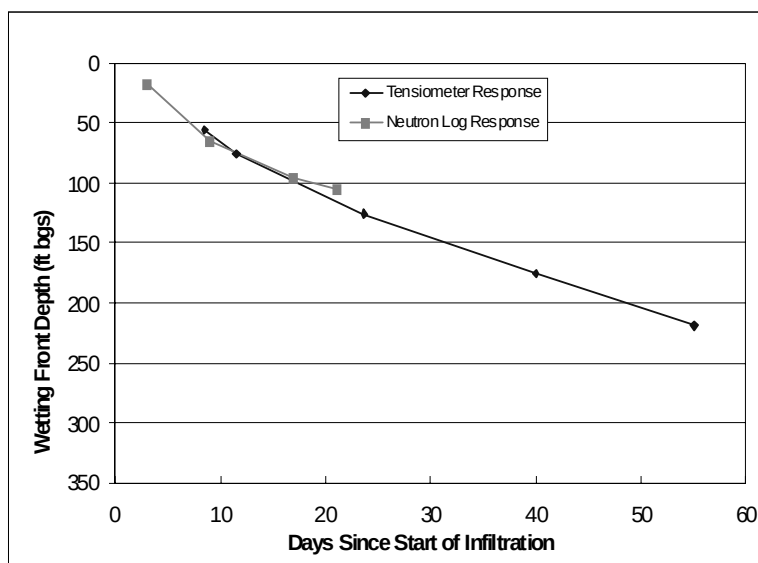
Applying the average cumulative recharge rate for a typical recharge season over the full Project area, the Project goal of 100,000 acre-ft ( $123,350,000 \text{ m}^3$ ) per recharge per season would have been obtained by mid March (83 days of recharge). Since 80 percent of the Project area is expected to have coarser textured surface soils, and coarser textured soils generally exhibit higher hydraulic conductivities (Todd, 1980), the results summarized above are expected to be conservative for the entire Project area.

### Measured Travel Times to the Water Table

Wetting front travel times were estimated from average cumulative recharge rates, matric potential and water content data collected at different depths, and groundwater elevation and EC data in MW-1. Estimated travel times from the pilot basin to the water table range from 74 to 124 days, with average vertical pore velocities of 4.5 to 2.7 ft (1.4 to 0.8 m) per day.

The wetting front was tracked with matric potential and water content data collected at different depths in the unsaturated zone from advanced tensiometers in well MW-1 and neutron logs in well N-1, respectively.

This wetting front response to a depth of 219 ft (66.8 m) is shown Figure 4. Extrapolating these data to 332 ft (101 m) bgs result in travel times of approximately 90 to 120 days and corresponding pore water velocities of 3.7 to 2.8 ft (1.1 to 0.85 m) per day.



**FIGURE 4. Observed wetting front response.**

Based on increased water levels in well MW-1, travel times were estimated to range from approximately 81 to 117 days. Observed water level increases were small (approximately 1 ft) and variable between early March and mid April 2006, most likely due to water being flushed from high permeability regions of the vadose zone ahead of the main recharge front.

### Average Specific Yield or Effective Porosity

Average specific yield was estimated by two methods from core samples collected from representative textural layers in the subsurface. The first method assumes that the effective porosity is the pore space that drains from a soil core between 0 bars (i.e. saturation) and minus 1/3 bar of matric potential. The second method assumes that effective porosity is the pore space where water flows at hydraulic conductivities greater than  $10^{-7}$  cm/s.

Effective porosity values for core samples collected over a range of depths in the unsaturated zone in wells MW-1, VW-1, and N-1 ranged from 0.07 to 0.21 with average estimated effective porosities of 0.18 and 0.16 for the 1/3 bar and  $10^{-7}$  cm/s methods, respectively (Table 1). These values are consistent with other specific yield estimates of 14% to 20% proposed by other workers who have studied the uppermost aquifer underlying the general Project area (DWR, 1977; USGS, 1978; Psomas, 1998; and USGS, 2003). Moreover, these values indicate there is at least 500,000 acre-ft of storage available in the dewatered portion of the aquifer. It should be noted that the near surface soil sample (MW-1) showed a low effective porosity of 0.07 to 0.09. This value is most likely due to the silty nature and higher moisture retention of that sample.

**TABLE 1. Estimated effective porosity of different sample cores**

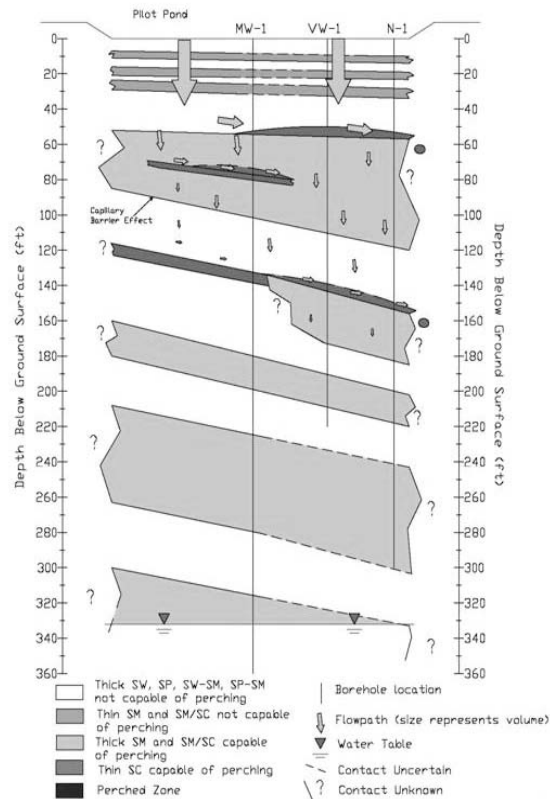
| Sample       | Saturated<br>Water Content<br>cm <sup>3</sup> /cm <sup>3</sup> | Volumetric Water<br>Content |                                           | Estimated Effective Porosity |                                                          |
|--------------|----------------------------------------------------------------|-----------------------------|-------------------------------------------|------------------------------|----------------------------------------------------------|
|              |                                                                | at 1/3<br>bar               | at $K_{\text{unsat}} =$<br>$10^{-7}$ cm/s | at 1/3 bar <sup>a</sup>      | at $K_{\text{unsat}} =$<br>$10^{-7}$ cm/sec <sup>b</sup> |
| MW1          | 0.28                                                           | 0.185                       | 0.208                                     | 0.095                        | 0.072                                                    |
| NL 286-286.5 | 0.308                                                          | 0.129                       | 0.162                                     | 0.179                        | 0.146                                                    |
| NL 54-54.5   | 0.28                                                           | 0.07                        | 0.097                                     | 0.21                         | 0.183                                                    |
| VW 120-121-2 | 0.267                                                          | 0.073                       | 0.082                                     | 0.194                        | 0.185                                                    |
| VW 160-161-2 | 0.334                                                          | 0.144                       | 0.146                                     | 0.19                         | 0.188                                                    |
| VW 220-2     | 0.295                                                          | 0.106                       | 0.12                                      | 0.189                        | 0.175                                                    |

<sup>a</sup> Effective porosity assumed to be the difference in water content saturation and at 1/3 bar.

## Perching and Lateral Spreading

Perched water was not observed at any depth prior to commencement of recharge, despite on-going furrow irrigation of the area since the 1960s. This is based on data from the following sources: drill and core logging, cone penetrometer tests (CPT), air and water piezometer monitoring, and advanced tensiometer and neutron logging data. The initial CPT survey data indicated relatively thin and laterally discontinuous fine grained layers are present from 0 to 60 ft (0 to 18 m) below ground surface. However, they were not observed to result in measurable perching or lateral spreading of recharged water in the follow-up CPT survey conducted during recharge operations.

Advanced tensiometers in VW-1 indicate that temporary perching of applied basin water did occur above various fine grained intervals at depths of approximately 55 and 75 ft (17 and 23 m); however these perched zones dissipated within four to six weeks after forming (Figure 5). This conceptual figure also shows that beginning at approximately 130 ft (40 m) bgs, more continuous and thicker fine-grained layers were present and the occurrence and frequency of thicker fine-grained layers increases with depth. These layers created sustained perched water zones throughout the recharge period however, mounding above these layers was limited in depth and dissipated 3 to 12 weeks after cessation of water application. The observed wetting front advancement and limited observed perching indicate that water is able to percolate around/through these deeper fine grained layers and that they should not negatively impact recharge operations.



**FIGURE 5. Conceptual subsurface lithologic layering.**

Although recharge water was observed to reach the water table, it is believed that some water also spread laterally and remains in the deeper vadose zone (i.e.

between 130 to 332 ft (40 to 100 m)). It is expected that full scale recharge operations will cause the water table to rise into and merge with recharge water stored within the deeper vadose zone. Since the groundwater table will be maintained below 130 ft (40 m) bgs through pumping of recovery wells, agricultural operations should not be impaired by recharge operations.

### **Water Quality Impacts**

Prior studies indicated the potential for arsenic and chromium to leach from subsurface sediments during recharge operations. Arsenic was not detected in any of the 66 suction lysimeter (vadose zone) and groundwater samples for this investigation, or in any prior investigations. Chromium was occasionally observed above the California primary Maximum Contaminant Levels (MCL) in the suction lysimeter samples. However, concentrations in water piezometer samples and groundwater have remained at background levels and speciation analyses of lysimeter samples indicated that soluble hexavalent chromium was not present at concentrations greater than 0.013 mg/l. Thus, the high chromium concentrations appear to consist of insoluble (i.e. colloidal) trivalent chromium that was pulled into the suction lysimeter.

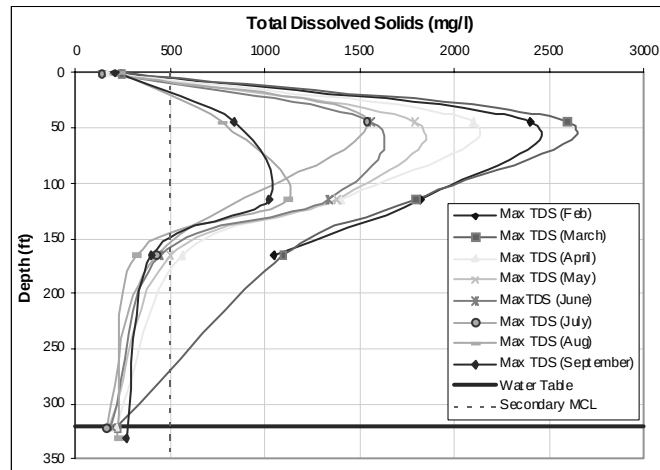
As expected, nitrate and total dissolved solids (TDS) were present in the vadose zone pore water above MCL and secondary MCL (SMCL) levels due to ancestral agricultural drainage. However, nitrate and TDS concentrations decreased with depth in suction lysimeters and baseline concentrations in groundwater in MW-1 were below MCL and SMCL levels. During recharge, TDS concentrations consistently decreased at all vadose zone depth intervals over time (Figure 6 shows TDS data).

Additionally, concentrations at the water table remained below MCL and SMCL levels as of mid October 2006, approximately 7 months after recharged water first reached the water table.

Mass balance calculations based on laboratory leaching tests and suction lysimeter data indicate that TDS and nitrate concentrations in the vadose zone should be diluted to below SMCL and MCL levels within approximately 2 and 5 recharge seasons, respectively. Mass balance calculations based on the estimated historical deposition of agricultural salts yield estimates of approximately one-half the suction lysimeter estimates under equilibrium mixing assumptions.

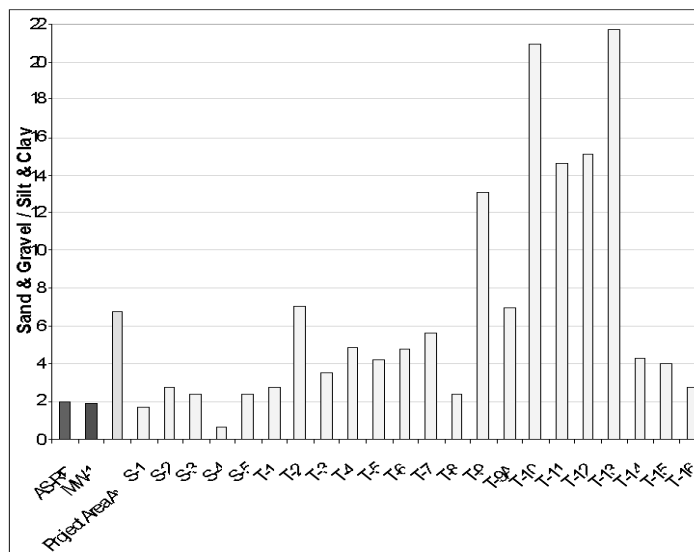
After approximately 1.5 recharge seasons of the pilot test, TDS and nitrate are below or slightly above the California MCL and SMCLs. During the life of the project, concentrations in agricultural and recovery wells are not expected to rise above MCL and SMCL levels because of dilution in the vadose zone and within the screened intervals of these wells.





**FIGURE 6. Vadose zone TDS concentrations vs. depth over time.**

### Representativeness of Pilot Recharge Basin Site



**FIGURE 7. Ratio of sand and gravel to silt and clay in surface samples.**

Data indicate that near surface soils (upper 10 ft) in the Pilot basin area are finer grained than soils underlying approximately 80 % of the full scale Project area. For example Figure 7 shows that the 2 pilot basin surface samples (AS-RP and MW-1) contain significantly less sand and gravel and more silt and clay than 19 of the other 22 surface samples collected

throughout the Project area. Since there is typically an inverse correlation between the amount of fines and the hydraulic conductivity of porous media, the pilot basin

test likely provides a conservatively low estimate of the infiltration capacity of near surface soils within the Project.

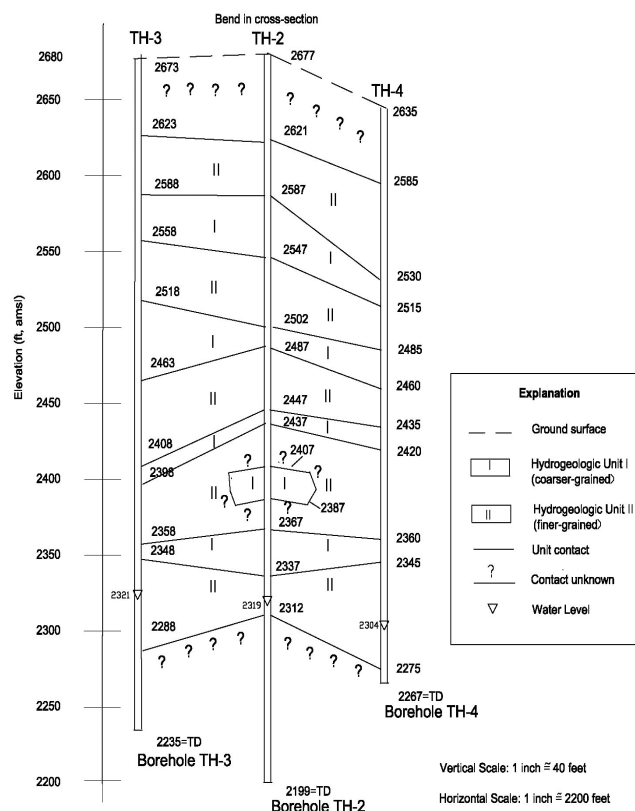
The subsurface lithologic conditions and recharge performance in the northern portion of the Project area should also be applicable to the central and southern portions of the Project area where exploratory and monitoring wells were not drilled for the following reasons.

- Nearly all irrigation wells spread over the Project area are perforated between 250 and 1000 ft (76 to 305 m) bgs and support pumping from 1000 to 2000 gpm (227,000 to 450,000 l/s) indicating that the deeper subsurface saturated sediments are coarse-grained and highly permeable throughout the Project area.

- Previous studies (Bloyd, 1967a and b) suggest that the overlying unsaturated sediments will also be similar or coarser-grained in texture throughout the Project area.

## CONCLUSIONS

The Antelope Valley Water Bank pilot test has demonstrated that water can be recharged to the water table at rates that exceed project requirements and that recharge and nearby agricultural operations should not be impaired by perching of percolating water. Based on other Project area surface and subsurface data, the pilot test results appear to provide a conservative indication of performance of the larger Project area. This test also corroborates specific yield assumptions that were used



**FIGURE 8. East-west lithologic cross section through northern Project area.**

to previously estimate an available storage capacity of 500,000 acre-feet (616,740,000 m<sup>3</sup>) in the dewatered portion of the aquifer.

Suction lysimeter water quality data showed that that arsenic was non-detect and that although chromium occasionally exceeded the MCL, the majority of chromium was insoluble (trivalent) chromium that had been pulled into the suction lysimeter. As expected, nitrate and TDS are present in the vadose zone at significant levels due to prior agricultural operations. Mass balance estimates indicate that vadose zone TDS and nitrate concentrations should be diluted to below SMCL and MCL levels within 2 to 5 recharge seasons. During this time, concentrations in agricultural and recharge recovery wells are not expected to rise above the SMCL and MCL levels because of dilution that will occur in the lower vadose zone and across several hundred feet of well screen within the aquifer below the recharge zone. As of February 2007, arsenic, chromium, TDS and nitrate concentrations in groundwater immediately below the pilot basin have remained below SMCL and MCL levels.

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## ACCUMULATION AND LEACHING DURING ARTIFICIAL RECHARGE COMPARED TO A NATURAL RECHARGE SYSTEM, IN THE NETHERLANDS

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### ABSTRACT

Basin recharge is applied in the Netherlands for drinking water supply since 1940. In order to evaluate the chemical sustainability of these systems, where mainly pretreated Rhine and Meuse River waters are being infiltrated, a geochemical study was carried out. The focus was on (a) the possible accumulation of inorganic and organic micropollutants (including trace elements like Ag, Ba, Cd, Hg, Sn, Tl, V and Zn, and organics like PAHs, PCBs and organochlorine pesticides), and (b) the leaching of reactive aquifer components like CaCO<sub>3</sub>, MnO<sub>2</sub>, Soil Organic Material (SOM) and pyrite.

In the ca. 45 years old artificial recharge areas of the cities of Amsterdam and The Hague 118 cores of sludge from recharge basins and of aquifer material were taken, in each along 2 transects between a recharge basin and its recovery. In addition, next to each artificial recharge area, in coastal dunes, also 2 drillings in 200-3500 years old, natural dune sand without artificial recharge were sampled and analyzed.

The data permitted to construct a map showing the penetration depth of pollutants (0.05 to 40 meters) introduced by either artificial recharge or atmospheric deposition. It appeared that Cd, Cu, Sn, Tl, Zn, PAHs, PCBs and pentachlorobenzene penetrated deeper in the systems with artificial recharge than in the systems with natural recharge only. The overall levels were, however, very low.

The leaching rate of CaCO<sub>3</sub> and MnO<sub>2</sub> in the artificial recharge areas was about 0.04 m/a, being about 65 times more rapid than in dunes without artificial recharge. The leaching of SOM and pyrite could not be demonstrated but could be quantified with a well-validated transport code (Easy-Leacher). These calculations indicate that Fe(OH)<sub>3</sub>, FeS<sub>2</sub> and SOM are leached at a very low rate.

Thousands of years appear to be needed for leaching all reactive aquifer components. This, in combination with reduced accumulation rates thanks to enhanced pretreatment, justifies the conclusion that the artificial recharge systems operate in a geochemically sustainable way.

## KEYWORDS

Artificial recharge, coastal dunes, accumulation, leaching, geochemistry, heavy metals, organic micropollutants, sustainability

## INTRODUCTION

Basin recharge systems are used in the Netherlands for drinking water supply since 1940, with the larger systems starting around 1955. Today they satisfy ca 13% of the annual total of drinking water demand, the latter being 1,250 Mm<sup>3</sup>/a. Most systems are situated in the coastal dunes which form an important nature reserve. In order to reduce the environmental impact of these recharge systems on the surrounding dunes various adaptations were introduced since 1974. The most important one was the extension of the pretreatment with a coagulation step, which improved the quality of the infiltration water and reduced the frequency of sludge removal from the basins.

Nevertheless the public and governmental organizations continue to question the sustainability of the recharge operations. Their incentives are based on: (a) the negative image of Rhine and Meuse river water which is used for infiltration (after pretreatment), due to quality problems; (b) the unnaturally high water flux in areas with artificial recharge; (c) problems with eutrophication of the recharge basins and phreatophytic plant communities in surrounding dune areas (Van Dijk & De Groot, 1987); (d) the general perception that man is anyhow causing unrepairable damage where intensively utilizing nature; and (e) mooring of the sustainability principle into national environmental policy.

After more than 45 years of artificial recharge the time was considered mature to make up a balance; this time not addressing the hydrochemical status (Stuyfzand, 1989, 1998) but the geochemical status of the flushed aquifer systems. The following questions were raised and will be answered in this contribution:

8. To which extent do pollutants accumulate and are the natural aquifer constituents being leached, in the basin recharge systems in the coastal dunes of the Western Netherlands?
9. How will the processes of accumulation and leaching evolve on the long term?
10. How do these processes relate to those in coastal dunes without artificial recharge?
11. Which are the consequences of the answers to questions 1-3 for the geochemical sustainability of the artificial recharge systems?

## MATERIAL AND METHODS

Two dune areas with artificial recharge were investigated on their geochemical characteristics: one north of The Hague (operated by Duinwatermaatschappij Zuid-Holland) and one south of Zandvoort (operated by Waternet supplying drinking water to the city of Amsterdam). In both areas sediment cores were taken by a core catcher during bailer drilling, in total 118. This was done, in each dune area, along a row in between 2 recharge basins and their recovery, and on 2 natural dune plots without artificial recharge (resp. just behind the foredune ridge and 3-4 km inland). The cored transects north of The Hague are shown in Fig.1.

The cores were stored at 4°C in the dark before analysis (within 1-7 days) on: their grain size distribution, 19 bulk parameters (like organic carbon, total C, N and S, CEC, CaCO<sub>3</sub>), elemental composition incl. 60 Inorganic Micropollutants (IMPs) like heavy metals, and 190 Organic Micropollutants (OMPs like PAHs, OCPs, PCBs, chlorinated hydrocarbons, BTEX, alkylphthalates and organophosphorus pesticides). Conventional techniques were applied: for the inorganics laser particle size analysis, thermogravimetry, pyrolysis, XRF, and ICP-MS after HNO<sub>3</sub> extraction; and for most OMPs either GC-MS or GC-ECD/FID after extraction with either acetone/hexane, methanol or chloromethane.

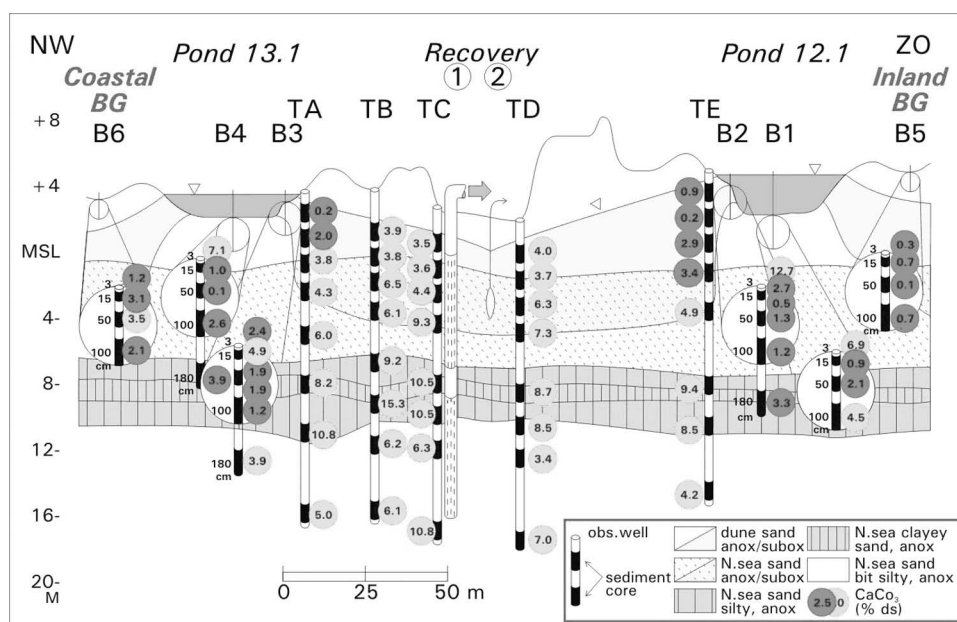
The accumulation and leaching have been quantified, after (a) determination of the natural background levels for coastal dune sand on the basis of ca. 100 samples from various studies in the past (Stuyfzand, 2003); and (b) determination of penetration and leaching depths of substances for selected soil types.

Subsequently the expert model Easy-Leacher 4.8 (Stuyfzand, 2005) was used, after calibration on already available water quality data and the new geochemical data, for calculating the advance of accumulation and leaching fronts in the aquifer, during infiltration of relatively polluted infiltration water (before 1975) and relatively clean infiltration water (since 1975).

## ACCUMULATION

Pollutants accumulate in sludge (neoformed sediment) at the bottom of recharge basins, and in the 'flushed' sandy aquifer material. The sludges investigated here, belong to the so-called second generation (Stuyfzand & Mosch, 2002): they formed after removal of first generation sludges that relate to the recharge period before 1974 when pretreatment was less intensive (aeration, sedimentation and slow sand filtration) and when the river waters were more polluted. Consequently the sludges investigated here are thinner (2-5 cm, accretion ca. 2mm/a) and much less polluted (close to natural background or target values).

Accumulation levels in the flushed aquifer are quite low (for all parameters at or below natural background), and can be only demonstrated for most substances within the basin proximal zone (<5 m distance from the basin banks; Fig.2). The trace elements Ag, Cd, Cr, Tl, V and Zn as well as some PAHs also accumulate beyond the basin proximal zone, but at a very low level (Fig.5).



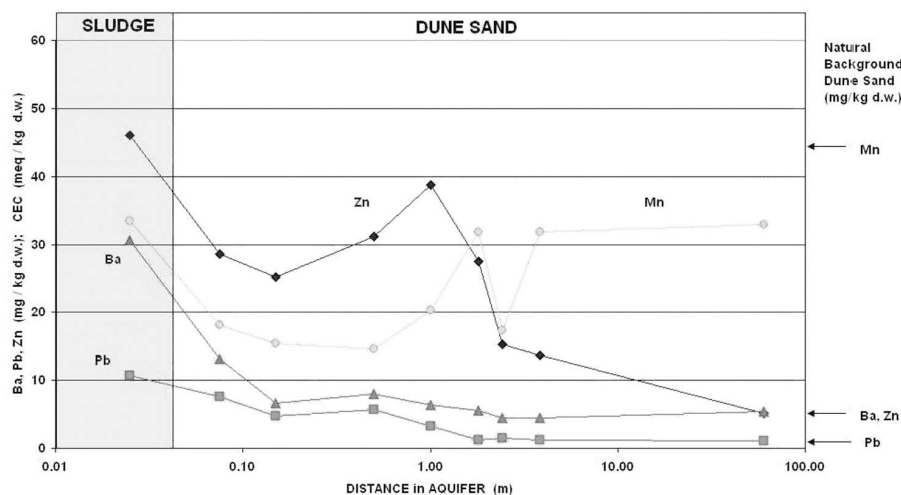
**FIGURE 1:** Transect near The Hague over 2 recharge ponds, with the observed calcium carbonate contents and redox zonation (suboxic =  $O_2$  and/or  $NO_3 > 0.5$  mg/L; anoxic =  $O_2$  and  $NO_3 < 0.5$  mg/L). Natural background logs are projected: B6 (calcareous young dunes just behind the foredune ridge) and B5 (decalcified old dunes 3 km more inland). Red/green bullets = yes/no evidence of decalcification.

## LEACHING

Hydrochemical research clearly demonstrates that dune infiltration should lead to the leaching of calcium carbonate, soil organic material (SOM), pyrite, manganese, iron and silicon compounds (Stuyfzand, 1989). The current geochemical survey confirms this regarding calcium carbonate (Figs.1, 4) and manganese (Fig.2), at least in dune sand. There are some weak indications of leaching of feldspars and biotite, but these are statistically not significant enough.

In the year 2002 calcium carbonate and manganese oxides were leached for 40-80% in the sandy basin banks (where most water infiltrates), over a distance of 1.5-3 meter.

Beyond this boundary both minerals still dissolve, but at a lower (Mn) and much lower rate (calcium carbonate).



**FIGURE 2: Plot of soil element contents along a flow line from recharge pond 12.1 towards the recollecting drain (Hoofdader, The Hague). Accumulation of barium, lead and zinc, and leaching of manganese (especially between 0.03 and 3 m). The zinc peak at 1 meter reflects the accumulation during a more polluted past.**

### Developments on the Long Term

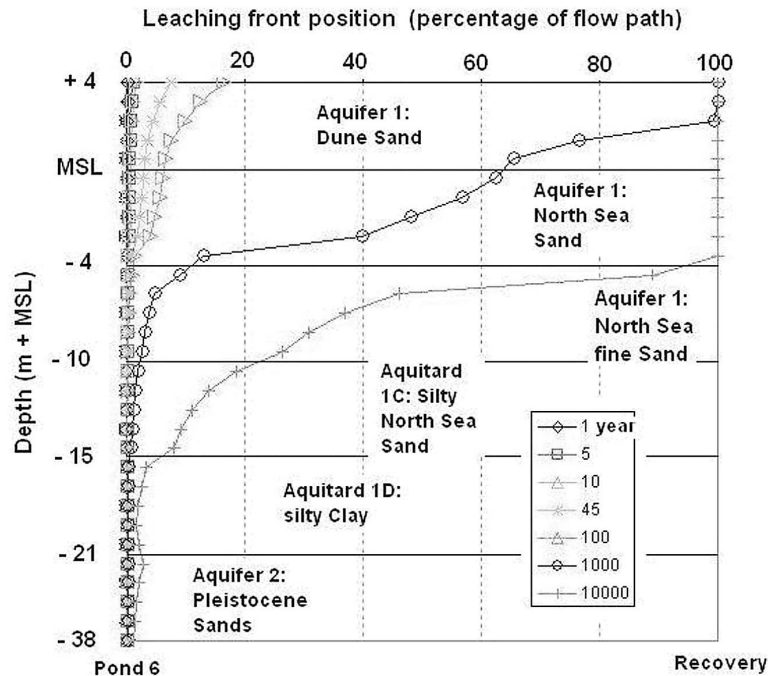
Calculations by the expert model Easy-Leacher yield the following results for both AR areas:

- The complete leaching of calcium carbonate from each of the flushed aquifer layers takes thousands to ten thousands of years (Fig.3). The leaching of pyrite in dune sand (negligible content) should have been completed long ago, in the underlying North Sea sands the leaching should be halfway by now, and below those marine sands (>5 m-MSL) it will take (tens of) thousands of years.
- The leaching of labile SOM and manganese probably requires several hundreds to thousands of years;
- The change over, around 1974, from relatively polluted infiltration water aggressive towards calcium carbonate, to less polluted and less aggressive infiltration water has led to a drastic decline of the leaching rate for calcium carbonate (with a factor of 10) and an acceleration of the leaching rate (1.5-2 times) for oxidizable materials like SOM and pyrite. In addition the composition of the sludge and dune sand improved significantly by a reduction in the



accumulation of IMPs (with a factor of 10-100) and OMPs (with a factor of 10-10,000).

- The composition of sludge and the flushed dune sand will remain in good chemical condition, provided that processes, like chemical precipitation (IMPs) and deep bed filtration of particles (IMPs and OMPs) suspended in the infiltration water, remain restricted to the basin proximal zone.



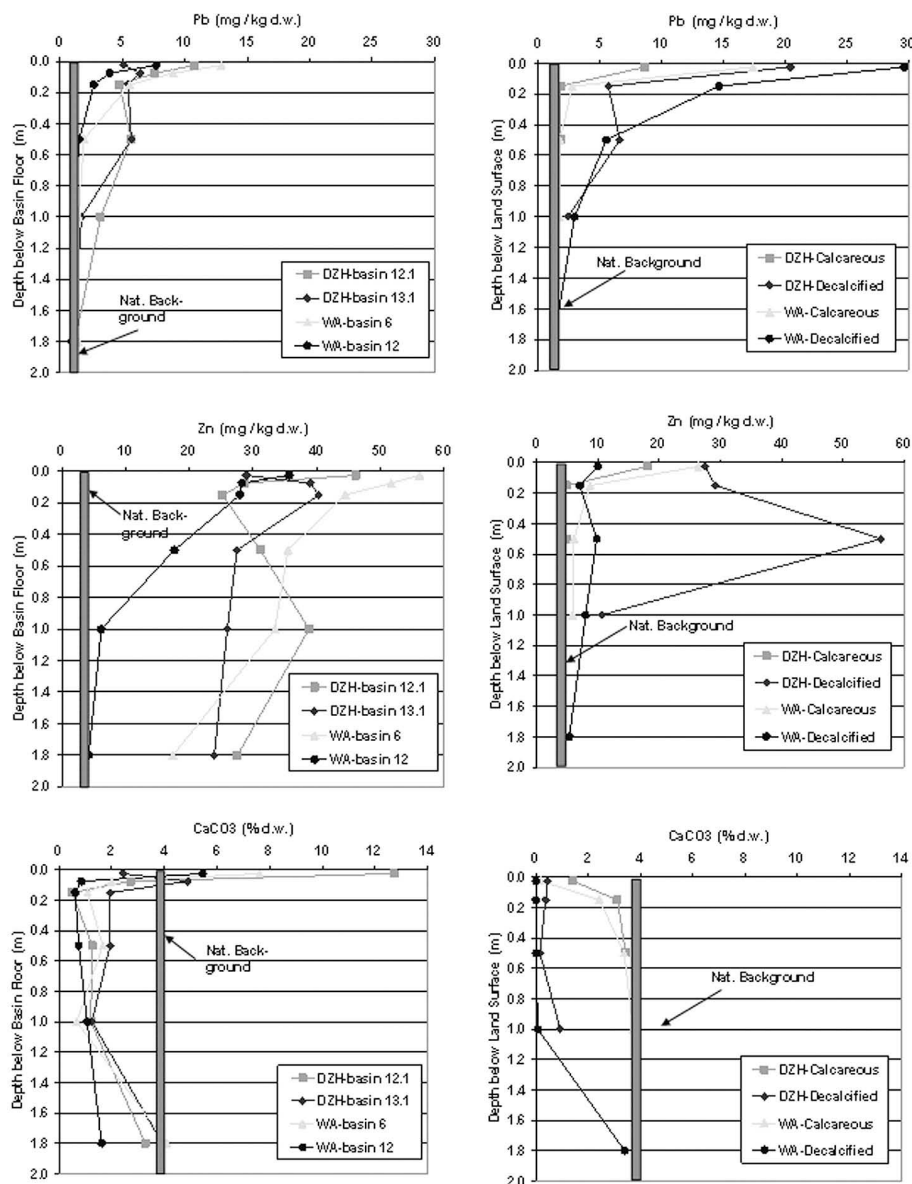
**FIGURE 3: Worst-case progress rate of the decalcification front between recharge basin 6 and the recovery in recharge area Zandvoort, if continuously recharged with low quality Rhine water as in the period 1970-1973. The current decalcification rate proceeds 10 times slower! X-axis: 0 = in recharge basin 6; 100 = in draining recovery canal (Westerkanaal). 1, 5, 10, 45, 100, 1,000, 10,000 = number of years since start of artificial recharge.**

### Comparison with Natural Dunes Lacking Artificial Recharge: Accumulation

Dune sand including its top layer (neoformed sludge) in AR systems (exclusively in saturated zone) was compared with natural dune soils (without AR, in unsaturated zone), up to a depth of 1.8 m below land surface.

In AR systems the dune sand and top layer exhibit a higher content of calcium carbonate (Fig.4) and associated trace elements (Sr, Mg, Co, Ni), higher concentrations of pollutants typically associated with surface water (Ag, Ba, Cu, Mo,

Ni, Tl, W, Zn [Fig.4], polychlorobiphenyls [PCBs], hexachlorobenzene [HCB] and pentachlorobenzene [QCB]), and a lower content of substances deriving from atmospheric deposition (Pb [Fig.4], polycyclic aromatic hydrocarbons [PAHs], 4,4-DDE, 4,4-DDT and various organochlorine pesticides [OCPs]).

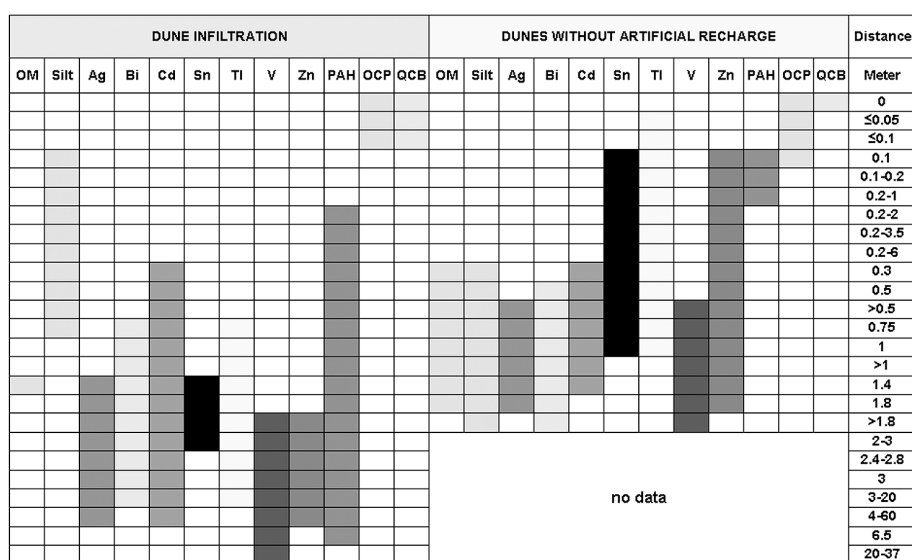


**FIGURE 4: Comparison of accumulation (Pb and Zn) and CaCO<sub>3</sub> leaching, between dunes with (left) and without artificial recharge (right). DZH = The Hague area; WA = Zandvoort area**

The range of penetration depths of pollutants is shown in a kind of soil chromatogram (Fig.5). Penetration depths are larger in AR systems than in natural dunes (without AR), regarding: Cd, Cu, Sn, Tl, Zn, PAHs, PCBs and QCB. This probably relates to the much higher flux and lower biodegradation rate of PAHs, QCB and PCB in a less aerobic environment in the saturated zone of infiltration systems.

When we compare the resulting SOPI-scores (SOPI = Soil Pollution Index; Stuyfzand, 2002) then AR systems do not score always better or worse than 'natural' dune systems. As far as the heavy metals, PCBs and OCPs are concerned, all cores can be denominated more or less 'clean' (better than target value). Only petroleum ether extractable organohalogenes [EOX] and PAHs exceed the target values, in dunes without and with AR.

So dunes with and without AR exhibit quite comparable accumulation levels of pollutants. However, the period it took to accumulate these levels, differs significantly (see below).



**FIGURE 5: Spectrum of penetration depths of substances introduced at the land surface (soil chromatogram) for on the one hand the 4 plots with artificial recharge and on the other hand the 4 dune plots with natural recharge. OCP = OrganoChlorinePesticides; OM = Organic Matter; QCB = pentachlorobenzene; Silt = particles <16 µm.**

### Comparison with Natural Dunes Lacking Artificial Recharge: Leaching

The differences in leaching depths and degree of leaching (%) are not large, between dunes with and without artificial recharge (AR), regarding calcium carbonate (Fig.4) and manganous oxides. However, the time needed for this leaching strongly

diverges. The mean decalcification rate in dunes with AR is 4 cm/a and in dunes without AR 0,06 cm/a. Decalcification in AR systems is therefore 67 times faster.

The leaching rates in AR-systems for calcium carbonate and manganese have strongly decreased, however, since 1975 when the infiltration water became much less aggressive towards  $\text{CaCO}_3$  and more aerobic. The leaching rates in dunes with AR are nevertheless still considerably higher (10-100 times) than in adjacent natural dunes. This fact also holds for pyrite and SOM, according to calculations with computer code Easy Leacher 4.8.

**Table 1: Processes with an impact on the geochemical sustainability of artificial recharge systems, with consequences and resulting problems.**

| PROCESS                               | CONSEQUENCES                                                                                                                                                                                                                                                                                                                                                                                                                   | PROBLEMS                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                                       |
|---------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| Accumulation of sludge in basins      | <ul style="list-style-type: none"> <li>• Stagnation of infiltration process</li> <li>• Eutrophication of surface water</li> <li>• Increase of chemical clogging of recovery system</li> <li>• More denitrification</li> <li>• More removal of (sub)oxic OMPs</li> <li>• Less leaching of aquifer below sludge</li> </ul>                                                                                                       | <p>None, because:</p> <ul style="list-style-type: none"> <li>• Easy to remove</li> <li>• Quality now already very good</li> <li>• Accretion rate slow</li> <li>• prognosis = in future even cleaner</li> </ul>                                                                                                                                                                                                                                                                                                                                                                                                                                                                 |
| Accumulation of pollutants in aquifer | <ul style="list-style-type: none"> <li>• Stagnation of infiltration process</li> <li>• Worsening of quality recovered water</li> <li>• Ecological threatening of impacted flow-through lakes in surroundings</li> <li>• Improved removal of pathogens</li> </ul>                                                                                                                                                               | <p>Hardly, because:</p> <ul style="list-style-type: none"> <li>• To a very limited extent only (low contents)</li> <li>• over a small area (proximal zone)</li> <li>• self purifying capacity of soil high</li> <li>• prognosis = inf.water becomes cleaner and flushes out ancient pollutants</li> </ul>                                                                                                                                                                                                                                                                                                                                                                      |
| Leaching of aquifer materials         | <ul style="list-style-type: none"> <li>• Loss of capacity to buffer acids and oxidants, and to sorb pollutants</li> <li>• Increase of <math>\text{NO}_3</math> and (sub)ox. OMPs#</li> <li>• Decrease of Fe, Mn, <math>\text{NH}_4</math> (less sludge)</li> <li>• Decrease of clogging of partly aerobic recovery systems</li> <li>• Possibly an increase of clogging of currently totally anoxic recovery systems</li> </ul> | <p>Not of serious concern, because:</p> <ul style="list-style-type: none"> <li>• Expected to decrease in consequence of EU-Water Framework Directive;</li> <li>• Raised SI-calcite and better quality of infiltration water already reduced <math>\text{CaCO}_3</math> leaching</li> <li>• Compensation by positive consequences: lower Fe and Mn concentrations of recovered water lead to reduced sludge production in post treatment</li> <li>• Although leaching in AR systems is relatively rapid, in absolute terms it is very slow</li> <li>• Loss of acid buffering capacity does not lead to acidification thanks to basic character of infiltration water</li> </ul> |

## **Conclusions Regarding the Chemical Sustainability of Dunes with Artificial Recharge**

The most important geochemical processes with an impact on the sustainability of AR systems are, according to Table I: (a) the accumulation of sludge in infiltration ponds, (b) the accumulation of pollutants (especially IMPs and OMPs) in the dune sand, and (c) the leaching of the dune sand.

The extent to which these processes manifest themselves is expected not to conduce to serious problems, which is explained in Table 1. Artificial recharge as applied here, is therefore considered a sustainable activity from a geochemical point of view!

In other words:

- the processes of leaching and accumulation are, in their present extent, comparable to the natural situation, but they proceed in AR systems much more rapidly. In the next decennia they will remain restricted to a small part of the flushed aquifer (<5 meter);
- the purifying processes during aquifer passage will continue to operate without significant changes in pH and redox environment, provided that sludge is not allowed to significantly hamper the recharge process;
- the existing infrastructure (notably the recharge basins and recovery means) will 'on average' not suffer from a (strongly) declining performance; and
- the water supply companies can continue, at least for decennia, with recharging and recovering the infiltrated pretreated surface waters, with in our opinion acceptable geochemical consequences.

Relatively new questions about the chemical sustainability of AR systems regard the behavior of emerging pollutants like pharmaceuticals and endocrine disrupting compounds during aquifer passage, and their impact on the ecosystem within and around the recharge areas.

## **ACKNOWLEDGEMENTS**

Water supply companies DZH, Waternet and PWN are acknowledged for their support in carrying out this research.

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## AQUIFER STORAGE AND RECOVERY IN FLORIDA: GEOCHEMICAL ASSESSMENT OF POTENTIAL STORAGE ZONES

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### ABSTRACT

In Florida, 34 aquifer storage and recovery (ASR) facilities are either in testing or operation. Approximately an equal number are pending or proposed, and the Comprehensive Everglades Restoration Plan (CERP) is considering up to 333 ASR wells. Eocene-Miocene carbonates of the Floridan aquifer system comprise the storage zone for these wells. Water-rock interactions resulting in mobilization of As, Mo, Sb, and U have been documented at several of the existing facilities. For example, As concentrations exceeding the federal drinking water standard have been observed in at least 14 ASR wells, and in some cases, associated monitor wells. Arsenian pyrite is the dominant As-bearing phase in these upper Floridan aquifer rocks. Desorption or oxidative dissolution is the most likely mechanism for As mobilization during initial recharge; however, additional reactions are involved due to variable redox conditions during subsequent phases of cycle testing (i.e., recharge, storage and recovery).

Bench-scale and other laboratory analyses have been employed to provide baseline information on the potential for mobilization of metals during ASR in Florida. Although not yet calibrated or validated by field-test results, bench-scale leaching experiments utilizing source water, native ground water and cores from wells at proposed ASR sites reveal desorption of As, Mo, Sb, Tl, U, V, and other metals from the aquifer matrix. Moreover, removal of As from leachates at high oxidation-reduction potential levels suggests re-sorption possibly during precipitation of hydrous ferric oxyhydroxides. Comprehensive geochemical characterization of the aquifer matrix, supplemented by bench and sequential extraction studies may serve as predictive tools regarding the potentially adverse water-quality changes at future ASR sites. This framework of knowledge can be supplemented by geochemical modeling to improve ASR system design, testing and operations.

### KEYWORDS

Arsenic, aquifer storage and recovery (ASR), bench-scale leaching, hydrogeochemistry, sequential extraction

### INTRODUCTION

Although aquifer storage and recovery (ASR) is viewed as a highly viable water-resource alternative across the globe, it is becoming more recognized that water-quality issues due to ASR activities exist in certain hydrogeological settings (e.g.,

Stuyfzand, 1998; Arthur et al., 2001; Roth, 2004; Mirecki, 2004; Arthur et al., 2005). On the other hand, ASR activities are known to yield water-quality improvements (Dillon and Toze, 2005). At present, 34 ASR facilities (some with multiple ASR wells) are operating in Florida. Cycle-test data from approximately 14 of these ASR wells reveal arsenic (As) concentrations exceeding the U.S. Environmental Protection Agency (EPA) maximum contaminant level (MCL) for drinking water (10 µg/L). Although less common, As exceedences have also been observed at monitor wells. The world's largest ecosystem restoration, the Comprehensive Everglades Restoration Plan (CERP), is underway in Florida. Implementation of up to 333 ASR wells is proposed as a key component of the plan.

During a typical ASR cycle test utilizing a confined Floridan aquifer system storage zone, recharge of oxygen-rich source water (SW) displaces oxygen-depleted (e.g., low oxidation-reduction potential [ORP]) native ground water (NGW). Equilibration of injection water pH with the aquifer materials (e.g., carbonate buffering) begins within the aquifer during ASR recharge and storage. The oxidation-reduction potential (ORP) also begins to decline as dissolved oxygen is consumed by redox reactions (Mirecki, 2004). During the ASR recovery phase a reversal occurs with regard to redox conditions: SW is pumped and NGW begins to migrate toward the ASR well, which is accompanied by increased reducing conditions.

Oxidation of organic matter (OM) or pyrite was believed to be the initial source of As in ASR waters (e.g., Arthur et al., 2001; Price and Pichler, 2006). Indeed analyses of pyrite in the upper Floridan aquifer (UFA) matrix and its confining sediments confirm As concentrations on the order of  $10^2$  to  $10^3$  ppm (e.g., Lasareva, 2004; Price and Pichler, 2006; Arthur, et al., in preparation). Electron probe microanalyses (EPMA) and sequential extraction analyses also confirm that sulfide minerals (as opposed to oxides or OM) are the dominant, but not only, sink for As in these rocks (Price and Pichler, 2006; Arthur et al., in preparation).

Based on the dynamic redox setting within ASR storage zones, a working hypothesis has emerged with regard to As fate and transport (Mirecki, 2006; Stuyfzand, 2006 [personal communication]; Arthur et al., submitted): 1) as recharged/injected oxic SW permeates the aquifer matrix, initial mobilization of As occurs via oxidative dissolution of pyrite and to a lesser extent, oxidation of OM; 2) if sufficient dissolved Fe is present, hydrous ferric oxide (HFO) precipitation occurs, on which As may sorb depending on competing ion concentrations (e.g.,  $\text{HCO}_3^-$ ,  $\text{PO}_4^{3-}$ ); 3) during recovery, HFOs experience As desorption as de-oxygenated SW (highly negative ORP) and/or an increasing proportion of reduced NGW migrates toward the ASR well. Gotkowitz et al. (2004) report an analogous As-mobilization scenario for pumping wells in a confined aquifer.

Certainly many variables and unknowns exist with respect to this hypothesis, including effects of pressure, temperature, microbial activity, dissolved organic



carbon, variable SW and NGW compositions, aquifer matrix chemistry/mineralogy and reaction kinetics in a dual-porosity system. Furthermore, this emerging hypothesis is not without contradiction. For example, geochemical modeling using mixtures of SW and NGW in southwest Florida (in the absence of aquifer matrix material) do not indicate HFO stability (Jones and Pichler, 2007). In addition, the behavior of As in the ASR hydrogeochemical setting is not the only potential concern. Other metals (e.g., Mn, Ni, V and U) have been observed in concentrations above background NGW during ASR recovery (Arthur et al., 2005). Analyses of water samples collected during cycle-tests at some Florida ASR sites reveals that Mo<sup>1</sup>, Sb and gross alpha are locally of concern. These observations, variables and uncertainties, underscore the importance of geochemical characterization of the aquifer matrix, as well as consideration of NGW and SW compositions in relation to water-rock interactions.

Acknowledging the above statement, comprehensive geochemical and mineralogical characterization of ASR storage zone rocks is a task in the CERP ASR Regional Study (U.S. Army Corps of Engineers and South Florida Water Management District, 2003). Whole-rock geochemical analyses of 315 aquifer matrix samples have been completed. Mineralogy and mineral chemistry were accomplished by lithologic descriptions, petrography, X-ray diffraction (XRD) analyses (bulk powder, clay separates and insoluble residues) and EPMA. Stable and radiogenic isotopic analyses of more than 40 samples were completed as well as a five-step sequential-extraction procedure, which identifies bond relations between minerals or OM and metals (including metalloids for discussion). Bench-scale leaching studies utilizing core chips, SW and NGW in sealed reaction vessels allow assessment of metals mobilization and precipitation of metals in response to changes in pH, dissolved oxygen (DO) and ORP.

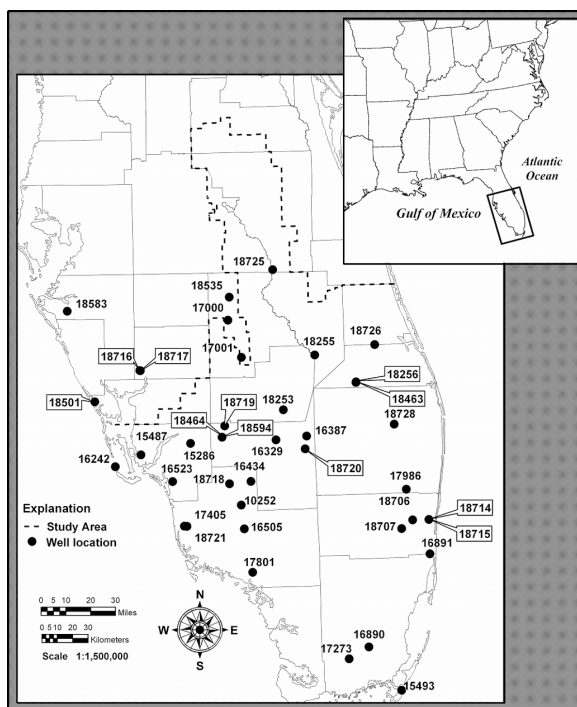
While not intending to understate the importance of each analytical method described above, this paper highlights a subset of the overall methods and results that are somewhat unique with respect to the CERP ASR geochemistry project (Arthur et al., in preparation). It is noteworthy that our approach to mineralogical and geochemical characterization of the aquifer storage-zone matrix and assessment of water-rock interactions at the bench scale is not a new concept (e.g., Pyne, 2005). On the other hand, these methods historically have not been collectively applied to ASR sites to the extent outlined herein.

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<sup>1</sup>. Although an EPA MCL does not exist for Mo, the World Health Organization 2004 Guideline Value for Mo = .07 µg/L.

## STUDY AREA

The CERP study area generally includes the southern third of peninsular Florida and encompasses 46,620 square km. Core samples from 39 wells in the region are included in this study (Figure 1). These wells provide rock samples ranging from the lower Hawthorn Group (~ 120 m below land surface [BLS]) to the Avon Park Formation (~400 m BLS). Where core coverage was limited, borehole cuttings were used; however, these samples are not preferred due to high probability of contamination (e.g., cavings and drilling mud). For this reconnaissance level approach, approximately 8 samples per well were collected per core.



**FIGURE 1: Study area and well locations. Labels are Florida Geological Survey well accession numbers.**

## HYDROGEOLOGIC SETTING

For many of Florida's ASR storage zones, overlying confinement includes intermediate aquifer system/intermediate confining unit strata, which correspond to the Oligocene to Pliocene Hawthorn Group. Storage zones utilized by ASR wells occur primarily within the UFA. In south Florida, the UFA consists of the Middle Eocene Avon Park Formation, the Upper Eocene Ocala Limestone, and the Lower Oligocene Suwannee Limestone; however, in some locations, the mixed carbonates-siliciclastics of the Oligocene-Pliocene Hawthorn Group are also part of the UFA (Reese, 2002). More commonly, Hawthorn Group rocks and sediments comprise the requisite overlying confinement of the ASR storage zone.

## METHODS

Several lithologic and hydrogeologic characteristics were considered during sampling, including permeability, porosity, and samples corresponding to

anomalous gamma-ray activity. Existing lithologic descriptions also helped identify wells and depth-intervals of interest. Although the sampling plan emphasized representative, homogenous lithologies, approximately 25% of the samples were collected to represent anomalous features (e.g., chemically reduced or oxidized zones, organic-rich sediments, sulfide minerals, interlayered carbonates/siliciclastics). Indurated core samples were trimmed to remove all exposed surfaces as well as potential drilling mud impregnated along the core exterior, post-drilling oxidized surfaces and precipitates (e.g., gypsum formed during core storage). After trimming, each sample was rinsed with de-ionized water and air-dried in a contamination-free environment. Crushing and pulverization were completed at a commercial laboratory.

*Whole-rock chemistry.* Whole-rock geochemical analysis was completed using a multi-method approach that included: 1) inductively coupled plasma - optical emission spectrometry (ICP/OES, 2) trace element fusion inductively coupled plasma - mass spectrometry (ICP/MS); 3) instrumental neutron activation analysis; 4) x-ray fluorescence spectrometry; 5) infra-red C and S analyzer and 6) a cold vapor flow injection mercury system analyzer. Accuracy and precision of all chemical analyses (for litho-geochemistry and hydro-geochemistry [below]) were carefully assessed and with minor exception, all were well within acceptable limits.

*Sequential Extraction.* Sequential extraction (step-wise leaching) of aquifer matrix samples followed a procedure modified from Ito (2001) and Thomas et al. (1994). The extractions steps were designed to isolate the following mineral-metal associations: (1) water-soluble (metals adsorbed on mineral surfaces; the leachate was distilled-deionized [DDI] water); (2) acid soluble (metals fixed primarily in carbonates; the leachate was 0.1 M acetic acid); (3) reducible (metals fixed in Fe- and/or Mn-oxides; the leachate was 0.1 M hydroxylamine hydrochloride solution, adjusting pH to 2 using  $\text{HNO}_3$ ); (4) organics (sample was decomposed using 0.1 M sodium pyrophosphate solution); and (5) insoluble residue (metals fixed mainly in sulfides, and less commonly in silicate and heavy minerals; dissolved by a mixture of concentrated  $\text{HNO}_3$  and  $\text{HClO}_4$ ). Sample agitation utilized a multi-wrist shaker. The extracts from the different steps were separated by centrifugation, filtered into polyethylene bottles using a 0.45 $\mu\text{m}$  Millipore syringe filter and analyzed at a commercial laboratory. Hydrogeochemical analyses for extraction leachate samples included ICP/MS and ICP/OES (the latter for over-range concentrations). Anions were analyzed by ion chromatography.

*Bench-scale leaching.* Bench-scale leaching studies are intended to provide an approximation of what may be observed in the field, specifically information on the relative mobility of metals and the potential changes in ASR storage zone water quality. Protocols are based on knowledge gained from previous studies and are designed to address hydrogeochemical hypotheses as they are defined or refined. Sealed reaction vessels (or mesocosms) with ports for collecting samples, taking

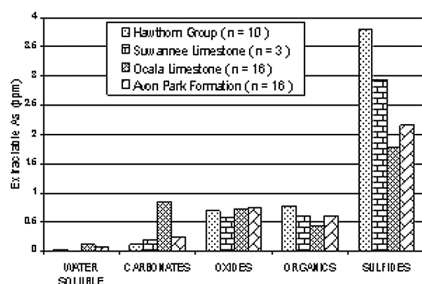
measurements and introducing gases were used to contain ~300g core chips and 1 L water (leachate). This ratio was established to yield leachate concentrations on the same order of magnitude as that observed in prior field studies. The leachate volume comprises approximately three pore volumes. Waters used in the reaction vessels vary according to the purpose of the bench study. Early experimental designs followed the model of the EPA leaching protocol, and used DDI water. Although this provided experimental control, the low ionic strength of DDI does not approximate the chemical “behavior” of Florida’s natural surface and ground water; therefore, bench-study protocols were modified to include use of SW and NGW. When possible, NGW is obtained from the same (or nearby) well as the core or cuttings to replicate the aquifer conditions. Source water for the proposed ASR well was used as leachate when it was available. Optimally, any treatment of the SW that is planned should be reproduced at the bench scale. In our leaching analyses, SW was sand-filtered followed by UV exposure. Adjustment for pH was not required. Leachate samples and physical parameters (DO, ORP, conductivity, pH, temperature) were collected per the protocol. Generally this involved initial daily measurement of parameters and collection of leachate samples for cation and anion analyses. After the first week, sample and data collection generally occurred three times per week for three weeks. Blank samples (i.e., leachate without rock) were included for control, as were duplicate reaction vessel set-ups (splits of core chips using same leachates) to assess total reproducibility. Hydrogeochemical analyses for bench study leachates were the same as sequential extraction.

## RESULTS AND DISCUSSION

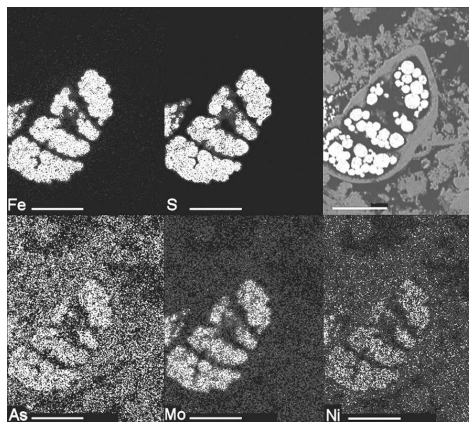
### Mineralogy and sequential extraction

Comprehensive XRD mineralogical characterization included identification of major constituents, clay minerals and insoluble residues. Pyrite was commonly observed in thin section and XRD analysis of residues; however, due to low volumetric concentrations, bulk XRD rarely detected pyrite. Based on scanning electron microscopy and petrographic analysis, the pyrites are inter- to intragranular, with morphologies ranging from euhedral to subhedral to framboidal masses. The crystals or masses rarely exceed 12  $\mu$ m in diameter. More than 175 EPMA analyses of calcite, francolite, and trace minerals were also completed. Nearly half of these analyses measured trace metals in pyrites, including As, Sb, Mo, Ni, U, Co, and Mn. Average concentrations of As and Mo in the pyrites equals 0.13 weight percent (wt %) and 0.64 wt %, respectively. Element mapping was also completed to visualize trace metal distributions. Arsenic in the pyrites appears to be incorporated into the mineral structure rather than adsorbed (Figure 2). Based on EPMA of 79 calcites, As, when detected, generally exceeds 300 ppm. Arsenic was not analyzed in dolomite due to X-ray peak interference.

Sequential analyses of rock samples confirm the trace metal associations of As, Mo, Sb and other metals with the sulfide fraction. For example, extractable As from samples of different lithostratigraphic units in the UFA (Figure 3) exhibit the aforementioned association. However, the results also indicate that Fe- and/or Mn-oxides, carbonates and organics may also contribute to the total As in the rocks. Although the carbonate-As association is clear in these analyses as well as in EPMA, the relation with oxides and organics requires further assessment due to incomplete carbonate dissolution in step 2. In general, the highest percentages of metals were extracted in step 5 (sulfides). Molybdenum is a notable exception as it is concentrated in the water-soluble and organic fractions. Bostick et al. (2003) report that Mo partitions strongly on pyrite surfaces in anoxic environments. Accordingly, Mo desorption is expected in strongly positive ORP conditions as suggested in the sequential extraction results (as well as bench-scale leaching results). Uranium detection in EPMA of pyrites is supported by a Fe-U correlation in step 5 extractants.



**FIGURE 2: Arsenic concentrations from five-step sequential extraction procedure relative to lithostratigraphic units in the study area.**

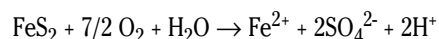


**FIGURE 3: EPMA element maps of framboidal pyrite clustered in a foraminifera; W-18253, Ocala Limestone (306 m BLS); upper right frame is corresponding backscatter electron image. Bar scale is 10 μm.**

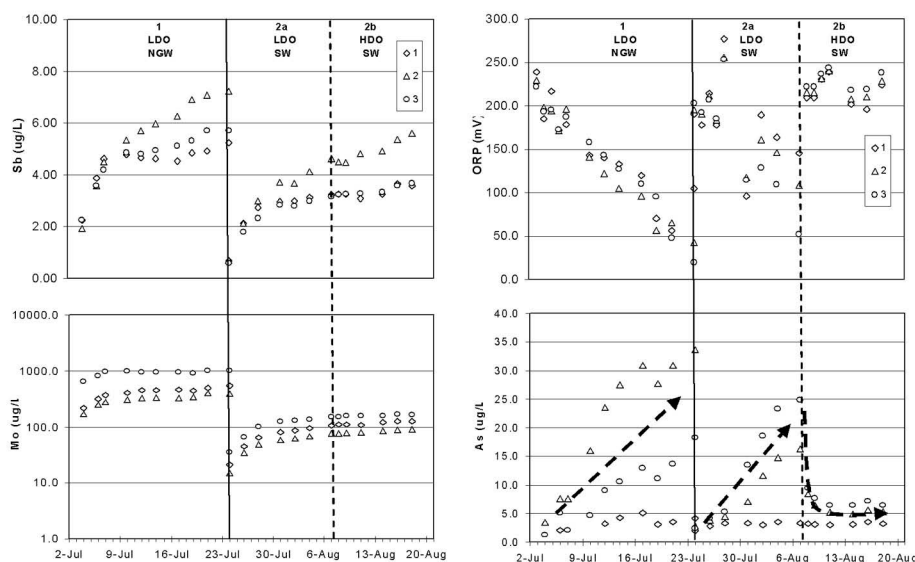
### Bench-scale Leaching

Bench-scale leaching has been completed on more than 40 carbonate samples of the UFA. Figure 4 represents selected hydrogeochemical data from three water-rock mesocosms, each having undergone the same protocol. In this particular experimental design, phase 1 includes core chips exposed to NGW in low-DO ( $DO < 0.3$  mg/L) conditions. Phase 2a represents a change of the leachate to SW, also under low-DO (LDO) conditions; whereas phase 2b represents the onset of high-DO

conditions (HDO; DO > 6.5 mg/L) with no exchange of leachate. Core chip samples represent the Hawthorn Group and the Suwannee Limestone (see caption of Figure 4). Phase 1 of the experiment was intended to mimic NGW conditions and establish experimental control (i.e., no metals mobilization). During this phase, however, As, Sb and Mo (note log-scale for Mo) are mobilized and ORP decreases. At positive ORP values pyrite is unstable; moreover, despite the LDO conditions there was apparently sufficient DO to either oxidize pyrite via:



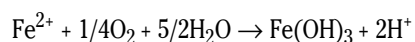
or desorb As and other metals. As this phase progresses, ORP declines whereas As, Mo and Sb rapidly desorb as equilibrium is approached. In other bench studies, as ORP stabilizes in the -200 mV range, As desorption discontinues. If pyrite oxidation has occurred, increases in products of reaction (1) should be observed, unless the trends are overprinted by the effects of other water-rock interactions or masked by ambient composition of the leachate water (NGW or SW). Iron and  $\text{SO}_4^{2-}$  increase in the leachate, but not until two weeks into phase 1.



**FIGURE 4: Leachate concentrations and ORP during phases of a bench-scale study. Leachate in phase 1 is LDO NGW collected from cored interval. Leachate in phase 2 is SW (treated) from nearby canal; low-DO during 2a was followed by high-DO (2b) without a change in leachate water. Core samples include: 1) W-18594 (166 m BLS), Hawthorn Group; 2) W-18594 (194 m BLS), Hawthorn Group; 3) W-18594 (262 m BLS), Suwannee Limestone.**

Similar As, Mo and Sb patterns are observed during phase 2a, again suggesting sufficient DO to oxidize or selectively leach As from pyrite. During phase 2b, however, while Mo and Sb continue to experience desorption, a sharp decrease in leachate As concentrations is observed (Figure 4).

Arsenic sorption during phase 2b has important implications with regard to understanding water-rock interactions during ASR. The sorption of As may have involved precipitation of a solid or colloidal form of HFO, which is a known As-scavenging phase (e.g., Nickson et al., 2000; Lu et al., 2005). If sufficient  $\text{Fe}^{2+}$  was available at the onset of the highly oxidized conditions in phase 2b, ferrihydrite [an HFO represented as  $\text{Fe}(\text{OH})_3$ ] may have precipitated:



Iron was indeed available at the end of phase 1 and was removed from solution as phase 2b began, similar to As. Normalized (C/Co) Fe and Mn concentrations indicate an increase in the rate of sorption at the beginning of phase 2b.

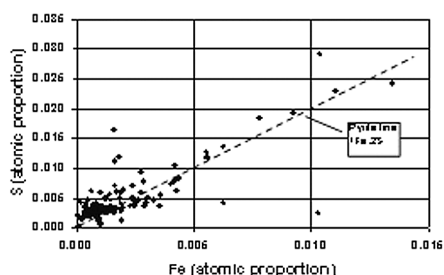
The average As:Fe atomic ratio = 0.0022 (pyrite) differs significantly from that of the maximum mobilized Fe and As in the leachates during phase 1, where the As:Fe atomic ratio ranges from 0.07 to 5.3. Arsenic concentrations are orders of magnitude greater than expected if bulk dissolution of pyrite is the only operative geochemical process. Several factors may account for this discrepancy, including: 1) selective leaching of As from pyrite, 2) desorption of As from OM or mineral phases other than pyrite, and 3) upon pyrite oxidation, rapid re-sorption of Fe as a different phase. With regard to the latter factor, Stüben et al. (2003) attribute a poor Fe-As correlation to resorption of As on freshly exposed HFOs, thereby changing the Fe:As ratio.

Experimental work by Kim et al. (2000) suggests that  $\text{HCO}_3^-$  in the presence of pyrite dissolution is effective in increasing soluble As concentrations. Results of modeling by Appelo et al. (2002) suggest that through competitive ion exchange,  $\text{HCO}_3^-$  displaces As sorbed to HFOs within an aerobic environment, and further increases in As (i.e., in the leachate) occur due to dissolution of HFOs in a strongly reducing environment (e.g., presence of dissolved OM). Appelo et al. (2002) also describe the effects of As sorption/desorption capacities in response to changes in  $\text{CO}_2$  pressure and the presence of dissolved Fe and  $\text{PO}_4^{3-}$  underscoring the dynamic nature of the system with respect to As and HFOs.

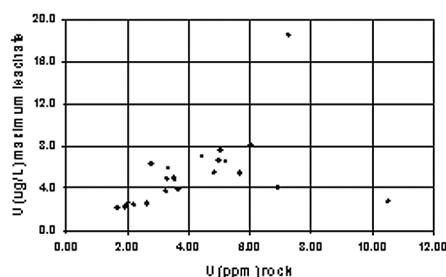
Pederson et al. (2006) further clarify the relation between HFOs and As, reporting that arsenic (as arsenate) is adsorbed onto the HFO surface rather than becoming incorporated in the crystal lattice. Based on their results, incongruent release of As and Fe is expected during reductive dissolution of HFOs, due in part to the change in HFO surface area as well as the kinetics of Fe reduction. They also found that more

than half of the Fe in HFOs must be reduced before significant arsenate desorption will occur. These dynamics may explain why As and Fe do not always “follow” in the leachate concentration patterns due to preferential leaching/desorption. Preliminary assessment of a bench study in our investigation supports this observation. Following phase 2b, strongly reducing conditions were induced in a water-rock mesocosm using a gas mixture of 5% H<sub>2</sub>S and 95% N<sub>2</sub>. In response, leachate Fe concentrations rapidly increased by two orders of magnitude, yet As did not increase until more than 48 hours later.

## WHOLE ROCK GEOCHEMISTRY



**FIGURE 5: Iron versus S for Hawthorn Group samples in study area. Pyrite control is indicated.**



**FIGURE 6: Maximum U concentrations in leachate for non-Hawthorn Group samples relative to U concentrations in corresponding rock samples.**

Assessment of bivariate relationships in whole rock chemistry can reveal mineralogical control. For example, a 1:1 slope of Sr – SO<sub>4</sub> (ionic proportions) in Avon Park Formation samples (not shown) suggests celestine (celestite) control. This control is also observed in sequential extraction data. Ionic proportions of Fe versus S in Hawthorn Group samples suggest pyrite control (Figure 5), which is an observation also reported by Lazareva (2004) for Hawthorn Group samples in west-central Florida. Deviations from the trend are likely due to the presence of gypsum, Fe-oxides, ferroan dolomite or heavy minerals in these samples. Multivariate statistics using cluster analysis (e.g., dendograms), as well as correlation matrices helped identify significant bivariate relationships and metal associations. For example, in some lithostratigraphic units, the association of As – Mo – Sb was particularly strong. Dendogram clusters in the Hawthorn Group delineated the influence of francolite (P-U) and heavy minerals (e.g., Ti-Zr-rare earth elements). Additional multivariate analyses of this dataset are pending.



Based on bench-scale leaching studies, maximum leachable As in the rock samples averages 2% and there appears to be no correlation between As in the leachate and As in the rocks. On the other hand, the study suggests an average of 0.15 % U is leachable and a strong correlation exists between average (n=3) maximum observed U in the leachate and U concentrations in the corresponding rock samples (Figure 6).

## CONCLUSIONS

Hydrogeochemical implications inferred from bench-scale leaching tests are intended to provide a cost-efficient approximation of what may be observed at the field scale. Specifically, the bench-scale results presented herein will hopefully provide information on relative mobility of metals and the potential order-of-magnitude changes in ASR storage zone water quality. In laboratory conditions, mobilization of metals (and metalloids) is clearly indicated; some metals are strongly desorbed from the aquifer matrix, some are apparently immobile, others sorb, and some metals exhibit dynamic behaviors that are highly responsive to changes in redox and solute compositions.

Noteworthy caveats exist with regard to interpretation of these results. Issues of volume and scale, physical aquifer characteristics, reaction kinetics during fluid flow and storage, etc. preclude direct transfer of the bench-study results to the field. While the study hopefully brackets the range of many hydrochemical processes during ASR, complicating variables include, but are not limited to temperature, pressure, redox conditions, water-rock surface area ratio, variability in source-water composition, source-water – ground-water mixing, effects of dual porosity, and artificial enhancement of trace-mineral exposure to the leachate solution (i.e., use of core chips rather than a flow-through core may have exposed pyrites to the leachate that would otherwise have been isolated from the matrix permeability). While beyond the scope of our present research, the importance of geochemical modeling as a potential predictive tool can not be understated.

## ACKNOWLEDGEMENTS

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## HYDROGEOCHEMISTRY OF AQUIFER STORAGE AND RECOVERY IN THE LOWER GREENSAND (LONDON, UK) FOR SEASONAL AND DROUGHT PUBLIC SUPPLY

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### ABSTRACT

Investigations are in progress to determine the potential for ASR, in the Cretaceous Lower Greensand aquifer at Horton Kirby, to meet demand during droughts of up to 2 years, whilst also meeting normal seasonal demands. The sands and sandstones are glauconitic and ferruginous so an understanding of the hydro geochemistry is needed to predict the responses to injection of aerobic water from the overlying Chalk aquifer. Pumped and pore water from the aquifers have been characterised, microcosm experiments undertaken and the results used to constrain geochemical modelling. The transmissivity of the 23 m thick aquifer is calculated to be 45 m<sup>2</sup>/d and it contains Ca-HCO<sub>3</sub> type groundwater with a pH of 7.6 and a SEC of 329 µS/cm. Concentrations of Fe(total) and Mn exceed the prescribed concentration value (PCV). The water to be injected is also a Ca-HCO<sub>3</sub> type with a pH of 7.4 and a SEC of 537 µS/cm, is aerobic and contains elevated concentrations of nitrate (22 mg/l), but not in excess of limits. Likely impacts of ASR are reactions with Fe-minerals (including small quantities of pyrite) resulting in an increase in dissolved iron and sulphate, and removal of injected nitrate through reduction.

### KEYWORDS

ASR, Hydrogeochemistry, Lower Greensand, UK

### INTRODUCTION

#### Objectives

An ASR scheme at the Horton Kirby site in south London would be used to store potable mains water for seasonal, peak and potentially drought demand use. London and south east England rely heavily on the Cretaceous Chalk aquifer for public water supply, and the population is predicted to increase significantly in the next 20 years requiring additional supplies. The local Chalk catchment is over-abstracted and

contains a sensitive low-flow river directly adjacent to the Horton Kirby site, and so to meet this additional demand Thames Water is investigating more innovative water resource management options such as ASR. A further complication is that the UK water regulator assesses Thames Water's supply capability throughout a drought period lasting 2 years. Consequently for ASR to contribute effectively to the supply capability a sufficiently large storage bubble must be maintained ready for use throughout such a drought period. Investigations are being undertaken to assess the hydraulic suitability and the likely geochemical impacts of injecting aerobic water into an anaerobic sand and sandstone aquifer containing glauconite, pyrite and other Fe-minerals. Core samples have been collected from the first investigation borehole and a second borehole is being drilled in early 2007. Flow and geochemical modelling will assist in optimising the well field configuration and recharge programme to enable seasonal use as well as allowing effective contribution to drought management.

### Hydrogeological Setting

Horton Kirby is situated to the east of London, south of the Thames near the axis of the syncline forming the London Basin. The major aquifer in the basin is the Cretaceous Chalk. This overlies the Lower Greensand aquifer, which is confined and separated from the Chalk aquifer by about 70 m of Gault Clay (Table 1). The top of the Lower Greensand at the site is at a depth of 252 m below ground level and the 23 m thick aquifer is represented by the Folkestone Beds, directly overlying the Weald Clay. There is not likely to be much groundwater flow at the site as it is only 10 m AOD, the groundwater level is 6.5 m AOD.

The aquifer thickens to the south towards the outcrop at a distance of over 10 km away, resulting in a mean transmissivity of about  $260 \text{ m}^2/\text{d}$ . The impact of the scheme on existing pumping stations, the nearest of which is 6.1 km away, is calculated to be less than a 1 m rise in water level given the planned injection regime. A rise in groundwater level of 0.07m is estimated at the nearest location where the river Darent flows across the Folkestone Beds, approximately 11.5 km from site.

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**Table 1. Geology at the Horton Kirby Borehole Site.**

| Geological Unit | Thickness (m) | Depth to top of Formation (mbgl) |
|-----------------|---------------|----------------------------------|
| River Gravels   | 4             | GL                               |
| Upper Chalk     | 41            | 4                                |
| Middle Chalk    | 71            | 45                               |
| Lower Chalk     | 65            | 116                              |
| Gault Clay      | 71            | 181                              |
| Lower Greensand | 23            | 252                              |
| Weald Clay      | N/A           | 275                              |

## SAMPLING AND TESTING METHODOLOGIES

The borehole was cored throughout the entire thickness of the Lower Greensand and the core analysed as follows:

- Lithological logging
- Thin section analysis
- Whole sample mineralogical analysis by quantitative XRD (X-ray diffraction) analysis
- Clay mineralogical analysis by quantitative XRD analysis
- XRF (X-ray fluorescence) chemical analysis of whole samples
- Assessment of cation exchange capacity
- SEM and EDX analysis
- Grain size analysis
- Hydrochemical analysis of porewater samples. The samples are extracted by centrifugation, a method which is efficient in separating the fluid from the sample but does subject it to aeration. Samples are therefore handled quickly to minimise oxidation.

Four samples of core were also selected, two from Aquifer Zone 2 and one from each of Aquifer Zones 1 and 3, for batch experiments which were conducted to examine the response of adding crushed samples of the host rock into water from the potential recharge source (Horton Kirby tap water). These experiments used polyethylene bottles, mounted on a shaker housed in an incubator and kept at constant temperature, 30 C. An increase in temperature relative to the in-situ temperature (16 C) was used in order to speed up and chemical reactions taking place. Similarly, the host rock was crushed before use (size  $\wedge$  125-250  $\mu\text{m}$ ) to increase the surface area for reaction. The experiments were run for approximately four weeks with sampling only at the end of the experiment. The experiments give an indication of the likely dissolution and precipitation reactions that may occur during storage of water in the aquifer.

The aquifer was also hydraulically tested with a series of step-tests followed by a 20-day continuous rate test. At the end of this period the drawdown was about 89 m induced by a pumping rate of 12 l/s. Analysis of these tests gave a transmissivity of 45  $\text{m}^2/\text{d}$  and an estimated storage coefficient of  $1 \times 10^{-4}$ . Samples were also collected during the test to characterise the groundwater in the Lower Greensand aquifer.

## RESULTS AND DISCUSSION

### Core Characterisation

The mineralogical, petrographic and particle-size analysis distinguished five aquifer zones within the cored interval of the borehole (Figure 1).

*Aquifer zone 1 (253-258.5 m)* — Sediments generally poorly-sorted, medium-grained quartz-rich sandstones, with irregularly distributed detrital clay (generally smectite and illite with minor chlorite and kaolinite). A lack of Fe-bearing phases identified by XRD analysis compared with the  $\text{Fe}_2\text{O}_3$  content identified by XRF geochemical analysis suggests amorphous Fe-oxyhydroxides may be present.

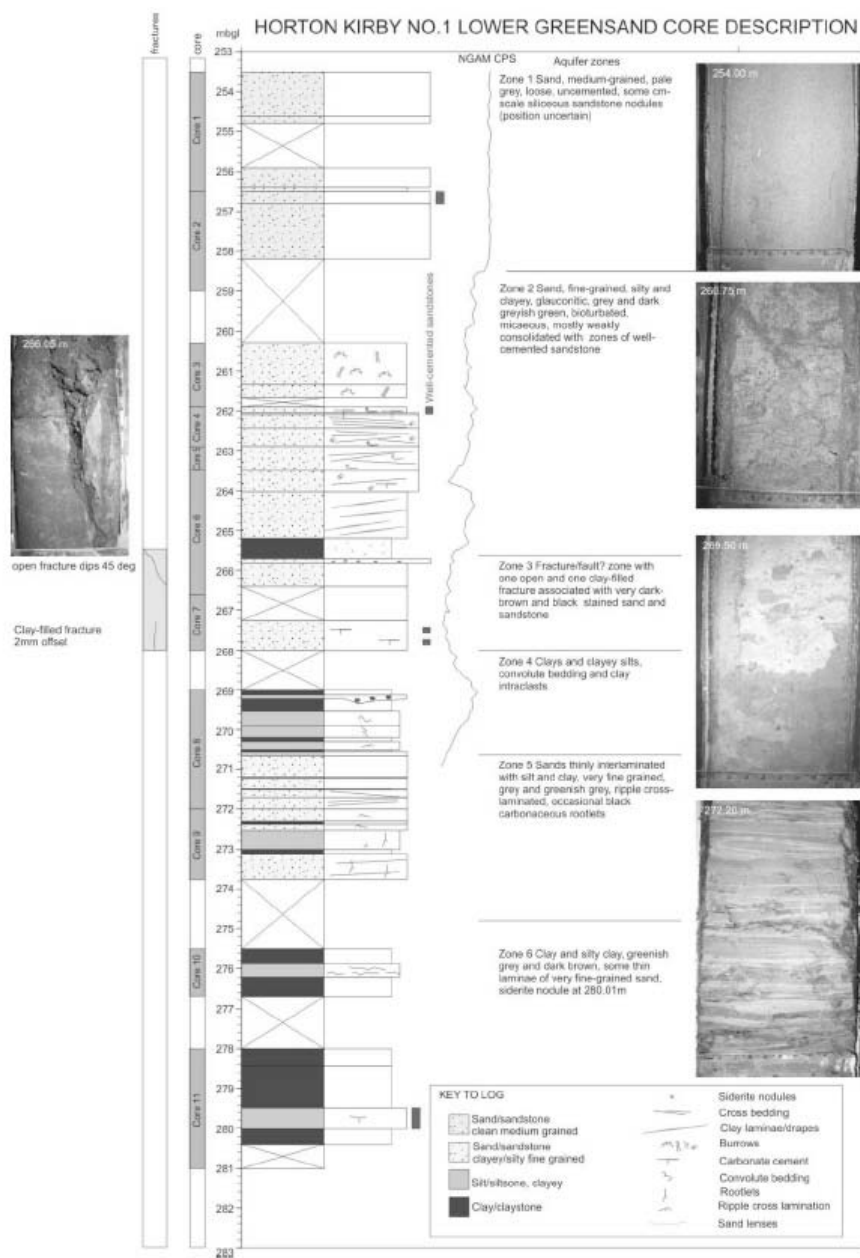
*Aquifer zone 2 (258-265.5 m)* — Upper interval (258-262 m) is similar to zone 1 but finer grained with progressively greater abundance of glauconite with depth. Again the sands have a low clay content. The zone contains some carbonate cement and traces of gypsum on surfaces, most likely to have precipitated when the samples were dried. The lower interval (262-265.5 m) comprises very poorly sorted very fine sands with abundant pellets of green glauconite. Kaolinite becomes more prominent at this depth. Carbonate cements (siderite and weakly ferroan calcite) are locally developed.

*Aquifer zone 3 (265.5-267.6 m)* — Sandstone similar to lower zone 2. However organic matter is abundant and associated with fine-grained clay, silica and authigenic Ti-oxides.

*Aquifer zones 4 & 5 (269.5-273.2 m)* — dominated by finely interbedded siltstones and mudstones. Characterized by kaolinite- and illite-dominated clay mineral assemblages with subordinate illite/smectite and chlorite.

Below aquifer zone 5 (275.4 — 280.4 m) the core largely comprises greenish grey and dark brown clay and silty clay, with some thin laminae of very fine sand.

The cation exchange capacities of two samples from Aquifer zone 2 were 39 (256.85 m) and 45 (260.48 m) meq/kg. A sample from Aquifer zone 3 (267.55 m) had a higher CEC (150 meq/kg), possibly as a result of its high glauconite content (8.5%).

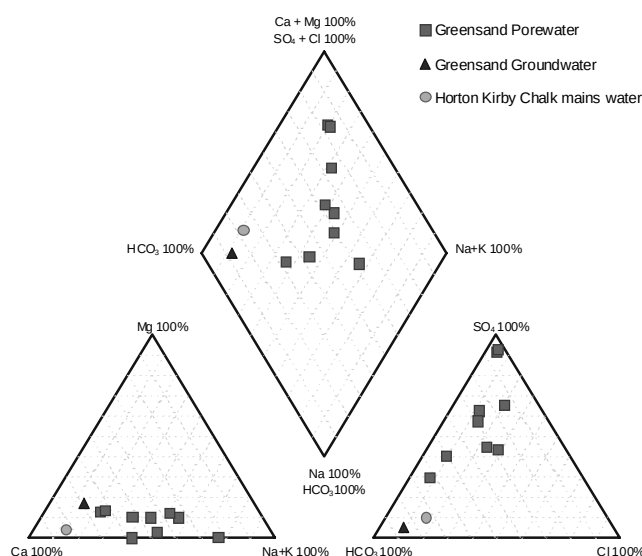


**FIGURE 1: Geological log of the Lower Greensand core at Horton Kirby.**



## Water Quality Analysis

The results of the analyses of the pore waters are shown in Table 2 together with analyses of groundwater pumped during the continuous rate test and water from the Chalk aquifer at Horton Kirby. These results are also plotted in Figure 2 and show the great variability in the composition of the pore water. Some of this variability may be due to contamination by the drilling and sampling process. For example, the very high pH values (up to 11.91) are likely be due to the Na-bentonite drilling fluid used when drilling the Gault Clay. A polymer fluid was then used but it was mixed with Horton Kirby water. This water has a distinctively higher Cl concentration which is seen in most of the analyses indicating that the core samples in the unconsolidated sands may also have been contaminated. However, the variability may also relate to the mineralogical and permeability variations described above.



**FIGURE 2: Trilinear plot of analyses of water from the Horton Kirby site.**

The pumped sample is taken to represent the aquifer water as it was collected after 20 days pumping so any drilling contamination will have been well flushed out. The water will also be derived from the most permeable and hence active parts of the aquifer, where recharge and recovery will take place. The proposed Chalk recharge water varies from the Greensand groundwater in that the former has higher SEC (537 vs. 329 uS/cm) and elevated levels of dissolved oxygen (c. 5.5 mg/l), nitrate (~22 mg/l), calcium, chloride and sulphate, and lower concentrations of iron (total and dissolved), manganese, potassium, magnesium and strontium. The pH values are similar (7.6 and 7.4).2. Trilinear plot of analyses of water from the Horton Kirby site.

**Table 2. Porewater and pumped groundwater sample analyses from the Lower Greensand aquifer at Horton Kirby, London, UK.**

| DETERMINAND                    | UNIT  | Porewater (na = not analysed) |              |              |              |              |
|--------------------------------|-------|-------------------------------|--------------|--------------|--------------|--------------|
|                                |       | S06-00031                     | S06-00032    | S06-00033    | S06-00034    | S06-00035    |
|                                |       | 254.7 m                       | 256.4-256.6m | 256.8-256.9m | 260.5-260.6m | 262.1-262.2m |
| PH VALUE                       |       | 11.91                         | 8.01         | 10.42        | 7.48         | 7.52         |
| CONDUCTIVITY AT 20°C           | µS/cm | 2274                          | 1266         | 988          | 1518         | 488          |
| TEMPERATURE                    | deg C | na                            | na           | na           | Na           | na           |
| CARBON TOTAL ORGANIC           | mg/l  | na                            | na           | na           | Na           | na           |
| NITRATE AS NO <sub>3</sub>     | mg/l  | 0.82                          | 5.40         | 0.31         | <0.22        | <0.22        |
| NITRITE AS NO <sub>2</sub>     | mg/l  | na                            | na           | na           | Na           | na           |
| AMMONIUM AS NH <sub>4</sub>    | mg/l  | na                            | na           | na           | Na           | na           |
| CHLORIDE AS CL                 | mg/l  | 43.5                          | 43.1         | 55.3         | 31.2         | 33.2         |
| SULPHATE AS SO <sub>4</sub>    | mg/l  | 210                           | 311          | 309          | 810          | 146          |
| ALKALINITY AS HCO <sub>3</sub> | mg/l  | 562                           | 165          | 197          | 42           | 40           |
| FLUORIDE                       | mg/l  | 0.256                         | 0.267        | 0.597        | 0.258        | 0.402        |
| BROMIDE                        | mg/l  | < 0.02                        | < 0.02       | < 0.02       | < 0.02       | < 0.02       |
| P (INORGANIC)                  | mg/l  | < 0.1                         | < 0.1        | 0.18         | <0.1         | <0.1         |
| ALUMINIUM                      | mg/l  | 0.31                          | < 0.01       | 0.32         | <0.01        | <0.01        |
| BARIUM                         | mg/l  | 0.316                         | 0.505        | 0.197        | 0.14         | 0.244        |
| BORON                          | mg/l  | < 0.1                         | 0.16         | 0.24         | 0.19         | 0.18         |
| CALCIUM                        | mg/l  | 250                           | 161          | 51.9         | 252          | 49.3         |
| IRON                           | mg/l  | < 0.005                       | < 0.005      | 0.0115       | <0.005       | <0.005       |
| IRON DISSOLVED                 | mg/l  | na                            | na           | na           | Na           | na           |
| Fe <sup>2+</sup>               | mg/l  | na                            | na           | na           | Na           | na           |
| Fe <sup>3+</sup>               | mg/l  | na                            | na           | na           | Na           | na           |
| MAGNESIUM                      | mg/l  | < 0.04                        | 6.3          | 0.198        | 30.4         | 6.04         |
| MANGANESE                      | mg/l  | < 0.002                       | 0.0107       | < 0.002      | 0.0926       | 0.0067       |
| SILICON                        | mg/l  | 2.94                          | 3.39         | 7.95         | 1.46         | 0.968        |
| SODIUM                         | mg/l  | 188                           | 186          | 191          | 83.6         | 31.8         |
| POTASSIUM                      | mg/l  | 33.6                          | 26.2         | 14           | 29.4         | 14           |
| STRONTIUM                      | mg/l  | 3.73                          | 2.8          | 0.608        | 5.33         | 1.25         |

**Table 2: Porewater and pumped groundwater sample analyses from the Lower Greensand aquifer at Horton Kirby, London, UK.**

| Porewater (na = not analysed) |                  |                  |                  | Pumped<br>ASR BH | HK CHALK |
|-------------------------------|------------------|------------------|------------------|------------------|----------|
| S06-00036                     | S06-00037        | S06-00038        | S06-00039        |                  |          |
| 263.0-<br>263.15m             | 264.5-<br>264.6m | 267.6-<br>267.7m | 271.4-<br>271.5m | Jan-06           | Mar-06   |
| 7.7                           | 7.37             | 7.71             | 8.17             | 7.6              | 7.4      |
| 318                           | 1473             | 354              | 508              | 329              | 537      |
| na                            | na               | Na               | na               | 16.1             | na       |
| na                            | na               | Na               | na               | 0.4              | 0.7      |
| 2.28                          | <0.22            | <0.22            | <0.22            | <1.4             | 22.2     |
| na                            | na               | Na               | na               | <0.01            | <0.01    |
| na                            | na               | Na               | na               | 0.12             | <0.05    |
| 31.6                          | 28               | 27.8             | 18.4             | 13               | 32       |
| 64                            | 838              | 70               | 103              | 8.8              | 26.1     |
| 51                            | 31               | 63               | 161              | 198              | 263      |
| 0.396                         | 0.261            | 0.361            | 0.374            | 0.06             | 0.1      |
| < 0.02                        | < 0.02           | < 0.02           | < 0.02           | 0.048            | <0.02    |
| <0.1                          | <0.1             | <0.1             | <0.1             | <0.071           | <0.071   |
| <0.01                         | <0.01            | <0.01            | <0.01            | 0.049            | <0.015   |
| 0.188                         | 0.31             | 0.0695           | 0.143            | 0.132            | 0.026    |
| 0.14                          | 0.19             | 1.19             | 0.26             | <0.07            | na       |
| 28.1                          | 234              | 21.9             | 37               | 50               | 108      |
| <0.005                        | <0.005           | <0.005           | <0.005           | 0.79             | <0.01    |
| na                            | na               | Na               | na               | 0.39             | na       |
| na                            | na               | Na               | na               | 0.57             | na       |
| na                            | na               | Na               | na               | 0.27             | na       |
| 3.62                          | 30.6             | 3.92             | 7.27             | 7.5              | 2.5      |
| 0.0047                        | 0.0402           | 0.0293           | 0.0243           | 0.052            | <0.003   |
| 0.873                         | 1.07             | 2.03             | 3.91             | 5.70             | 5.05     |
| 25.5                          | 76               | 37.5             | 53               | 9.1              | 16.4     |
| 10.3                          | 51.2             | 6.61             | 11.8             | 4.2              | 1.4      |
|                               |                  |                  |                  |                  |          |

## Geochemical modelling

The mineral saturation states of all the reacted fluids collected from the microcosm batch experiments were examined using the EQ3/6 (version 7.2c) geochemical modelling software package (Wolery, 1992) using the 'data0.cmp.v8.R6' thermodynamic database.

Examination of the saturation indices (SI) for the minerals shows that, in general, the fluids are not usually in equilibrium, other than with kaolinite, and are undersaturated with respect to gypsum and anhydrite. Thus if these minerals are present in the solid samples they will tend to dissolve, as suggested by the fluid chemistry results.

For the most part, the chemical and mineralogical analyses have revealed little evidence for reaction between the fluids and the solid samples. However, the sample taken to be representative of Aquifer Zone 2 (Sample MPLL720, depth 260.45-260.50 m) did react with the Horton Kirby tap water to produce an acid fluid ( $\text{pH} \approx 3$ ), and in this sample, some evidence for mineral dissolution was seen, with the removal of the previously observed gypsum/anhydrite. The presence of pyrite in the sediment could also affect the fluid chemistry.

In order to model the possible chemical reactions between recharge and native water and the Greensand aquifer material a number of scenarios have been developed using the geochemical modelling package PHREEQC, developed by Parkhurst and Appelo (1999). These scenarios were based on the equilibrium reactions of aqueous solutions interacting with minerals, gases, solid solutions, exchangers and sorption surfaces, and simulate chemical reactions and transport processes in natural or contaminated water (Appelo and Postma, 2005).

*MixModel 1:* A simple bucket model simulating the mixing of Horton Kirby Chalk water with Lower Greensand water. The resultant solution is allowed to reach thermodynamic equilibrium with respect to  $\text{Fe}(\text{OH})_3$ , calcite and dolomite (i.e. if any these minerals are supersaturated within the solution they are allowed to precipitate). The results show that virtually all the dissolved iron in the Greensand water is oxidised and precipitates as Fe-hydroxide ( $\text{Fe}(\text{OH})_3$ ), and that calcite will precipitate from the supersaturated recharge water.

*MixModel 2:* Pyrite and calcite are added to MixModel 1, thereby simulating reaction between these aquifer minerals, recharge water and native water. The model assumes that the aquifer comprises 0.1% pyrite. The results indicate that the injection water will oxidise pyrite resulting in an increase in the concentration of sulphate and dissolved iron and a reduction in pH within the recharge water. The pyrite will be oxidised both by dissolved oxygen and nitrate within the recharge water. It is unlikely that  $\text{NO}_3$  will oxidise pyrite directly by chemical means and such

denitrification has yet to be demonstrated in the laboratory (Kinniburgh et al, 1993). It is more likely that the reaction will proceed as a result of bacterially-mediated denitrification.

*MixModel 3:* The Fe stability diagram predicts that at pH (7.3-7.6) and Eh (-0.16 to -0.20V) of the solutions generated by MixModel 2, siderite ( $\text{FeCO}_3$ ) would be the solubility control of Fe. MixModel 3 therefore adds siderite to the phase assemblage, allowing it to precipitate. The results indicate that under thermodynamic equilibrium siderite will precipitate resulting in lower dissolved Fe concentrations than in MixModel 2. In the example outlined above, 22.5 mg of pyrite dissolves generating 17.5 mg of siderite (precipitate) and 1.9 mg/l dissolved Fe.

*MixModel 4:* This model develops MixModel 3 with the addition of other minerals to the phase assemblage. These minerals, noted in the XRD and thin section analyses, include: K-feldspar, gibbsite, chalcedony and chlorite14A (closest approximation to glauconite). The model predicts that at thermodynamic equilibrium, there will be an increase in K and Mg (reaction with K-feldspar and chlorite) and a decrease in Fe (siderite precipitation), Ca and alkalinity as  $\text{HCO}_3$  (precipitation of calcite). These clay mineral reactions are likely to be slow and may take many years to reach thermodynamic equilibrium. However the simulations indicate the expected direction of change.

All the models discussed above assume thermodynamic equilibrium between the various phases and do not take into account reaction rates. It is unlikely that, over the period between recharge and recovery, the reactions simulated in the models outlined above will go to completion. It is possible to model kinetics within PHREEQC. However this requires the user to define rate expressions which are poorly understood for the required reactions. Additionally the models do not take into account that iron precipitate (Fe-oxyhydroxide and siderite), formed as a result of reaction between pyrite and recharge water, may coat pyrite grains thereby restricting further reaction (Nicholson et al., 1990). The models are therefore likely to over-predict the amount of pyrite that will be oxidised through reaction with chalk injection water.

In addition to redox reactions, ion exchange is likely to have an impact on the quality of recharge water. Calcium concentrations in the recharge water exceed those in the native water; as a result it is likely that ion exchange will take place between dissolved Ca and Na in Na-rich clays. This ion exchange would result in an increase in dissolved Na and a decrease in dissolved Ca.

## CONCLUSIONS

The main mineral of concern within the Greensand aquifer is pyrite ( $\text{FeS}_2$ ), oxidation of which would result in the generation of dissolved ferrous iron ( $\text{Fe}^{2+}$ ) and sulphate. The ferrous iron may then be further oxidised and precipitate out of solution as iron-oxyhydroxide ( $\text{Fe}(\text{OH})_3$ ) or, depending on the Eh and pH of the recharge water, as siderite ( $\text{FeCO}_3$ ), possibly causing some clogging of the aquifer. Acidity produced by pyrite oxidation may react with auxiliary minerals in the Greensand such as glauconite and other micaceous minerals, Na and K feldspar and calcite, resulting in an increase in concentrations of Mg, Ca, K and Na.

Further investigations at the Horton Kirby site, including the implications for long-term bubble storage for drought use, will be guided by the results to date. Improved understanding of the injection water-aquifer-native water interactions will be gained through monitoring key determinants, including:

*Cl*: Since the Cl concentration of native LGS groundwater (13 mg/l) is lower than the concentration in the potential injection water (~30 mg/l), the Cl concentration of the abstracted water will provide an indication of recovery efficiency.

*Fe (total and dissolved)*: Concentrations will give an indication as to the degree of interaction of injection water with Fe-minerals within the Greensand (and interaction with dissolved Fe within native groundwater).

*$\text{SO}_4$ , Mg, Ca, Mn, Sr, Na, K and  $\text{HCO}_3$  (alkalinity)* – Concentrations will give an indication of the degree of interaction of injection water with aquifer minerals and native water.

*N-species*: Nitrate in the injection water is likely to be reduced so measurements of  $\text{NO}_3$ ,  $\text{NO}_2$  and  $\text{NH}_4$  will therefore be useful in identifying the degree of reduction within the LGS.

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## ARSENIC MOBILITY UNDER VARIABLE REDOX CONDITIONS INDUCED DURING ASR

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### ABSTRACT

Arsenic release from aquifers is a major issue for aquifer storage and recovery (ASR) schemes supplying drinking water, especially as water quality targets for arsenic in drinking water are very stringent. Understanding the processes that release and attenuate arsenic during ASR is the first step towards managing this issue. This study utilised a full scale field trial to examine the fate of arsenic within the injectant plume during all stages of the ASR cycle, and the resultant water quality that was produced. The major influence on the recovered water arsenic concentration was oxidation of reduced minerals soon after injection observed in close proximity to the point of injection. There was evidence of attenuation via adsorption to iron oxides which reduced concentrations on the outer fringes of the injectant plume. However this process was reversed during recovery and the average recovered concentration was, on average, 14 µg/L; 11 µg/L above the source concentration. A reduced zone directly around the ASR well produced the greatest arsenic concentration and illustrated the importance of iron oxides for controlling arsenic concentrations. Given the small spatial extent of this zone, this process had little effect on the overall recovered water quality.

### KEYWORDS

Arsenic, aquifer storage and recovery (ASR), carbonate, pyrite oxidation, iron oxide dissolution

### INTRODUCTION

Arsenic release from sedimentary aquifers is a prevalent health issue affecting safe drinking water supplies (Chatterjee *et al.* 1995; Smedley and Kinniburgh, 2002; Bhattacharya *et al.* 2004; Duker *et al.* 2004). The processes postulated as leading to elevated concentrations of arsenic in groundwater include oxidation of reduced arsenic-bearing minerals, most commonly arsenian pyrite ( $\text{FeAs}_x\text{S}_{2-x}$ ) or arsenopyrite ( $\text{FeAsS}$ ) (Schreiber *et al.* 2000); reductive dissolution of iron oxides and the associated release of adsorbed arsenic (Ahmed *et al.* 2004; Zheng *et al.* 2004; Smedley *et al.* 2005); ligand exchange or displacement from sorption sites (Appelo *et al.* 2002; Smedley *et al.* 2003); and desorption following the reduction of arsenate to arsenite

(Bose and Sharma 2002; Oremland and Stolz 2003). Conversely, arsenic concentrations are reportedly controlled by attenuation mechanisms such as sorption to iron oxides, typically amorphous iron oxides, goethite or hematite, under oxidising conditions (Dixit and Hering 2003; Foster 2003; Stollenwerk 2003) and precipitation of sulphides (Kirk *et al.* 2004) under sulphate reducing conditions.

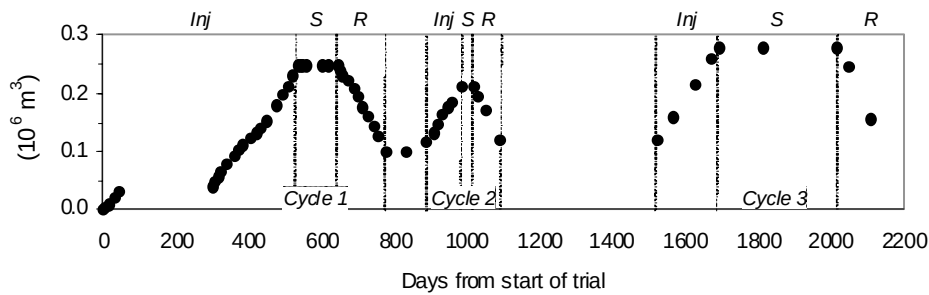
Aquifer Storage and Recovery (ASR) is a technology for storing water in aquifers via an injection well, balancing water supply with demand. The recharge source, whether it is a surface water, groundwater, urban runoff or waste water, is generally oxygenated. Well injection allows access to deeper confined anaerobic storage zones, often within limestone and sandstone aquifers that are not suited to infiltration (Pyne 2006). Thus by oxygenating the anoxic storage zone, injection can result in oxidation of reduced minerals and release of trace elements such as arsenic from the aquifer (Stuyfzand 1998; Arthur *et al.* 2001; 2003). Once released, the mobility of these species depends on the chemical environment within the aquifer, which can also result in subsequent removal prior to recovery. In an ASR scheme the chemical environment can be quite transient, varying spatially within the plume of injected water and also temporally throughout the various stages of the cycle. In addition, the use of reclaimed waste water in ASR can amplify the redox gradients that develop (Vanderzalm *et al.* 2006).

Despite being reported as a prevalent issue for the water recovered from an ASR scheme (Arthur *et al.* 2001; 2003), the processes responsible for arsenic mobilisation and immobilisation during the ASR cycle have received little attention in the literature. This paper uses a full scale field trial in a carbonate aquifer to examine the processes impacting on the mobility of arsenic during the various stages of the ASR cycle and their influence on the quality of water that is recovered.

## METHODS

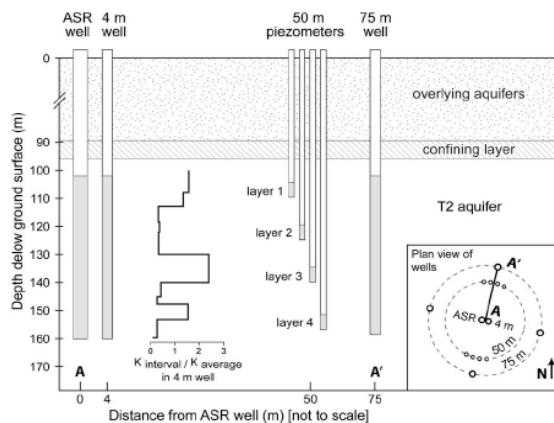
A full-scale ASR trial at Bolivar, South Australia between October 1999 and July 2005 investigated the mobilisation of arsenic from a carbonate aquifer. Three ASR cycles were undertaken, injecting  $0.52 \times 10^6 \text{ m}^3$  and recovering  $0.37 \times 10^6 \text{ m}^3$ , in total (Figure 1). The ASR field site was situated adjacent to a supply of reclaimed water for injection, from the Bolivar Water Reclamation Plant, and in close proximity to a horticultural region utilising the reclaimed water for a seasonal irrigation source.





**FIGURE 1. Cumulative volume during three ASR cycles at Bolivar (Inj = injection, S=storage, R=recovery).**

The storage zone was the lower Tertiary sediments (T2) of the Port Willunga Formation, an approximately 60 m thick calcarenite aquifer. The ASR well was completed and open over most of the T2 aquifer thickness, approximately 100 to 160 m below ground surface (bgs) (Figure 2). The mineralogy is dominated by calcite and quartz, with minor contributions from ankerite, hematite, microcline and albite. Pyrite framboids have been identified with scanning electron microscopy (Rinck-Pfeiffer *et al.* 2000) but not quantified. Horizontal hydraulic conductivity varies over almost three orders of magnitude ( $10^{-1}$  to  $10^2$  m/day) when measured on core samples over a 10 cm scale (Wright *et al.* 2002), and the storage zone can be represented by four horizontal layers, alternating between high and low permeability (Pavelic *et al.* 2006). The average hydraulic conductivity derived from a pumping test over the whole aquifer thickness is 3 m/day (Martin *et al.* 1998).



**FIGURE 2. Simplified vertical section (see inset) of the T2 aquifer showing the depth intervals intersected by the ASR well, 4 m fully penetrating well and the 50 m piezometers (Vanderzalm *et al.*, 2006).**

This work reports hydrogeochemical data from the ASR well, a fully penetrating observation well 4 m from the ASR well and one piezometer 50 m (50 m n-3) from the ASR well, intersecting a high permeability layer from 134 to 139 m bgs (layer 3). Travel time to the 4 m radius is approximately 1-2 days, representing changes soon after injection, while travel time to the 50 m radius is of the order of 100 days and is close to the outer fringe of the injectant plume. This examination focuses on the first and third ASR cycles undertaken, where arsenic concentrations were measured.

Ground water samples were collected after stabilisation of pH, temperature, DO, EC and Eh, in accordance with appropriate well purging (approximately 3 well volumes). These physical parameters were measured *in-situ* using a TPS-FL90® probe in a flow-through cell, directly from the pump discharge to prevent atmospheric exposure. Sampling, preservation and analysis methodology was based on the Standard Methods for the Examination of Water and Wastewater (APHA, 1998), with samples maintained below 4°C and transported to the Australian Water Quality Centre (AWQC), Bolivar, South Australia for analysis within 24 hours of sampling. Metal and metalloid samples were acidified immediately after sampling with nitric acid to below pH 2. Dissolved metal and metalloid samples were filtered through a 0.45 µm filter immediately after sampling and prior to acidisation. Aggregated samples of recovered water were collected in cycle 3 using an ISCO® 6700 refrigerated auto sampler. These samples were maintained below 4°C but were not acidified prior to transport to the laboratory. Arsenic was analysed by ICP-MS with a detection limit of 1 µg/L.

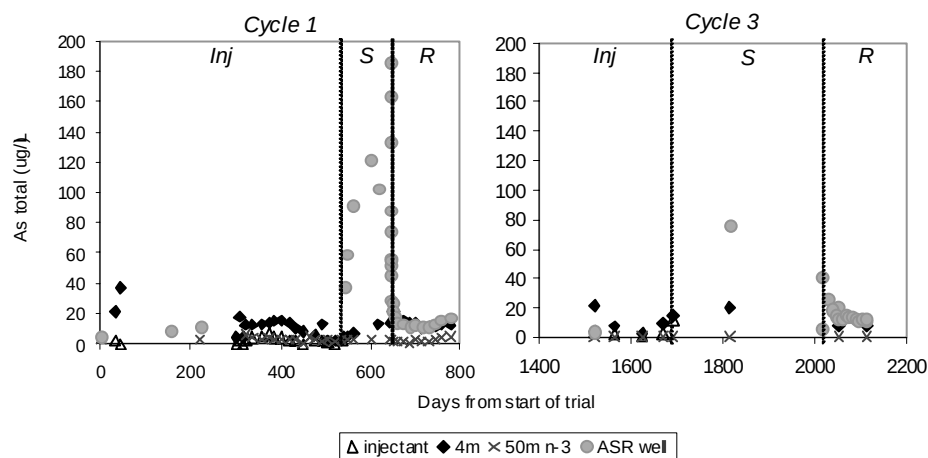
The calculation of arsenic excess, or release from the aquifer, was based on the difference between the measured concentration and the concentration expected from conservative mixing alone, using chloride as a conservative tracer.

## RESULTS

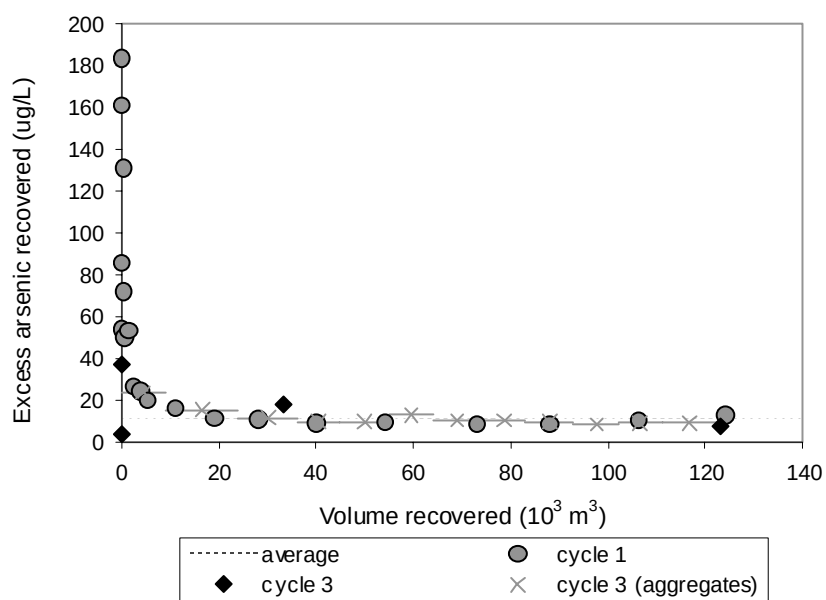
Prior to the commencement of ASR at Bolivar, the arsenic concentration in the undisturbed groundwater of the T2 aquifer was on average 5 µg/L, and largely soluble (3 µg/L). Arsenic within the reclaimed water injectant was also predominantly soluble, with an average concentration of 3 µg/L.

The presence of arsenic in the groundwater implies the occurrence of an arsenic-bearing mineral phase within the sediments. The highest ambient concentrations, 7-14 µg/L, were found in less permeable layers of the aquifer, from approximately 119-124 m bgs and 152-156 m bgs. Groundwater samples taken from discrete intervals of the 4 m fully penetrating observation well, using packer sampling, after  $20 \times 10^3 \text{ m}^3$  of injection and full penetration of the injectant in this layer, indicated elevated total iron (32.7 mg/L) and total arsenic (436 µg/L) concentrations and a sulphur-34 signature depleted by 2 ‰ in the 120-125 m bgs interval when compared with the other intervals. The soluble concentrations were typical of the ambient levels in this layer.

This layer does not contribute largely to the open hole signature, but these observations are supporting evidence of a significant arsenic source in the less permeable zones of the aquifer.



**FIGURE 3. Arsenic behaviour during the first and third ASR cycles at Bolivar.**



**FIGURE 4. Excess arsenic in water recovered from the ASR well during the first and third cycles at Bolivar.**

The arsenic concentration in water recovered from the ASR well following the first and third cycles was greater than that injected or within the ambient groundwater (Figure 3), indicating release through the cycle. On average, the arsenic concentration recovered from the ASR well was 11 µg/L above that expected for mixing between the injectant and the ambient groundwater (Figure 4). However the excess was far greater in the early stages of recovery, followed by a decline to plateau around the average where the total concentration was 14 µg/L. The peak concentration (and excess) measured, of 186 µg/L, occurred with 80 m<sup>3</sup> of recovery in cycle 1. While the early stage of recovery in cycle 3 was not monitored in great detail, an arsenic concentration of 40 µg/L after recovery of 18 m<sup>3</sup>, was lower than at the same point in cycle 1.

The processes that lead to increased arsenic concentrations in the recovered water can be explained through the examination of the arsenic variation during the injection and the storage phases. Therefore these stages of the ASR cycle are examined in more detail. Release of arsenic was evident during both the injection and storage phases of the ASR cycle. Recovery of arsenic at greater concentrations than injected was evident at the 4 m radius, while the concentrations at the 50 m radius remained similar to the ambient condition.

During the injection phase, the increase in arsenic was evident in the 4 m observation well, however the extent of the increase declined toward the end of the first injection cycle and was also diminished in magnitude by the third injection cycle. The average arsenic release between injection and 4 m was 8 µg/L in cycle 1, producing concentrations consistently an order of magnitude higher than the injected and ambient signatures. Over the duration of the first injection, arsenic illustrated a peak at around 30 days of injection, also coincident with peaks in iron, zinc and nickel. This was followed by a trend of relatively high arsenic concentration at 4 m, up to around 450 days, equivalent to an excess of 10 µg/L. The arsenic excess subsequently dropped to below 5 µg/L at a stage when nitrate was higher in the injectant and persisted to the 4 m well (NO<sub>3</sub>-N above 1 mg/L). In the third cycle the average arsenic release was reduced to 3 µg/L, similar to the latter part of cycle 1 (4 µg/L). This variation in arsenic at 4 m was independent of the injected arsenic concentration, which remained reasonably constant throughout the injection. The increase evident at 4 m was not witnessed at the 50 m radius despite full penetration of the injectant. Arsenic at 50 m, the edge of the injectant plume, remained similar to the ambient condition, on average 3 µg/L.

A more reduced condition developed around the ASR well once flow halted during the storage phase (Vanderzalm *et al.* 2006). Here the redox state progression reached methanogenesis in cycle 1 with a 109 day storage phase, and sulphate reduction in cycle 3 with a 325 days storage phase. In contrast, sulphate remained stable at the 4 m radius during the storage phase of both cycles.

There was evidence of arsenic release at both the ASR and 4 m wells during the first cycle storage phase. The effect was clearly greatest near the ASR well where arsenic peaked at 121 µg/L after 80 days of storage, before declining to 102 µg/L. The maximum arsenic concentration reached at 4 m, 14 µg/L, at the end of the storage phase remained an order of magnitude lower than in the ASR well, and similar to the concentrations in the early stages of injection. During the storage phase of the third cycle, an elevated arsenic concentration of 75 µg/L was seen after 123 days in the ASR well. The frequency of monitoring in this cycle did not allow the magnitude of the peak to be ascertained.

## DISCUSSION

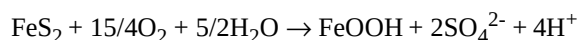
Arsenic mobilisation was apparent through the ASR cycle which necessitates the investigation of the dominant mechanisms behind this process. An average increase of 11 µg/L As in the recovered water when compared with the injected concentration of approximately 3 µg/L did not hinder use of recovered water for irrigation as the long-term trigger value for arsenic in water used for irrigation is 100 µg/L (Table 1).

**TABLE 1. Comparison of water quality guideline values for arsenic recovered from ASR schemes.**

| Arsenic(µg/L)          | Drinking water | Agriculture |           |
|------------------------|----------------|-------------|-----------|
|                        |                | Irrigation  | Livestock |
| USEPA, 2004            | 10             |             |           |
| WHO, 2004              | 10             |             |           |
| NHMRC & NRMCMC, 2004   | 7              |             |           |
| ANZECC & ARMCANZ, 2000 |                | 100†        | 500       |

† long-term trigger value

One possible explanation for the arsenic mobilisation is the oxidation of pyrite (eqn 1) in an oxic environment. The oxic environment was observed essentially during the early stage of injection. However, aside from an early peak in iron (also nickel and zinc) the concomitant increases in iron expected from pyrite oxidation were not observed. In an oxic environment the Fe(II) released from pyrite oxidation will be oxidised to Fe(III) which forms FeOOH or Fe(OH)<sub>3</sub>, depending on pH.



Alternatively, as attested by the presence of ankerite in the sediments, iron concentrations will also be controlled by carbonate equilibrium. The magnitude of the sulphate addition was not significant enough to be distinguished from the large

pools of native and injected sulphate at 270 mg/L and 210 mg/L respectively, nor to influence the sulphur-34 signature of sulphate.

The variability in arsenic concentration observed at 4 m was influenced by the spatially and temporally variable redox conditions encountered, with a trend toward higher arsenic concentrations under marginally more reducing conditions. In the first cycle, Eh measurements in the groundwater at 4 m were indicative of this relationship between arsenic mobility and redox state. Elevated arsenic was evident below a threshold Eh value of approximately 200 mV and attributed to the dominance of the uncharged reduced arsenite species,  $\text{H}_3\text{AsO}_3$  whereas under more oxidic conditions the charged species arsenate,  $\text{HAsO}_4^{2-}$ , could be removed via adsorption. However the limited Eh measurements in the third cycle did not illustrate the same relationship, as arsenic concentrations remained low ( $< 10$  g/L) under the 200 mV threshold during the injection phase. The lower arsenic concentrations at 4 m in the latter part of cycle 1 and again in cycle 3 were also coincident with nitrate persisting to 4 m, when concentrations above 1.4 mg/L  $\text{NO}_3\text{-N}$  were injected (Vanderzalm *et al.* 2006). Intuitively, higher oxidant concentrations could be expected to increase arsenic release through a greater potential for pyrite oxidation. However, in this case study while an oxidising environment induced pyrite oxidation it also allowed adsorption of As(V) in the charged  $\text{HAsO}_4^{2-}$ , while a lower Eh favoured release of As(III) in the uncharged  $\text{H}_3\text{AsO}_3$ .

The elevated arsenic concentrations in the near well zone were not evident on the outer fringe on the injected plume at the 50 m radius. Similar attenuation of arsenic with aquifer passage has previously been documented in dual well schemes via adsorption (de Ruiter and Stuyfzand, 1998). Given the variable behaviour of arsenic at 4 m it is likely that removal occurred soon after 4 m. Unfortunately, in this single well scheme, removal was reversed during recovery under a slightly more reducing condition where the more mobile neutral molecule,  $\text{H}_3\text{AsO}_3$ , again dominated. As a result, arsenic was mobilised from a zone extending at least 4 m from the ASR well (but not to 50 m).

Arsenic mobilisation through reducing conditions at neutral pH (Smedley and Kinniburgh, 2002), also explains the extreme increase in concentrations during the storage phase. Under these conditions, a concomitant increase in iron suggests reductive dissolution of arsenic-containing iron oxide surfaces. This mechanism provides the potential to exceed guideline values for irrigation (cycle 1), but this can be managed by discarding the initial volume of recovered water. The magnitude of this arsenic increase indicates that a considerable amount of arsenic was immobilised by iron oxides in the vicinity of the ASR well.

Comparison of arsenic concentrations at similar stages in each storage period gave a slightly higher arsenic concentration in cycle 3 than in cycle 1. In cycle 3 the arsenic

concentration was 75 µg/L after 123 days of storage (the end of the storage interval), while in cycle 1 the arsenic concentration was 50 µg/L after 109 days (or ~1800 days of the trial). At the same point in time, sulphate reduction was evident in both cycles, but was far more significant in the first storage phase where sulphate concentrations were reduced by 75%. The arsenic that was released was thought to have accumulated after pyrite oxidation and iron oxide precipitation on the wall face. The storage samples suggest that iron oxide dissolution released a similar (or greater) magnitude of arsenic in the third full ASR cycle as in the first. However localised sulphide precipitation under sulphate reducing conditions may have had more effect in reducing the aqueous arsenic concentration in the ASR well in the first cycle than the third.

In contrast, the arsenic concentration in recovered water after 15 minutes of pumping was 40 µg/L in cycle 3, compared with 88 µg/L in cycle 1. This may suggest the accumulation of arsenic on iron oxide surfaces in a zone surrounding the ASR well may have declined with successive ASR cycles.

## CONCLUSION

The injection of reclaimed water into the carbonate aquifer mobilised a source of arsenic through oxidation of traces of pyrite within the sediments. The mobility of this arsenic was then controlled by slight changes in the aqueous redox condition, which effectively attenuated arsenic as the injectant was transported through the aquifer. However, with reversal of flow during the recovery phase and return to the native redox condition arsenic was again rendered mobile and remained above the injected and ambient concentrations in the recovered water.

The importance of iron oxides for control of arsenic concentrations was illustrated by the dramatic increase of arsenic coincident with reductive dissolution of iron oxides in the ASR well during storage. While this example is not utilised for drinking water supply, it exemplifies the processes that are likely to impact on ASR schemes in carbonate aquifers. Despite ambient and injected water quality that meets guideline values, there is potential to exceed drinking water quality standards when arsenic is released from the sediments during ASR. It may be possible to maintain a sufficient redox condition within the storage zone to promote arsenic adsorption. However, in this example a shift to reducing conditions was induced by organic carbon within the reclaimed water injectant. Some net removal of arsenic adsorbed to iron oxide may be achieved during regular backwashing to remediate clogging of the ASR well.

## ACKNOWLEDGEMENTS

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# MODELING THE REDOX-DEPENDENT FATE OF A PHARMACEUTICALLY ACTIVE COMPOUND DURING MANAGED AQUIFER RECHARGE

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## ABSTRACT

The processes controlling the hydrochemistry and the related fate of the pharmaceutical residue phenazone as a result of pond infiltration were revealed and quantified by reactive transport modelling. A detailed hydrogeochemical dataset from 6 monitoring wells derived from a preceding field study in Berlin (Germany), provided the necessary constraints for the development and calibration of the model. The calibrated model reproduced the hydrochemical changes at all monitoring wells in great detail. Sensitivity analysis identified seasonal temperature changes as the key factor for the observed temporal and spatial dynamics of the redox chemistry at the investigated site. Variability in infiltration rates seems to be of less importance for the observed redox dynamics. More importantly, the model results indicate that the fate of phenazone is only indirectly dependent on the temperature variations and can mainly be attributed to the seasonal dynamics of dissolved oxygen concentrations within the aquifer. This explains why, rather surprisingly, phenazone is more persistent during summer months, leading to a seasonal occurrence at all monitoring wells during warmer conditions.

## KEYWORDS

Artificial recharge, pharmaceutically active compounds, redox zoning, temperature, reactive transport, PHT3D

## 1. INTRODUCTION

In recent years, pharmaceutically active compounds (PhACs, e.g., antibiotics, analgesics, lipid regulators, beta blockers and tranquilisers) have increasingly been detected in surface water and groundwater bodies (e.g., Heberer *et al.*, 2002; Ternes *et al.*, 2004) and their occurrence has been recognized as a critical issue for the feasibility and sustainability of managed aquifer recharge (Reddersen *et al.*, 2002;

Khan and Rorije, 2002; Heberer and Adam, 2004). Since the degradability of many PhACs depends on the prevailing redox conditions (e.g., Holm *et al.*, 1995; Khan and Rorije, 2002; Ternes *et al.*, 2004), detailed knowledge about the redox system and its controlling factors is fundamental in order to understand and quantify the fate of PhACs in managed aquifer recharge systems.

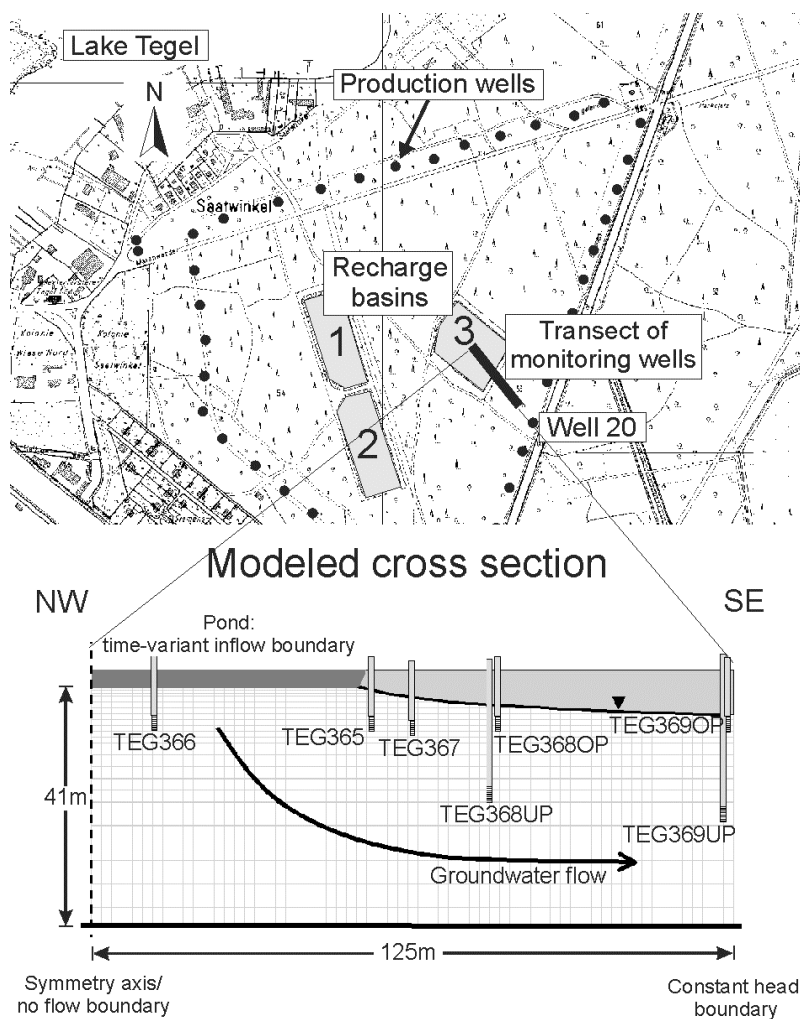
Rigorous, process-based reactive multi-component transport modelling can assist in the analysis and quantification of redox processes in artificial recharge systems. This has been successfully demonstrated for both laboratory (e.g., Amirbahman *et al.*, 2003) and field studies (e.g., Lensing *et al.*, 1994; van Breukelen *et al.*, 1998; Saaltink *et al.*, 2003; Prommer and Stuyfzand, 2005). However, none of these studies has comprehensively investigated the implications of redox dynamics on the fate of trace organic compounds. Furthermore, most of the studies neglected the significant effect of seasonal temperature variations of the infiltration water, although their impact on redox reaction rates in the subsurface has been reported previously (von Gunten *et al.*, 1994; Bourg and Bertin, 1994).

The present paper summarises a modelling study for an artificial recharge pond near Lake Tegel (Berlin, Germany), where temperature effects appeared to play an important role for the water quality changes downstream of the pond. Modelling has been carried out in order to: (i) give a consistent, process-based interpretation of the field observations reported by Massmann *et al.* (2006) and (ii) understand and quantify the impact of transient groundwater flow, transport and biogeochemical processes in combination with seasonal temperature changes on the observed redox dynamics and the corresponding fate of the PhAC phenazone. Phenazone is an analgesic substance that has been detected in the surface waters, groundwater and drinking water of Berlin (Reddersen *et al.*, 2002). The modelling concept reported herein may also be applied for simulating the fate of other redox-dependent trace organic compounds in comparable hydrogeochemical systems. Full details of the present study can be found in Greskowiak *et al.* (2006).

## 2. BACKGROUND

The investigated area is located inside a triangular shaped well-field operated by the *Berliner Wasserbetriebe* (BWB), as shown in Figure 1. In its centre, three recharge ponds are continuously used for infiltration of surface water derived from the nearby Lake Tegel. Infiltration occurs into Quaternary sediments of the upper aquifer consisting mainly of fluvial and glacio-fluvial, medium-sized sand deposits. The operation of the three ponds started in the 1960s. The infiltration area of the investigated recharge pond (pond 3) is approximately 8700 m<sup>2</sup>. The recharge rate varies considerably and ranges from 0 – 3.5 md<sup>-1</sup> (Darcy velocity) shown in Figure 2. These changes result from the repeated formation of a clogging layer at the bottom of the pond and its regular removal by BWB, approximately 3-4 times per year.

Between November 2001 and September 2004, Massmann *et al.* (2006) closely monitored the hydrochemistry of the groundwater at a number of observation wells that were installed along a transect between pond 3 and production well 20.



**FIGURE Figure 1: Study area (top) and schematic representation of the cross-sectional model (bottom). Note that the investigated/ modeled cross section (black line, left) does not extend to production well 20.**

The observations along the flow path from pond 3 towards well 20 (Massmann *et al.*, 2006) indicate increasingly reducing conditions, generally varying from aerobic and nitrate-reducing near the pond to manganese-reducing conditions further downstream. However, the position and the extent of each redox zone was found to

depend strongly on the seasonal temperature variations of the infiltration water. In summer, dissolved oxygen (DO) and nitrate become rapidly depleted and manganese reducing conditions develop in most parts of the investigated cross section. During winter, when the microbial activity is much lower, redox conditions in the aquifer generally remain aerobic. Anoxic conditions were only present in the deep monitoring well TEG369UP, located furthest from the pond. The observed temporal variability of phenazone concentrations suggested that its degradation and/or disappearance would strongly correlate with oxygen availability (Massmann *et al.* 2006). Phenazone degradation was previously reported to occur preferentially under oxic conditions (Sauber *et al.*; 1977, Zühlke, 2004), facilitated by the aerobic bacteria *Phenylobacterium immobile* (Lingens *et al.*, 1985). Sorption of phenazone to filter material, sludge or soil samples is reported to be negligible (Redderson *et al.*, 2002; Zühlke, 2004).

The present study investigates the groundwater flow and reactive transport along a vertical cross section indicated by the transect in Figure 1. Note that the modeled cross section does not extend to production well 20.

### 3. MODEL DEVELOPMENT

#### 3.1 Modelling Framework

Groundwater flow was simulated with the standard USGS flow model MODFLOW (McDonald and Harbaugh, 1988). Based on the computed flow-field the reactive multi-component transport model PHT3D (Prommer *et al.*, 2003) was used to simulate the fate of all major groundwater constituents and their interaction with solid phases. PHT3D couples the transport simulator MT3DMS (Zheng and Wang, 1999) and the geochemical reaction model PHREEQC-2 (Parkhurst and Appelo, 1999) through a sequential split operator technique. A reaction network representing all major, site-specific biogeochemical processes was developed and incorporated into the model's reaction database. For simplicity, heat transport was approximated by retarded advective-dispersive transport using a site-specific retardation factor of 2.1 (Prommer and Stuyfzand, 2005; Massmann *et al.*, 2006) across the model domain.

#### 3.2 Conceptual and Numerical Model

The aquifer surrounding the pond was simulated by a radial-symmetric two-dimensional vertical cross-sectional model. It was assumed that (i) the pond's shape could be approximated as a circle and (ii) the recharged water spreads radially from the pond. The model domain, discretisation and boundary-conditions are shown in Figure 1. For simplicity the model aquifer was assumed to be isotropic and homogeneous. The comparison between the simulated and measured temperature

changes provided the necessary constraints to evaluate the flow and physical transport processes.

The development of the conceptual hydrogeochemical model was based on the field investigations reported earlier in Greskowiak *et al.* (2005) and Massmann *et al.* (2006). The reaction network of the numerical model included (i) microbially mediated oxidation of soil organic matter (SOM) and the corresponding sequential consumption of the electron acceptors oxygen, nitrate and pyrolusite (Mn-oxide), (ii) dissolution/precipitation of calcite and rhodochrosite ( $\text{MnCO}_3$ ) and (iii) biodegradation of phenazone. Modeled phenazone degradation rates depended on the (simulated) concentration of dissolved oxygen and no degradation was assumed to occur under anaerobic conditions. Cation exchange reactions were not considered. While the model assumed that the groundwater is in equilibrium with respect to calcite and rhodochrosite, biodegradation of SOM and phenazone were treated as kinetic reactions and described by standard Monod-type rate expressions. To account for the temperature dependence of the biodegradation reactions, both the rate-expressions for SOM and phenazone degradation were extended by a term containing an empirical function  $f_T$ , as proposed by Kirschbaum (1995) and others:

$$f_T = \exp \left[ \alpha + \beta T \left( 1 - 0.5 \frac{T}{T_{\text{opt}}} \right) \right] \quad (\text{EQ 1})$$

where  $T$  is the water temperature in  $^{\circ}\text{C}$ ,  $\alpha$  and  $\beta$  are fitting parameters, and  $T_{\text{opt}}$  is the optimal temperature for biodegradation of organic matter and phenazone.

The total simulation time was 825 days, which was divided into 165 stress periods, starting from 17 June 2002. The time-varying fluxes and seasonal temperatures in the pond were defined for each stress period as 5-day-averages that were calculated from daily recharge and temperature values, respectively. The time-varying surface water quality has been accounted for by adjusting the inflow concentrations for each stress period based on monthly measurements. As the simulation starts in summer 2002, the initial groundwater composition represented a typical summer situation.

## 4. RESULTS AND DISCUSSION

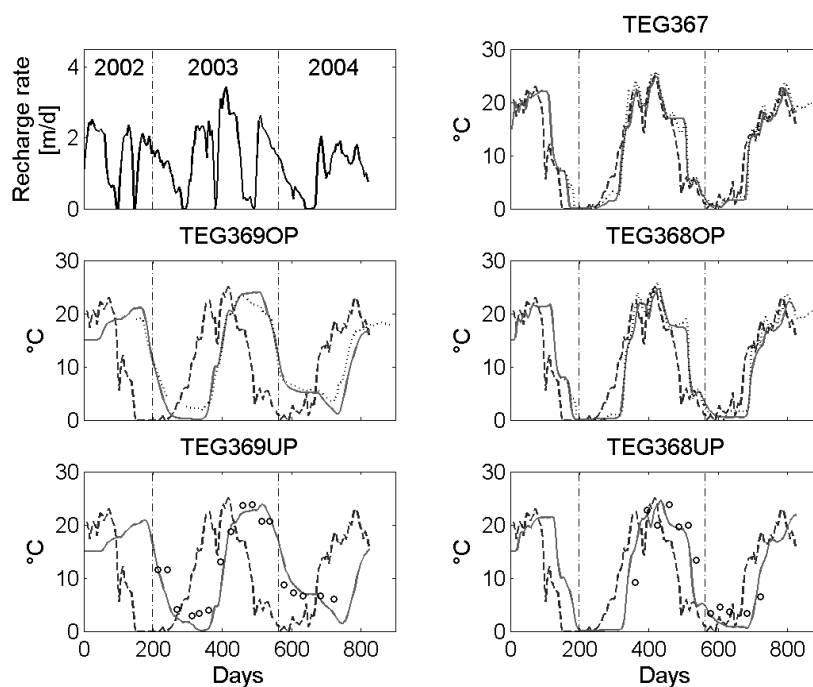
### 4.1 Non-reactive Transport

The non-reactive transport model, i.e., temperature simulations, closely describe the observed temperature variations at all of the monitoring wells (Figure 2). Note, that temperature data from the monitoring wells TEG365 and TEG366 were not used for calibrating the non-reactive transport model, since the measured temperatures were not clearly distinguishable from the pond water temperatures due to the short travel times to those wells. The accurate simulation of the temperature breakthrough

curves supports our simplifying assumptions with respect to the aquifer's hydrogeology.

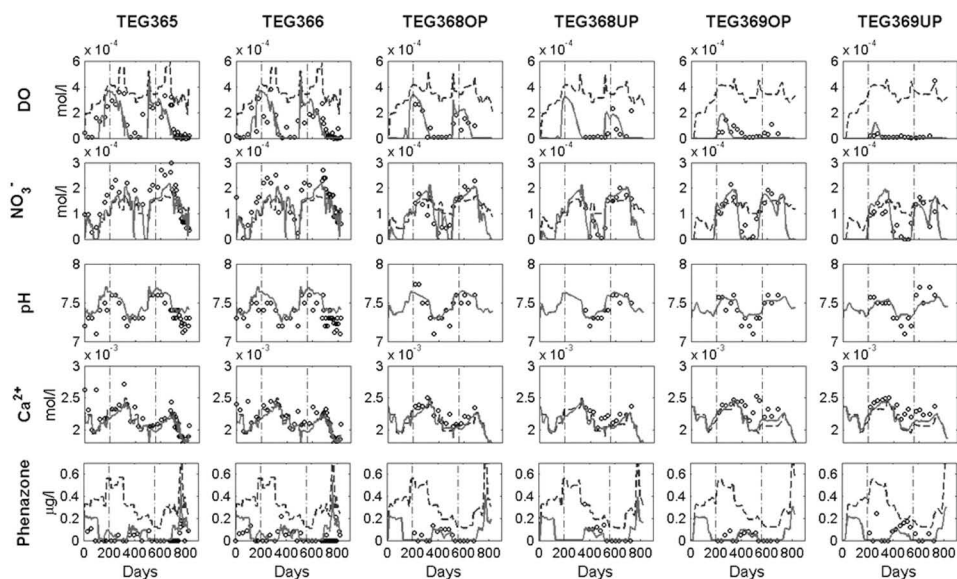
## 4.2 Simulated Redox Dynamics and Fate of Phenazone

The final calibrated reactive transport model reproduced the observed hydrogeochemical data at all of the monitoring wells in great detail (Figure 3). From the comparison with the corresponding non-reactive simulations, where all reactions were switched off, it can be seen that the fate of dissolved oxygen (DO), nitrate, dissolved  $Mn^{2+}$  and phenazone was considerably affected by reactive processes. During winter oxygen consumption within the aquifer is rather low. Although DO concentrations decline gradually with increasing distance from the pond, oxygen breakthrough could still be observed at almost all of the monitoring wells. The prevailing aerobic conditions during winter allow nitrification of organic nitrogen, which results in elevated nitrate concentrations at all monitoring wells compared to the pond water.



**FIGURE Figure 2: Temperatures of the pond water (dashed lines), and observed temperatures (circles for manual measurements and dotted lines for data logger) and simulated temperatures (solid lines) at the monitoring wells. Upper left plot: Infiltration rate at pond3.**

During the summer months DO concentrations became depleted before reaching TEG365 and TEG366, leading to anoxic conditions over almost the entire aquifer transect. However, only partial nitrate depletion occurred within the study zone. During summer months in particular, the model indicated that the observed and simulated breakthrough pattern of nitrate concentrations resulted from drastic changes in travel times due to the transient recharge rates. Overall the redox dynamics were well reproduced by the model for the entire simulation period. In addition, the simulated phenazone concentrations were in good agreement with the observed concentrations at all monitoring wells. The comparison with the non-reactive simulations showed that less phenazone was removed at higher temperatures. This result is somewhat surprising, since the degradation of phenazone is known to be microbially mediated.



**FIGURE 3. Simulated and observed concentration data in the years 2002-2004. The solid lines represents final calibrated simulations. The dashed lines display the corresponding non-reactive simulations. The circles show the observed concentrations.**

### 4.3 Carbonate Equilibrium

A comparison between the reactive and corresponding non-reactive simulation showed that the carbonate (not shown) and calcium concentrations (Figure 3) are not as dramatically affected by reactive processes as the redox components. Supplemental simulations, where the temperature of the pond water was held constant, showed that calcium and carbonate concentrations are insensitive to temperature variations. In contrast, pH showed considerable temperature

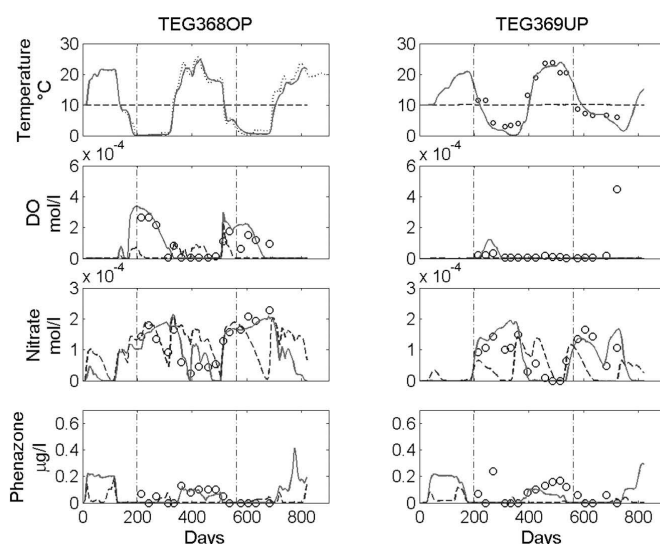


dependence. The lower pH at high temperatures (Figure 3) is thought to be due to the fact that protons (produced by the degradation of SOM) were buffered less, as the solubility of calcite is lower for higher temperatures. Further sensitivity analysis (not shown) revealed that the seasonal variability of the solubility constant of calcite is the main factor for the observed pH changes, rather than the seasonal variability of the generated protons due to SOM degradation: The observed pH variations can still be reproduced by the model when the temperature dependency of SOM degradation is inactivated.

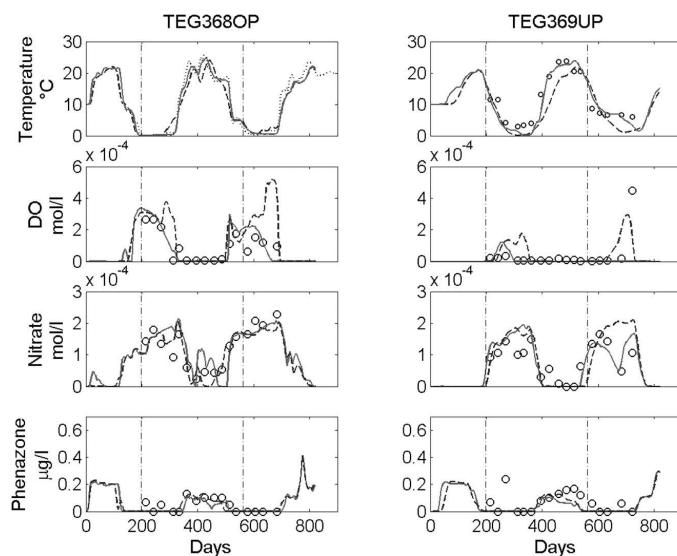
#### 4.4 Sensitivity Analysis

In order to evaluate the impact of temperature and time-varying recharge rates on the proposed biogeochemical reaction network separately, two additional simulations were carried out. In the first simulation the temperature of the pond water was held constant at the average value of 10 C. Typical results are illustrated by the monitoring wells TEG368OP and TEG369UP (Figure 4). It can be seen that the constant temperature simulation did not reproduce the seasonal dynamics of the redox components and phenazone at all. Even a re-adjustment of the calibrated rate constants for SOM and phenazone degradation was not able to improve the fit. In the second simulation the recharge rate was held constant and fixed to the average value of  $1.3 \text{ md}^{-1}$ . The temperature breakthrough curves of the constant recharge simulation slightly differ from the observed breakthrough curves and those of the final calibrated model, where transient flow was considered (Figure 5). However, the overall transport processes at the investigated site can still be reasonably well reproduced by the model when neglecting the dynamic behaviour of the infiltration rates. This is also reflected by the simulation of the redox-sensitive components. Although the constant recharge simulation could not replicate the measured concentrations as accurately as the final calibrated model, it could still reproduce the observed data to a higher degree than the constant temperature model.

Finally, sensitivity studies for which the redox-dependency and the temperature dependency of the phenazone degradation rate were activated/inactivated, revealed that the poor simulation of phenazone concentrations in the constant temperature simulation (Figure 4) solely results from the inadequate reproduction of the redox conditions (results not shown). It can be concluded that an explicit simulation of temperature dependence of the phenazone degradation rate is not essential to predict phenazone concentrations at this site. Thus, the degradation of phenazone seems to be far more sensitive to the availability of oxygen than to temperature variations, which explains why breakthrough of phenazone can be observed at higher temperatures during summer when DO becomes depleted, and vice versa.



**FIGURE 4.** Comparison of constant temperature simulation, final calibrated model, and observed concentrations and temperature data at TEG368OP and TEG369UP. The dashed lines display the constant temperature simulation at 10°C. The solid lines represent the final calibrated simulations. In the temperature plots, the dotted line and the circles represent the measured water temperatures from the data logger and manual measurements, respectively.



**FIGURE 5.** Comparison of constant recharge simulation, final calibrated model, and observed concentrations and temperature data at TEG368OP and TEG369UP. The dashed lines display the constant recharge simulation for an infiltration rate of 1.3 md<sup>-1</sup>. The solid lines represent the final calibrated simulations. In the temperature plots, the dotted line and the circles represent the measured water temperatures from the data logger and manual measurements, respectively.

## 5. CONCLUSIONS

Reactive multi-component reactive transport modeling has been carried out in order to identify and quantify the processes affecting the water quality changes downstream of an infiltration pond in Berlin, Germany. It was demonstrated that the seasonal changes in pond water temperature were largely responsible for the observed dynamic changes in the redox zonation rather than the high variability of infiltration rates. It was further shown that phenazone degradation rates are more sensitive to the availability of dissolved oxygen, rather than temperature changes. However, the former leads to an indirect temperature dependency, which explains why degradation of phenzone was observed to be higher during winter. Moreover, the seasonal pH variations could mainly be attributed to the temperature dependence of the solubility of calcite.

In general terms, this study clearly demonstrates the capability of mechanistic multi-component reactive transport modelling to assist in the interpretation and in-depth analysis of hydrogeological and hydrogeochemical field data, and thus to verify developed/existing hypotheses of how physical and biogeochemical processes impacting the water quality evolution during managed aquifer recharge.

## ACKNOWLEDGEMENTS

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## BEHAVIOR OF PHARMACEUTICALS AND OTHER EMERGING POLLUTANTS IN VARIOUS ARTIFICIAL RECHARGE SYSTEMS IN THE NETHERLANDS

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### ABSTRACT

Basin recharge and deep well recharge systems contribute ca 15% to the annual total of drinking water (1,250 Mm<sup>3</sup>/a) produced in the Netherlands. With 2 snapshot surveys in summer 2005 and 2006, of 10 monitoring transects of 2-4 observation wells in 3 basin artificial recharge (BAR) and 2 deep well injection (DWI) systems, insight has been gained in the behavior of 140 organic micropollutants (<0.001 – 5 µg/L). 80 of these are pharmaceuticals and other emerging pollutants (PEPs) and 60 regulated organic pollutants (ROPs).

Most PEPs (71%) and ROPs (60%) were everywhere below the limit of quantification. Concentrations of detected pharmaceuticals and X-ray contrast agents were low (0.01-0.2 µg/L). They showed significant (bio)degradation (>90%) during aquifer passage regarding phenazone, iohexol, iomeprol and iopamidol in suboxic environment, and of sulfamethoxazole and amidotrizoic acid in anoxic environment. Carbamazepine, MCPP, bentazone, tertiary octylphenole, iso-nonylphenole, PFOA, PFOS, TCEP, 1,4-dioxane and diglyme showed a very persistent behavior, both in young, suboxic infiltrates and in 1-36 years old, anoxic infiltrates. Phenazone and iopamidol showed a persistent behavior only in anoxic environment, and sulfamethoxazole, amidotrizoic acid and carbendazim only in suboxic environment.

Uncertainty levels in half lives (as a relative measure of decay rates) of organic micropollutants in BAR and DWI systems with low input levels, as deduced from a snapshot survey, are related to (a) analytical noise, (b) uncertainties in (reconstructed) input concentration, travel time of the water and sorption (retardation factor), and (c) variations in temperature and redox environment. For iopamidol in suboxic environment the uncertainties result in a range of half lives down to half and up to twice the expected mean value (58 days).

### KEYWORDS

Artificial recharge, coastal dunes, pharmaceuticals, emerging pollutants, degradation rate, redox environment, uncertainty analysis

## INTRODUCTION

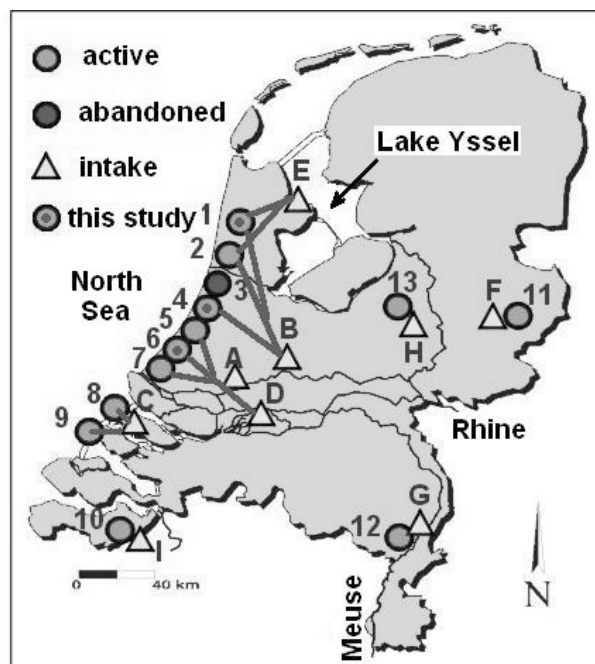
Emerging pollutants are defined as pollutants that are currently not included in routine monitoring programmes nor in regulation, but which deserve attention due to their suspected (eco)toxicity, potential health effects, public perception and detection in various environmental compartments. Pharmaceuticals and other emerging pollutants (together PEPs) have been detected in low concentrations (0.001 -10 µg/L) internationally, in decreasing order, in (treated) sewage effluents, surface waters, surface waters pretreated for artificial recharge (AR), river bank filtrate (RBF), the recovered infiltrates after AR, and drinking water (Ternes & Hirsch 2000; Drewes et al. 2002; Zühlke et al. 2004). This has been demonstrated also in the Netherlands (Noy et al. 2003).

Gradually insight is gained in the occurrence and behavior of various PEPs in RBF and AR groundwater flow systems, as a function of the redox environment and travel time, especially in Germany (Massmann et al., 2006). No such information was available, however, on PEPs in AR systems in the Netherlands, which contribute 15% to the annual total of drinking water supply of 1,250 Mm<sup>3</sup>/a. This study aims at filling up this knowledge gap in order to (a) predict concentrations in the recovered water also on the long term; and if needed (b) optimize the existing drinking water production systems utilizing AR, by either changing the redox environment and travel time or intensifying the pre- or post-treatment.

## MATERIAL AND METHODS

Three dune areas with artificial recharge were investigated on the occurrence and their elimination capacity of PEPs (Fig.1): one close to Castricum (operated by Provincial Waterworks North-Holland), one south of Zandvoort (operated by Waternet supplying drinking water to the city of Amsterdam) and one north of The Hague (operated by Duinwatermaatschappij Zuid-Holland).

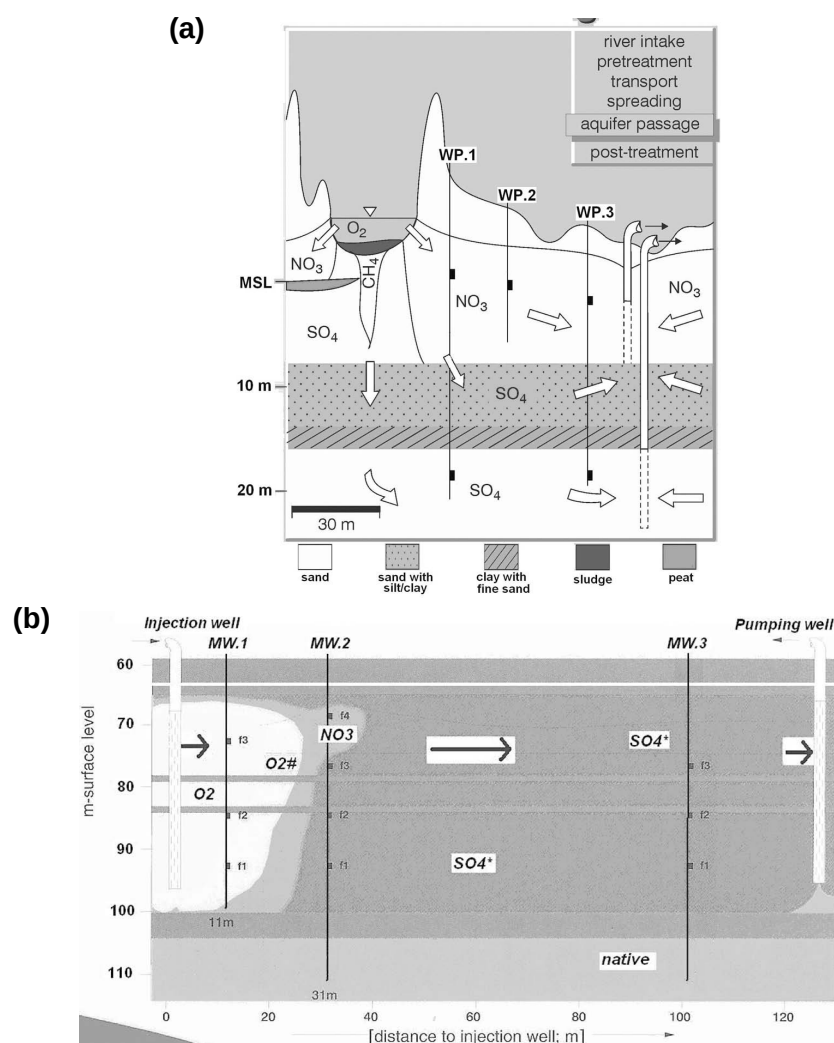
Eight rows of observation wells, in between different recharge basins and recovery wells or drains (Fig.2-a), and 2 rows of observation wells in between an injection and recovery well (Fig.2-b) were sampled to determine in 2 snap-shot surveys (summer 2005 and 2006) the spatial distribution of ca 80 PEPs, 60 regulated organic pollutants (ROPs: pesticides, volatile organohalogenes and volatile benzenes), DOC (Dissolved Organic Carbon) and inorganic chemistry along well defined flow paths in diverging redox systems. Most observation wells (screen length 0.5-1 m) have a long monitoring record (10-40 years) on water levels, macroparameters and various pollutants (not PEPs), thus supplying background information on groundwater flow and hydrogeochemical processes. For some PEPs a 2-5 years input record was available, of the water after pretreatment and/or of the Rhine and Meuse Rivers at national monitoring points.



**FIGURE 1. Site map showing all basin aquifer recharge systems in the Netherlands, incl. the 3 studied recharge areas (numbered 1,4 and 6 on map) and their water intake (resp. E+B, B, A+D). Intake A abandoned in 1978. Red lines = transport mains from pretreatment plant near intake to AR area.**

Monitoring wells were selected that were installed >4 years ago and are regularly pumped, thus avoiding problems due to PVC interaction. Samples were taken after evacuation of >3 times the volume of well screen (length 1 m) plus riser (diameter 2.5 cm), using a rotary peristaltic pump. Ultraclean bottles were used for PEPs. The samples for main cations, Si, P and TEs were filtrated in the field over 0.45 µm and subsequently acidified. On site, the samples were immediately cooled down to 4°C and kept in the dark. Temperature, EC and pH were measured on site, main constituents by routine methods, main cations, Si, P and TEs by ICP-MS and ICP-AES.

The following PEPs were analysed within 1 week in an extract of each sample that was obtained within 1-2 days after sampling: pharmaceuticals and X-ray contrast fluids by Solid Phase Extraction (SPE) HPLC-MS-MS, nitrosoamines, alkylphenols, perfluoro compounds, various individual industrial compounds, hormones and Endocrine Disruptive Compounds (EDCs) by Liquid Liquid Extraction (LLE) with ethylacetate and detection by LC-GC-MS, volatile organics by purge and trap isolation and GC-MS detection, diglyme by SPE-GC-MS.



**FIGURE 2. Typical monitoring system (piezometer nests) and redox zonation in basin aquifer recharge (a) and deep well injection systems (b).  $O_2$  = Oxidic (>90%  $O_2$  saturated);  $O_2\#$  /  $NO_3$  = suboxic ( $O_2$  10-100% depleted,  $NO_3 > 0.5$  mg/L,  $Fe = Mn < 0.1$  mg/L);  $SO_4(*)$  = Anoxic ( $O_2 = NO_3 < 0.5$  mg/L,  $Fe^{2+} > 0.1$  mg/L,  $SO_4 < 10\%$  depleted,  $CH_4 < 0.5$  mg/L);  $CH_4$  = deep anoxic ( $O_2 = NO_3 < 0.5$  mg/L,  $SO_4$  10-100% depleted or  $CH_4 > 0.5$  mg/L).**

Travel times in the aquifer (in our observation wells ranging from 1 day to 40 years) were determined by history matching of Cl,  $SO_4$ ,  $^3H$  and temperature (retardation factor 1.8-2) in both the input and observation wells, and by hydrological



calculations using piezometric levels and well known hydraulic conductivities. The redox environment was assessed by data on  $O_2$ ,  $NO_3^-$ ,  $SO_4^{2-}$ ,  $Fe^{2+}$ ,  $Mn^{2+}$  and  $NH_4^+$ , using the classification system indicated in Fig.2 (Stuyfzand, 1993, 2005).

**TABLE 1. The 10 monitoring transects of observation wells with important background information.**

| Recharge area               | Transect No. | Recharge Type \$ | AR start | Source water                                       | No. wells | distance [m] | depth [m+MSL] | travel time | redox envir # | Geol. Formation @ |
|-----------------------------|--------------|------------------|----------|----------------------------------------------------|-----------|--------------|---------------|-------------|---------------|-------------------|
| <b>CASTRICUM (PWN)</b>      |              |                  |          |                                                    |           |              |               |             |               |                   |
| ICAS - shallow              | 1            | BAR              | 1957     | R <sup>1</sup> , R/Y <sup>2</sup>                  | 5         | 1-64         | 1 / -3        | 2-56 d      | SO            | D                 |
| ICAS - deep                 | 2            | BAR              | 1957     | R <sup>1</sup> , R/Y <sup>2</sup>                  | 4         | 34-340       | -34 / -52     | 3->36 y     | A             | E                 |
| Watervlak                   | 3            | DWI              | 1989     | R <sup>1</sup> , R/Y <sup>2</sup>                  | 3         | 10-73        | -67 / -97     | 18-103 d    | SO-A          | En                |
| <b>ZANDVOORT (Watermel)</b> |              |                  |          |                                                    |           |              |               |             |               |                   |
| Basin 6 - shallow           | 4            | BAR              | 1957     | R <sup>3</sup>                                     | 2         | 4-96         | 5 / -3        | 2-49 d      | SO            | D                 |
| Basin 6 - deep              | 4A           | BAR              | 1957     | R <sup>3</sup>                                     | 2         | 4-96         | -8 / -14      | 75-351 d    | A             | NS                |
| Basin 12 - shallow          | 5            | BAR              | 1957     | R <sup>3</sup>                                     | 4         | 3-57         | 5 / 4         | 8-57 d      | SO            | D                 |
| Van der Vlietkan.           | 6            | BAR              | 1957     | R <sup>3</sup>                                     | 5         | 95-500       | -2 / -20      | 3.5->26 y   | A             | NS                |
| <b>THE HAGUE (DZH)</b>      |              |                  |          |                                                    |           |              |               |             |               |                   |
| Basin 13.1 - shallow        | 7            | BAR              | 1956     | R <sup>4</sup> , R/M <sup>5</sup> , M <sup>6</sup> | 4         | 3-37         | -2            | 20-48 d     | SO            | D                 |
| Basin 13.1 - deep           | 8            | BAR              | 1956     | R <sup>4</sup> , R/M <sup>5</sup> , M <sup>6</sup> | 3         | 2-100        | -16 / -54     | 1.7-11.4 y  | (D)A          | NS/E/U            |
| DWAT                        | 9            | DWI              | 1989     | M <sup>6</sup>                                     | 3         | 23-120       | -31 / -34     | 21-112 d    | SO-A          | E/U               |

\$: BAR = Basin Aquifer Recharge; DWI = Deep Well Injection

1 = Rhine River till 1981, filtrated and since 1974 also coagulated

2 = Mix of Rhine River and Lake Yssel since 1981, both coagulated, Y also with activated carbon filtration since 1989

3 = Rhine River, coagulated since 1974

4 = Rhine River till 1978, filtrated; 5 = Mix of Rhine and Meuse Rivers 1978-1978, both coagulated;

6 = Meuse River since 1978, coagulated; 7 = Meuse River water, twice coagulated, since 1986 also activated carbon filtration;

#: SO = (Sub)Oxic; A = Anoxic; DA = Deep Anoxic; (D)A = (Deep) Anoxic

@: D = Holocene dune sand; NS = Holocene North Sea deposits; E = Eemian Formation (marine, Pleistocene);

U = Urk and Sterksel Formation (fluvial, Pleistocene); En = Enschede Formation (fluvial, Pleistocene)

## HYDROGEOLOGICAL AND GEOCHEMICAL SETTING

Relevant background information on the 10 transects is given in Table 1. There are 4 shallow transects in the upper phreatic aquifer (1, 4, 5, 7), which is recharged via basins and composed of Holocene coastal dune sands on top of North Sea sand deposited in a beach and shallow marine environment. These sands are mostly suboxic, calcareous, poor in organic carbon (ca. 0.01-0.1%), medium grained, well sorted and relatively homogeneous. Travel times and distances are relatively short (<100 d, <100 m).

The 4 deep transects (2, 4A, 6, 8) reach basin infiltrate in anoxic, calcareous, pyritiferous, semiconfined aquifer layers below the upper aquifer. These layers consist of Pleistocene coarse grained, marine and fluvial sands (2, 8) or heterogeneous, fine grained, silty North Sea sands deposited during the Holocene in a tidal or estuarine environment (4A, 6). Travel times and distances are relatively long (75 d - >36 y, 100-500 m). The deep transects 2 and 8 consist of dispersed monitoring wells projected on an imaginary cross section, the others are based on a row of observation wells perpendicular to a basin's bank.

Both transects of a deep well injection system (3, 9) address a deep, semiconfined, coarse sandy, Pleistocene aquifer which is calcareous, pyritiferous and low in organic carbon (0.05-0.11%). The system is suboxic close to the injection well and turns anoxic at a distance of 20-40 m. Travel times and distances are relatively short (<112 d, <120 m).

A typical cross section with the position of monitoring wells and redox zonation, is presented for basin aquifer recharge systems and deep well injection systems in Fig.2.

## PRESENTATION OF ANALYTICAL RESULTS

The analytical results for all detected and quantified PEPs and ROPs during the 2 snapshots are given in Table 2. The results refer to the snapshots in 2005 and 2006, each of which totalling 30 samples (4-5 input, 25-26 monitoring wells). In total 23 out of 80 PEPs and 24 out of 60 ROPs have been detected and quantified during at least one of both snapshots.

Differences in detection frequency and concentration levels of the PEPs and ROPs, between the snapshot of 2005 and 2006, are mainly caused by: (a) differences in analytical program; (b) small differences in the analytical methods applied, also leading to different limits of quantification (LOQs; samples of 2005 by OMEGAM in Amsterdam, those of 2006 by TZW in Karlsruhe); (c) a different selection of sampled wells (21 wells identical, 9 wells different); and (d) fluctuating input levels.

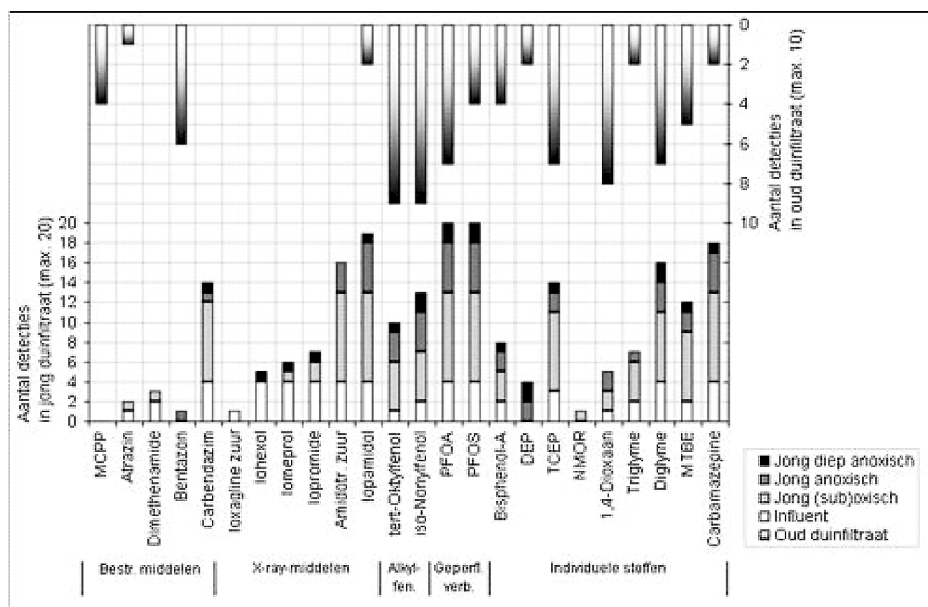
Nevertheless quite comparable results have been obtained for the pharmaceuticals, X-ray contrast agents, diglyme and MTBE. Bisphenol-A is the only exception, with significantly higher concentrations in 2006, probably due to a better isolation and quantification with the method applied in 2006.

Somewhat lower frequencies of detection in 2006 (Table 2) are related to more focus on relatively 'old' infiltration waters by addressing 9 more observation wells in transects 2, 6 and 8. A more detailed picture of the number of detections in 2006 is given in Fig.3, where young (n=20) and old dune infiltrates (n = 10) are distinguished. The young group is further subdivided into the categories influent, suboxic, anoxic and deep anoxic (for redox criteria see subtitle of Fig.2).

It can already be concluded from Fig.3 that the following pollutants, with frequent detection in old infiltrates, are very persistent and mobile, and had a higher input in the past (>3 years ago): MCP, bentazone, tertiary octylphenole, iso-nonylphenole, PFOA, TCEP, 1,4-dioxane and diglyme. Also persistent but with possibly a lower input in the past (>3 years ago) or less mobile/persistent than the previous pollutants are: carbendazim, iopamidol, PFOS and carbamazepine.

TABLE 2. Detected Pharmaceuticals and other emerging pollutants (PEPs) and regulated organic micropollutants (pesticides, volatile organohalogenes, benzenes), with their Limit of Quantification (LOQ), frequency of detection (No. > LOQ; max = 30), and maximum concentration levels. Cell colouring: black = No.  $\geq 10$ , Max  $\geq 0.1$   $\mu\text{g/L}$ ; red = No.  $\geq 10$ , Max < 0.1  $\mu\text{g/L}$ ; blue = No. < 10, Max  $\geq 0.1$   $\mu\text{g/L}$ ; white = No. < 10, Max < 0.1  $\mu\text{g/L}$ .

| Type                     | PEP                               | 2005            |       |                 | 2006            |       |                 |
|--------------------------|-----------------------------------|-----------------|-------|-----------------|-----------------|-------|-----------------|
|                          | or                                | LOQ             | No.   | Max             | LOQ             | No.   | Max             |
|                          | organic micropollutant            | $\mu\text{g/L}$ | > LOQ | $\mu\text{g/L}$ | $\mu\text{g/L}$ | > LOQ | $\mu\text{g/L}$ |
| Pharmaca                 | carbamazepine                     | 0.01            | 26    | 0.10            | 0.03            | 20    | 0.14            |
|                          | phenazone                         | 0.01            | 10    | 0.20            |                 |       |                 |
|                          | primidone                         | 0.02            | 3     | 0.07            |                 |       |                 |
|                          | sulfamethoxazole                  | 0.01            | 18    | 0.04            | 0.03            | 0     |                 |
| X-ray contrast agents    | amidotrizoic acid                 | 0.01            | 25    | 0.20            | 0.01            | 16    | 0.25            |
|                          | iohexol                           | 0.01            | 4     | 0.08            | 0.01            | 5     | 0.04            |
|                          | iomeprol                          | 0.01            | 5     | 0.07            | 0.01            | 6     | 0.08            |
|                          | iopamidol                         | 0.01            | 30    | 0.20            | 0.01            | 21    | 0.14            |
|                          | iopromide                         | 0.01            | 6     | 0.10            | 0.01            | 7     | 0.09            |
|                          | ioxagline acid                    | 0.01            | 1     | 0.01            | 0.01            | 1     | 0.01            |
|                          | ioxitalamine acid                 | 0.01            | 2     | 0.01            | 0.01            | 0     |                 |
| EDCs                     | Bisphenol-A                       | 0.05            | 0     |                 | 0.005           | 12    | 0.16            |
|                          | Diethylphthalate (DEP)            |                 |       |                 | 0.5             | 6     | 0.87            |
|                          | iso-Nonylphenol                   |                 |       |                 | 0.025           | 22    | 0.65            |
|                          | tert-Octylphenol                  |                 |       |                 | 0.005           | 19    | 0.09            |
| Various                  | 1,4-Dioxane                       |                 |       |                 | 0.1             | 13    | 4.8             |
|                          | diglyme                           | 0.01            | 29    | 0.39            | 0.1             | 23    | 0.74            |
|                          | methyl-tert-butyl-ether (MTBE)    | 0.02            | 30    | 0.86            | 0.05            | 17    | 0.86            |
|                          | Perfluorooctanoic acid (PFOA)     |                 |       |                 | 0.001           | 27    | 0.017           |
|                          | Perfluorooctanosulfonate (PFOS)   |                 |       |                 | 0.001           | 24    | 0.039           |
|                          | Triglyme                          |                 |       |                 | 0.1             | 9     | 0.57            |
|                          | Tris(chloroethyl)phosphate (TCEP) |                 |       |                 | 0.025           | 21    | 0.34            |
|                          | n-nitrosomorpholine (HMOR)        | 0.003           | 0     |                 | 0.001           | 1     | 0.001           |
| Pesticides               | Atrazine                          |                 |       |                 | 0.01            | 3     | 0.01            |
|                          | Carbendazim                       |                 |       |                 | 0.01            | 14    | 0.06            |
|                          | Dimethanamide                     |                 |       |                 | 0.01            | 3     | 0.01            |
|                          | Bentazone                         |                 |       |                 | 0.05            | 7     | 0.55            |
|                          | MCPP                              |                 |       |                 | 0.05            | 4     | 0.25            |
| Volatile organohalogenes | trichloromethane                  | 0.02            | 19    | 0.37            |                 |       |                 |
|                          | bromodichloromethane              | 0.02            | 2     | 0.15            |                 |       |                 |
|                          | dibromochloromethane              | 0.02            | 1     | 0.12            |                 |       |                 |
|                          | tribromomethane                   | 0.02            | 4     | 0.10            |                 |       |                 |
|                          | 1,2-dichloroethane                | 0.02            | 4     | 0.28            |                 |       |                 |
|                          | cis-1,2-dichloroethylene          | 0.02            | 4     | 0.29            |                 |       |                 |
|                          | trans-1,2-dichloroethylene        | 0.02            | 2     | 0.05            |                 |       |                 |
|                          | trichloroethylene                 | 0.02            | 2     | 0.06            |                 |       |                 |
|                          | 1,2-dichloropropane               | 0.02            | 3     | 0.06            |                 |       |                 |
| Benzenes                 | benzene                           | 0.02            | 1     | 0.03            | 0.1             | 0     |                 |
|                          | methylbenzene                     | 0.02            | 12    | 0.50            |                 |       |                 |
|                          | 1,2-dimethylbenzene               | 0.02            | 2     | 0.10            |                 |       |                 |
|                          | 1,3 and 1,4-dimethylbenzene       | 0.02            | 14    | 0.13            |                 |       |                 |
|                          | 1,2,4-trimethylbenzene            | 0.02            | 5     | 0.36            |                 |       |                 |
|                          | 1,3,5-trimethylbenzene            | 0.02            | 3     | 0.13            |                 |       |                 |
|                          | ethylbenzene                      | 0.02            | 3     | 0.26            |                 |       |                 |
|                          | iso-propylbenzene                 | 0.02            | 1     | 0.09            |                 |       |                 |
|                          | n-propylbenzene                   | 0.02            | 2     | 0.16            |                 |       |                 |
|                          | chlorobenzene                     | 0.02            | 1     | 0.02            |                 |       |                 |



**FIGURE 3. Detection frequencies of the 19 most important PEPs plus 5 pesticides in 2006, in young dune infiltrates (4 inputs + 16 monitoring wells; water age < 1 year) and old dune infiltrates (10 monitoring wells, water age 3-40 years). The young dune infiltrate is further subdivided into influent (max. 4), suboxic (max. 9), anoxic (max. 5) and deep anoxic (max. 2). The old dune infiltrates are anoxic.**

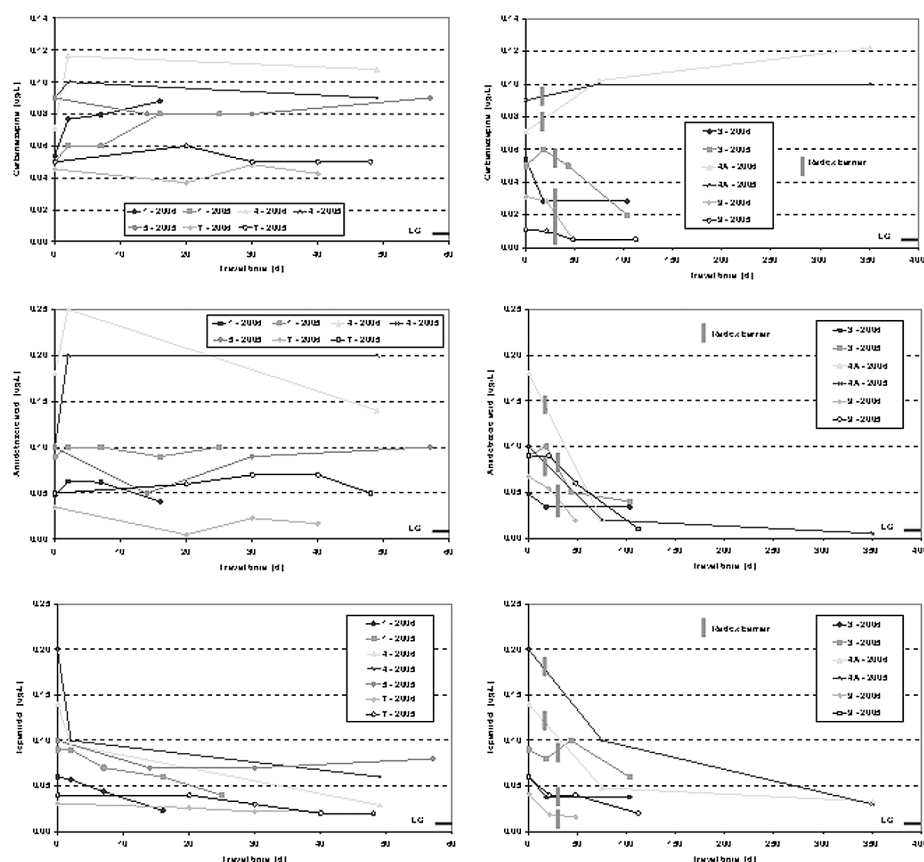
## BEHAVIOR IN YOUNG INFILTRATES, IN SUBOXIC AND ANOXIC ENVIRONMENT

Young infiltrates are defined here as those with a travel time <1 year. The behavior of PEPs in these groundwaters can only be deduced from snapshot surveys when their input signal is known, and when retardation by sorption and effects of filtration of suspended matter can be ignored or accounted for.

The PEPs with a well known input record are indicated in Table 3. In general, pharmaceuticals and X-ray contrast agents appear to be much less variable in the input than pollutants like MTBE and diglyme. The physicochemical properties of most PEPs (Table 3) justify the assumption that sorption to suspended matter and to the aquifer matrix can be neglected in most cases. Carbamazepine is an exception, showing a calculated retardation coefficient for dune sand  $R = 2.2$ . Its sorption to suspended and dissolved organic material can be neglected however.

The behavior of carbamazepine, amidotrizoic acid and iopamidol is shown in Fig.4, for both the suboxic and anoxic transects with a travel time <1 year. In the anoxic

transects the position of the redox barrier is indicated, where suboxic infiltrate (upgradient) is reduced to anoxic infiltrate. At this point the graphs can be misleading: the changes between 2 points on both sides of this barrier probably take place exclusively on one side. The patterns for carbamazepine are mainly dictated by fluctuations in the input indicating that degradation is insignificant. This is confirmed by the persistence in anoxic old infiltrate (see below) and observations by Massmann et al. (2006). Amidotrizoic acid and iopamidol show a contrasting redox dependent behavior: the first is clearly only degraded in anoxic environment, and the latter mainly in suboxic environment.



**FIGURE 4. Left: concentration profile along the shallow, suboxic transects 1 (Castricum ICAS), 4 (Zandvoort Basin 6), 5 (Zandvoort, Basin 12) and 7 (The Hague, Pond 13.1), for summer 2005 and 2006. Right: concentration profile along the deep, anoxic transects 3 (Castricum DWAT), 4A (Zandvoort Basin 6), and 9 (The Hague, Waalsdorp), for summer 2005 and 2006.**

The behavior of other PEPs and 3 pesticides is summarized in Table 3. It can be concluded that phenazone is rapidly degraded in suboxic conditions and very slowly in anoxic environment. This matches the conclusions of Massmann et al. (2006). Iohexol, iomeprol and iopromide show such a fast degradation in suboxic environment, that their behavior in anoxic environments, which in our study are preceded by suboxic conditions, could not be determined here. Grünheid & Jekel (2006) noticed that iopromide hardly degraded in anoxic environment.

**TABLE 3. Preliminary conclusions regarding the behavior of PEPs and 3 pesticides in all studied transects. In yellow: pollutants with much higher loads in the remote past (1980s).**

|       | PEP               | logK <sub>OW</sub> | pKa | Input signal# | Removal process |            |          |         | T <sub>1/2</sub> [d] † |            |
|-------|-------------------|--------------------|-----|---------------|-----------------|------------|----------|---------|------------------------|------------|
|       |                   |                    |     |               | Basin##         | Filtration | Sorption | Biodegr | (Sub)oxic              | Anoxic     |
| Phac  | Carbamazepine     | 2.45               |     | 1             | ?               | no         | R=2.2 \$ | <10     | 9999                   | >7300      |
|       | Phenazone         | 0.38               |     | ?             | ?               | no         | no       | 0-100   | 2                      | >> 730     |
|       | Sulfamethoxazole  | 0.89               | 6   | 1             | ?               | no         | no       | 0-70    | 9999                   | 30         |
|       | Amidotrizoic acid | 1.37               |     | 1             | 45              | no         | no       | 0-70    | 9999                   | 25 - 55    |
| X-ray | Iohexol           | -3.05              |     | 1             | 0-40            | no         | no       | >90     | <1 - <7                | no data    |
|       | Iomeprol          | low                |     | 1             | 0-20            | no         | no       | >90     | <0.5 - <6              | no data    |
|       | Iopamidol         | -2.42              |     | 1             | 30              | no         | no       | 0-90    | 25 - 85                | 140 - 9999 |
|       | Iopromide         | -2.05              |     | 2             | 20-40           | no         | no       | >90     | 0.7 - <7               | no data    |
| Var   | 1,4-Dioxane       | -0.27              |     | ?             | prob            | no         | no       |         | 9999                   | 9999       |
|       | Diglyme           | -0.36              |     | 3             | ?               | no         | no       | <20     | 9999                   | 9999       |
|       | MTBE              | 1.07               |     | 3             | prob            | no         | no       | 0-70    | 9999?                  | 9999?      |
|       | Bentazone         | -0.46              | 2.9 | 4             | ?               | no         | no       | <20     | ?                      | >2200      |
| Pest  | Carbendazim       | 1.56               |     | ?             | ?               | no         | no       | 0-90    | 9999                   | 25         |
|       | MCP               | 0.1                | 3.8 | 4             | ?               | no         | no       | <20     | ?                      | >2200      |

# : 1-3 = well known since 2004; 1 = low variability; 2 = moderate variability; 3 = high variability; 4 = well known since 1988, low variability

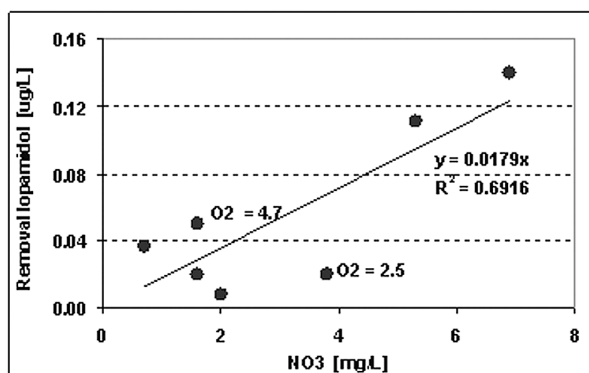
## : removal by volatilization or photolysis [%]; prob = probable but not quantified

\$ = in dune sand

† : 9999 = >365 days probably, hard to quantify

Sulfamethoxazole and carbendazim are both slowly/hardly degraded in suboxic environment and much more rapidly in anoxic environment. Grünheid & Jekel (2006) demonstrated this for sulfamethoxazole as well.

The elimination rate of iopamidol in suboxic environment seems to depend on the concentration level of nitrate (Fig.5): the higher nitrate the higher the

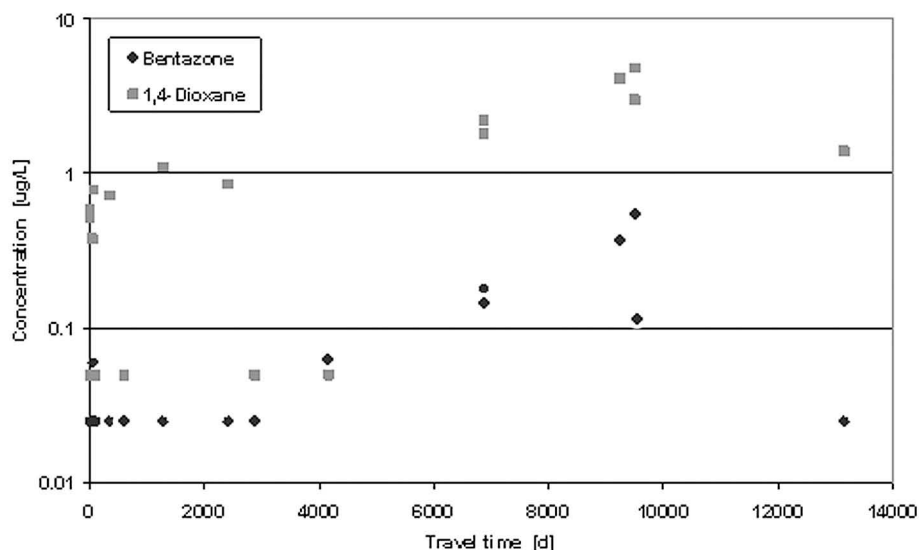


**FIGURE 5. Mean removal of iopamidol as a function of mean nitrate concentration, at the shallow, suboxic transects 1 (Castricum ICAS), 4 (Zandvoort Basin 6), 5 (Zandvoort, Basin 12) and 7 (The Hague, Pond 13.1), for summer 2005 and 2006**

elimination rate. Probably nitrate is a surrogate here for the oxidation capacity of water (in this environment being  $4\text{O}_2 + 5\text{NO}_3^-$  in me/L)

## BEHAVIOR IN OLD INFILTRATES, IN ANOXYC ENVIRONMENT

Old infiltrates are defined here as those with a travel time >1 year. All 14 pollutants listed in Table 3 have been present in the source and infiltration waters for more than at least 5 years. Their exact input signal is largely unknown, although the earliest data on bentazone, MCPP and diglyme data back from the late 1980s. Obviously the input levels of bentazone and 1,4-dioxane were much higher in the 1980s than today (Fig.6). Other pollutants that are present in older, anoxic infiltrate, are MCPP (mecoprop), diglyme, MTBE and carbamazepine. Their presence demonstrates their resistance against (bio)degradation.



**FIGURE 6. Concentration of bentazone and 1,4-dioxane in dune infiltrates as a function of the travel time of water along all 9 transects. Their presence in old infiltrates (travel time >365 days) demonstrates their persistent character.**

## QUANTIFYING DEGRADATION RATES

The rate of (bio)degradation of a pollutant is commonly and conveniently defined by a single parameter, the half life ( $T_{1/2}$ ) under the assumption of first order decay. Taking  $R_{t_{H_2O}}$  as the residence time of the pollutant in the aquifer system then yields:

$$T = 0.3 R t_{H2O} / \log (C_0 / C_t) \quad [d] \quad (EQ 1)$$

With:

- $C_0$  = input concentration at  $t = 0$  [ug/L]
- $C_t$  = concentration after  $t$  days [ug/L]
- $T_{1/2} = (\ln 2) / \lambda$  where  $\lambda$  = decay constant of pollutant [1/d]
- $t_{H2O}$  = residence (travel) time of the water since infiltration [d]
- $R$  = Retardation factor of the pollutant due to sorption to the aquifer matrix [-]
- The factor  $R$  can be estimated by:

$$R = 1 + \rho_s (1-n) f_{ND} K_D / n \quad [-] \quad (EQ 2)$$

With:

- $\rho_s$  = density porous medium, solids [kg/L]
- $n$  = effective porosity [-]
- $f_{ND}$  = non-dissociated fraction =  $1 / \{1 + 10^{(pH - pK_A)}\}$
- $pK_A = -\log K_A$ , with  $K_A$  being the 1<sup>st</sup> acid dissociation constant of the pollutant
- $K_D$  = distribution coefficient of pollutant, soil/water (L/kg) often to be calculated as follows Appelo & Postma, 2005):

$$K_D = f_{OC} 10^B K_{OW}^A \quad [L/kg] \quad (EQ 3)$$

Where:

- $f_{OC}$  = fraction of bulk organic carbon in aquifer matrix [-]
- $K_{OW}$  = octanol water distribution coefficient [-]
- $A, B$  = constants depending on pollutant (type), listed in literature.

## UNCERTAINTY ANALYSIS FOR IOPAMIDOL IN SUBOXIC ENVIRONMENT

The nearly exponential decline of iopamidol concentration levels with increasing travel time (Fig.4, left) suggest that the half life concept is valid enough. A closer inspection of Fig.4 and the apparent relation of iopamidol removal with the oxidation capacity of water ( $NO_3^-$ ; Fig.6) reveal, however, that the removal rate of oxidants by bulk organic matter in the aquifer is more important in determining the degradation rate than the initial amount of iopamidol. Although this does disrupt the validity of the half life concept, it does not invalidate calculated half lives as long as they are considered as relative measures for comparison.

When we accept  $T$  as a valuable parameter, then it is worthwhile to make a kind of uncertainty analysis.



In this analysis we try to quantify the combined effects of estimated errors in the 4 terms in Eq.1:  $R$ ,  $t_{H_2O}$ ,  $C_0$  and  $C_t$  ( $C_0 > C_t$ ), by taking:

$$(T)_{\text{MIN}} = 0.3 R_{\text{MIN}} (t_{H_2O})_{\text{MIN}} / \log \{(C_0)_{\text{MAX}} / (C_t)_{\text{MIN}}\} \quad [d] \quad (\text{EQ } 4)$$

$$(T)_{\text{MAX}} = 0.3 R_{\text{MAX}} (t_{H_2O})_{\text{MAX}} / \log \{(C_0)_{\text{MIN}} / (C_t)_{\text{MAX}}\} \quad [d] \quad (\text{EQ } 5)$$

We estimate the errors in the individual terms as follows:

- $R$ : if  $R = 1$  then 0%, else if  $R = 1-10$  then +/- 10%, else if  $R = 10-100$  then +/- 20% etc.
- $t_{H_2O}$ : if based on history matching of environmental tracers then +/- 15%, otherwise if calculated with a hydrological model well validated on water balances, piezometric levels and high quality KD and effective porosity values, then +/- 30%
- $C_0$  and  $C_t$  dealing with relative analytical errors only: if  $C > 4 \cdot \text{LOQ}$  then +/- 10%, else +/- 20%

When we apply these estimates to Eqs.4 and 5 we obtain, with the mean  $T$  values for iopamidol in 6 transects with travel times < 100 days in 2005 (Table 4), minimum values that are about half the value calculated using Eq.1 (mean  $T$ ), and maximum values that double that value (see Table 4).

Not included in this uncertainty analysis are the effects of temporal and spatial changes in redox environment and temperature of the infiltration water. These effects can be very large.

**TABLE 4. Range of calculated half lives for iopamidol in suboxic environment along 6 transects in 2005.**

| Iopamidol, suboxic            | $T_{1/2}$ Min | $T_{1/2}$ Mean | $T_{1/2}$ Max |
|-------------------------------|---------------|----------------|---------------|
| Basin Recharge pan 13         | 28            | 48             | 94            |
| Basin Recharge G6             | 30            | 45             | 73            |
| Basin Recharge G12            | 48            | 85             | 194           |
| Basin Recharge Castricum 8    | 30            | 53             | 120           |
| Deep Well Injection Waalsdorp | 20            | 36             | 81            |
| Deep Well Injection dWat      | 26            | 82             | 205           |
|                               |               |                |               |
| Mean Range / Values           | 30            | 58             | 128           |
| Max Range / Values            | 20            | 58             | 205           |

## CONCLUSIONS

Two snapshot surveys of 10 monitoring transects of observation wells in 3 basin artificial recharge (BAR) and 2 deep well injection (DWI) systems yielded insight in

the behavior of 140 organic micropollutants ( $<0.001 - 5 \mu\text{g/L}$ ). These consist of 80 pharmaceuticals and other emerging pollutants (PEPs) and 60 regulated organic pollutants (ROPs). At this stage our interpretation is still in progress, which means that various results presented here are preliminary.

Most PEPs (71%) and ROPs (60%) were everywhere below the limit of quantification. This low score is partly explained by a very effective sanitation of the Rhine and Meuse fluvial basins, and by a thorough pretreatment of the infiltration waters (consisting of coagulation, rapid sand filtration and in several cases activated carbon filtration).

Concentrations of detected pharmaceuticals and X-ray contrast agents were low ( $0.01-0.2 \mu\text{g/L}$ ). Various of these pollutants showed significant (bio)degradation ( $>90\%$ ) during aquifer passage, in either suboxic or anoxic environment or both. Others were not removed at all, not even after 36 years. These findings again demonstrate a strong influence of redox conditions on removal rates (cq persistency) of many organic micropollutants, in addition to others mapped for artificial recharge and river bank filtration systems in the Netherlands by Stuyfzand (1998).

All together, artificial recharge significantly contributes to water quality improvements, also regarding PEPs and ROPs. Even better achievements of aquifer passage can be realized, however, by increasing the travel times and forcing the infiltrates to flow through first suboxic and then anoxic environments, for instance by installing the recovery system in a deeper anoxic aquifer.

Further research should focus on the relation between the oxidation capacity and temperature of water and the removal of organic pollutants in suboxic and anoxic environments.

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# INDIGENOUS GROUNDWATER MICROORGANISM PROCESSES THAT INFLUENCE THE DECAY OF ENTERIC VIRUSES

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## ABSTRACT

Microbial pathogens are the most immediate health risk associated with water reuse methods including managed aquifer recharge. Research has shown that microbial pathogens do decay in groundwater and that the activity of indigenous groundwater microorganisms is the factor having the largest influence on the decay of bacterial and viral pathogens. The exact nature of the groundwater microorganism influence on the inactivation of pathogens is unclear. This paper provides details on research that shows that some groundwater bacteria are able to produce antiviral effects that can pass across a permeable membrane. The results also show that 25% of indigenous groundwater bacteria isolated from groundwater were able to enhance the degradation of enteric viruses. The antiviral effects produced by these groundwater bacteria are heat labile and can be influenced by enzyme inhibitors.

## KEYWORDS

Decay; enteric viruses; groundwater microorganisms; managed aquifer recharge; microbial activity

## INTRODUCTION

With global freshwater supplies under pressure, water reuse is being examined to assist in improving water sustainability. Municipal effluent is an ideal source for water reclamation as it is consistent in quality and quantity. The health aspects of water reuse have been identified as an issue of concern, in particular the potential presence of enteric viruses. Managed Aquifer Recharge (MAR) is a method that can aid water reclamation by recharging water such as treated effluent into a suitable aquifer. Research into the removal of pathogenic contaminants by natural processes within aquifers, namely the action of autochthonous bacteria, has led to the consideration that MAR could be used to assist in the removal of microbial

pathogens (Dillon and Toze, 2005). Pathogens have been demonstrated to be removed during residence in groundwater, but the presence of active autochthonous groundwater bacteria are required for significant removal rates to occur (Gordon and Toze, 2003).

The potential for the contamination of groundwater by enteric viruses is a concern that increasingly requires management. Other activities involving groundwater such as water reclamation and managed aquifer recharge schemes have also become more prominent which require careful management to prevent groundwater contamination (Dillon *et al.*, 1999). Enteric viruses have been shown to have a greater resistance to decay in groundwater than other microbes such as faecal coliforms (Gordon *et al.*, 2002; Nasser and Oman, 1999; Toze and Hanna, 2002). MAR has been demonstrated to be able to remove or reduce contaminants in groundwater (Dillon and Toze, 2005).

A number of studies into viral decay in groundwater have alluded to the importance of the autochthonous groundwater microorganisms (Jansons *et al.*, 1989; Keswick and Gerba, 1980; Toze and Hanna, 2002; Yates *et al.*, 1985). It has been shown that viral decay is greater in the presence of other microorganisms (Gordon and Toze, 2003). The mechanisms by which this occurs, however, are still not well understood. It remains unclear whether the principal mode of action by autochthonous microorganisms on viral decay is a result of direct predation, the production of virucidal compounds or other unknown activities. If the mechanisms by which groundwater microorganisms cause the decay of enteric viruses could be determined, the risk from these viruses as a contamination source in groundwater as well as for activities such as managed aquifer recharge may be able to be better managed.

The aim of this study was to examine how groundwater microorganisms can influence viral decay by determining whether or not direct contact was required between viruses and groundwater microorganisms for decay to occur, and what the characteristics of the antiviral activity might be.

## **METHODS AND MATERIALS**

### **Groundwater Source and Collection**

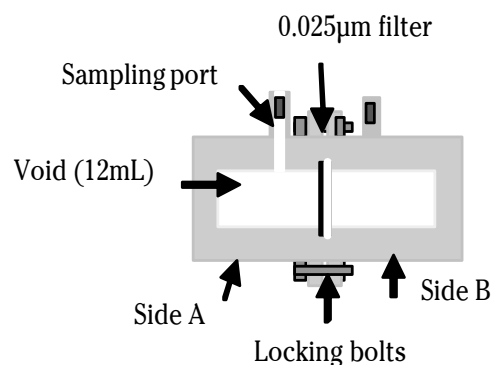
Groundwater was obtained from a bore located in a shallow (~10 m) unconfined, karstic limestone aquifer on the Swan Coastal Plain, Perth, Western Australia. The bore was purged by pumping for 30 minutes using a submersible pump to prevent the collection of stagnant water prior to the collection of groundwater samples. Groundwater was collected into sterile 1 L borosilicate glass bottles and immediately stored on ice until processed.

## Viruses

The antiviral activity of the groundwater microorganisms was tested on the RNA viruses poliovirus and coxsackievirus and the DNA virus adenovirus. Viral stocks were cultured and processed as previously described in Gordon and Toze (2003).

### Determination of Antiviral Activity in Contact and Non-Contact with Groundwater Microorganisms

Separation chambers (Figure 1) were used to separate either poliovirus or coxsackievirus and indigenous groundwater microorganisms as described in Dillon and Toze (2005). The chambers were set up with a 250 000 molecular weight cut-off membrane (PALL) dividing the two halves of the chamber. This pore size is sufficient to prevent the passage of poliovirus across the membrane. The chambers were set up as described in Table 1. Filtered groundwater was prepared by twice passing portions of collected groundwater through sterile 0.2  $\mu\text{m}$  syringe filters. Thus the groundwater microorganisms were either in direct contact with the virus (Direct Contact in Table 1) or separated from the viruses by the filter (Indirect Contact in Table 1). No contact was tested where filtered groundwater (ie, no groundwater microorganisms present) was used to fill both sides of the chamber. The chambers were sampled weekly to determine the number of detectable viruses present and for the continuing presence or absence of active groundwater bacteria.



**FIGURE 1. Plan of separation chamber.**

**TABLE 1. Conditions tested in separation chambers**

| Contact conditions tested | Side A      |                  | Side B (+virus) |                  |
|---------------------------|-------------|------------------|-----------------|------------------|
|                           | Groundwater | Groundwater      | Groundwater     | Groundwater      |
|                           | Filtered    | Microbes Present | Filtered        | Microbes Present |
| Direct contact            | No          | Yes              | No              | Yes              |
| Indirect contact          | No          | Yes              | Yes             | No               |
| No Contact                | Yes         | No               | Yes             | No               |

## Screening of Groundwater Isolates for Viral Degradation

Groundwater bacterial strains were isolated from groundwater by inoculating R2A agar with 100  $\mu$ L of freshly collected groundwater. Inoculated plates were incubated aerobically at 28 °C from 1 to 7 days depending on growth. Plates were checked at 24 hours, 4 and 7 days. One hundred colonies were selected from the incubated plates and subcultured onto fresh R2A plates and given a number from 1 to 10 and a letter from A to J as a culture identity. The selection of colonies was based firstly on colony morphology in order to select isolates of different species, and secondly by selecting random colonies from the different plates to make a total of 100. Isolates were re-streaked at least twice on R2A to ensure the purity of the isolates. Isolates which were observed to have identical colony morphologies were identified and only one of these “paired” isolates was subsequently used. This resulted in a total of 63 isolates with different characteristics to be tested for antiviral activity. All of these 63 selected groundwater bacterial strains were initially screened for antiviral activity against coxsackievirus and poliovirus. Those groundwater bacterial isolates that showed the greatest activity against these two viruses were then also tested against adenovirus after this virus had become available for use in experiments.

Antiviral activity was tested by suspending a groundwater isolate in filtered groundwater which was then seeded with one of the viruses. This was done by picking a single colony of a groundwater isolate (cultured overnight on R2A agar at 28 °C) using a 10  $\mu$ L inoculation loop and suspending this colony in 9.5 mL groundwater which had been filter sterilised twice through sterile disposable 0.2 mm syringe filters. The inoculated groundwater suspension was then incubated aerobically at 28 °C for 24 hours. After incubation, 500 mL of poliovirus, coxsackievirus, or adenovirus was added to the tube. A negative control for each groundwater source was also carried out by adding a virus to a tube of double filter sterilised groundwater which had not been inoculated with any of the groundwater isolates. The tubes inoculated with the viruses were then mixed and a 1 mL sample taken immediately (time 0 sample). Tubes were then incubated at room temperature up to day 28 when a second sample was collected. All collected samples were stored at -80 °C until RNA extractions were undertaken. The presence or absence of viable groundwater bacterial isolate cells in all collected samples was monitored by inoculating 20 mL of sample into 180 mL of nutrient broth in a microtitre tray immediately on collection of the samples. Trays were incubated at 28 °C for 24 hours and then examined for the presence of bacterial cell growth. This was undertaken to ensure that any observed viral decay was due to the presence of active cells of each groundwater bacterial isolate and to ensure that the negative controls had remained sterile (apart from the virus) over the experimental period.

### Investigation of Groundwater Microorganisms Antiviral Activity

The four groundwater bacterial isolates which had the greatest activity against the three viruses were then tested to determine the nature of the antiviral activity. To achieve this, five treatments were carried out following by the following procedure.

The inoculum was prepared by suspending one colony of a bacterial isolate in a 1:10 diluted R2A broth and incubating the inoculated broth at 28 C for 24 hours with shaking. A 100 iL volume was then inoculated into 100 mL of 1:10 diluted R2A broth and incubated for approximately 3 hours to reach a cell density of approximately  $10^6$  cfu mL<sup>-1</sup> (incubation time had been determined previously by growth curves for each isolate). A 1mL volume of this bacterial suspension was then treated as described in Table 2 and brought to a total volume of 10 mL using 1:10 R2A broth. One of the three viruses was then added to this treated suspension and incubated aerobically at room temperature with samples collected immediately (time 0) and at day 35. A negative control was established for each treatment where the viruses were suspended in 1:10 diluted R2A broth which had not been inoculated with any groundwater isolates but were treated in the same manner as the broths inoculated with the groundwater isolates. On each sampling occasion a 1mL sample was taken and analysed for detectable viral numbers using quantitative PCR or RT-PCR. The presence or absence of active bacterial cells was tested by inoculating 180 µL of nutrient broth with 20µL of the collected sample.

**TABLE Treatments Used to Characterise Antiviral Activity**

**TABLE 2. Treatments used to Characterise Antiviral Activity**

| Treatment Name     | Treatment                                            |
|--------------------|------------------------------------------------------|
| No Treatment       | untreated inoculum                                   |
| Filtered           | filtered (0.8/0.2 “m) inoculum                       |
| Heated             | filtered (0.8/0.2 “m) then heated at 60°C for 10 min |
| RNase Inhibitor    | inoculum + RNase inhibitor                           |
| Protease Inhibitor | inoculum + Protease inhibitor                        |

### Viral Nucleic Acid Extraction

Viral RNA or DNA was extracted from experimental samples using a NucleoSpin® RNA and Virus Purification kit (Clontech) using the method described by the manufacturer. All extracted nucleic acid samples were stored at -80 C to prevent degradation until analysed by quantitative PCR or RT-PCR.



## PCR/RT-PCR for Enumeration of Viral Pathogens

Quantitative PCR/RT-PCR reactions and post reaction quantitation was undertaken using the methods previously described by Gordon and Toze (2003) using an iCycler Real time PCR system (Bio-Rad). Poliovirus and coxsackievirus numbers were determined using the Titan RT-PCR Enzyme System (Roche Diagnostics) and the Ent-Up and Ent-Down primers (Abbaszadegan *et al.*, 1993). Adenovirus numbers were determined using iQ Supermix enzyme system (Bio-Rad) and the HexAA 1885 and HexAA 1913 primers (Allard *et al.*, 1992; Allard *et al.*, 1990). The DNA gel stain SybrGreen 1 (Molecular Probes) was used as a DNA specific fluorophore to track real-time results for all PCR and RT-PCR reactions and a passive dye (fluorescein) (BioRad) was used as a background dye.

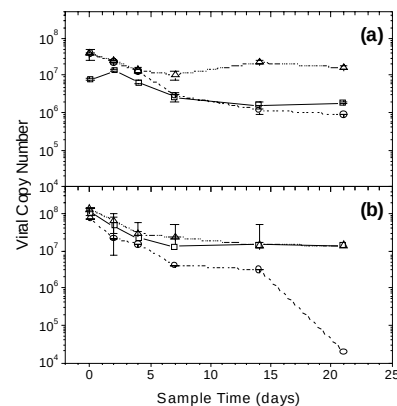
## Determination of Viral Decay Rates

The time for a  $1\log_{10}$  loss ( $T_{90}$ ) of each of the viruses under all of the conditions tested was determined as previously described by Wall *et al.* (2006).

## RESULTS AND DISCUSSION

### Separation Chambers

Figure 2 shows the survival times of poliovirus and coxsackievirus when in direct contact, when separated from autochthonous groundwater microorganisms by a size excluding membrane (Indirect Contact) and when groundwater microorganisms had been removed completely from both sides of the chamber (No Contact). Poliovirus showed a similar reduction in survival when in contact, and separated from autochthonous groundwater microorganisms. The decay of poliovirus under both conditions where groundwater microorganisms were present on at least one side of the chamber (Direct and Indirect Contact) was observed to be faster than when the groundwater microorganisms had been completely removed (No Contact).



**FIGURE Decay of (a) poliovirus and (b) coxsackievirus in separation chambers when in direct contact (●) with groundwater microorganisms; indirect contact (■) with groundwater microorganisms; and in chambers (▲) with no groundwater microorganisms present.**

Coxsackievirus however only showed significant decay when in direct contact with autochthonous groundwater microorganisms, as decay when separated from autochthonous groundwater microorganisms by the membrane was not significantly different to the decay shown in the “No Contact” chamber where the groundwater microorganisms had been completely removed from both sides of the chamber.

Despite their genetic similarity the different decay patterns observed for poliovirus and coxsackievirus in the separation chambers indicated that the antiviral activity of the groundwater microorganisms was different for each of these viruses. The fact that poliovirus decayed when separated from the groundwater microorganisms by an excluding membrane indicated that the antiviral activity was cell free and released by the groundwater microorganisms into the water surrounding them. The antiviral activity was produced in sufficient quantity that it could diffuse across the membrane and increase the decay of the poliovirus particles. In contrast, increased decay of coxsackievirus only occurred when the viral particles were in close association with the groundwater microorganisms. This suggests that either the groundwater microorganism antiviral activity against coxsackievirus was either cell bound or not produced in sufficient quantity to cross the excluding membrane.

### **Isolation of Groundwater Bacterial Isolates Capable of Increasing the Decay of Viruses in Groundwater.**

All 63 groundwater bacterial isolates were found to be gram negative rods apart from one isolate which was a gram positive rod that showed feathery colony growth. All isolates were catalase positive, 47 (74) % were oxidase positive and 23 (37%) of the isolates were able to ferment glucose. On testing, 27% of the isolates were able to increase the decay of poliovirus and coxsackievirus. The ten isolates that showed the greatest influence on coxsackievirus and poliovirus were also tested on adenovirus. The influence of these ten isolates on the three viruses produced variable degradation times (T90) (Table 3) between the viruses as had been observed for with the different decay patterns for poliovirus and coxsackievirus in the separation chambers.

**TABLE 3. T90 value (days) of poliovirus, coxsackievirus and adenovirus in the presence of the top ten active groundwater isolates.**

| Isolate | Poliovirus | Coxsackievirus | Adenovirus |
|---------|------------|----------------|------------|
| 3A      | -28.3      | -4.1           | -11.7      |
| 4B      | -11.7      | -4.1           | -4.8       |
| 4F      | -38.1      | -3.9           | ND         |
| 10G     | -70.7      | -19.1          | -27.5      |
| 10B     | -59.2      | -24.9          | ND         |
| 10E     | ND         | -49.2          | -17.7      |
| 3J      | ND         | -4.5           | -101.6     |
| 1G      | -9.3       | -10.4          | -4.7       |
| 6J      | -62.0      | -88.0          | -190.7     |
| 9G      | -97.0      | -88.9          | ND         |

ND: no decay (T90 beyond -500 days)

It was also observed that the different groundwater isolates varied in their ability to increase the degradation (smaller T90 values) each particular virus. Despite these observations on the variability of decay, coxsackievirus was the virus which was most likely to have the fastest degradation rate of the three viruses (7 out of 10). In contrast, poliovirus and adenovirus tended to have much slower degradation rates in the presence of the groundwater microbial isolates compared to coxsackievirus.

### Characteristics of Antiviral Agents Produced by Four Groundwater Isolates.

The outcomes of the effect of the different treatments on the ability of the four selected groundwater isolates to increase the rate of decay of the three viruses are given in Table 4.

**TABLE 4. T90 values (days) for treatments against poliovirus, coxsackievirus and adenovirus.**

| Treatment            | T90 (days) Values for Virus Decay |            |                |            |
|----------------------|-----------------------------------|------------|----------------|------------|
|                      | Isolate                           | Poliovirus | Coxsackievirus | Adenovirus |
| No treatment         | 3A                                | -18.0      | -62.1          | -27.1      |
|                      | 4B                                | -14.0      | -26.1          | -16.2      |
|                      | 1G                                | -15.7      | -37.8          | -27.4      |
|                      | 9G                                | -19.7      | -35.0          | -27.0      |
|                      | neg                               | -469.4     | ND             | -228.2     |
| Filtered             | 3A                                | -86.7      | -46.9          | -39.8      |
|                      | 4B                                | -245.8     | -78.9          | ND         |
|                      | 1G                                | -64.0      | -47.4          | ND         |
|                      | 9G                                | -43.9      | -324.5         | -25.8      |
|                      | neg                               | -61.2      | -150.5         | ND         |
| Filtered then Heated | 3A                                | -470.7     | ND             | -57.0      |
|                      | 4B                                | -88.8      | ND             | ND         |
|                      | 1G                                | ND         | ND             | -397.3     |
|                      | 9G                                | ND         | ND             | -261.5     |
|                      | neg                               | -481.7     | ND             | -258.8     |
| RNase inhibitor      | 3A                                | -55.9      | -125.0         | NA         |
|                      | 4B                                | -80.9      | -468.8         | NA         |
|                      | 1G                                | -71.4      | ND             | NA         |
|                      | 9G                                | -59.7      | -229.0         | NA         |
|                      | neg                               | -45.4      | ND             | NA         |
| Protease inhibitor   | 3A                                | ND         | -37.2          | -217.1     |
|                      | 4B                                | ND         | -40.4          | ND         |
|                      | 1G                                | ND         | -39.2          | ND         |
|                      | 9G                                | ND         | -43.0          | -70.0      |
|                      | neg                               | -87.2      | ND             | -45.2      |

ND = no decay observed or that the T90 was greater than 500 days.

In the No Treatment control where the broth had not been treated, all four of the groundwater isolates showed an ability to increase the reduction of poliovirus, coxsackievirus and adenovirus numbers compared to the negative control with T<sub>90</sub> values ranging from 14 days to 62.1 days. It is also notable that the decay of poliovirus in the No Treatment controls was faster than for coxsackievirus or adenovirus. This is different from the results reported above for the top ten isolates (Table 3) where coxsackievirus had the fastest degradation rates.

The experiments undertaken to determine which isolates had detectable antiviral activity were undertaken in filtered groundwater whilst the experiments done to characterise the antiviral agents were done in diluted medium. The differences noted suggest that the additional nutrients present in the diluted medium had an influence on the production of these agents by the groundwater microorganisms. The presence of the additional nutrients appears to stimulate the production of the agents active against poliovirus, particularly for groundwater isolates 3A and 9G. In contrast, the additional nutrients slowed the decay of coxsackievirus and adenovirus for all of the groundwater bacterial isolates apart from isolate 9G.

The removal of the bacterial cells from the diluted medium by filtration prior to inoculation with poliovirus, coxsackievirus and adenovirus had varying influence on overall viral decay, with the filtrate from different isolates influencing the decay of different viruses. For example, the filtrate from Isolate 3A increased the decay of coxsackievirus and adenovirus but not poliovirus compared to the negative control. In comparison, the filtrate from Isolate 1G increased the decay of coxsackievirus only while the filtrate from Isolate 9G increased the decay of adenovirus and poliovirus but not coxsackievirus. This suggests that the four isolates each produce a different antiviral activity and that the individual antiviral activity produced by the isolates have different effects on the three viruses tested.

The effect of heating the filtrate prior to seeding with the viruses prevented the decay of all of the viruses (compared to the negative control) except for the influence of Isolate 4B on poliovirus and Isolate 3A on adenovirus. In all cases, apart from the influence of Isolate of 4B on poliovirus, heating significantly decreased the decay of the viruses when compared to the T<sub>90</sub> values obtained for the No Treatment controls and filter treated samples. The effect of heating in significantly reducing the decay of the three viruses indicates that the antiviral activities of the four isolates were heat labile.

Like the outcomes of the filtration and heating treatments, the addition of enzyme inhibitors to the broths caused a different effect for each of the viruses. There was little difference, however, of the influence of the enzyme inhibitors on the antiviral activity of each groundwater isolate on one specific virus. For example, the addition of RNase inhibitor significantly reduced the decay of coxsackievirus (T<sub>90</sub> times of 125 days to >500 days) but had minimal influence in the decay of poliovirus, which had

decay times ( $T_{90}$  times of 45 to 80 days) that were comparable to the samples that had been treated by filtration. (Note that RNase inhibitor was not tested on adenovirus as it is a DNA virus and thus should not be affected by the presence of RNase enzymes). In contrast, the addition of protease inhibitor significantly reduced the decay of poliovirus and adenovirus but had no effect on the decay of coxsackievirus, which had  $T_{90}$  times that were comparable to the times in the No Treatment controls. The use of the enzyme inhibitors showed that at least part of the antiviral activity of the four groundwater bacterial isolates was enzymatic in nature but that there were other factors that were unaffected by the use the presence of the inhibitors for some of the viruses. In addition, the use of the enzyme inhibitors reinforces the finding that the antiviral activity produced by groundwater microorganisms is different for different viruses, even those closely related genetically such as poliovirus and coxsackievirus.

## CONCLUSIONS

The results of this study have shown that groundwater microorganisms can produce antiviral activity that can have an influence on some viruses even when not in the direct presence of the cells of the groundwater microorganisms. The antiviral activity produced by the groundwater microorganisms are heat labile and at least part of the activity may be enzymatic in nature although it also appears that some of the antiviral activity may not be enzymatic. The results also showed that the antiviral activity was virus specific with different decay times observed for coxsackievirus, poliovirus and adenovirus, regardless of the treatment method tested.

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## REDUCTION OF MICROORGANISMS AND NATURAL ORGANIC MATTER IN UNSATURATED ZONE — A COLUMN EXPERIMENT

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### ABSTRACT

The removal efficiency of a typical west-Swedish glaciofluvial formation was measured for Natural Organic Matter (NOM), bacteria and bacteriophages in a pilot column plant. The solutions of microorganisms, *E.coli*, coliforms, *Cl.perfringens*, enterococci, bacteriophages MS-2 and ...X174 were added to the raw water which fed the columns. Seven columns were constructed to represent cases of 1-4 m of unsaturated zone, 2 and 4 m of saturated zone and 2 m unsaturated zone including 0.3 m of Iron Oxide Coated Olivine Sand (IOCO-sand). The microbiological barrier of 4 m of unsaturated zone reduced bacteriophages by 3 log<sub>10</sub> and bacteria by = 2.4 log<sub>10</sub>. For bacteriophages the reduction efficiency was higher in unsaturated zone. The overall reduction of organic matter was = 63 %. IOCO-sand strongly enhanced the reduction of bacteriophages and NOM and could be favorable considering its use in a thinner layer of unsaturated zone.

### KEYWORDS

Managed aquifer recharge, microbiological barrier efficiency, NOM, removal efficiency, IOCO, column experiment

### INTRODUCTION

Between 1980 and 2004, 142 waterborne outbreaks of disease were reported in Sweden with at total of 63 000 cases. The most common agents were campylobacter, norovirus and *Giardia lamblia*. In Sweden, the groundwater is considered microbiologically safe

and rarely needs treatment for bacterial reduction. 50% of the water supplies are surface water and another 25% are managed aquifer recharge (MAR).

In order to achieve microbiologically safe water the disinfection plays an undisputable role. The introduction of chlorine in water treatment enabled quick and efficient processed-based water production. However, the disinfection with chlorine has shown to have several weaknesses, e.g. a varying efficiency on different microorganisms targeted, a potential of producing disinfection by products (DBP) and relatively fast depletion within the distribution network. Compared to the treatment and production of drinking water direct from a surface source, MAR is often easier to operate. In Sweden, MAR is accepted as a microbiological barrier, which is, however, based rather on operational experience than on quantifiable studies.

The glaciofluvial formations in Sweden offer favorable hydrogeological conditions for MAR water production by increasing the infiltration rate with sprinklers or infiltration ponds. However, the often limited extent of these formations implies relatively short water transport time due to a high capacity demand, especially in highly populated areas. MAR operated under such conditions, should be regarded as a treatment with its pros and cons, somewhere between the surface water treatment and the use of groundwater. In Sweden e.g. it is common with a retention time of less than 10 days (Hanson, 2000), which sets focus on the microbiological safety of the treatment and the reduction of natural organic matter (NOM).

This study was performed during an investigation of the glacial delta formation in Gråbo in Sweden, a location with potential of becoming a complementary treatment facility for the region of Göteborg. The study was financed by The Cities of Göteborg and Lerum, EC and The Swedish Water & Wastewater Association (SWSA).

## MICROBIOLOGICAL BARRIER EFFICIENCY

A microbiological barrier is a step in the treatment process that prevents pathogens from reaching the consumer. In Sweden, there is no specific regulation on how efficient a microbiological barrier should be. The guidance, however, recommends the treatment operator to achieve turbidity less than 0.1 FNU and a 99 % ( $2 \log_{10}$  when applying disinfection with chlorine) reduction of bacteria, which is regarded as minimum efficiency for a barrier (Livsmedelsverket 2004).

In an investigation (Persson *et al.*, 2005) of chemical precipitation and rapid sand filtration, the reduction of virus and faecal indicator bacteria was between 3 and  $4 \log_{10}$  and 1 to  $2 \log_{10}$  for natural particles with a size of bacteria and protozoa ( $>1 \mu\text{m}$ ). Ultra filtration with a membrane pore size of 100 nm generated a reduction of 98-99.9 % of natural particles with a size of bacteria, and a reduction of 4.7 to  $5.5 \log_{10}$



of amended bacteriophage (Persson *et al.*, 2005). Slow sand filtration gives a reduction of 1 to 4 log<sub>10</sub> of protozoa, and often higher reductions of virus (Logsdan *et al.*, 2002).

The microbiological barrier efficiency depends on two major reduction principles: physical/chemical removal and inactivation or die-off. When water is filtered through sand material the major part of the reduction takes place in the first 2 meters of the material and in MAR there is commonly an early reduction of micro organisms (Huisman, 1983). However, due to e.g. protozoa's high resistance, there might be a risk for possible breakthrough (Schijven, 2001). Micro organisms have been observed to travel tens to thousands of meters (Pang *et al.*, 2005). However, a major part is reduced within 40 meters from the infiltration basin (Engblom and Lundh, 2006). The low infection-dose of parasites and viruses combined with the fact that both have a high resistance to disinfectants and high survival in subsoil environments makes them important factors in all water treatment applications.

Jørgensen *et al.* (2001) found that after 10 days of transport in the subsurface the reduction of bacteriophages was 6 log<sub>10</sub> and after 25 days 8 log<sub>10</sub>. Schijven (2001) measured an 8 log<sub>10</sub> reduction of MS-2 bacteriophages after a transport time of 38 days in saturated soil. Studies in unsaturated zone show a higher efficiency in the reduction of virus (e.g. Jin *et al.*, 2000; Schijven 2001). A hypothesis is that the interface between the gases and liquid plays an important role (Jin *et al.*, 2000). A Swedish study (Blomberg *et al.*, 1999) measured an 80 % reduction of bacteriophages in 4 meter of unsaturated soil, which implies a great variance in efficiency.

## NATURAL ORGANIC MATTER

During the last decades many waterworks in Sweden have experienced increasing problems with organic matter and colour in the raw water. A study showing a statistically verified increase of natural organic matter in typical Swedish forest lakes during the 1990s was presented by Löfgren (2002). The increase is believed to be related to climatic variations, even though this has not been verified. Sundlöf and Kronqvist (1992) presented a reduction of approx. 30 % of organic matter at Swedish MAR plants with pond infiltration. More detailed investigation of a few plants showed a 20% to 30% removal of COD<sub>Mn</sub> in the unsaturated zone during summer and approx. 10 % during winter (Blomberg *et al.*, 1999). Conclusively, MAR plants in Sweden have problems in removing the increasing NOM concentrations and several plants have experienced an increase of NOM in the treated drinking water (Katrineholm, Eskilstuna and Uppsala).

## IOCO-SAND

Iron oxide coated olivine (IOCO) sand is a filter material with an improved ability of removing NOM compared to conventional filter sand (Chang *et al.*, 1997, McMeen & Benjamin 1997). The material consist of olivine sand  $[(\text{Mg}, \text{Fe})_2\text{SiO}_4]$  that is coated with a thin layer of iron oxide. Studies (Jonsson 2003, Berggren *et al.*, 2004) have shown a great potential of removing NOM in the unsaturated zone by adding a 30-cm layer of IOCO-sand on the top of the filter bed. It is also possible to overlay the IOCO-layer with ordinary filter sand as a maintenance layer without the efficiency declining. Hypothetically, there should also be an improved efficiency in reducing pathogens.

## OBJECTIVES

The main objective of the study was to investigate the microbiological barrier efficiency and the reduction of NOM in the unsaturated zone in artificial recharge of groundwater by pond infiltration by:

1. Quantifying the reduction of bacteria and viruses utilizing pulse-response amending with indicator bacteria and bacteriophages as models for viruses,
2. Quantifying the reduction of NOM
3. Investigating the reduction efficiency of Iron Oxide Coated Olivine sand (IOCO)

## METHOD

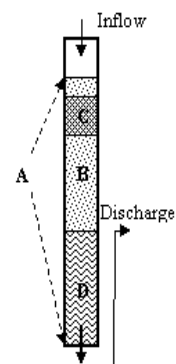
### Experimental Setup

**TABLE 1. Column properties and case studies**

| Column | Total depth (A) | Unsat. zone <sup>1</sup> (B) | IOCO (C) | Sat. zone (D)            |
|--------|-----------------|------------------------------|----------|--------------------------|
| K0     | 5.4 m           | -                            | -        | 2 and 4 m <sup>1,2</sup> |
| K1     | 5.4 m           | 4 m                          | -        | 1.5 m                    |
| K2     | 5.4 m           | 2 m                          | -        | 3.5 m                    |
| K3     | 5.4 m           | 2 m                          | -        | 3.5 m                    |
| K4     | 5.4 m           | 1 m                          | -        | 4.5 m                    |
| K5     | 2.5 m           | 2 m                          | 0.3 m    | 0.5 m                    |
| K6     | 2.5 m           | 2 m                          | 0.3 m    | 0.5 m                    |

1) was actually approx. 10 cm less but is rounded off for readability reasons.

2) top 2 and 4 m of saturated zone was studied.



Five 6-m PVC-plastic pipes with a diameter of 0.3 m were raised and filled with 5.4 m of sand from the GrÅbo glacial delta formation. Study cases were configured of 1, 2 and 4 m of unsaturated zone overlaying varying depths of saturated zones. (Table 1). Two columns contained 2.5 m of GrÅbo sand including 0.3 m layer of IOCO-sand in study cases with 2 m of unsaturated zone.

The sand was analysed by KIWA Water Research in Holland within the EU-project ARTDEMO, and at G-teborg Water, Sweden. The hydraulic loading was 25 ml/min, which resulted in an infiltration rate of 0.5 m/d. The raw water was taken from the Lakes Delsj-arna in G-teborg, Sweden, and brought to the top of the column in soft transparent PVC-hoses where it was distributed by dripping on the sand infiltration surface.

### Reduction of Bacteriophages

The reduction of bacteriophages was measured during three pulse response experiments (experiment 1, 2 and 3), using bacteriophages MS-2 (ISO 10705-1, International Organization for Standardization, 2002b) and ΦX174 (ISO 10705-2, International Organization for Standardization, 2002a) as model for viruses. A solution of phages was added to the raw water during a 40-min period in experiments 1 and 2, while in experiment 3 the amending was continuous for 7 days in order to reach a steady state of reduction. The concentration of bacteriophage in the added solution in experiment 3 was only 0.5 % of those in experiments 1 and 2 due to practical experimental reasons.

The calculation of the reduction in the unsaturated zone was based on the measured peak concentration in the response curve for each column during one experiment. All columns received the same pulse concentration of bacteriophages. The  $\log_{10}$ -reduction =  $-\log_{10} (C_{\max}/C_0)$  was calculated for each case, where  $C_{\max}$ =maximum concentration and  $C_0$ =initial pulse concentration. As  $C_0$  varied, e.g. in experiment 3, the  $C_0$  is represented by the peak of the pulse concentration and regarded as the maximum load that the columns experienced. The uncorrected filter factor  $C_f$  was calculated from:

$$-\log_{10} (C_{\max}/C_0) = C_f \times \text{depth of zone} + \text{Constant.} \quad (\text{EQ 1})$$

The dispersion in the water, calculated from tracer experiments with NaCl, was handled as a dispersion factor  $C_d$  from the relationship:

$$-\log_{10}(C_{\max}/C_0) = C_d \times \text{depth of zone} + \text{Constant,} \quad (\text{EQ 2})$$

Finally, the accurate filter factor F was gained through the correction formula:

$$F = C_f - C_d. \quad (\text{EQ 3})$$

## Reduction of Bacteria

| TABLE 2. Analysis of bacteriological parameters |                 |                       |
|-------------------------------------------------|-----------------|-----------------------|
| Analysis                                        | Detection limit | Method                |
| Coliform bacteria 35°C MPN                      | <1 No/100 ml    | Colilert18 (IDEXX)    |
| <i>E.coli</i> 35°C MPN                          | <1 No/100 ml    | Colilert 18 (IDEXX)   |
| Coliform bacteria 35°C MF                       | <1 CFU/100 ml   | SS 0281672            |
| <i>E.coli</i> 44°C MF                           | <1 CFU/100 ml   | SS 0281672 mod        |
| Enterococci 35°C MF                             | <1 CFU/100 ml   | SS EN ISO 78992 utg 1 |
| <i>Cl.perfringens</i> (pres) MF                 | <1 CFU/100 ml   | ISO/CD 64612 021220   |

Experiment 4 was performed as a pulse response experiment with a duration of 12 days to reach steady state in reduction. The pulse suspension consisted of pre-treated wastewater from Ryaverket in G-teborg, in which the analysed (Table 2) microorganisms exist in high numbers. In order to keep a relatively steady concentration in the pulse solution, a new batch was prepared every day during the experiment. Pre-treated wastewater was added to the raw water to reach a 2% wastewater contamination, which is a higher concentration than commonly encountered in the raw water from the G-ta lv River; the raw water supply for G-teborg Water. Furthermore, the concentration of wastewater was kept low to prevent high concentrations of suspended solids that could clog the infiltration surface of the column.

Coliforms, *E.coli*, enterococci and *Clostridium perfringens* were chosen as model organisms in this study. *Cl. perfringens* is especially interesting due to its spore forming ability. Spores show a resistance comparable to the parasites but are of a size equal to bacteria. A  $\log_{10}$ -reduction was calculated by using the maximum detected effluent concentration for each model organism,  $C_{\max, \text{column}}$ , which was divided with the minimum;  $C_{\min, \text{pulse}}$ , and the maximum;  $C_{\max, \text{pulse}}$ , concentration in the amended pulse concentration. Values <1 cfu or No/ml was treated as no detection=0. The relationships used were minimum reduction=  $-\log_{10} (C_{\max, \text{column}}/C_{\min, \text{pulse}})$  and maximum reduction=  $-\log_{10} (C_{\max, \text{column}}/C_{\max, \text{pulse}})$ .

## Reduction of NOM

The results of the analyses of  $\text{COD}_{\text{Mn}}$ , TOC and  $\text{UV}_{254}$  were statistically compiled and the reduction was calculated as the difference between the median concentration in raw water and the median concentration in the effluent of each studied case. Finally, the reduction was related to soil passage and transport time.

## RAW WATER QUALITY AND HYDRO GEOLOGICAL CHARACTERIZATION OF SAND MATERIAL

**TABLE 3. Experiment parameters**

| Parameter              | Unit        | Exp. 1 | Exp. 2 | Exp. 3 |
|------------------------|-------------|--------|--------|--------|
| COD <sub>Mn</sub>      | mg /l       | 2900   | 3400   | 15     |
| TOC                    | mg/l        |        |        | 20     |
| UV 254nm               | ae/cm       | 1.78   | 8.9    | 0.15   |
| pH                     |             | 6.9    | 7      | 6.8    |
| El. Conduct.           | mS/m        | 1348   | 5500   | 36     |
| COD <sub>Mn</sub> /TOC |             |        |        | 0.8    |
| UV <sub>254</sub> /TOC | l/(mg x m). | 0.6    | 0.3    | 0.8    |

The raw water from Delsj~arna was soft; 10 mg Ca /l , and had a temperature variation from 2 C in the winter to 20 C in the summer. The columns, however, were placed indoors and exposed to a constant temperature of 13 C giving the water a similar temperature with only minor variations. Salinity (11 mS/m) and concentrations of metals

were low. The microbiological quality was good with low concentrations of faecal indicators. The turbidity was less than 1 FNU, pH was about 7.1 and alkalinity was approx. 0.25 mmol/l. The concentration of organic matter was 4 -7 mg/l O<sub>2</sub> for COD<sub>Mn</sub>, 4 - 6 mg/l TOC and 0.092 - 0.202 ae/cm measured as UV- absorbance at 254 nm.

The material from Gråbo was found to originate from granite/gneiss with 40% quartz and 27% Na-silicates and with a clay content of 2-3%. The bulk density of the material was measured to be 1526 kg/m<sup>3</sup> with a total porosity of 41%. The specific yield (drainage porosity) was measured to 24% and the theoretical specific surface was calculated to be  $1.23 \times 10^5 \text{ } \bar{\text{m}} \text{ } 2.51 \times 10^5 \text{ m}^2/\text{m}^3$  (Rådmark and Svensson 2003). The grain size of d<sub>10</sub> was between 0.044 mm and 0.074 mm and d<sub>60</sub> between 0.210 and 0.500 mm, which gives d<sub>60</sub>/d<sub>10</sub> = 6-7.

The water transport time was 13-44 h for 1-4 m of unsaturated zone and 39 h for 2 m of saturated zone. The hydraulic conductivity was  $0.4 \times 10^{-4}$  to  $1.5 \times 10^{-4}$  m/s and the unsaturated water head was -25 cm. The analysis of the water transport time for both unsaturated and saturated conditions (Lundh *et al.*, 2006), suggests a rough estimation of the saturation of 56% of total porosity. With a total porosity of 41% that would result in a saturation of 23%.

## RESULTS AND DISCUSSION

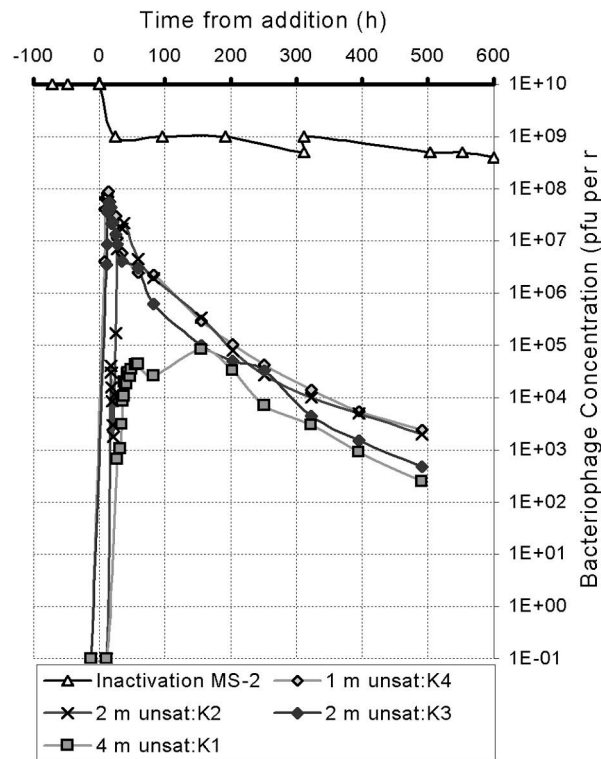
### Reduction of Bacteriophages

Figure 1 presents the result of MS-2 in experiment 1. The results of the other phage and experiment 2 have been presented in Lundh *et al.* (2006).

The result from experiment 1 and 2 show a reduction in phage concentration for all columns, observed as the difference between pulse concentration (e.g. Fig.1, curve for inactivation MS-2) and peak concentration (time ~20h from addition) in the response. The highest reduction was observed for the deepest unsaturated zone (4 m; K1).

In experiment 1 the saturated case (K0) did not work properly due to clogging but in experiment 2 the 2-m saturated case had a similar performance as the cases with 2 m and 1 m of unsaturated zone. The unsaturated zone did thus not seem to have a higher efficiency than the saturated zone.

During experiment 1 and 2 the salinity increased in the raw water due to the added pulse solution. The salinity increased from 11 mS/m in the unaffected raw water to 1300-5500 mS/m when the pulse solution with bacteriophages was added. Also the organic content increased from approx. 6 mg/l to 2900-3400 mg/l. The pulse solution had also an effect on pH that was below 6 during the addition. During experiment 3 these effects were much smaller (Table 3) and the results, referring to the filter factor, also shows similarities to experiment 1. Thus, these effects might have had an impact on experiment 2, for which the observed filter factor was smaller than for experiment 1 and 3.



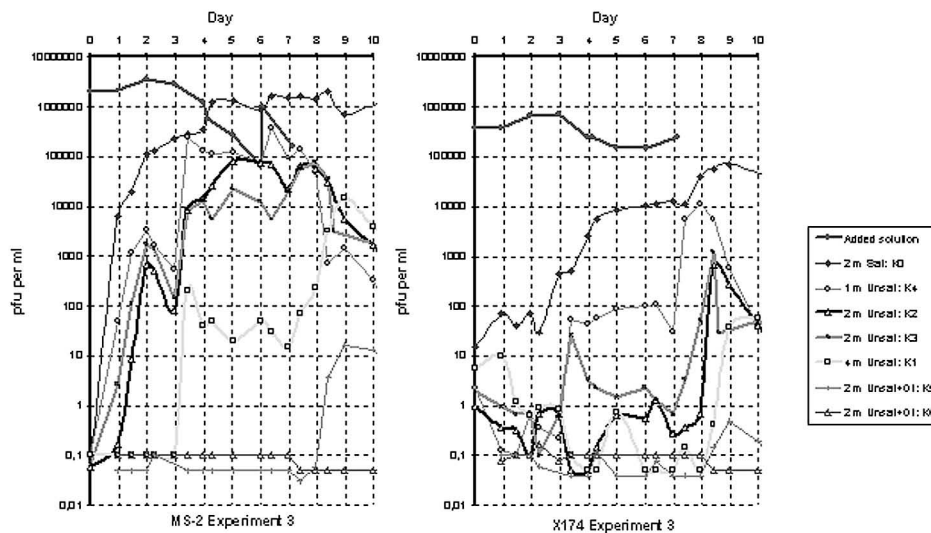
Phages MS-2 Experiment 1

**FIGURE 1. The response curves of a 40 minute-pulse with bacteriophage MS-2 in experiment 1.**

The reduction in 4 m unsaturated zone in experiment 2 displayed a less efficiency than in experiment 1, which was likely due to the high salinity, and the high content of organic matter and virus in the preceding experiment 1, that possibly occupied adsorption sites in experiment 2.

The pulse concentration changed in time for the different experiments, assumed to be a result of inactivation of phages and mixing procedures. The curve that shows the concentration in the added solution was the result of the monitoring for inactivation in the batch that was used for the pulse and comprised of a longer time period than the actual pulse.

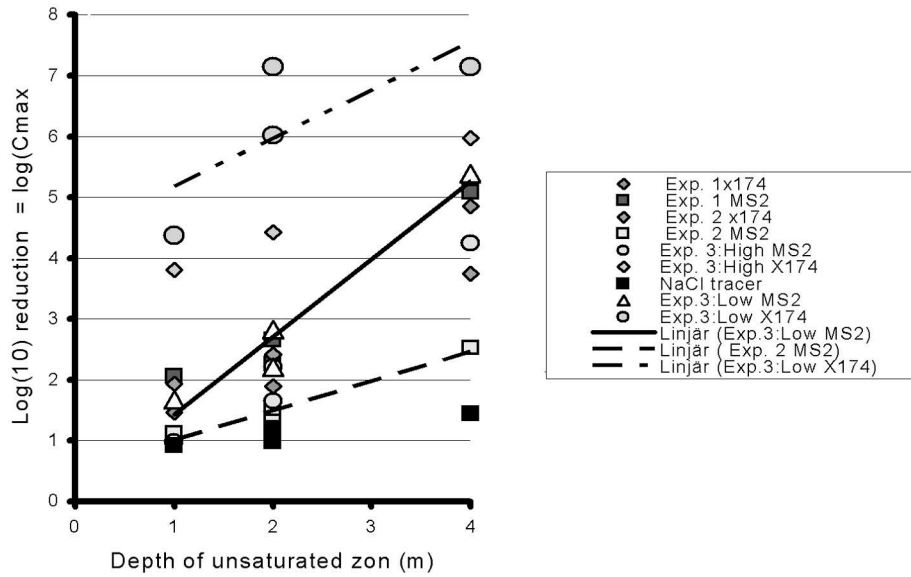
Experiment 3 presented distinct differences in phage concentration in the effluent from the columns (Figure 2). The results display the highest effluent concentration from the 2-m case with saturated zone: 1-4  $\log_{10}$  higher than for the 2-m case of unsaturated zone. The result suggests that the reduction during the experiment increased with depth of unsaturated zone with the 4-m case showing the highest reduction.



**FIGURE 2. Experiment 3 with bacteriophages MS-2 och ÖX174 with the intention of reaching steady state. The pulse was added continuously during 7 days.**

The results from experiment 3 were compared to those in experiments 1 and 2 with the same calculation procedure as described in the Method section. Instead of using the peak concentration, however, a time period of estimated steady state (5 days) was used. Two reduction conditions were calculated; high and low. These are

refereeing to a maximum and minimum organism load, which are the maximum and minimum concentration in the amended pulse.



**FIGURE 3.** The log<sub>10</sub>-reduction of bacteriophages in relation to depth of unsaturated zone. C<sub>0</sub> is the maximum pulse concentration.

| TABLE 4. The filter factor (regression analysis) based on log <sub>10</sub> -reduction for each column and bacteriophage. |                |                                                            |                  |                                        |                                                             |
|---------------------------------------------------------------------------------------------------------------------------|----------------|------------------------------------------------------------|------------------|----------------------------------------|-------------------------------------------------------------|
| Experiment                                                                                                                | Experiment     | y<br>log <sub>10</sub> (C <sub>max</sub> /C <sub>0</sub> ) | R <sup>2</sup>   | C <sub>f</sub><br>log <sub>10</sub> /m | F = C <sub>f</sub> - C <sub>d</sub><br>log <sub>10</sub> /m |
| 1 MS-2                                                                                                                    | Pulse response | 1.0717x + 0.602                                            | 0.93             | 1.07                                   | 0.90                                                        |
|                                                                                                                           | 1 ΦX174        | Pulse response                                             | 1.163x + 0.1045  | 0.94                                   | 1.16                                                        |
| 2 MS-2                                                                                                                    | Pulse response | 0.4872x + 0.5197                                           | 0.96             | 0.49                                   | 0.32                                                        |
|                                                                                                                           | 2 ΦX174        | Pulse response                                             | 0.6275x + 1.1571 | 0.96                                   | 0.63                                                        |
| 3:1 MS-2                                                                                                                  | Steady state   | 1.1056x - 0.217                                            | 0.97             | 1.11                                   | 0.94                                                        |
|                                                                                                                           | 3:2 MS-2       | Steady state                                               | 1.2781x + 0.1458 | 0.96                                   | 1.28                                                        |
| 3:1 ΦX174                                                                                                                 | Steady state   | 0.6488x + 3.5953                                           | 0.54             | 0.65                                   | 0.48                                                        |
|                                                                                                                           | 3:2 ΦX174      | Steady state                                               | 0.7904x + 4.392  | 0.57                                   | 0.79                                                        |
|                                                                                                                           |                |                                                            |                  |                                        | Filter Factor F<br>log <sub>10</sub> /m                     |
| Dispersion exp.                                                                                                           |                | y (log <sub>10</sub> (C <sub>max</sub> /C <sub>0</sub> ))  | R <sup>2</sup>   | C <sub>d</sub> (log <sub>10</sub> /m)  | min medel max                                               |
| NaCl (Grey)                                                                                                               | Pulse response | 0.1749x + 0.7426                                           | 0.85             | 0.17                                   | 0.32 0.73 1.11                                              |

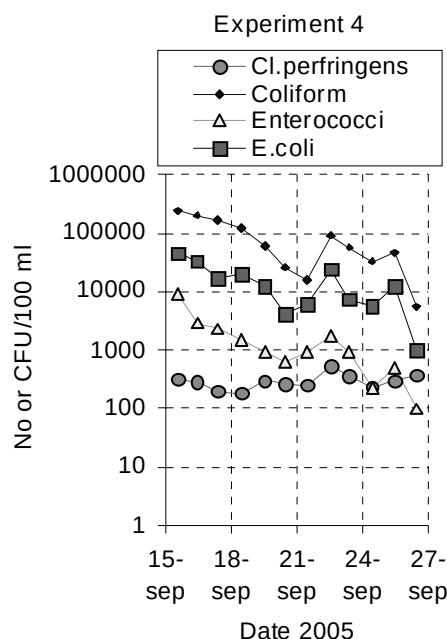


The average reduction, expressed as the filter factor, gained by analysing the inclination from 1 m to 4 m for each case in figure 3, was calculated to  $0.73 \pm 0.4 \log_{10}/\text{m}$  (Table 4). The filter factor was higher in experiment 1 ( $0.90\text{--}0.99 \log_{10}/\text{m}$ ) than in experiment 2 ( $0.32\text{--}0.46 \log_{10}/\text{m}$ ), which could be explained by the high concentrations of phages and organic matter (experiment 1), which might have occupied sorption locations in the sand. In both experiments 1 and 2 the filter factor for MS-2 was lower than for  $\Phi\text{X174}$ .

### Reduction of Bacteria

The number of bacteria was expectedly higher in the pre-treated wastewater than in the raw water from Delsjöarna and the variation was significant in the mixed pulse concentration. The content of coliforms was between 5 500 and 240 000 No/100 ml and the numbers of *E.coli* varied between 1000 and 46 000 No/100 ml. Enterococci varied between 100 and 9000 CFU/100 ml and *Cl.perfringens* between 180 and 520 CFU/100 ml (Fig.4).

Neither coliforms nor *E.coli* were detected in any effluent during experiment 4 (Table 5). For 2 m of unsaturated zone (K2) 1 CFU/100 ml enterococci and 1-2 CFU/100 ml of *Cl.perfringens* was detected for 2 m and 4 m unsaturated zone and 2 m saturated zone. No bacterial parameter could be detected in any effluent from the IOCO-columns. Due to the low numbers of bacteria it was not possible to distinguish whether unsaturated condition was better than a saturated one or if 4 m of soil passage gave a higher efficiency than 1 m. Thus the  $\log_{10}$ -reduction in Table 5 is related to a depth of 1-4 m of soil passage.

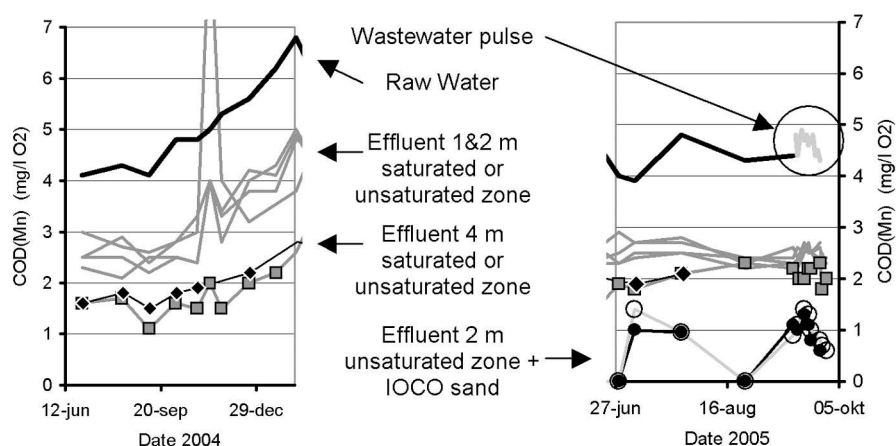


**FIGURE 4. Pulse concentrations for all columns in experiment 4.**

### Reduction of NOM

Figure 5 shows the reduction performance of NOM of the different columns. The graphics, (left diagram), suggests a 25% to 40% reduction in the first 2 meters and then another 25% the following 2 meters. The greater reduction in the first 2 meters

could be contributed to surface effects, such as biomass causing cake filtration and bacteria that consume organics. 3.



**FIGURE COD<sub>Mn</sub> in real time before and after the experiments 1, 2 and 3. Result from experiment 4 with a pulse of wastewater is included in the right diagram.**

**TABLE 5. Log<sub>10</sub>-reductionen of studied bacteria for 1-4 m soil passage.**

|                                 | Coliforms | E.coli    | Enterococci | Cl.perfr   |
|---------------------------------|-----------|-----------|-------------|------------|
|                                 | No/100 ml | No/100 ml | CFU/100 ml  | CFU/100 ml |
| No of samples                   | 12        | 12        | 12          | 12         |
| Average influent                | 88958     | 15675     | 1808        | 297        |
| Median influent                 | 59500     | 12000     | 910         | 290        |
| Minimum influent                | 5500      | 1000      | 100         | 230        |
| Maximum influent                | 240000    | 46000     | 9000        | 520        |
| Detection effluent              | 0         | 0         | 1           | 2          |
| log <sub>10</sub> reduction min | -         | -         | 2,0         | 2,1        |
| log <sub>10</sub> reduction max | >5,4      | >4,7      | 4,0         | 2,4        |

> Indicate that the reduction efficiency might be higher as there was no detection

During the period before the phage experiments 1, 2 and 3, which is shown in the left diagram, the raw water concentration of organic matter increased and the effluent quality in all columns followed. That would suggest that the removal capacity has been reached in terms of grams of organics removed per meter. For the 4-m cases, however, the difference between in and out concentration does seem to have

increased along the time period. A possible explanation could be that the removal capacity had been reached for the 2-m cases but not for the 4-m cases. Moreover, the result strongly suggests that the reduction of NOM seems independent of saturation rate as both conditions gave the same result.

The results in Table 6 present a reduction efficiency of 52-63% for 4 m of unsaturated zone. Reduction values depended on what parameter that was measured. The saturated case had more or less the same efficiency except for the reduction of UV-absorbance that was much lower for the saturated condition. That could indicate that humic-related (aromatic bindings) organic matter is more easily removed in unsaturated zone.

**TABLE 6. Resulting reduction of organic matter calculated for 4 meter of zone.**

|                     | Unsaturated zone  |                    |                  | Saturated zone    |                    |                  |
|---------------------|-------------------|--------------------|------------------|-------------------|--------------------|------------------|
|                     | COD <sub>Mn</sub> | UV <sub>254</sub>  | TOC              | COD <sub>Mn</sub> | UV <sub>254</sub>  | TOC              |
| Reduction over 4 m  | 63 %              | 54 %               | 52 %             | 58 %              | 28 %               | 48 %             |
| Reduction per meter | 0.67 mg/l/<br>m   | 0.016 ae/cm/<br>m  | 0.58 mg/l/<br>m  | 0.63 mg/l/<br>m   | 0.008 ae/cm/<br>m  | 0.47 mg/l/<br>m  |
| Reduction per hour  | 0.061 mg/l/<br>h  | 0.0014 ae/<br>cm/h | 0.052 mg/l/<br>h | 0.032 mg/l/<br>h  | 0.0004 ae/<br>cm/h | 0.027 mg/l/<br>h |

The reduction in unsaturated zone was measured to 0.58-0.67 mg/l per meter and 0.47-0.63 mg/l per meter of saturated zone (Table 6). The initial reduction could probably be as much as 1.25 mg/l per meter in the first 2 meters, which was observed in a previous investigation (Lundh *et al.*, 2006). The same investigation also found that the removal appears to be linear for the investigated 1 to 4 meters.

The 30-cm layer of IOCO-sand proved to have a high reduction efficiency regarding organic matter (Fig.5, right diagram). The UV-absorbance was better reduced (by almost 5 times) than under solely unsaturated conditions and the reduction of COD<sub>Mn</sub> was almost twice as good as under solely unsaturated conditions (Lundh *et al.*, 2006).

The analysis of the reduction efficiency was done for the period before experiment 1, 2 and 3 (Figure 5, left diagram). During this period the IOCO-columns were not in operation. The period after the experiments (right diagram) presents a different situation. The difference between the 2-m cases and the 4 m-cases was much less and the reduction efficiency in the 4-m cases seems to have decreased.

One explanation could be that many of the sorption locations in the material had been occupied, preventing more organic matter from attaching to the sand. During the experiments the columns had been loaded with high concentrations of organic

matter. The initial part of the reduction, seen in Fig.5, right diagram as the drop from approximately 4.5 mg/l down to 2.5 mg/l, could perhaps be related to the easy removable part of the organic matter, removed in the vicinity of the infiltration surface. The remaining part might have been more difficult to remove and if many of the sorption locations were occupied, only a small fraction would have been removed deeper down in the column. However, a material with reactive surfaces, like the IOCO-sand, would perhaps attract the remaining part of organic matter and the result (Fig.5, right diagram) might be evidence for such theory.

## CONCLUSIONS

The microbiological barrier regarding virus was efficient in 4 m of unsaturated zone ( $3 \log_{10}$ ). It was higher than disinfection with chlorine ( $2 \log_{10}$ ) and as high as chemical precipitation ( $3-4 \log_{10}$  for bacteria and virus). The efficiency was higher in unsaturated zone compared to saturated zone. IOCO-sand strongly enhanced the reduction of virus (in total  $4-6 \log_{10}$ ) and could be favorable in enhancing the reduction in a thinner layer of unsaturated zone.

The microbiological barrier efficiency regarding bacteria was sufficient in 4 m of unsaturated zone ( $\approx 2.4 \log_{10}$ ). It was similar to disinfection with chlorine ( $2 \log_{10}$ ) and slightly lower than chemical precipitation ( $3-4 \log_{10}$  for bacteria and virus). There was no obvious difference in efficiency between an unsaturated and a saturated condition.

The reduction of organic matter was sufficient in 4 m of unsaturated zone with up to 63 % reduction in  $\text{COD}_{\text{Mn}}$ . The reduction was slightly lower in the saturated zone, especially regarding organic matter measured as UV-absorbance. IOCO-sand strongly improved the reduction and should be favorable in enhancing the reduction in a thinner layer of unsaturated zone.

The time frame for the experiment was narrow and the temperature relatively constant. A study on long-term consequences and temperature related effects is highly warranted. Such a study could be valuable for better understanding of the mechanisms involved in managed aquifer recharge.

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# EFFECT OF WASTEWATER QUALITY AND PROCESS PARAMETERS ON REMOVAL OF EFFLUENT ORGANIC MATTER (EfOM) DURING SOIL AQUIFER TREATMENT

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## ABSTRACT

Soil Aquifer Treatment (SAT) is an emerging natural treatment technology which, in combination with other available wastewater treatment technologies, can produce effluent of acceptable quality for indirect potable reuse. The effect of water quality and process parameters on the removal of wastewater effluent organic matter (EfOM) during SAT was analysed by conducting soil column and batch tests at laboratory-scale using primary and secondary effluents from a wastewater treatment plant. The DOC concentrations of primary and secondary effluents were 35-37 mg/L and 12-15 mg/L, respectively. Silica sand of size 0.8 - 1.25 mm was used as a filter (soil) media. DOC, UVA<sub>254</sub>, SUVA, oxygen and nitrogen profiles along the depth of the column were monitored. After acclimation (i.e., ripening) of the soil columns, steady-state DOC removal was 46 - 54 % for settled primary effluent compared to 15 - 30 % for secondary effluent. When the hydraulic loading rate was increased from 1.25 m/day to 2.5 m/day, the effluent DOC concentration of the primary effluent increased from 15.2 mg/L to 18.4 mg/L, and the effluent DOC of the secondary effluent increased from 10.3 mg/L to 11.6 mg/L. For batch studies with bioacclimated sand, the DOC removals from non-settled primary effluent were 58 % and 49 % under anoxic and aerobic conditions, respectively. The DOC removal behavior followed first order kinetics and a three parameter model was fitted for both batch and soil column experimental data. It was concluded that extending the limits of SAT from secondary to primary effluents is feasible.

## KEYWORDS

Soil aquifer treatment, organic matter, dissolved organic carbon, biodegradation, biomass, and hydraulic loading rate

## INTRODUCTION

Soil Aquifer Treatment (SAT) is a managed aquifer recharge as well as wastewater treatment technology which, in combination with other available wastewater treatment technologies, can produce effluent of acceptable quality for indirect potable reuse. This technology uses physical, chemical and biological processes in the soil matrix and aquifer for wastewater treatment. It is a low cost and appropriate option for wastewater reclamation in developing countries as well as developed countries. It is also considered appropriate for replenishment of underground water to avoid exhaustion of groundwater resources and lowering of groundwater levels.

Furthermore, artificial recharge of groundwater basins with SAT effluent contributes to the sustainability of surface water and groundwater resources within the context of integrated water resources management (Asano and Cotruvo, 2004).

The characteristics of organic matter present in the wastewater used for recharge affects water quality after groundwater recharge during SAT. Organic carbon present in reclaimed groundwater may adversely affect subsequent treatment unit operations through formation of disinfection byproducts, membrane fouling and biological growth in the distribution systems (Rauch *et al.*, 2005). Organic pollutant removal efficiency during SAT depends on the processes conditions, the organic transformations during groundwater recharge, and hydrogeological conditions. Detailed information on organic matter removal by SAT systems subject to different processes and hydrogeological conditions is lacking, making it difficult to predict the degree of purification by SAT at various sites.

Several studies have been conducted on removal of effluent organic matter (EfOM) and its transformations during SAT of conventional wastewater treatment effluents, however, very little is known about organic matter removal characteristics of SAT for non-nitrified primary effluent. Furthermore, pertinent information on organic matter removal by SAT systems under different process and hydro-geological conditions is lacking yet it is essential for rational design of SAT systems and to predict the degree of purification provided by SAT at various sites (Sattler *et al.*, 2005). Therefore, there is a need to investigate removal of organic compounds during SAT by analyzing effects of influent wastewater quality, soil properties, retention time, and hydraulic loading rate under aerobic and anoxic conditions. This study focused on analyzing the potential and suitability of SAT technology for removal of EfOM from wastewater effluents under different process conditions. The results of this study will provide more data and knowledge that will contribute to future rational design and operation of SAT systems and hence future full-scale operation of SAT systems.

The main goal of this study was to investigate the potential and suitability of SAT technology for removal of organic matter from treated wastewater (primary and secondary effluents). This was achieved by investigating the effect of wastewater quality on removal of organic matter during SAT and assessing the performance of SAT under different process conditions. Finally, the degradation process of organic carbon during SAT under various process conditions was modelled.

## MATERIALS AND METHODS

The study was carried out by conducting soil column and batch tests at laboratory-scale under different process conditions using primary and secondary effluents from a wastewater treatment plant located in Hoek van Holland, The Netherlands. The DOC concentrations of primary and secondary effluents were 35 - 37 mg/L and 12 - 15 mg/L, respectively. The average wastewater quality parameters are included in Table

1. Silica sand of size 0.8 - 1.25 mm was used as a filter (soil) media. Aerobic conditions were maintained by aeration of influent wastewater before entering the column and anoxic conditions were achieved in the presence of adequate nitrate by stripping off oxygen present in wastewater with continuous flushing of the influent tank with nitrogen gas.

**TABLE 7. Wastewater quality parameters of primary and secondary effluents used for batch and soil column studies**

| Parameter            | Units    | Primary Effluent - Non-settled (Average) | Primary Effluent - Settled (Average) | Secondary Effluent (Average) |
|----------------------|----------|------------------------------------------|--------------------------------------|------------------------------|
| TSS                  | mg/L     | 127                                      | 45                                   | 18.8                         |
| O <sub>2</sub>       | mg/L     | 1.5                                      | 8.5                                  | 4.0                          |
| pH                   | -        | 7.5                                      | 7.95                                 | 7.2                          |
| Temperature          | °C       | 7.1                                      | 20.1                                 | 9.6                          |
| NH <sub>4</sub> -N   | mg/L     | 25                                       | 23.5                                 | 0.4                          |
| PO <sub>4</sub> -P   | mg/L     | 5.1                                      | 6.8                                  | 2.2                          |
| EC                   | µS/cm    | 1939                                     | 1646                                 | 1213                         |
| DOC                  | mg/L     | 35.3                                     | 35                                   | 13.5                         |
| UVA <sub>254nm</sub> | /cm      | 0.75                                     | 0.76                                 | 0.4                          |
| SUVA                 | L/(mg.m) | 2.1                                      | 2.2                                  | 3.0                          |
| BOD                  | mg/L     | 114                                      | 103                                  | 5.7                          |
| TOC                  | mg/L     | 133                                      | 98.7                                 | 16.8                         |
| NO <sub>3</sub> -N   | mg/L     | 2.55                                     | 2.55                                 | 2.6                          |
| TKN                  | mg/L     | 34                                       | 34                                   | 4.2                          |
| COD                  | mg/L     | 330                                      | 267                                  | 56.8                         |

### Soil Column Experimental Set Up

The soil column setup consisted of 2 PVC columns of 54 mm internal diameter and 2.5 m length each connected in series, with 11 sampling points at intervals of 0.5 m. The hydraulic loading rate was controlled by adjusting the valves. For soil column studies, hydraulic loading rate (HLR) varied from 1.25 m/day to 2.5 m/day. DOC, UVA, SUVA, oxygen and nitrogen profiles along the depth of the column were monitored. DOC for non-filtered and filtered samples were measured using Model 700 TOC analyzer of O-I corporation, USA. Each time after changing the process conditions, the soil column system was allowed to attain steady state with respect to DOC removal before taking samples to study the effect of a particular parameter.



## Acclimation/Ripening of the Soil Columns

First of all, the soil columns were biologically acclimated (ripened) using the settled primary effluent under aerobic conditions. Pre-settling of primary effluent 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 TSS concentration was reduced from 128 to 45 mg/L by pre-settling the primary effluent for at least 2 hours before transferring it to the influent tank.

Redox conditions were maintained by continuous aeration of the settled primary effluent using a fine bubble diffuser in the influent tank. The average influent DOC concentration during this ripening period was  $35.7 \pm 1.61$  mg/L and the filtration rate was 1.25 m/day. 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 50 % DOC removal was obtained after 30 days of soil column ripening.

## Batch experimental set up

**Preparation of the bio-sand:** In order to conduct batch experiments in the laboratory using media with biofilm (simulating soil aquifer treatment), bio-sand was prepared in the laboratory using 0.8 ñ 1.25 mm silica sand and secondary effluent from the wastewater treatment plant. This was done by putting 3 kg of silica sand in a 10-litre container and applying secondary effluent to this reactor. The secondary effluent in the reactor was continuously stirred without mixing the filter media. The secondary effluent was renewed by withdrawing the supernatant and feeding the reactor with a new batch of secondary effluent after every 4 days for 30 days. This was done to allow growth of heterotrophic bacteria (biofilm) on the media to produce bio-sand.

Batch experiments were conducted by transferring 25 g of the bio-sand (prepared earlier) to each of triplicate 0.5-litre glass bottles that were later fed with primary or secondary effluent and aerated to obtain a dissolved oxygen concentration of ~6.9 mg/L. The triplicate batch reactors were operated under similar conditions, anoxic and aerobic, as desired. Samples were taken periodically and DOC and UVA were measured.

## Modeling DOC removal in batch experiments and soil columns

A three-term model (given below) was fitted to batch and soil column experimental data in order to explain the removal of different fractions of DOC during SAT.

$$C_t = C_1 * e^{-\lambda_1 * t} + C_2 * e^{-\lambda_2 * t} + C_3 \quad (\text{EQ 1})$$

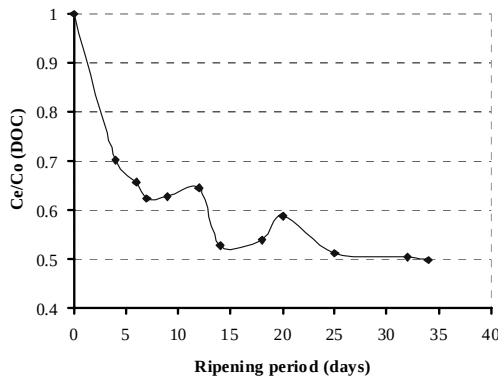
Where:

- $C_t$  is the DOC concentration (mg/L) at any time  $t$
- $C_1$  is the easily biodegradable DOC concentration (mg/L)
- $C_2$  is the slowly biodegradable DOC concentration (mg/L)
- $C_3$  is the non-biodegradable DOC concentration (mg/L)
- $\lambda_1$  and  $\lambda_2$  are rate constants of the biodegradation process (/h)
- $t$  is the biodegradation time (h)

## RESULTS AND DISCUSSION

### Soil column studies with primary effluent

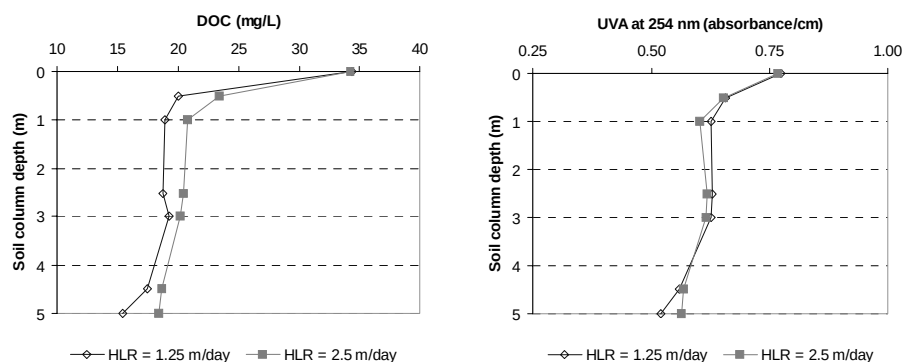
In column studies, it took about 30 days to reach steady state (i.e. acclimation) with respect to DOC removal when the soil column was operated with primary effluent with the HLR of 1.25 m/day. This ripening can be attributed to the development of a biofilm on the filter media which subsequently increased the DOC removal efficiency. Figure 1 shows the increase in DOC removal with the ripening of the soil column.



**FIGURE 1. DOC removal during acclimation or ripening period (Hydraulic loading rate = 1.25 m/day; Influent: settled primary effluent; Bed depth = 5m, media size = 0.8 - 1.25 mm; under aerobic conditions).**

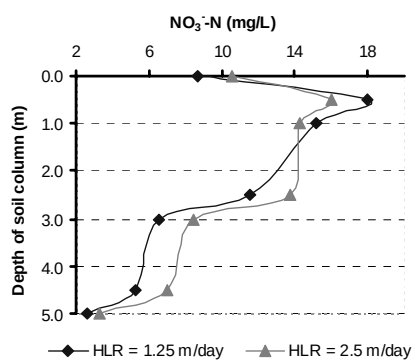
After the ripening period, the soil column system was operated at the HLR of 1.25 m/day and 2.5 m/day to determine the effect of HLR on DOC removal. First the system was operated at HLR of 1.25 m/day and the samples were for taken for analysis. After changing the HLR from 1.25 m/day to 2.5 m/day, the system was again allowed to attain the steady state with respect to DOC removal before taking the samples for

analysis. Figure 2 shows the DOC and UVA along the depth of the column after ripening at different loading rates. The effluent DOC concentration of the primary effluent increased from 15.2 mg/L to 18.4 mg/L when the hydraulic loading rate was increased from 1.25 m/day to 2.5 m/day. The corresponding DOC removal rates were 54 % and 46 %, respectively. Thus, there was about an 8 % reduction in DOC removal when the hydraulic loading rate was increased from 1.25 m/day to 2.5 m/day. Furthermore, it was observed that effluent DOC at HLR of 1.25 m/day was about 15 mg/L which is close to that obtained from activated sludge systems. The overall trend regarding UVA is the same for both HLR values showing some decrease in the first 0.5 m of the column due to high removal of non-humic organic matter in the top layers. The SUVA values of the primary effluent increased from less than 2.3 L/mg m to greater than 2.9 L/mg m in first 1 m of the soil column. This indicates the preferential removal of non-humics in the top part of the soil column.

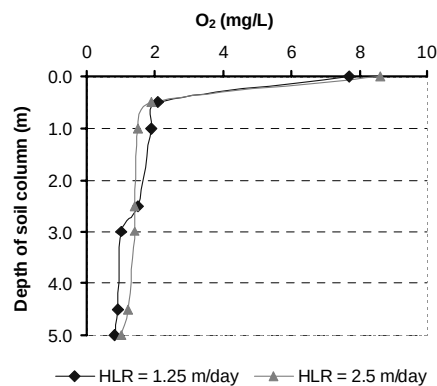


**FIGURE 2. DOC and UVA profile along the depth of the soil column at different hydraulic loading rates with primary effluent, under aerobic conditions.**

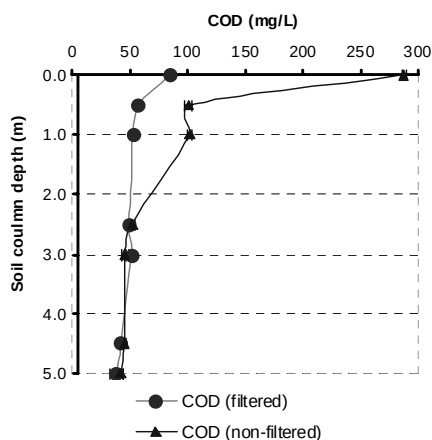
Figures 3 and 4 present nitrate and oxygen profiles along the depth of the column, respectively. It is clear from Figure 3 that in the top one meter of the column there is nitrification of ammonium present, leading to an increase in nitrate concentration. As there is depletion of oxygen in the subsequent depth of the column, the denitrification of the nitrate starts. As expected, at higher HLR, nitrification and denitrification activities shifted deeper down in the soil column. It can be concluded that when proper redox conditions and adequate depth of the soil column is provided, both nitrification and denitrification can be achieved during SAT.



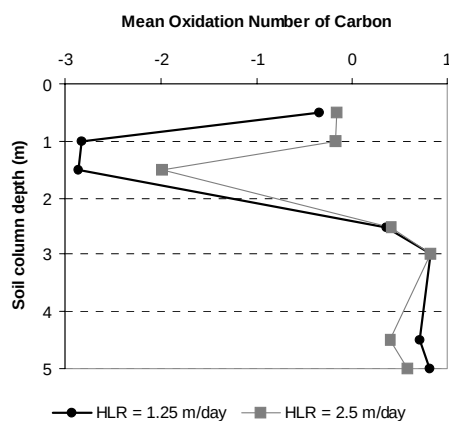
**FIGURE 3.**  $\text{NO}_3^-$ -N profile along the soil column at different HLR when operated under aerobic conditions with settled primary effluent.



**FIGURE 4.** Oxygen profile along the soil column at different HLR when operated under aerobic conditions with settled primary effluent.



**FIGURE 5.** COD profile along the soil column when operated with settled primary effluent at HLR = 1.25 m/day under aerobic conditions.



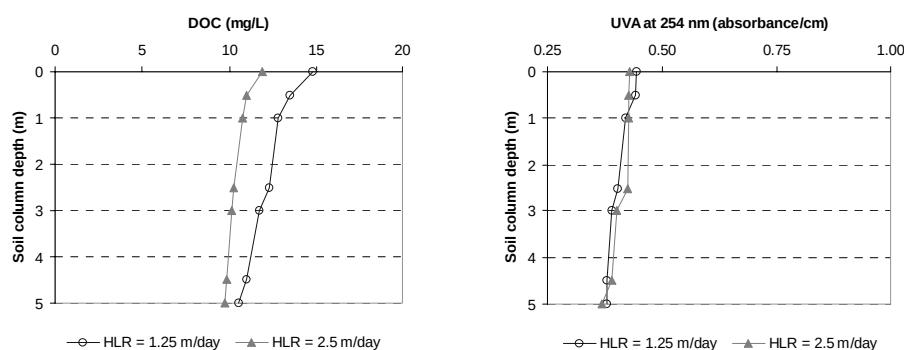
**FIGURE 6.** MOC profile along the soil column when operated with settled primary effluent at different HLR under aerobic conditions.

Figure 5 presents the COD profile along the soil column for filtered (using 0.45 mm filter) and unfiltered samples. The non-filtered COD removal was 86 % while filtered COD removal was 55 %. The filtered and non filtered COD levels of the soil column effluents were 38 mg/L and 42 mg/L respectively implying that both physical straining and degradation processes are significant with regard to COD removal and the difference in effluent DOCs is small due to the absence of suspended solids. This is in agreement with Idelovitch *et al.* (2003) who found that there is virtually no difference between  $COD_{non-filtered}$  and  $COD_{filtered}$  after SAT with partially treated effluent from a wastewater treatment plant.

The mean oxidation number of carbon (MOC) profile along the depth of the soil column at different HLR for experiments with primary effluents are presented in Figure 6. This shows that MOC are in the range -3 to 1, which indicates that DOC removal during SAT are chemical degradation process (Vogel *et al.* 2000).

#### *Soil column experiments with secondary effluent*

As in the case of primary effluent, the soil columns were first acclimated for 30 days to achieve steady state with respect to DOC removal and then DOC removals at different hydraulic loading rates were studied. At HLR of 1.25 m/day the effluent DOC concentration was  $10.3 \pm 0.7$  mg/L with an average DOC removal of 30 %. When operated at HLR of 2.5 m/day with the average influent concentration of  $11.6 \pm 0.3$  mg/L, the soil column effluent DOC concentration was  $9.7 \pm 0.1$  mg/L and the corresponding average DOC removal rate was 15 %. In general, there was no significant change in DOC removal and UVA along the depth of the soil column with secondary effluent when the hydraulic loading rate was increased from 1.25 m/day to 2.5 m/day (Figure 7).

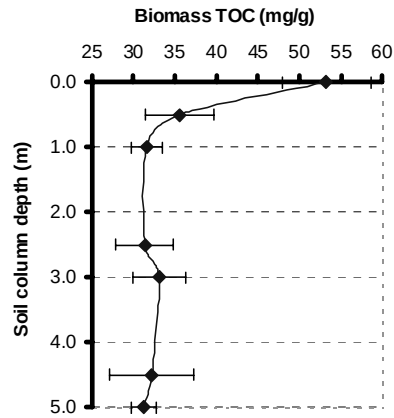


**FIGURE 7. DOC and UVA profile along the depth of the soil column at different hydraulic loading rates with secondary effluent, under aerobic conditions.**

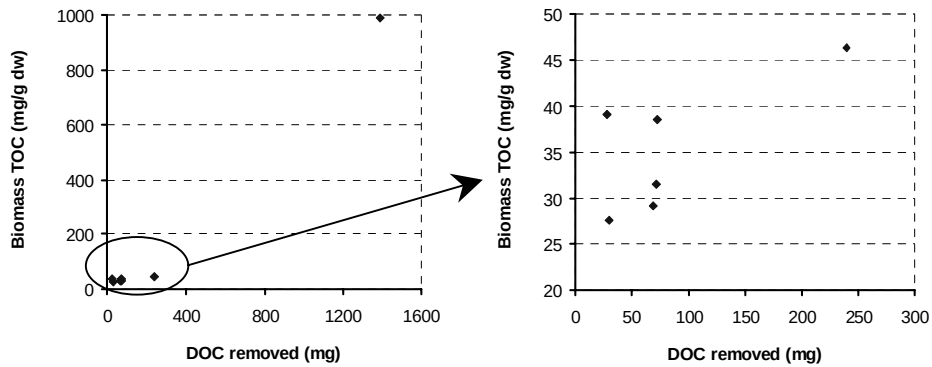
The preferential removal of non-humic substances in the first hours of filter bed contact time (at the top of the soil column) led to an increase in the SUVA values (from 3 to 3.3 L/mg.m) along the depth of soil column though not as big as for the case with primary effluent. The observed increases in SUVA values in soil columns are in agreement with observations made by Fox *et al.* (2001) during their studies on reclaimed water from a nitrifying -denitrifying activated sludge treatment system treatment plant.

#### *Soil biomass and DOC removal*

At the end of the soil column experiments, aliquots of sand were taken from different depths of the column and the TOC of the biomass on the sand samples were measured after extraction of the biomass using 2M H<sub>2</sub>SO<sub>4</sub>. As expected biomass TOC was high at top 0.5 m of the column. Biomass development was much less at the depth below 1 m of the soil column (Figure 8). This is in agreement with high TOC removal at the top of the soil column and no significant DOC removal below 1 m depth. Figure 9 shows the relationship between DOC removed in the soil column and biomass measured as TOC. The general trend was that the higher the DOC removed, the higher the biomass formed on the sand media. No linear correlation was established between biomass TOC and DOC removed as there was one point corresponding to top 0.5 m of the column with high values of both of these parameters whereas at other depths both of these parameters were relatively low. A similar trend was observed in soil column studies with secondary effluent.



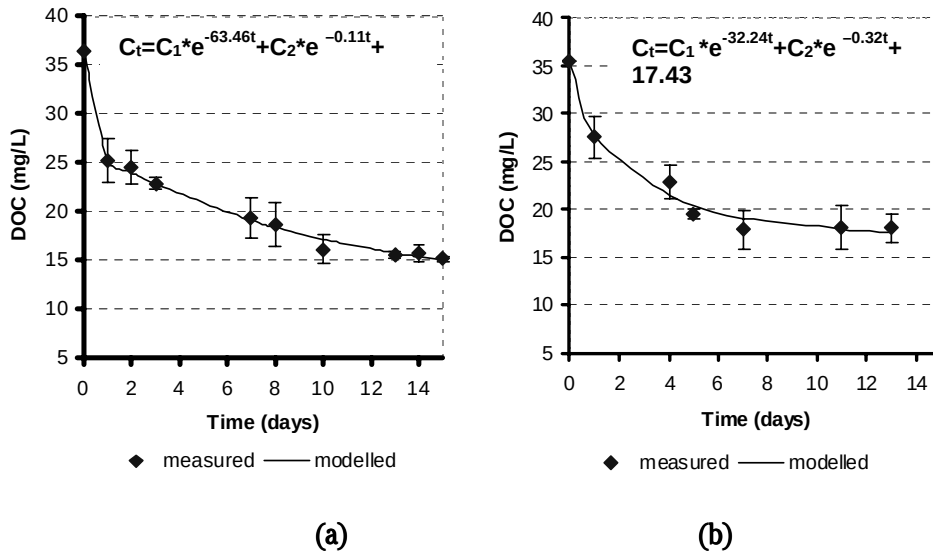
**FIGURE 8. Biomass TOC along the soil column depth (HLR = 2.5 m/day; Influent: settled primary effluent).**



**FIGURE 9. Relationship between DOC removed and biomass TOC along the soil column (HLR = 2.5 m/day; Influent = settled primary effluent).**

### *Batch experiments*

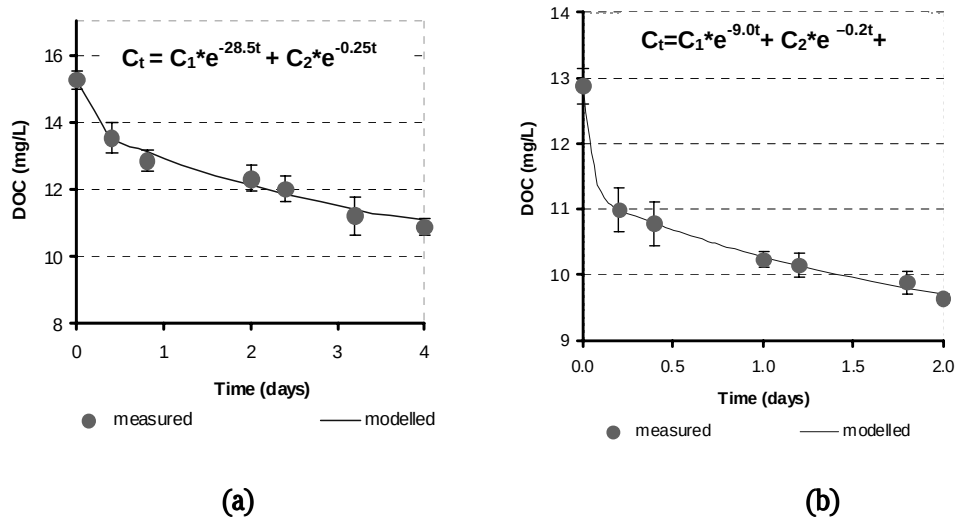
Batch experiments were conducted using the bio-sand and non-settled primary effluent under aerobic and anoxic conditions. The changes in DOC concentrations in the batch reactors with time under different conditions were monitored and the three parameter model was fitted to the experimental data as shown in Figure 10.



**FIGURE 10. Change in DOC concentration with time during batch experiments with bio-sand (0.8 - 1.25 mm) using non-settled primary effluent (a) aerobic conditions and (b) anoxic conditions.**

The DOC removals from non-settled primary effluent based on batch tests were 58 % and 49 % under aerobic and anoxic conditions, respectively. For primary effluent, the DOC concentration after SAT under aerobic conditions with both soil column and batch tests averaged around 15 mg/L although the biodegradation time with batch tests was three times that with soil column tests. This may be due to relatively lower amount of biomass available in batch system for biodegradation compared to soil columns. In the case of soil batch tests with primary effluents, the DOC removal was higher by about 10% under aerobic conditions as compared to anoxic conditions. As in the case of soil columns, SUVA values increased with time for both conditions. However the increase in SUVA values were higher under aerobic conditions (from 2.3 to 3.5 L/mg.m) compared to that under anoxic conditions (from 2.3 to 2.9 L/mg.m). These results clearly show that during SAT, DOC present in the wastewater effluent can be removed under both aerobic as well as anoxic conditions. As expected, data fitting of the model showed that primary effluent had a higher biodegradable DOC component than secondary effluent. It was concluded that extending the limits of SAT from secondary to primary effluents was feasible, when proper process conditions are provided.





**FIGURE 11. Measured and modeled change in DOC concentrations with time in soil columns with using secondary effluent under aerobic conditions at (a) HLR = 1.25 m/day and (b) HLR = 2.5 m/day.**

Figure 11 shows the three parameter model fitted to the soil column data for experiments with secondary effluent under aerobic conditions. It is clear from the model parameters that low DOC removal from secondary effluent in soil columns is due to the presence of a fraction of non-degradable DOC.

## CONCLUSIONS

- Average primary effluent DOC was reduced from 35 mg/L to 15 mg/L and 18 mg/L at HLR of 1.25 m/day and 2.5 m/day, respectively.
- There was only slight change in DOC removal for secondary effluent (from 10.3 mg/L to 11.6 mg/L) when HLR was increased from 1.25 m/day to 2.5 m/day.
- Oxygen concentration decreased from 8 mg/L to less than 2.5 mg/L in the top 1 m implying that most of the bioactivity, especially the biodegradation process, takes place in this upper part of the soil column.
- It was found that both nitrification and denitrification can be achieved during SAT when proper process conditions are provided.
- Batch studies showed that for the given primary effluent, aerobic DOC removal was about 10% higher than anoxic DOC removal.

- DOC removal behavior followed first order kinetics and a three-term model fitted to measured data showed that primary effluent has a higher BDOC.
- When adequate depth (travel time) and appropriate process conditions are provided, SAT is equally effective in treatment of primary effluent from wastewater treatment plants as in the case of treatment of secondary effluent. The implication is that, rather than just being a secondary effluent polishing step, SAT may also provide the equivalent of secondary biological treatment.

## ACKNOWLEDGEMENTS

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## TRACING RECHARGE WATER FROM SPREADING PONDS: A DECADE OF STUDIES

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### ABSTRACT

Determining hydraulic connections and travel times between recharge facilities and production wells has become increasingly important for permitting and managing MAR operations. This knowledge is critical for examining soil aquifer treatment and assessing the potential for future contamination. During the last decade a number of geochemical approaches have been developed to trace the movement of recharge water. Tritium/ $^3\text{He}$  dating can be used to estimate "far field" transport. Transport in the "near field" can be imaged with deliberate tracer experiments. Because the volumes of spreading basins are very large, non-reactive tracers that can be economically introduced at a sufficient concentration to allow quantification after at least a 1000:1 dilution must be used. Studies have shown that sulfur hexafluoride, boron isotopes, and isotopes of noble gases (e.g.,  $^3\text{He}$ ,  $^{124}\text{Xe}$ , and  $^{136}\text{Xe}$ ) satisfy these conditions. Results from five experiments, which employed gas tracers, indicate that retardation and gas loss is small at sites with a long history of recharge, shallow to moderately deep water tables (<15 m), and high recharge rates (>0.3 m/day). However, three experiments have shown that at new sites experiencing their first wetting, gas loss occurs indicating that gas tracers do not give reliable travel time information in these settings.

### KEYWORDS

Travel time, hydraulic connection, trapped air, deliberate tracer experiments, sulfur hexafluoride

### INTRODUCTION

Groundwater has been a primary source of potable and irrigation water for centuries. During the last 50 years, the soaring demand for freshwater has placed unprecedented stresses upon many aquifers throughout the developed and developing world. The projected growth in population combined with uncertainties associated with a changing climate will only aggravate these problems. A recent cost effective advancement in groundwater/surface water management aimed at augmenting water supplies is managed aquifer recharge (MAR), the practice of artificially recharging imported surface water, recycled (reclaimed) wastewater, or surplus runoff into aquifers for storage and later extraction (e.g., Bouwer, 2002; Dillon, 2005).

For a MAR operation to be successful, a few challenges must be overcome. First, a source of surface water must be found. Second, facilities, which can maintain recharge rates  $>10$  cm/day, must be engineered. Third, because the available water for the recharge operation is often of lesser quality than generally used in potable supplies, there is the potential to degrade the existing groundwater. The introduction of infective micro-organisms, disinfection by-products, or trace organic compounds with unknown health risks such as pharmaceuticals, is a concern, especially when urban runoff or reclaimed wastewater is a large component of the source water (NRC 1998). In the near future, these low quality sources are likely to become a larger portion of the recharge water at most MAR operations because the availability of higher quality water is limited and may shrink due to shifts in climate and the diversion of other uses such as maintaining riparian ecosystems. Because of water quality concerns, it is vital to understand the fate and transport of potential contaminants near MAR sites. Only from this understanding can cost effective and appropriate regulations be developed.

Recent results of detailed water quality studies near MAR operations have shown that many potential contaminants such as dissolved organic carbon (DOC), nitrate, some pharmaceuticals, and most infective micro-organisms are naturally removed or become inactive with time and distance in the subsurface (e.g., Yates and Yates, 1987; Fox et al., 2001; Drewes et al., 2002; Hiscock and Grischek, 2002). These water quality changes, which are considered one of the benefits of MAR (e.g., Dillon, 2005), are generally observed very soon after recharge, near the facility.

Despite these earlier studies, water quality concerns remain the focus of many regulations and an obstacle for the permitting of new MAR operations that includes a reuse component. Development of field methodologies to evaluate flow near MAR facilities is critical for the effective management of these operations. This knowledge is needed to establish hydraulic connections and travel times between the recharge location and production wells and to evaluate in situ biogeochemical reactions that change the quality of the recharged water within the subsurface. A fundamental approach for investigating travel times, flow paths, recharge rates, and dispersivity in groundwater systems relies on geochemical tracers. Tritium/ $^3\text{He}$  ( $\text{T}/^3\text{He}$ ) dating is the best technique for examining flow over spatial scales of kilometers and temporal scales of years to decades near MAR operations (Clark et al., 2004; Avisar et al., 2006; Massman et al., 2006). Other environmental tracer dating techniques such as CFCs are likely to be unreliable near MAR operations because non-atmospheric sources of these tracers may complicate the interpretation of apparent ages, especially when reclaimed water is being recharged (e.g., Schultz et al., 1976; Clark et al., 1995).

Because of the typical uncertainty associated with geochemical dating techniques ( $\pm 2$  years), these methods are not well suited for examining short-term transport. Deliberate tracer experiments using gases such as sulfur hexafluoride ( $\text{SF}_6$ ) or noble

gases (i.e. He or Xe isotopes) below spreading ponds have become acceptable methods for evaluating transport from recharge locations to wells over periods of weeks to a few years (Gamlin et al., 2001; Clark et al., 2004, 2005). These tracer experiments are well suited for following artificial recharge plumes below spreading ponds where large volumes of water ( $>10^5$  m<sup>3</sup>) need to be tagged.

The results from a decade of studies are discussed here. Most of the work has been conducted near spreading ponds in Southern California where in excess of ten billion liters of surface water is typically recharged each year.

### Principle of Tracer Methods

Environmental tracers that have been commonly used to investigate flow characteristics of relatively young groundwater include dating with tritium and its daughter <sup>3</sup>He (T/<sup>3</sup>He) (e.g., Schlosser et al. 1989; Cook and Solomon 1997; Solomon and Cook, 2000). This method uses pre-existing signatures to determine groundwater apparent ages (up to 50 years) that typically have uncertainties of  $\pm 2$  years. Tracer ages are defined as the length of time that the groundwater has been isolated from the soil atmosphere (i.e., isolated from the water table). While it is convenient to interpret the groundwater age using the piston flow model, dispersion and mixing of different aged flow lines can complicate the age interpretation (Tompson et al., 1999; Bethke and Johnson, 2002; Weissmann et al., 2002; McDermott et al., 2007).

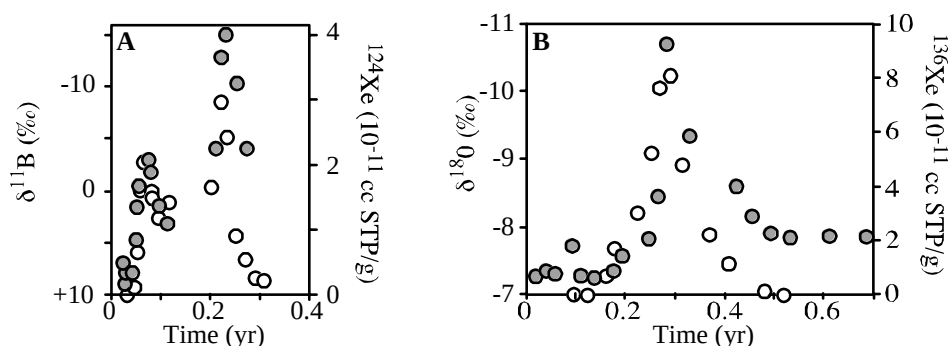
The T/<sup>3</sup>He and other environmental dating methods are unable to resolve groundwater ages between 0 and 2 years because of analytical uncertainties. The most common approach to evaluate flow over these short time scales, which are critical for the management of MAR sites, is to perform deliberate tracer experiments. This type of experiment has been used to study advection and dispersion in groundwater aquifers for more than two decades (e.g., Mackay et al., 1986, LeBlanc et al., 1991). Commonly used tracers include trace ions (e.g. Br, Li), fluorescent dyes (e.g. Rhodamine), and conservative gases (e.g. He, SF<sub>6</sub>). The experimental design consists of introducing a tracer into a volume of water, which is subsequently injected into the groundwater flow field and measured at selected wells down gradient.

The scale of deliberate tracer experiments is defined by the quantity of water that can be "tagged" and the signal to noise ratio of the tracer being used. The two factors that often limit their size are (1) the cost of tracer and (2) the ability to introduce a sufficient amount of tracer without significantly changing the buoyancy of the tagged water. The cost of the tracer can be a particular problem when large volumes of water ( $>10^5$  m<sup>3</sup>) need to be tagged, as is often the case near MAR operations. Gamlin et al. (2001), Clark et al. (2004, 2005), Avisar and Clark (2005) and Quast et al. (2006) have recently demonstrated that gas tracers (sulfur hexafluoride, SF<sub>6</sub>, and noble gas isotopes) and isotopes of boron (as <sup>10</sup>B enriched borate) can be used economically

to tag large volumes of water, in excess of 106 m<sup>3</sup>. Furthermore, they demonstrated groundwater flow over spatial lengths greater than 4 km and temporal periods greater than 4 years can be evaluated using these tracers. Thus, these tracers are ideal for tracing the movement of artificially recharge water at MAR sites.

Two time scales of transport can be determined from the tracer breakthrough curves at down gradient wells. The initial arrival time of tracer represents the fastest flow paths between the well and MAR site and is the most relevant time scale when considering contaminate transport. This is especially true in heterogeneous aquifers (with preferential flow paths) and when examining breakthrough curves at production wells. At monitoring wells with narrow screened intervals, the arrival time of the center of mass represents the mean travel time of the tracer patch and can be used to calculate groundwater velocities. However, because production wells draw water in from the entire length of screen, multiple flow lines with presumably different travel times are sampled. Therefore, these wells typically mix groundwater that contains tracer with unknown amounts of water that has not been tagged. Because the fraction of water tagged with tracer can change with time, velocities determined with either the peak concentration or center of mass do not necessarily represent the mean groundwater velocity at production wells.

The costs of the analytical system and measurements are significantly less for SF<sub>6</sub> than for the noble gas isotopes and boron isotopes. Thus, its application as a deliberate tracer at MAR sites is more cost-effective. SF<sub>6</sub> is a synthetic gas used primarily in the electrical industry as a gas insulator and has been used as a tracer in the atmosphere and natural waters for more than two decades. It is an ideal tracer for the following reasons. (1) SF<sub>6</sub> is nontoxic (Lester and Greenberg, 1950) and permission has been granted to use it as a tracer in potable aquifers in California and South Australia. (2) Background concentrations of SF<sub>6</sub> in natural waters are extremely low (<0.2 pmol/l; 1 pmol = 10<sup>-12</sup> mole). (3) It can be measured precisely in water (±5% or better) over a concentration range of 6 orders of magnitude using a gas chromatograph equipped with an electron capture detector (Wanninkhof et al., 1991; Clark et al., 2004). (4) Laboratory scale experiments have shown that breakthrough curves of SF<sub>6</sub> and Br are identical in saturated porous media, which contains either high contents of organic material or clays, demonstrating that SF<sub>6</sub> is not retarded (Wilson and Mackay, 1993, 1996). SF<sub>6</sub> differs from ionic and dye tracers in that it is a gas and is lost from solution across the air - water interface to the atmosphere. Thus, it is important to monitor the SF<sub>6</sub> concentrations in the recharge water to determine the amount of tracer lost due to gas exchange. This is especially true if the artificial recharge is taking place in a shallow river or spreading pond. Insufficient monitoring of spatial variability can lead to erroneous interpretations of travel times (e.g., Clark et al. 2004; Avisar and Clark, 2005). Furthermore, laboratory column experiments have shown that its transport is slowed (retarded) when trapped air is contained within the porous media (Fry et al., 1995; Vulava et al., 2002).



**FIGURE 1. Comparison of solute and gas tracer breakthrough curves from tracer experiments conducted at (A) the Montebello Forebay redrawn after (Quast et al., 2006) and (B) Orange County (after Clark et al., 2004). In (B), the  $\delta^{18}\text{O}$  composition of the native groundwater ranges between  $-7\text{‰}$  and  $-8\text{‰}$ . Open circle = Xe isotope; Filled circle = conservative isotope.**

Boron isotopes provide a cost effective method for examining the movement of recharge water near MAR sites with a conservative ion tracer. Natural boron has two stable isotopes,  $^{10}\text{B}$  (relative abundance of 19.8%) and  $^{11}\text{B}$  (80.2%). Boric acid enriched to 96%  $^{10}\text{B}$  is commercially available through Eagle-Picher Industries, Inc. Recently, Quast et al. (2006) demonstrated the potential of using  $^{10}\text{B}$  enriched borate as a tracer at MAR spreading basin. They showed that on the order of one kilogram of enriched boric acid is needed to sufficiently alter the B isotope ratio of recharge water, even when it contains a large percentage of B-rich reclaimed wastewater. Using their methodology, the isotope ratio change will be on the order of 100‰, while the borate concentration change will be less than 5 ppb. The non-personnel costs of enriched boron tracer experiments are approximately an order of magnitude greater than SF<sub>6</sub>.

## RESULTS AND DISCUSSION

Gas tracers have been used during at least nine experiments conducted at seven different MAR sites. The results have been summarized by Gamlin et al. (2001), Clark (2002), Fram et al. (2003); Clark et al. (2004, 2005), Heilweil et al. (2004), Avisar and Clark (2005), Avisar et al., (2006), Quast et al. (2006) and McDermott et al. (2007). Five of the experiments have been conducted from surface spreading sites at large southern California MAR locations, which have a long history of use. Three have been conducted from pilot (new) surface spreading sites. The final experiment was conducted at an ASR well (Fram et al., 2003). Results from these experiments will be briefly summarized and used as a guide for determining when gas tracer experiments

are a good approach for determining travel times and hydraulic connections at MAR sites and when they are not.

### **Tracer Release**

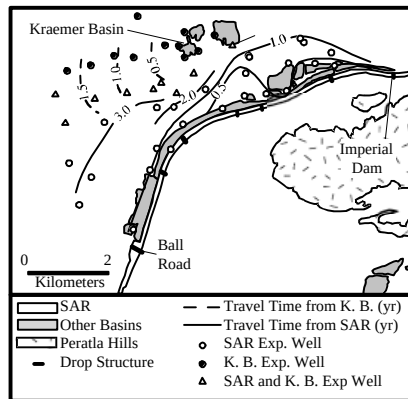
Surveys of recharge ponds and rivers following the introduction of tracer have shown significant spatial variation in concentration. This is due to a number of factors including the type of release-instantaneous vs. continuous, the recharge rate relative to mixing time within the basin, and air-water gas exchange. Along rivers, variations can be more than an order of magnitude (Gamlin et al., 2001, Clark 2002). Smaller variations are typically observed within spreading basin, though concentrations still range by a factor of two to four (Avisar and Clark, 2005; Quast et al., 2006, McDermott et al., 2007). These studies have shown that careful monitoring of the surface water prior to recharge is necessary.

### **Breakthrough Curves**

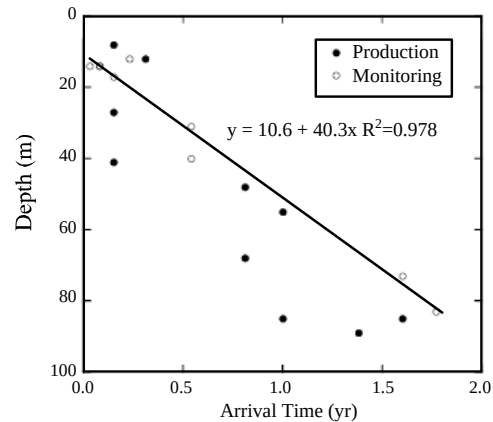
The application of gas tracer experiments at surface recharge sites such as spreading ponds creates a potential problem-the loss of the tracer from the percolating water to soil air trapped above the water table. During experiments conducted at three MAR locations in California (Orange County, Montebello Forebay, and El Rio), gas tracers have been observed at down gradient wells (Clark et al., 2004, 2005; Avisar and Clark, 2005; Avisar et al., 2006; Quast et al., 2006; McDermott et al., 2007). In fact, during two experiments in which both gas and solute tracer were used, the breakthrough curves look remarkably similar (Fig. 1). These results indicate that gas loss and retardation during percolation is minimal. Both of these phenomena are expected when trapped air is present because gases will partition strongly into the immobile air volume, retarding their transport and reducing their concentrations in the infiltrating water. These effects have been documented previously in laboratory column experiments (Fry et al., 1995; Donaldson et al., 1997; Vulava et al., 2002).

In the cases where gas tracers are transported across the water table by the infiltrating water, valuable hydrologic information can be inferred from their breakthrough curves. In an area with an abundance of wells, such as near the Orange County and Montebello Forebay MAR sites, the migration of the recharge water can be imaged in both the vertical and horizontal directions. During the summer of 1998, two tracer experiments were performed from two different recharge areas within the Orange County, CA, MAR facility: the Santa Ana River (SAR) and Kraemer Basin (Fig. 2).





**FIGURE 2. Tracer arrival times from Kraemer Basin and the SAR. In the SAR, recharge occurs between the Imperial Dam and Ball Road. Redrawn after Clark et al. (2004).**



**FIGURE 3. First arrival of tracer to wells within 150 m of spreading ponds and the Montebello Forebay. The depth is the distance between the pond bottom and top of screen. Redrawn after McDermott et al. (2007).**

These gas tracer experiments revealed a rapid flow system in the upper 100 to 200 m of the aquifer near the recharge facilities. Mean horizontal velocities calculated with the arrival times of the center of mass at monitoring wells were on the order of 1 km yr<sup>-1</sup>, indicating that the tracer plumes were flowing through material with a hydraulic conductivity on the order of 100 m day<sup>-1</sup> (Clark et al., 2004). Given the very heterogeneous hydro-stratigraphy of the aquifer (Tompson et al. 1999), it is likely that the tracer plumes were moving preferentially through the most conductive layers. Because different spreading areas were tagged with different tracers, the experiment revealed that the recharge plumes were filling distinct areas of the aquifer and little mixing was occurring between plumes (Fig. 2).

The vertical transport of tracer was successfully examined at the Montebello Forebay Spreading Area, Los Angeles County, CA, where the focus was on the travel times to both monitoring and production wells within 150 m (500 ft) of spreading ponds. Production wells this close are not in compliance with the draft water reuse rules of the Department of Health Services of the State of California. At both monitoring and production wells, there was a strong relationship between tracer arrival time and depth to the top of perforation (Fig. 3) and no correlation between horizontal distance and travel time (Avisar et al., 2006; McDermott et al., 2007). Although pumping rate was a poor predictor, travel times to production wells screened at the same depth as monitoring wells were always the same or faster. This

indicates that while production is not the primary factor, it does have an influence on travel time. Depth may be the most important factor influencing travel time because the deeper the well perforation is, the more likely is it for the screen to be situated below layers with low hydraulic conductivity.

### Trapped Air

Based on the success of the Orange County tracer experiments, Clark et al., (2004) hypothesized that the bulk of the tracer and therefore recharge water passed through saturated columns, which exist below these spreading areas, and were transported across the water table without interacting with soil air. This finding contradicts conventional wisdom, observations of dissolved noble gases (Clark et al., 2005; Clark and Hudson, 2006), and gas tracer experiments performed at three other locations: Littlerock Creek (Clark, 2002), the Big Bear pilot basins (Clark and Avisar, 2005) and the Sandy Hollow Reservoir experimental basin (Heilweil et al., 2004).

At the three sites listed above, gas tracer experiments have failed because the tracer have not been observed at down gradient wells. While the non-detections could be explained hydrologically-e.g., the down gradient wells were not hydrologically connected with the recharge sites, a more likely explanation is gas loss and retardation by trapped air. This is especially true for the Sandy Hollow Reservoir experiment because Heilweil et al. (2004) employed a dual tracer method using He gas and bromide. The latter tracer was detected at down gradient wells while the former was not.

Below spreading sites, surface water infiltrates into the ground and flows through a vadose or unsaturated zone prior to reaching the water table. While conservative solute tracers will pass through this zone without loss or slowing due to sorption processes (i.e., retardation), relatively insoluble gases will partition between the percolating water and in-mobile soil air due to gas transfer. The net effect of this gas transfer is the slower the movement of the gas relative to the water through the unsaturated zone or retardation (e.g., Gupta et al., 1994; Fry et al., 1995; Vulava et al., 2002).

Fry et al. (1995) proposed that the retardation factor,  $R$ , for a gas flowing through porous media containing trapped air can be calculated from the dimensionless Henry's Law constant,  $H$ , and the ratio between the air,  $V_{air}$  and water,  $V_{water}$  pore volumes:

$$R = 1 + H (V_{air} / V_{water}) \quad (EQ 1)$$

In formulating this relationship, Fry et al. (1995) assumed that gas transfer was sufficiently fast to maintain the solubility equilibrium. Equation (1) accurately predicts retardation factors determined during laboratory column experiments at

low  $\tilde{V}_{\text{air}}V_{\text{water}}$  ratios, but under predicts retardation at high  $\tilde{V}_{\text{air}}V_{\text{water}}$  ratios (Fry et al., 1995; Vulava et al., 2002). Kinetic models that included diffusion controlled gas transfer do a much better job matching laboratory experiments at high  $\tilde{V}_{\text{air}}V_{\text{water}}$  ratios (Donaldson et al., 1997; Vulava et al., 2002). The effect of non-instantaneous gas exchange include earlier breakthrough of gases (first arrival), longer tails, and non-equilibrium between the percolating water and soil air.

The failure of these three experiments was most likely due to gas loss into trapped air during infiltration. Thus, these results clearly show that trapped air can exist below recharge sites at sufficient quantities to effect gas tracer experiments. It is important to note that all of the experiments that show gas loss took place during the first wetting at new spreading sites while the five experiments that show little gas loss or retardation took place at MAR facilities with long histories (more than three decades) of use.

## CONCLUSIONS

Why some gas tracer experiments indicate flow beneath spreading basins is saturated and others indicate unsaturated is unknown. Clark et al. (2005) hypothesized that recharge rate and history of basin use are the dominant factors determining the conditions below spreading basins. Saturated flow will occur beneath basins with long service histories because the volume of the initial trapped air will decrease with time due to its dissolution by excess air formation (e.g., Clark and Hudson, 2006). Given enough time, portions of the vadose zone will become saturated because the initial soil air will have completely dissolved. If this is the case, then travel time information from gas tracer experiments is only reliable at relatively old MAR operations. At relatively young or pilot spreading areas alternative tracers such as boron isotopes need to be considered.

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# IMPACTS OF A LONG-TERM SHUTTING DOWN ON THE AQUIFER RECHARGE MANAGEMENT: CASE OF THE AQUIFER RECHARGE OF GENEVA, SWITZERLAND

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## ABSTRACT

The drinking water supply for the Geneva area comes partly from a large trans-boundary aquifer called «Genevois aquifer». Due to severe overpumping during the 70's, an artificial recharge (A.R.) system has been carried out in 1980 to recharge water from one of the main river crossing Geneva (the Arve river) into the Genevois aquifer in order to maintain the groundwater level and to enable continued use of the groundwater. During 25 years of exploitation, the A.R. system of Geneva has brought over 225 Mm<sup>3</sup> of treated water into the Genevois aquifer. The building of a new dam on the Arve river started in September 2005 on the site where the water-intake structure for the A.R. system is located. In order to take advantage of this building site to displace and improve the water-catchment, it has been necessary to shut down the A.R. plant for 15 months. This project had an important impact on the A.R. and drinking water supply management. A good balance between pumping and recharging has demonstrated its efficiency. This strategy proved to be effective in some situation with the help of the natural aquifer recharge.

## KEYWORDS

Aquifer recharge management, case study, Geneva, natural recharge, shutdown

## INTRODUCTION

The drinking water supply for the Geneva area comes partly (80%) from Lake Geneva and partly (20%) from a large aquifer called « Genevois aquifer ». As a trans-boundary aquifer, the Genevois aquifer is exploited for drinking water supply by 10 wells on the Swiss side and 5 wells on the French side (fig. 1). On the French side, there are several operators, but the total withdrawal is usually between 2 and 3 Mm<sup>3</sup>/year. On the Swiss side, the operator is SIG (Services Industriels de Genève), in charge of 2 water pumping plants in lake Geneva as well. A special agreement between the State of Geneva and French communities regulates the use of water as well as the price.

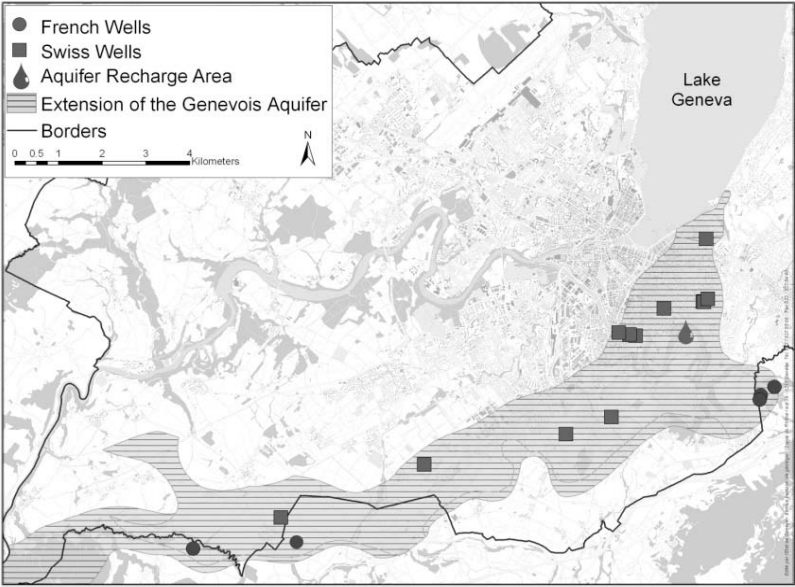


FIGURE 1. Location of the Genevois aquifer.

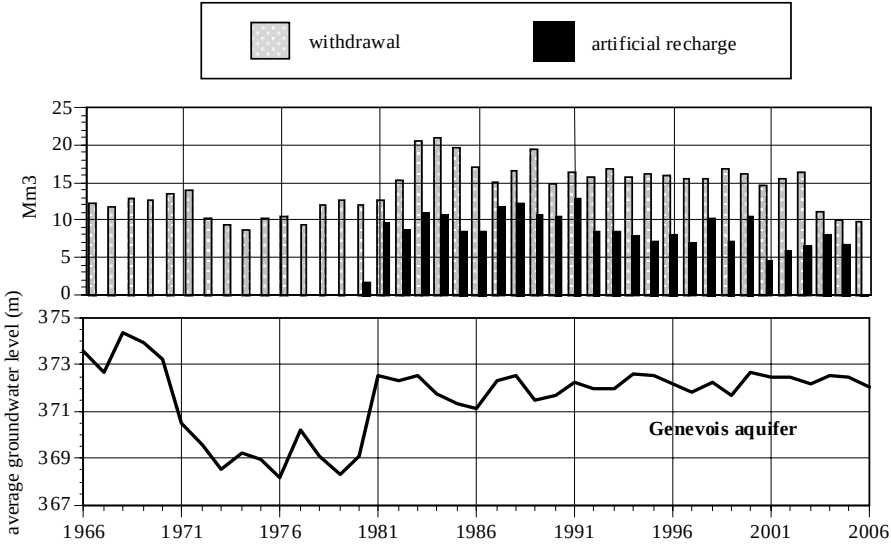
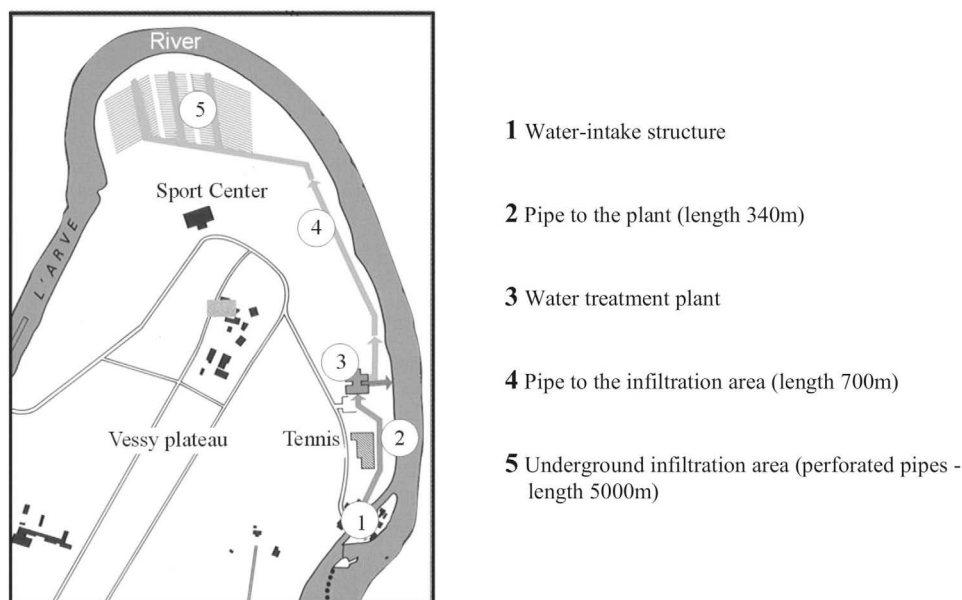


FIGURE 2. Impacts of withdrawals and artificial recharge on the aquifer level.

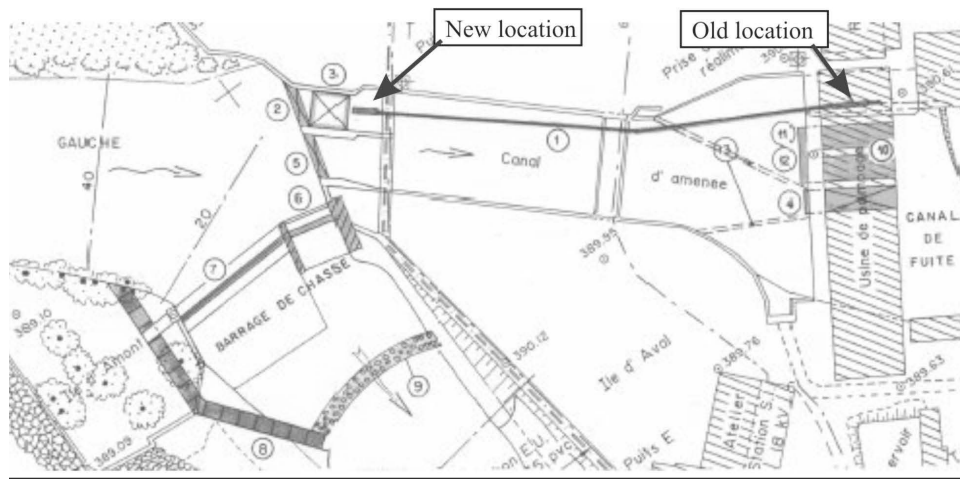
During the 70's, overpumping lowered the groundwater level by an average of 7 meters or more (fig. 2). In some places a drop of up to 12 m lead to the closing of some wells. Consequently, an artificial recharge (A.R.) system was established to recharge water from the Arve river into the Genevois aquifer in order to maintain the groundwater level and enable continued use of the groundwater. This A.R. plant, located near the Arve river (fig. 3), started its activity in 1980. During 25 years of exploitation, the A.R. system of Geneva has brought over 225 Mm<sup>3</sup> of treated water into the Genevois aquifer. The A.R. system also brought positive results to the global quality of water resource, especially its hardness and nitrate contents (de los Cobos, 2002).



**FIGURE 3. Description of the artificial recharge area.**

In September 2005, the building of a new dam on the Arve river started on the site where the water-intake structure for the A.R. system is located (fig. 4). The water-intake maintenance becoming too tricky, it has been decided to take advantage of this project to displace and improve the water-intake structure. In order to minimize the impacts of pumping on the water level and to prepare the Genevois aquifer to be used without A.R. during the 15 month- period, it has been necessary to plan groundwater management more than a year in advance. The main fear was to face another scorching summer in 2006, as experienced in 2003, and to be deprived of the assistance provided by A.R.





**FIGURE 4. Project of the displacement of the water-intake structure for A.R. in the new dam site (~80m).**

### **Project of Groundwater Management Strategy**

In order to prepare the aquifer to be used without A.R. and to keep a high piezometric level, the strategy was to start the 15 month-work in September 2005 to spend only one summer period (summer 2006) without A.R., and to restart the A.R. plant during a winter period. This is due to the fact that, winter is known to be one of the best period regarding the turbidity of the Arve river on one hand. On the other hand, restarting in winter gives the opportunity to save several months for the reconstruction of groundwater storage before summer comes (2007).

### **Preparation of the Aquifer Before the Shutdown**

In order to “fill” the aquifer by A.R. to its highest level by September 2005, two main strategies were adopted:

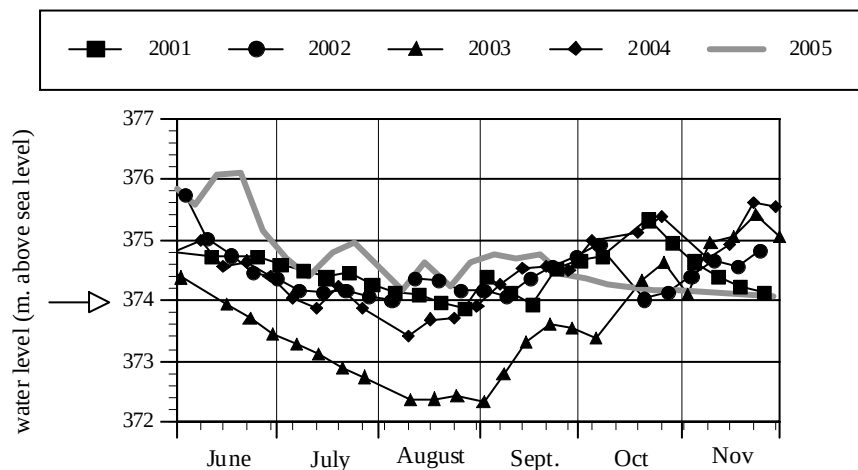
- 1) Maximum artificial recharge up to September
- 2) Minimum withdrawals during summertime

There were two unknown important elements: 1) the ability of A.R. to work without shutdowns due to high level of turbidity of Arve river. The reason is the plant is not working when turbidity exceeds 150 NTU, which can occur during floods in the river, after summer storms or when the thaw sets in, particularly during spring time. 2) the

summer heat which increases the water consumption (global withdrawals are highest during the summertime).

September is the starting point from which an optimistic or a pessimistic vision for the groundwater management during the 15 month-period without A.R. can be given. If the summertime is good enough in terms of no extra pumping and artificial recharge, the piezometric level can be maintained at a sufficient high level to overcome the year without A.R.. Otherwise, a crisis management has to be applied to manage a severely depleted aquifer.

In order to get a realistic vision of different piezometric situations, the piezometric level monitoring in the recharge area has been compared to the piezometric levels recorded during past years for the same period (June to November). Various target values of groundwater levels have been chosen: a piezometric level at 374m height has been considered as a minimum level to have a good groundwater reserve to complete the year without A.R. (fig. 5).



**FIGURE 5. Piezometric levels in the A.R. area during summer & fall seasons for the last 4 years before the shutdown of the A.R.**

### Pumping Management During the Shutdown

During the 15 month-period without A.R., it has been decided to have a restrictive pumping, limited to 10'000 m<sup>3</sup>/day. In order to reach this goal, and avoid a crisis situation, SIG decided to unplug a district of Geneva from water supply coming from the aquifer and plug it to the lake-water net during the dam construction period.

Despite the good water level conditions in September 2005, we were still fearing another scorching summer in 2006.

### **Re-starting of A.R. Plant Once the Dam Work is Completed**

The completion of construction has been scheduled for January 2007. Once the A.R. is operational, the strategy consists in "filling" the aquifer as much as possible. However, past experience has shown that artificial recharge in the underground perforated pipes after an inactive period can create "gaseous embolisms". These embolisms come from the reduction of flow capacity caused by trapped air in the pipes. Therefore, it is necessary to recharge gradually.

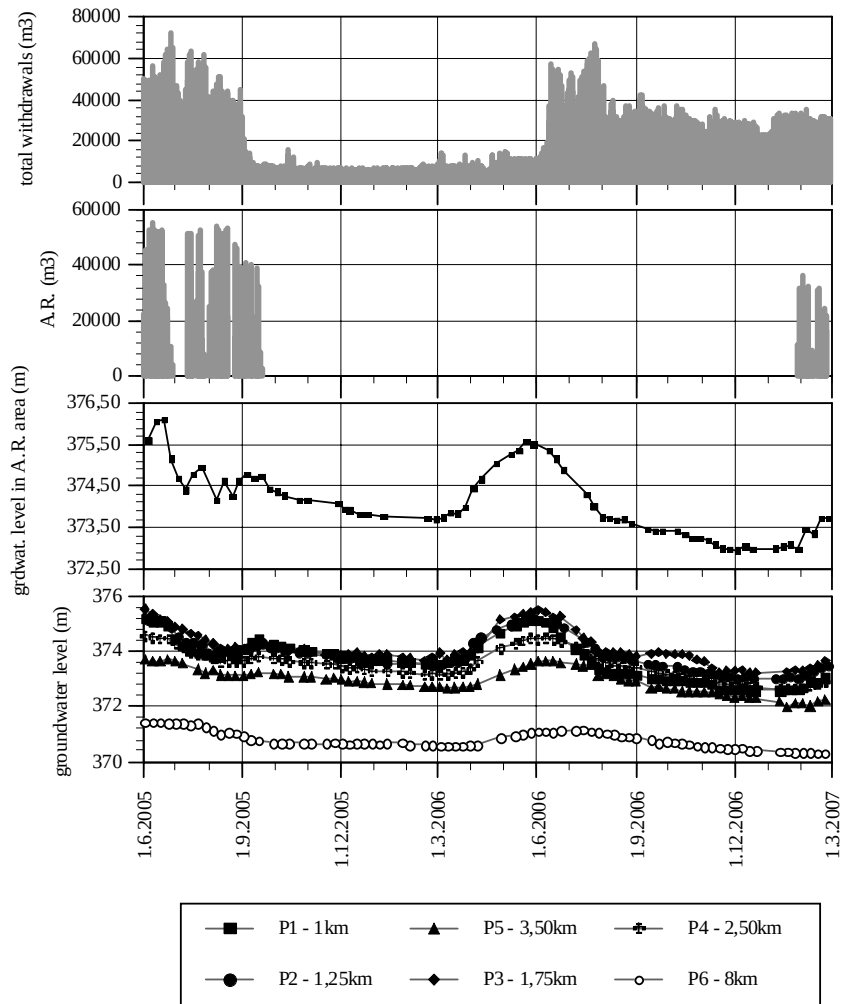
### **Description of Facts**

#### **Preparation of the Aquifer Before the Shutdown.**

Between June and September 2005, the withdrawals reach an average of 40'000 m<sup>3</sup>/day, with some peaks at 60'000 and even 70'000 m<sup>3</sup>/day. The A.R. is working almost every day in June with more than 50'000 m<sup>3</sup>/day (fig. 6). The total recharge for this month is more than 800'000 m<sup>3</sup>. The groundwater level in the recharge area is at 376 m above sea level (a.s.l.), which is quite high and better than projected.

During July and August 2005, frequent stops of the A.R. caused the piezometric level to decrease. In the beginning of September, pumping started to decrease to 10'000 m<sup>3</sup>/day and A.R. continued to work with an injection of 40'000 m<sup>3</sup>/day up to 19<sup>th</sup> of September, last day before shutting down the plant for 15 months. The piezometric level is approximately at 374.75 m a.s.l., higher than the target goal of the project.

Total withdrawal for this period starting from June 1st to September 1st is more than 4 Mm<sup>3</sup> and the total A.R. is around 2 Mm<sup>3</sup>, 50% of the pumping.



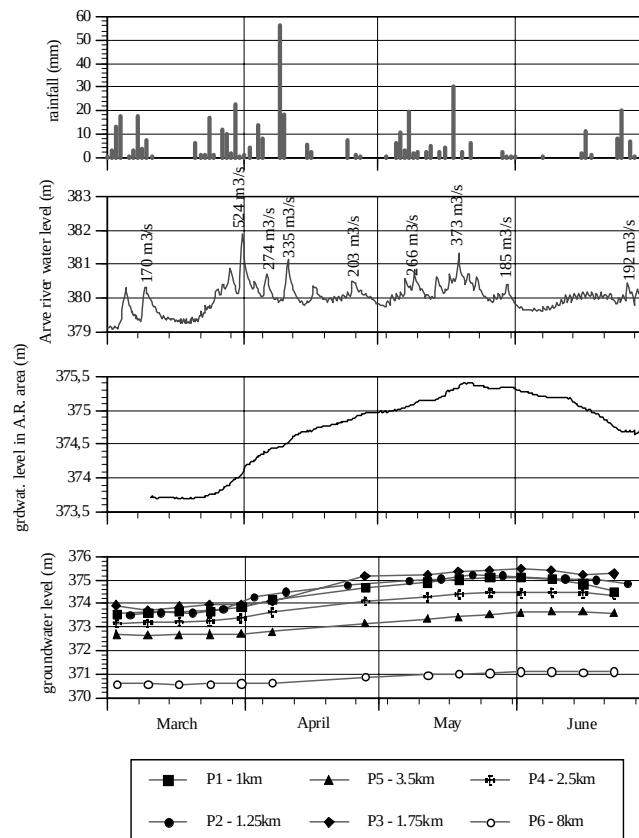
**FIGURE 6. Groundwater levels in the monitor wells between June 2005 to March 2007 including the A.R. shutdown period.**

### Groundwater Variations During the Shutdown

Without artificial recharge, the pumping is limited from 8 to 10'000 m<sup>3</sup>/day. The water level is slightly decreasing between September 2005 to Spring 2006. On March 1<sup>st</sup>, 2006, the piezometric level in the recharge area is approximately 373.75 m a.s.l., more than 2 m lower than the value recorded in June 2005. At that time, the summer 2006 is expected with some fear even if meteorological records for spring 2006 showed a normal year with temperatures close to the average.

### The Chance Factor

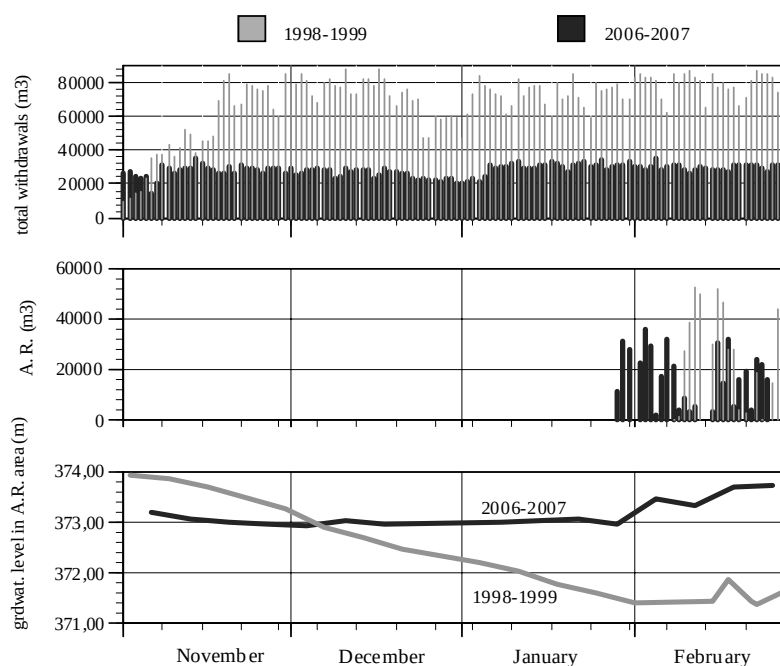
During March 2006, the piezometric monitoring showed the start of an important increase of the groundwater level (fig.7). This slope continued on every piezometer of the aquifer up to beginning of June. The A.R. being shutdown, the only explanation to this increase was a natural recharge by the Arve river. Actually, an analysis of the river yield shows an important succession of various floods between end of March and end of May 2006. This was due to several rainfalls in the catchment area of the Arve river. The natural recharge was very impressive and was estimated to give a recharge volume exceeding 2 Mm<sup>3</sup>. Groundwater levels increased more than 2 meters in the recharge area and 1m in some wells located 4km far from the river. This was the first time where a natural recharge of groundwater has been monitored so well without any interference due to A.R.



**FIGURE 7. Impact of the natural recharge on the aquifer level during spring 2006.**

This phenomenon has been a great chance with respect to the groundwater management and to the uncertain summer weather conditions in 2006. July 2006 has been the hottest month of July in the meteorological history of Switzerland (average of 23.7 C against 18.3 C for the 10 year-normal average). The total pumping during this month was more than 1,5 Mm<sup>3</sup>, almost the draw-off recorded in 2003. Nevertheless, due to the good management of withdrawals and monitoring of the water level, and due to the exceptional natural recharge during the spring time, the water level has been stabilized in its decrease.

At the end of 2006, the piezometric level is at 373 m height in the recharge area. This is around 2 m lower than usually for this period compared to previous years, but higher than the worst level recorded since 1981 (1998-1999) (fig. 8). The A.R. restarted end January 2007 with some technical stops. The total recharge volume for the first month (end of January - end of February) has been around 450'000 m<sup>3</sup>.



**FIGURE 8. Comparison between 98-99 and 06-07 groundwater levels, with different A.R. and total withdrawals conditions.**

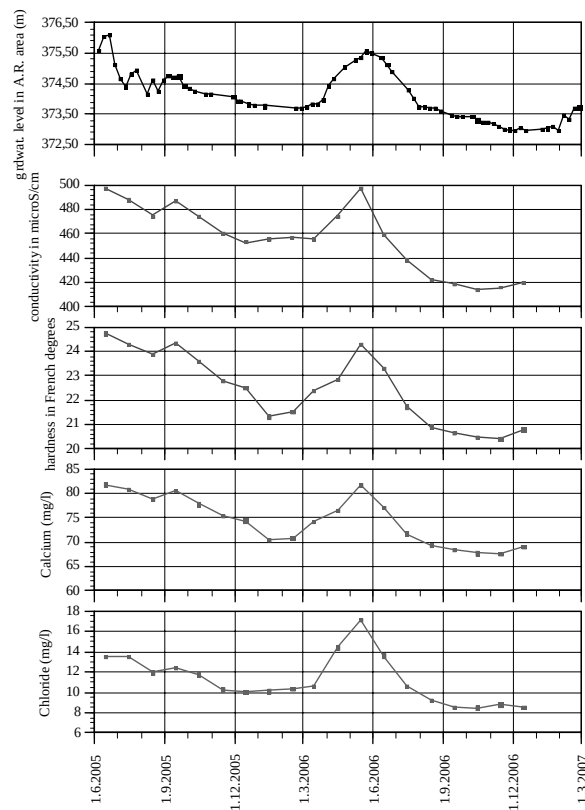
### Impacts of the Long-term Shutdown on Water Quality

The monitoring of water quality in the recharge area and in some wells during this period has not shown significant impacts except a very clear correlation between the

recharge of the aquifer and the variation of concentration for some parameters. The first results show a decrease in concentration (water conductivity, hardness, calcium and chloride) when the recharge is not working (fig. 9). The significant natural recharge between April and May 2006 has had an important impact on the increase of these chemical parameters, particularly for chloride.

The correlation is working at the opposite of what is expected; water of the river should have less concentration than the groundwater. In fact it seems that the Arve river has a very high quality variation depending on the flow. In the flooding period, the quality of the groundwater shows the same concentration than during A.R. period. It is even higher as shown by the chloride concentration. This is demonstrated by the variations of concentration (~10%) for various parameters at the same time.

These considerations are currently under investigation.



**FIGURE 9. Impact of the natural recharge on the quality of the aquifer.**

## CONCLUSIONS

The combination of a slight pumping during the entire duration of A.R. shutdown and a sound scheduling have been proven successful in the management of groundwater. Of course one should take into account the chance factor as an amazing and unexpected natural recharge of the aquifer has been monitored during spring 2006. But even without the natural recharge recorded during this time, with the same pumping conditions, the groundwater level would not have reached the 371m mark in December 2006. This level is considered as a limit from which the groundwater storage is started. The big advantage of the artificial recharge is that it can recharge the aquifer anytime, except during floods of the river. Artificial recharge and natural recharge are very complementary, as they bring very interesting solutions to the groundwater management of the Genevois aquifer.

These works have also shown that 30 years ago, when the designers of the artificial recharge in Geneva started to work on this project, the studies on the river-aquifer relationship and on the recharge area have been very well done. Almost a quarter of a Century later the A.R. of Geneva has demonstrated that it is a great tool with a fantastic potential in the groundwater management.

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## AQUIFER STORAGE AND RECOVERY IN A COMPARTMENTALIZED BASALT AQUIFER SYSTEM, WALLA WALLA, WASHINGTON

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### ABSTRACT

The City of Walla Walla developed an Aquifer Storage and Recovery (ASR) project in response to local overdraft of groundwater supplies and to provide an alternative source of water when their primary supply (Mill Creek) is unavailable. The City operates two ASR wells, each with a recharge capacity of about 1,600 gpm (100 L/s) and a pumping capacity of about 2,800 gpm (175 L/s). The ASR system is developed in a confined basalt aquifer system consisting of thin (less than 10 m) interflow zones within thick dense basalt flows (Columbia River Basalt). The basalt aquifer has been faulted and folded, offsetting interflow zones and creating compartments that are hydraulically isolated, forming excellent reservoirs for water storage. To evaluate changes in the groundwater system during ASR operations, and the overall effects of recharge and pumping of the basalt aquifer system, a MODFLOW groundwater flow model was developed. The model simulates the complex interaction between individual compartments within the local basalt aquifer, regional groundwater flow in the basalt aquifer, and with the overlying unconsolidated aquitard and aquifer. ASR provides an overall net benefit to the groundwater and surface water resources.

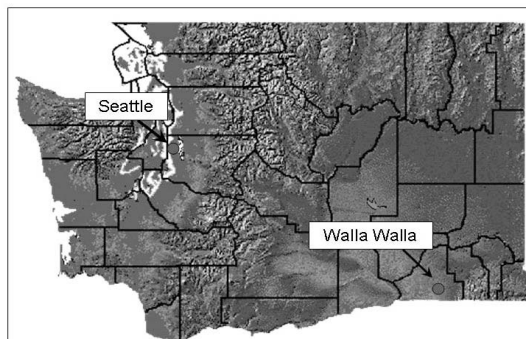
### KEYWORDS

ASR, basalt aquifer, faulting, groundwater flow model

### INTRODUCTION

The City of Walla Walla (City) is located in southeastern Washington, USA, about 40 km east of the confluence of the Walla Walla and Columbia Rivers (Figure 1). The City has a population of about 30,000, and the neighboring City of College Place has a population of about 8,000. The primary source of drinking water for the City is a diversion on Mill Creek. The diverted water is treated at an ozonation plant and sent to the distribution system. The capacity of the ozonation plant is 24 million gallons per day (MGD; 1,050 L/s). The City also operates seven deep wells completed in the Columbia River Basalt (CRB) aquifer system to provide water during peak demand periods or when the surface water source is limited because of low flows or is unavailable because of high turbidity. Other area water purveyors, including College Place, also obtain their water from the CRB aquifer.

As the Walla Walla area grew, the demand on the basalt aquifer system increased, resulting in long-term groundwater level declines of 50 to 200 feet in portions of the aquifer between the 1930's and 1990's. At the same time, concerns about low streamflows during the summer months, coinciding with peak demand for water, have lead to restrictions on surface water withdrawals. The City was also concerned that a fire in their watershed could take their surface water source offline for up to ten years because of significantly increased turbidity and other water quality problems. To increase the reliability of it's water supply, the City decided to evaluate Aquifer Storage and Recovery (ASR) in 1999. ASR in Walla Walla was initially evaluated by the U.S. Geological Survey and the City in the 1950's (Price, 1960). The initial evaluations indicated ASR was feasible; however, no further work was done until the late 1990's



**FIGURE 1. Location Map.**

## **CITY OF WALLA WALLA ASR PROGRAM**

The City of Walla Walla started evaluation of ASR in 1999. The City converted Well No. 1, an existing supply well completed in the CRB aquifer, to an ASR well. Well No. 1 has a recharge capacity of about 1,300 gpm (80 L/s) and a pumping capacity of about 2,800 gpm (175 L/s). A series of pilot ASR cycle tests ranging in duration from about three to about 160 days were performed in Well No. 1 to evaluate the aquifer response to recharge, changes in water quality, and storage of water (CH<sub>2</sub>M-Hill, 1999, 2000). The pilot testing indicated ASR was feasible, and Well No. 1 was placed into service as an ASR well in 1999.

Based on the success of ASR at Well No. 1, the City converted Well No. 6 to an ASR well in 2003 (Golder 2004). Well No. 6 has a recharge capacity of 1,600 gpm (100 L/s) and a pumping capacity of 2,800 gpm (175 L/s). The City completed a 135-day ASR pilot test in Well No. 6, including recharge for 49 days, a 43-day storage period, and a 43-day pumping period. During the pilot test, about 290 million liters were recharged to the basalt aquifer, and 521 million liters were pumped from the basalt aquifer.

Treated water from the City's Mill Creek ozonation plant is used as recharge water in both wells. Recharge water is typically available from about October through May, when excess surface water is available after meeting demand and instream flows and turbidity is acceptable for recharge (greater than one NTU). Pilot testing and ongoing

water quality monitoring indicate that dispersion is the primary process affecting the concentrations of the recovered water. Major ion water quality during the Well No. 6 pilot test is summarized in Table 1. .

**TABLE 1. Key Water Quality Parameters.**

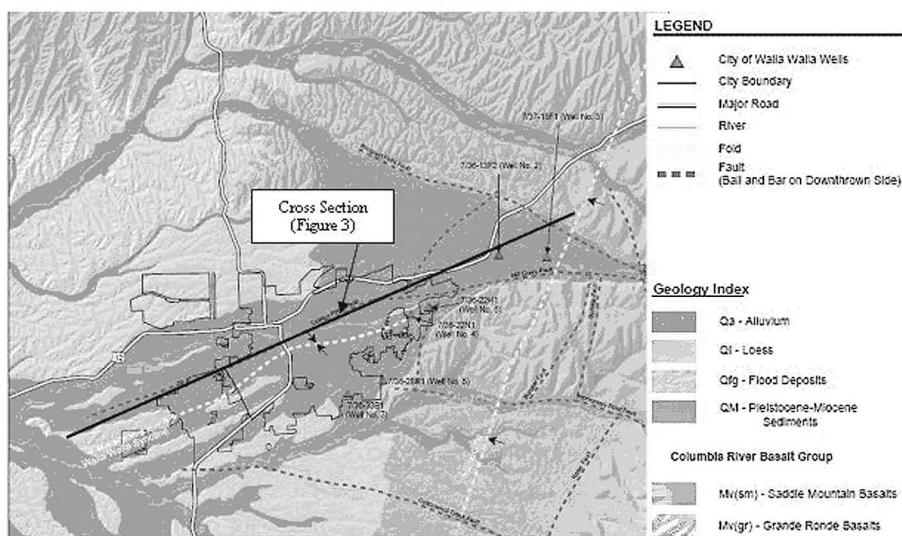
| <b>Parameter</b>                | <b>Recharge Water<br/>(mg/L)</b> | <b>Native<br/>Groundwater<br/>(mg/L)</b> | <b>Recovered Water<br/>(mg/L)</b> |
|---------------------------------|----------------------------------|------------------------------------------|-----------------------------------|
| Calcium                         | 5.6 to 6.4                       | 17.5                                     | 9.9 to 15                         |
| Magnesium                       | 1.9 to 2.5                       | 7.3                                      | 3.8 to 5.8                        |
| Sodium                          | 2                                | 24                                       | 10 to 19                          |
| Silica                          | 27 to 29                         | No data                                  | 46 to 59                          |
| Iron                            | <0.1                             | <0.05                                    | <0.1                              |
| Manganese                       | <0.001 to 0.0007                 | 0.011                                    | 0.0041 to 0.0055                  |
| Chloride                        | 1.3 to 1.5                       | 2.3                                      | 1.5 to 1.6                        |
| Alkalinity as CaCO <sub>3</sub> | 25 to 30                         | 120                                      | 56 to 110                         |
| Sulfate                         | 1.0 to 1.2                       | 5.5                                      | 2.1 to 4                          |

The City's ASR projects were developed prior to development of formal ASR regulatory guidelines. In 2003, the State of Washington promulgated regulations for ASR projects. The City submitted an ASR reservoir application to the State for storage of 7.7 million m<sup>3</sup> of water from the operation of Wells No. 1 and 6, with the potential for future expansion of the system. Since the City started the ASR project in 1999 with Well No. 1, over 3.8 billion gallons (14.4 billion liters) of water has been recharged to the CRB aquifer. To date, annual recharge has ranged from about 226 million liters in 1999 to over 3.5 billion liters in 2004.

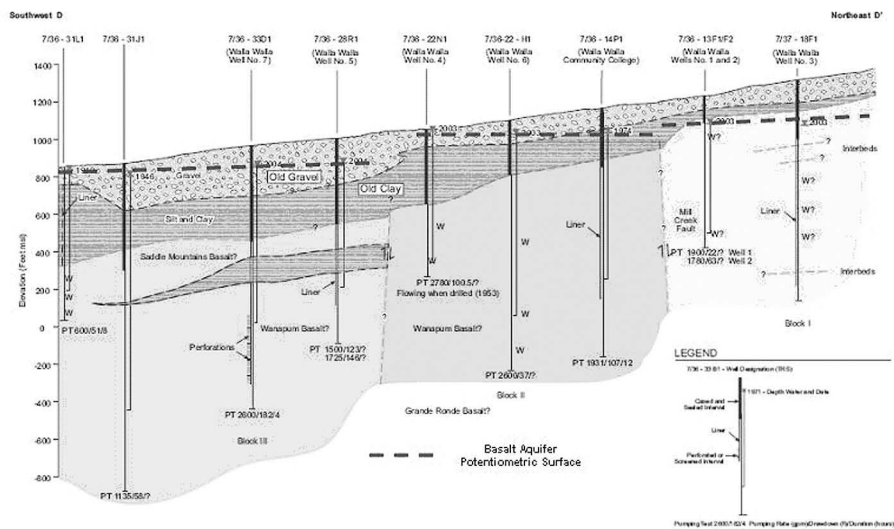
The City also operates five other supply wells completed in the basalt aquifer. These wells have not been converted to ASR wells.

## **HYDROGEOLOGIC SETTING**

The geologic units in the vicinity of Walla Walla consist of unconsolidated alluvial, wind-blown, and flood-deposited materials overlying bedrock consisting of Columbia River Basalt flows (Figure 2). The unconsolidated alluvial deposits form an unconfined aquifer. The unconfined aquifer is underlain by a thick section of silt and clay that forms an aquitard. The basalt bedrock faulted and folded into compartmentalized aquifers that are hydraulically separate (Figure 3). A generalized stratigraphic column and the principal hydrogeologic units are summarized in Table 2.



**FIGURE 2. Geologic map of Walla Walla area showing faults that separate basalt aquifer into hydraulically separate compartments. Wells No. 1 and 6 are ASR wells.**



**FIGURE 3. Geologic cross section, showing faults that separate basalt aquifer into hydraulically separate compartments.**

**TABLE 2. Stratigraphic Column and Hydrogeologic Units.**

| Age                         | Geologic Unit                                          | Lithology                     | Hydrogeologic Unit                                               | Thickness (m)                         |
|-----------------------------|--------------------------------------------------------|-------------------------------|------------------------------------------------------------------|---------------------------------------|
| Unconsolidated Materials    |                                                        |                               |                                                                  |                                       |
| Pleistocene -Recent         | Alluvial Deposits                                      | Gravel, Silt, Sand Uncemented | Sand and Gravel Aquifer                                          | up to 3?                              |
|                             | Flood Deposits (including Touchet Beds)                | Silt to Sand                  | Aquifer or Unsaturated                                           | up to 30?                             |
|                             | Loess                                                  | Silt to Fine Sand             | Unsaturated or Saturated, Low Permeability                       | up to 3                               |
|                             | Palouse Formation                                      | Silt                          |                                                                  | up to 30                              |
| Pliocene-Pleistocene        | Old Gravel                                             | Gravel and Sand, cemented     | Sand and Gravel Aquifer                                          | up to 90                              |
| Miocene-Pliocene            | Old Clay                                               | Silt and Clay                 | Confining Unit                                                   | up to 150                             |
| Columbia River Basalt Group |                                                        |                               |                                                                  |                                       |
| Miocene                     | Saddle Mountains Basalt                                |                               |                                                                  | Greater than 600 m in total thickness |
|                             | Lower Monumental Member                                | Basalt Flows                  | Interflow zones aquifers. Dense flow interiors act as aquitards. |                                       |
|                             | Ice Harbor Member                                      |                               |                                                                  |                                       |
|                             | Umatilla Member                                        |                               |                                                                  |                                       |
|                             | Saddle Mountains - Wanapum Interbed                    | Silt, Sand                    | Aquitard                                                         |                                       |
|                             | Wanapum Basalt                                         |                               |                                                                  |                                       |
|                             | Frenchman Springs Member                               | Basalt Flows                  | Interflow zones aquifers. Dense flow interiors act as aquitards. |                                       |
|                             | Eckler Mountain Member                                 |                               |                                                                  |                                       |
|                             | Wanapum -Grande Ronde Interbed                         | Silt, Sand                    | Confining Unit                                                   |                                       |
|                             | Grande Ronde Basalt                                    | Basalt Flows                  | Aquifers and Aquitards                                           |                                       |
|                             | Imnaha Basalt                                          | Basalt Flows                  | Unknown                                                          |                                       |
| Older Geologic Units        |                                                        |                               |                                                                  |                                       |
| Pre-Miocene                 | Not exposed at ground surface or intersected in wells. |                               |                                                                  |                                       |

## Hydrogeologic Units

### Sand and Gravel Aquifer

The sand and gravel aquifer occurs in the upper 30 to 100 m of the unconsolidated materials overlying the basalt bedrock. Numerous domestic and irrigation wells are completed in the sand and gravel aquifer. Some wells are also completed in sand or gravel lenses within the lower silt and clay portion of the unconsolidated materials that forms an aquitard underlying the sand and gravel aquifer.

Groundwater in the sand and gravel aquifer occurs under unconfined to semi-confined conditions. The depth to water in the sand and gravel aquifer generally ranges between about 2 m below ground to 15 to 20 m below ground. Using specific capacity data, the transmissivity of the sand and gravel aquifer was estimated to range from 3 m<sup>2</sup>/d to 9,300 m<sup>2</sup>/d, with a mean and median transmissivity of 195 m<sup>2</sup>/d and 72 m<sup>2</sup>/d, respectively. The storativity of the sand and gravel aquifer is estimated to be about 0.1 to 0.25. Recharge to the sand and gravel aquifer is about 40 to 80 mm/year (Golder, 2006).

### Basalt Aquifer System

The Walla Walla ASR system is developed in the Columbia River Basalt aquifer system, the most productive aquifer in the Walla Walla area. The basalt aquifer system consists of the Saddle Mountains, Wanapum, and Grand Ronde basalt and sedimentary interbeds between these units (Table 2). Within the basalt aquifer system, there are individual basalt flows and interflow zones that individually may act as aquitards or aquifers.

Groundwater in the basalt aquifer system generally occurs in the contacts between basalt flows (interflow zones), which are about 5 to 10 m thick. These contacts are typically broken or fractured, vesicular, or may contain interbedded sedimentary materials such as sand or silt, or pillow basalts. Newcomb (1965) estimated that the water bearing zones represent about ten percent of the total thickness of the basalt aquifer system. Little groundwater occurs in the basalt flow interiors because the flow interiors are generally dense and unfractured. Groundwater occurs in the basalt aquifer system under confined conditions. The basalt aquifer is primarily recharged in the Blue Mountains, east of Walla Walla. Groundwater flow is from the Blue Mountains west to the Columbia River (Figure 4), but is locally disrupted by folds or faults within the basalt aquifer.

The basalt aquifer system is bounded by lateral changes in interflow thickness and permeability, regional faults or folds, and regional groundwater discharge areas such as the Snake and Columbia Rivers. Within these regional aquifer boundaries, the basalt aquifer system is locally bounded by structural features such as faults, folds,

or dikes that separate the basalt aquifer into discrete blocks or compartments. In the Walla Walla area, faults or dikes have separated the basalt aquifer system into several discrete blocks or compartments (Figure 3). Adjacent blocks have a limited hydraulic connection from each other by either the development of low-permeability fault gouge or rubble, precipitation of secondary mineralization such as silica, carbonates, or iron and manganese hydroxides in the fault zone, or the offset of permeable interflow zones against low-permeability flow interiors. Because of these low-permeability boundaries, groundwater levels in one block do not appear to respond to pumping or ASR operations in adjacent blocks.



**FIGURE 4. Groundwater Flow in Basalt Aquifer System.**

#### Inter-Aquifer Flow

Groundwater levels in shallow monitoring wells completed in the sand and gravel aquifer adjacent to the City's ASR wells are above the potentiometric surface in the basalt aquifer under static (non-recharge) conditions, indicating a downward component of hydraulic gradient at these locations, indicating some recharge of the basalt aquifer occurs from leakage from the sand and gravel aquifer. During recharge, groundwater elevations in Well No. 1 remain below the groundwater elevation in the shallow aquifer. Thus, there is a downward component of hydraulic gradient at all times at Well No. 1.

At Well No. 6, the groundwater elevation in the basalt aquifer is above the ground surface and above the groundwater elevation in the sand and gravel aquifer during recharge. Thus, in the immediate vicinity of the well, there is an upward component of hydraulic gradient during recharge. The groundwater elevation in the immediate vicinity of the well drops below the groundwater elevation in the sand and gravel aquifer as soon as recharge stops. In other areas of the basalt aquifer near Well No. 6, flowing artesian conditions develop during recharge at Well No. 6. Thus, there is an upward component of hydraulic gradient in these areas.

## GROUNDWATER FLOW MODELING

A groundwater flow model was developed using the USGS MODFLOW finite-difference code (McDonald and Harbough, 1988) to evaluate the pumping and storage capacity of the basalt aquifer system and changes to the groundwater system balance as a result of ASR operations.

### Model Setup

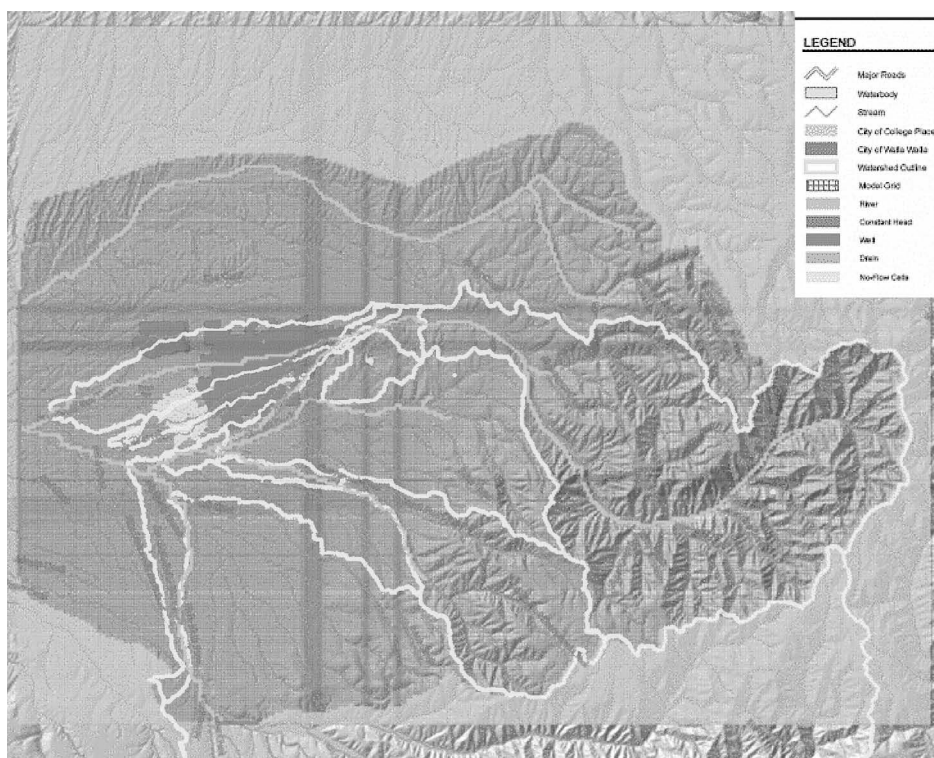
A five-layer finite difference model grid was set up over the model area. The model grid area is 35 by 45 km. The model layer elevations were interpreted from Digital Elevation Models (top of layer 1) and geologic information obtained from well logs. The model grid cells range from 150 by 150 meters to about 30 by 30 meters near the City wells. The model layers are:

- **Layers 1 and 2** represent the sand and gravel aquifer on the west side of the model and the upper part of the basalt aquifer system on the east side of the model, where basalt is exposed at the ground surface in the Blue Mountains. The sand and gravel portion of the aquifer is subdivided into two zones. There is a zone of higher hydraulic conductivity around the stream and river channels. The combined thickness of layers 1 and 2 ranges from less than 12 m to about 210 m in the sand and gravel portion of the model to about 15 m to over 600 m on the eastern portion of the model where the basalt outcrops;
- **Layer 3** represents the silt and clay aquitard on the western side of the model domain, and the basalt aquifer system on the eastern part of the model domain (Blue Mountains). The silt and clay aquitard ranges from less than 8 m thick to about 150 m thick; and
- **Layers 4 and 5** represent the basalt aquifer. Layers 4 and 5 are each 300 m thick, for a combined thickness of 600 m. All of the basalt aquifer wells that are represented in the model are included in layer 4.

Constant head boundaries were used to represent regional groundwater inflow to the model, and outflow from the model. The major rivers and streams in the model domain were represented using MODFLOW's river package, and springs discharging



from the sand and gravel aquifer were represented using the drain package. The model grid and boundaries (layer 1) are shown on Figure 5. Faults in the basalt aquifer were represented using the horizontal flow barrier package in MODFLOW. The conductances of the model boundaries were adjusted during the model calibration process.



**FIGURE 4. Model grid and boundaries (layer 1).Calibration and Sensitivity**

The model was calibrated under steady-state conditions to groundwater elevations and the vertical component of hydraulic gradient in a monitoring well network maintained by the City, to estimated stream baseflows, and to spring discharge. The model was calibrated under transient conditions to pumping tests and ASR pilot tests completed in the City's wells.

In the area of the City monitoring well network, the model calibration is good, with a residual mean of 2.5 feet and a residual standards deviation of 29 feet. The model simulated the observed groundwater elevations, vertical component of hydraulic gradient, baseflows, and spring discharge. The model also simulated pumping tests and ASR pilot tests under transient conditions.

The sensitivity analysis indicates the model is most sensitive to the downstream constant head boundary. The model is also sensitive to changes in the hydraulic conductivity of the sand and gravel aquifer and the streambed conductance in some of the river reaches. The model was moderately sensitive to changes in the vertical hydraulic conductivity of the aquitard, changes in horizontal hydraulic conductivity of parts of the basalt aquifer, the conductance of selected river reaches, and the conductance of some of the fault barriers.

## Model Simulations and Results

The model was used to evaluate the effects of aquifer storage and recovery operations on the groundwater flow system and water balance, especially changes in storage in both the basalt and sand and gravel aquifers and changes in induced recharge from rivers and intercepted groundwater discharge to rivers.

### Aquifer Storage and Recovery

The model was used to compare the water balance and groundwater system under non-ASR and ASR operations for yearly operations. The non-ASR condition (Scenario 1) assumed that five City wells were pumped (245 L/s) for five months (assuming surface water was used for the remaining seven months). The ASR condition (Scenario 2) assumed that Wells No. 1 and 6 were recharged (180 L/s) for a period of seven months followed by five months of pumping (245 L/s). In this scenario, annual recharge is virtually equivalent to annual pumping (water balance neutral).

The water balances for Blocks I and II of basalt aquifer (layers 4 and 5) for the two scenarios are summarized on Table 3.

The mass balance for Blocks I and II of basalt aquifer for a typical year of pumping (Scenario 1) indicates the following:

- At the end of one year, after about 3.29 million m<sup>3</sup> of water are pumped from the City's wells in the basalt aquifer. During pumping about 0.47 million m<sup>3</sup> (15%) of water is derived from storage in the basalt aquifer. Leakage from other compartments of the basalt aquifer is 1.07 million m<sup>3</sup> (or about 35%). Leakage from the overlying unconsolidated materials is 1.76 million m<sup>3</sup> (or about 50%). Of the 1.76 million m<sup>3</sup> leakage that occurs during pumping, about 0.22 million m<sup>3</sup> (16 L/s) is induced recharge (streamflow loss) or interception of groundwater discharge to surface water.

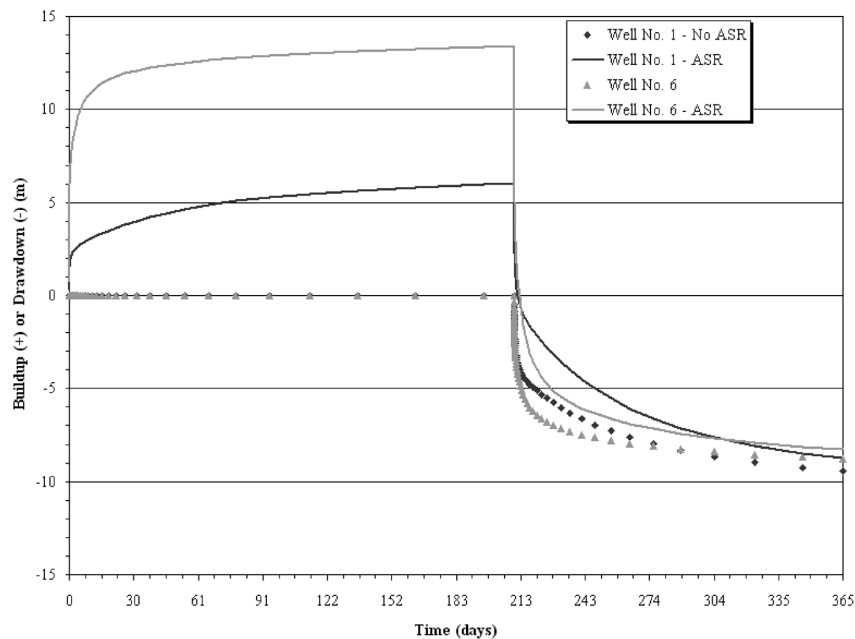
The mass balance for Blocks I and II the basalt aquifer for a typical year of ASR (Scenario 2) indicates the following:

TABLE 1. 3: Water Balance Basalt Aquifer Blocks I and II

| Mass Balance Component                                | Scenario 1 Pumping Only                         |                                         |                                                 |                                         | Scenario 2 Aquifer Storage and Recovery         |                                         |                                                 |                                         |
|-------------------------------------------------------|-------------------------------------------------|-----------------------------------------|-------------------------------------------------|-----------------------------------------|-------------------------------------------------|-----------------------------------------|-------------------------------------------------|-----------------------------------------|
|                                                       | End of Non-Pumping Period (7 months)            |                                         | End of Pumping Period (12 months)               |                                         | End of Recharge Period (7 months)               |                                         | End of Pumping Period (12 months)               |                                         |
|                                                       | Outflow (-) or Inflow (+) ( $m^3 \times 10^6$ ) | Change in Storage ( $m^3 \times 10^6$ ) | Outflow (-) or Inflow (+) ( $m^3 \times 10^6$ ) | Change in Storage ( $m^3 \times 10^6$ ) | Outflow (-) or Inflow (+) ( $m^3 \times 10^6$ ) | Change in Storage ( $m^3 \times 10^6$ ) | Outflow (-) or Inflow (+) ( $m^3 \times 10^6$ ) | Change in Storage ( $m^3 \times 10^6$ ) |
| <b>Pumping</b>                                        | -                                               | -                                       | -3.29                                           | -                                       | -                                               | -                                       | -3.29                                           | -                                       |
| <b>Well Recharge</b>                                  | -                                               | -                                       | -                                               | -                                       | +3.32                                           | -                                       | +3.32                                           | -                                       |
| <b>Change in Storage - Basalt</b>                     | -                                               | -                                       | -                                               | -0.47                                   | -                                               | +0.42                                   | -                                               | -0.3                                    |
| <b>Leakage To/ From Unconsolidated Materials*</b>     | -                                               | -                                       | -1.76                                           | -                                       | +1.70                                           | -                                       | -0.02                                           | -                                       |
| <b>Leakage To/ From Other Parts of Basalt Aquifer</b> | -                                               | -                                       | -1.07                                           | -                                       | +1.21                                           | -                                       | +0.35                                           | -                                       |

\*includes changes in streamflow.

- Over the 7-month recharge period, about 3.32 million m<sup>3</sup> of water has been recharged to the basalt aquifer. About 0.42 million m<sup>3</sup> (15%) of water goes into storage in the basalt aquifer. There is leakage from the basalt aquifer to the overlying unconsolidated materials of 1.70 million m<sup>3</sup> (or about 50%), and leakage into other compartments of the basalt aquifer of 1.21 million m<sup>3</sup> (or about 35%) because of increased water levels in the basalt. At the end of the recharge period water levels are higher in the sand and gravel and basalt aquifers.
- At the end of one year (after seven months recharge and five months ASR recovery (pumping) period), about 3.29 million m<sup>3</sup> of water has been pumped from the City's wells in the basalt aquifer—thus a net annualized input to the groundwater system of 0.03 million m<sup>3</sup>. Storage in the basalt has been reduced by about 0.30 million m<sup>3</sup>. There is leakage from the unconsolidated materials to Blocks I and II of 0.02 million m<sup>3</sup>. There is leakage from Blocks I and II to other basalt blocks amounting to 0.35 million m<sup>3</sup>. Over a full year of ASR operations, the average streamflow *gain* is 0.21 million m<sup>3</sup> (7 L/s). ASR also results in a minor decrease in drawdown of about 0.5 m in the ASR wells at the end of the pumping period (Figure 6).



**FIGURE 6. Hydrographs for ASR Wells.**

## CONCLUSIONS

The City of Walla Walla has developed an ASR system in a basalt aquifer. There are several favorable attributes of the City's water system and the hydrogeologic conditions that lead to the successful development of an ASR system. The City constructed an ozonation plant to treat their surface water source, resulting in the availability of high-quality recharge water. The City has excess surface water available in winter and spring, allowing recharge to the basalt aquifer for up to seven months each year. The City's two ASR wells were converted from existing wells. The ASR modifications included significant upgrades to the wells, increasing the reliability of the City's water supply.

The City's ASR wells are completed in a confined, transmissive basalt aquifer. Faulting has resulted in the development of hydraulically-separate aquifer blocks or compartments. The faults effectively contain the recharge water, limiting seepage to other parts of the basalt aquifer. Dense, low-permeability basalt flow interiors and the aquitard overlying the basalt aquifer limit leakage to the overlying sand and gravel aquifer.

A groundwater flow model was developed to evaluate aquifer storage and recovery. The modeling work indicates that operation of the City's ASR system provides a net environmental benefit to the groundwater and surface water system. ASR results in an increase in storage in parts of the basalt aquifer and unconsolidated materials, less leakage from the unconsolidated materials and other parts of the basalt aquifer during pumping, and an increase in discharge to surface water.

Compared to the typical annual pumping scenario, an annual ASR cycle of recharge and pumping results in the following annual average changes:

1. A net increase to the groundwater and surface water system of 3.32 million  $m^3$  ( $3.29+0.03$ ); This results in the following benefits/impacts on an annual average basis:
  - A increase in storage within basalt Blocks I and II of 0.17 million  $m^3$ ;
  - An increase in storage in other basalt blocks of 1.42 million  $m^3$  ( $1.07+0.35$ ); and
  - An increase in leakage to the unconsolidated materials of 1.74 million  $m^3$  ( $1.76-0.02$ ). Of this leakage, there is an increase in discharge to surface water of 0.43 million  $m^3$  (14 L/s). The remainder of the water enters storage.

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## AQUIFER STORAGE AND RECOVERY AT THE SOUTH FLORIDA WATER MANAGEMENT DISTRICT – CERP AND BEYOND”

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### KEYWORDS

Aquifer Storage and Recovery, Florida, Comprehensive Everglades Restoration Plan (CERP), Lake Okeechobee and Estuary Recovery (LOER)

### ABSTRACT

The South Florida Water Management District (SFWMD) is responsible for overall water management across a huge area of southern Florida including the cities of Miami, Fort Myers, Naples, West Palm Beach, and southern Orlando. As part of their overall responsibilities, the SFWMD also is charged with water supply planning for a rapidly growing population of over five million persons. Due to the rapidly growing population, water supply demands have soared over the last decade. The SFWMD has funded several Aquifer Storage and Recovery (ASR) projects – through an “Alternate Water Supply Grant” program - for utilities to conduct water banking and managing peak flows during high demand periods.

Concurrently, the Comprehensive Everglades Restoration Plan (CERP) is moving ahead at a rapid clip. The CERP includes a proposal to construct 330 ASR wells to provide a flexible water storage and supply for Lake Okeechobee and other urbanized areas as well as important natural areas. The CERP ASR Program includes construction of several ASR pilot projects and a south-Florida-wide ASR Regional Study, including the development of a regional groundwater model. The schedule for the eventual construction of the CERP ASR system will likely take place through 2020.

During the past few years, south Florida has also experienced numerous hurricanes and wet weather periods that have caused massive degradation of the water quality in Lake Okeechobee and necessitated the release of huge quantities of water into the estuary systems, damaging the ecosystems there. In October 2005, Governor Jeb Bush initiated the Lake Okeechobee and Estuary Recovery (LOER) plan to help

restore the ecological health of Lake Okeechobee and the St. Lucie and Caloosahatchee Estuaries. The SFWMD is one of the key agencies charged with implementing this plan, which includes construction and operation of new ASR systems in tandem with other measures.

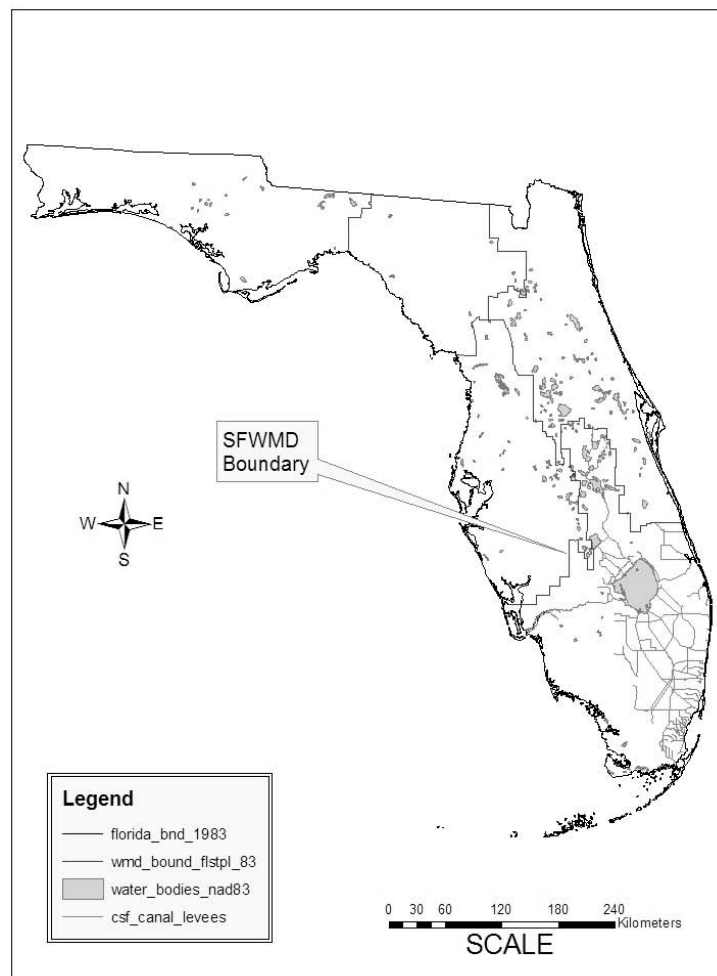
This paper will endeavor to discuss the ASR programs along with the progress each one has made. It will also focus upon the unique technical challenges of each program and why cutting edge science has been required to solve some of the more complex issues. Lastly, the paper will discuss and comment upon ongoing and future policy issues that have been identified.

## **THE SOUTH FLORIDA WATER MANAGEMENT DISTRICT**

SFWMD's boundaries extend from central Florida to Lake Okeechobee, and from coast to coast, from Fort Myers to Fort Pierce, south through the Everglades to the Florida Keys. The South Florida Water Management District (SFWMD) manages and protects regional water resources by balancing and improving water quality, flood control, natural systems and water supply. Figure 1 depicts the political boundary and area of operations for SFWMD.

The "seeds" for the creation of the SFWMD were planted in the late 1940s, by flood and drought. Today, the agency's responsibilities include regional flood control, water supply and water quality protection as well as ecosystem restoration. The region's subtropical extremes of hurricane, flood and drought – combined with efforts to safely populate this "new frontier" – led the U.S. Congress to adopt legislation creating the Central and Southern Florida Flood Control Project (C&SF) in 1948. In 1949, the Florida Legislature created the Central and Southern Florida Flood Control District, the predecessor to the South Florida Water Management District (SFWMD), to manage the huge project being designed and built by the U.S. Army Corps of Engineers. In 1972, with the Florida Water Resources Act ([Chapter 373](#)), the state created five water management districts, with expanded responsibilities for regional water resource management and environmental protection. In 1976, voters approved a constitutional amendment giving the districts the authority to levy property taxes to help fund these activities. Today, the South Florida Water Management District operates and maintains approximately 1,800 miles of canals and levees, 25 major pumping stations and about 200 larger and 2,000 smaller water control structures. Major canals are shown on Figure 1.





**FIGURE 1. South Florida Water Management District Boundary.**

## HYDROGEOLOGY OF SOUTH FLORIDA

The unconfined Surficial Aquifer System (SAS) extends from land surface (top of the water table) to a depth of between 100 to 200 feet bls. It consists of Holocene and Pliocene-Pleistocene aged sands and shelly limestone. Below the SAS lies an "Intermediate Confining Unit" extending to between 700 to 900 feet bls (Miller, 1997). The Peace River and Arcadia Formations of the Miocene-Pliocene aged Hawthorn

Group act as confining units separating the FAS from the SAS. The Hawthorn Group sediments consist predominately of soft, non-indurated detrital clays, silts and poorly to moderately indurated mudstones/wackestone with minor amounts of sand and shell (Scott, 1988).

Beneath the Hawthorn Group lies the Floridan Aquifer System – the prime zone for ASR in south Florida. The Floridan aquifer system consists of a series of Tertiary limestone and dolostone units. The system includes sediments of the lower Arcadia Formation, Suwannee and Ocala Limestones, Avon Park Formation, and the Oldsmar Formation. The top of the FAS, as defined by the Southeastern Geological Society AdHoc Committee on Florida Hydrostratigraphic Unit Definition (1986) coincides with the top of a vertically continuous permeable early Miocene to Oligocene-aged carbonate sequence. The upper Floridan aquifer consists of thin, high permeability water-bearing horizons interspersed with thick, low permeability units of early Miocene to middle-Eocene age sediments, including the basal Arcadia Formation, Suwannee and Ocala Limestones, and the Avon Park Formation. Generally, two predominant permeable zones exist within the upper FAS with the uppermost typically lies between 700 and 1,200 feet bls. The most transmissive part usually occurs near the top, coincident with an unconformity at the top of the Oligocene or Eocene aged formations. The Paleocene-age Cedar Keys Formation – consisting of low permeability evaporitic gypsum and anhydrite - forms the lower boundary of the FAS. The hydrogeology of south Florida is generally compatible with the application of ASR technology – particularly in the “deep” artesian Floridan Aquifer System (FAS). Multiple ASR projects are currently operating in south Florida with most utilizing some portion of the FAS as a storage repository.

## **ASR DEVELOPMENT IN SOUTH FLORIDA**

ASR technology has been explored at over 30 sites in southern Florida beginning the 1980's. Initial ASR concepts were developed in 1983 at the Peace River site located outside of the SFWMD boundary south of Tampa, Florida (Pyne, 1996). Additional ASR studies in Florida at the Cocoa Beach and Hialeah ASR sites were conducted by the United States Geological Survey (USGS) and the U.S. Army Corps of Engineers (USACE) from 1983 to 1986 (Merritt, 1983; Merritt, 1986). Most of the ASR systems have been installed by public water supply utilities, with the intent of banking fully treated drinking water, reclaimed water, raw groundwater or partially treated surface water. Most of these facilities were constructed in the 1990's. Some sites have been in operation for at least 10 years including the Boynton Beach and Port Malabar/Palm Bay (Brown, 2005) projects. Other sites are relatively new including multiple sites located in southwest Florida such as Olga. ASR projects in Florida have been successful for the most part but some projects have performed sub-optimally (Reese, 2002). Isolated ASR problems documented in Florida include salt-water upconing, excessive mixing between ambient brackish water and recharged freshwater, and in-

situ geochemical reactions related to release of pyrite (Brown, 2005; Price and Pichler, 2006).

## **SFWMD ALTERNATIVE WATER SUPPLY PROGRAM**

The SFWMD has been cooperatively developing, funding and implementing “alternative” water supply projects – including ASR - for 10 years under the various state initiatives and grant programs. Between 1997 and 2004, these projects have created more than 340 million gallons of water per day through development of saline water sources, reclaimed water and ASR. In the past decade, the SFWMD has funded 28 ASR-related construction projects across seven counties at over a dozen public utility water facilities. The projects have involved construction of exploratory wells, ASR wells and associated monitoring wells. The projects involved reclaimed water, potable water and treated surface water. Many of the projects have expanded to become multi-well facilities and undergone multiple cycles of recharge, storage and recovery.

## **THE COMPREHENSIVE EVERGLADES RESTORATION PROGRAM (CERP)**

The Comprehensive Everglades Restoration Plan (CERP) is a framework and guide to restore, protect, and preserve the water resources of central and southern Florida.

- The Plan has been described as the world's largest ecosystem restoration effort and includes more than 60 major components.
- Because the region's environment and economy are integrally linked, the Plan provides important economic benefits.
- Thus, the Plan will result in a sustainable south Florida by restoring the ecosystem, ensuring clean and reliable water supplies, and providing flood protection.

The Comprehensive Everglades Restoration Plan provides a framework and guide to restore, protect, and preserve the water resources of central and southern Florida, including the Everglades. It covers 16 counties over an 18,000-square-mile area, and revolves around a major re-plumbing of the entire C&SF Project. The C&SF Project provides water supply, flood protection, water management and other benefits to south Florida. For close to 50 years, the C&SF Project has performed its authorized functions well. However, the project has had unintended adverse effects on the unique and diverse environment that constitutes south Florida ecosystems, including the Everglades and Florida Bay. The Water Resources Development Acts in 1992 and 1996 provided the U.S. Army Corps of Engineers with the authority to re-evaluate the performance and impacts of the C&SF Project and to recommend

improvements and or modifications to the project in order to restore the south Florida ecosystem and to provide for other water resource needs. The resulting Comprehensive Plan (USACE & SFWMD, 1999) was designed to capture, store and redistribute fresh water previously lost to tide and to regulate the quality, quantity, timing and distribution of water flows.

The Plan was approved in the Water Resources Development Act of 2000. It includes more than 60 elements, will take more than 30 years to construct, and will cost an estimated \$7.8 billion. The major Plan components include reservoirs, stormwater treatment areas, and approximately 333 ASR wells located throughout the study area. The CERP ASR components are expected to take surplus freshwater (collected in surface reservoirs during wet periods), treat it as required and then store it deep underground in the FAS for subsequent recovery during dry periods. Implementation of regional ASR technology within the south Florida basin is anticipated to significantly increase freshwater storage capacities. It is also forecasted to help minimize damaging high-volume freshwater releases from surface water sources (such as Lake Okeechobee) to the estuaries. During dry periods, water recovered from ASR wells would be utilized to augment surface water supplies and maintain the surface water levels and/or flows within Lake Okeechobee, the St. Lucie and Caloosahatchee Rivers and associated canals throughout southern Florida. Of the 68 project components recommended in the CERP, several recommended inclusion of ASR wells. The six project locations authorized in the plan are:

- Lake Okeechobee ASR (component GG) – 200 wells
- Caloosahatchee (C-43) Reservoir ASR (component D) – 44 wells
- L-8 Basin ASR (components K and GGG) – 10 wells
- C-51 Canal ASR (component LL) – 34 wells
- Central Palm Beach County (Agricultural Reserve) Reservoir ASR (component VV) – 15 wells
- Site 1/Hillsboro ASR (component M) – 30 wells

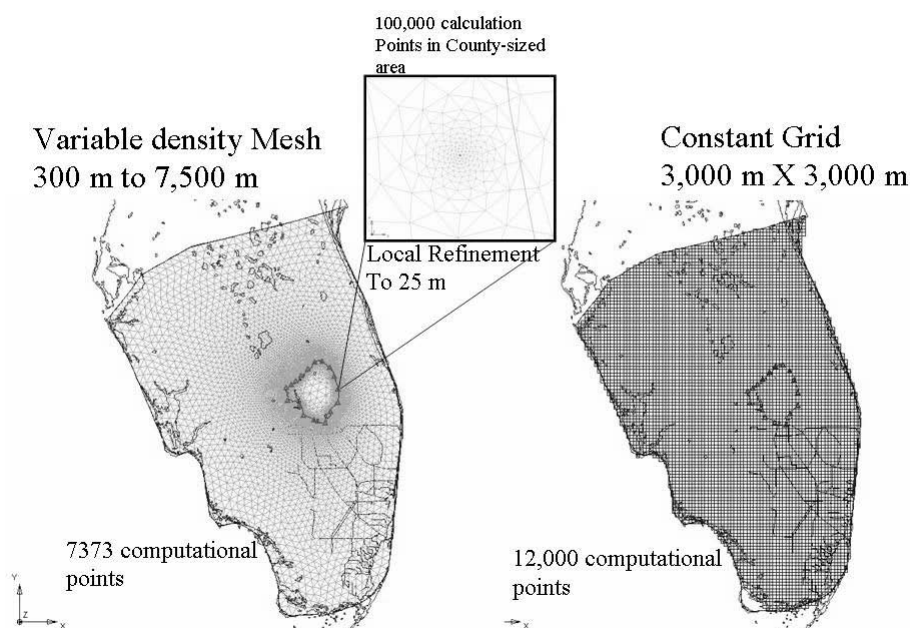
Additional ASR wells are currently under consideration for inclusion in conjunction with the CERP ACME Basin B reservoir and the C-9 Water Preserve Area (WPA). The exact number of wells has not been finalized. In addition, the final number and disposition of *all* proposed ASR wells will be determined through further scientific investigations conducted under a comprehensive regional study. The ASR Regional Study, the associated ASR Pilot Projects and required Project Implementation Report (PIR) studies for each CERP ASR component will be under development until 2010 or later. The primary focus of the ASR Regional Study is to provide additional scientific studies of the proposed CERP ASR project to ensure that the project is feasible at the scale originally conceived in 1999. The ASR Regional Study is outlined in a comprehensive project management plan prepared by the USACE and the SFWMD

and subsequently reviewed and critiqued by the National Research Council (NRC, 2002; USACE & SFWMD, 2004). The ASR Regional Study will address 8 major issues or areas of uncertainty including:

- Characterization of the quality of prospective source waters, spatial
- and temporal variability,
- Characterization of regional hydrogeology of the Upper Floridan
- Aquifer: hydraulic properties and water quality,
- Analysis of critical pressure for rock fracturing,
- Analysis of site and regional changes in head and patterns of flow,
- Analysis of water quality changes during movement and storage in
- the aquifer,
- Potential effects of ASR on mercury bioaccumulation for ecosystem
- restoration projects, and
- Relationship among ASR storage interval properties, recovery
- rates, and recharge volume.
- Ecological affects and studies related to recovery and use of stored ASR water

One large component of the overall ASR Regional Study is the development of a peninsular numerical model of the FAS so that simulations of the proposed CERP ASR operation may be conducted. Several model types and resolutions are under consideration in order to efficiently address the study issues. Figure 2 depicts the anticipated model domains under consideration for the study.

## Three Scales of Model Resolution



**FIGURE 2. Preliminary Domains and Resolutions for ASR Regional FAS Model.**

### CERP ASR PILOT PROJECTS

The scale of the proposed ASR program in the Comprehensive Everglades Restoration Plan (CERP) is unprecedented, and “pilot projects” were envisioned to address the technical and regulatory uncertainties associated with its implementation at a local level. The pilot tests were designed to reduce uncertainties relative to ASR design and operation by investigating options for surface water withdrawal, injection and pumping cycles, water treatment technology, and effects of these pumping cycles on the groundwater and ecosystem in the test region. Operating pilot tests will also provide insight into construction and operational costs, aiding in a comparison of ASR technology with conventional storage technologies such as surface water reservoirs.

Three (3) ASR pilot projects were initially envisioned as comprising the CERP ASR Program – at Lake Okeechobee, the Hillsboro Canal and at the Caloosahatchee River. Because the extent of ASR envisioned around Lake Okeechobee was so large, the Lake Okeechobee Pilot Project was subsequently split into 3 distinct project

localities – at Moore Haven, the Kissimmee River, and at Port Mayaca. The Project Management Plans for the Lake Okeechobee and Hillsboro Pilot Projects - which contained the work plan and schedule at each of the pilot project locations were developed and finalized in 2001. The Caloosahatchee River ASR (CRASR) Pilot Project Management Plan was subsequently approved in February 2002. The PMPs included steps to construct exploratory wells, conduct source water (the water that would be pumped into the wells) sampling, design studies, permitting, construction and operation (cycle testing).

## **EXPLORATORY WELLS AND INITIAL PILOT PROJECT TASKS**

After the PMPs were approved, exploratory wells were constructed at each of the 5 pilot project locations. The exploratory wells were used to characterize the hydrogeology of each of the sites, to confirm that ASR could be implemented in the Floridan Aquifer. Additionally, water quality sampling, design evaluations and permitting were also conducted. Concurrent with the exploratory wells, designs for the treatment and pumping systems were developed at each of the pilot project locations.

## **PILOT PROJECT DESIGN REPORTS AND ENVIRONMENTAL IMPACT STATEMENTS**

When the initial designs were complete, a combined Pilot Project Design Report and Environmental Impact Statement was prepared (USACE & SFWMD, 2004). Those documents contained the recommended design, construction and operation of each of the pilot projects and an evaluation of the potential environmental effects of the pilot projects. A Record of Decision accepting the findings and recommendations of the PPDR/EIS was released in October 2005. As part of the PPDR/EIS document, a cost estimate for the eventual construction of each pilot project was prepared. The cost estimate indicated that the construction of the three Lake Okeechobee sites would exceed the initial authorization – hence it was determined that construction of the Moore Haven pilot project would be deferred. Hence, the Kissimmee and Port Mayaca pilot systems would be the only pilot projects to be constructed in the near term.

### **Pilot Project Details**

The Kissimmee River ASR pilot project has been designed to be a one-well, 5 million gallon per day (MGD) system on the western bank of the Kissimmee River, just north of Lake Okeechobee. The system will use a “pressure media” filter coupled with ultraviolet radiation disinfection to treat river water before it is recharged into the ASR well. The system is currently under construction, and will become operational in late 2007. The Port Mayaca pilot project has been designed to be a multi-well ASR

facility, with 3 ASR wells planned. This project will be useful in determining the effect that multiple wells will have on each other during operation and recovery. Currently, one large diameter ASR well and a dual-zone monitor well have been constructed at the site. The USACE will be constructing additional wells and the surface facilities for this system in late 2007 and through 2008. The Hillsboro or Site 1 is located west of Boca Raton about six miles west of S.R. 7 (U.S. Hwy 441) on Loxahatchee Road. The site is on SFWMD-owned land referred to as "Site 1", adjacent to Loxahatchee Road and the Hillsboro Canal.

The Site 1 ASR system has been designed to provide a capacity to recharge and recover approximately 5 MGD from FAS. The ASR system will withdraw surface water from the Hillsboro Canal through construction of a proposed new inlet and outlet structure, pumps and piping. The raw surface water will be treated to meet primary drinking water standards via screen filtration with ultraviolet (UV) disinfection prior to recharge into the ASR well. During recovery, the water will be treated via aeration prior to discharge back into the Hillsboro Canal. It is anticipated that the system will begin cycle testing in late 2007.

The Caloosahatchee River ASR (CRASR) pilot project was sited at Berry Groves in Hendry County, the future site of an impoundment related to the C-43 Reservoir Project. The exploratory well was constructed between May 2003 and July 2004. The well was completed with a final casing cemented to 640 feet below land surface (bls) – the "top" of the FAS. The open hole was extended to a total depth of 900 feet bls, developed, and subjected to a series of pumping tests. During these tests, the well produced water at rates of up to 2,450 gallons per minute (equivalent to approximately 3.5 MGD). However, copious quantities of fine-grained quartz sand were also produced, and the open hole would collapse and "backfill" with sand.

As a result of the difficulties encountered at the exploratory well – a continuous core boring was installed at the CRASR site – about 1,000 feet west of the exploratory well. The objective of the continuous core boring was to obtain detailed stratigraphic information, to confirm the lateral persistence of the problematic sand interval encountered at the top of the FAS and to stay within the footprint of the conceptual design. The continuous core boring was constructed between October 2004 and January 2005 and succeeded in recovering a nearly complete interval of sediments from 200 to 1,000 feet bls. The continuous core boring has confirmed that the problematic sand interval is also present at this location, albeit at a somewhat shallower depth (between approximately 550 and 660 feet bls). Below that depth, the formation appears more stable – however, the scope of work for the core boring did not include hydraulic testing that would verify the well capacity. The core boring has since been completed with a temporary cap on the wellhead.

At present, the USACE and SFWMD have collected extensive subsurface information at the Berry Groves site, but have not determined the ASR well capacity or its ability to produce water without problematic quantities of sand. This uncertainty has



resulted in a delay in finalizing the surface facility design – primarily related to the uncertainty of the ASR well to produce 5 mgd. It was originally anticipated that this project would be constructed in FY05 and operated for a two year cycle testing period through FY07 – well in advance of construction of the impoundment at Berry Groves. However, due to these technical delays and CERP program budgetary constraints, construction is scheduled to start no earlier than FY06 – which could impinge upon the construction of features related to the C-43 impoundment, which is now part of the Acceler8 program.

## **THE LAKE OKEECHOBEE AND ESTUARY RECOVERY PLAN (LOER)**

Announced October 10, 2005 by Governor Jeb Bush, LOER was a response to identified water resource needs, legislative directives, and demands of Florida citizens. This action plan was developed to help restore the ecological health of Lake Okeechobee and the St. Lucie and Caloosahatchee Estuaries. Key state agencies charged with carrying out this plan include the South Florida Water Management District (SFWMD), the Department of Environmental Protection (FDEP), the Department of Agriculture and Consumer Services (FDACS) and the Department of Community Affairs (FDCA). Through the leadership of the Governor and Legislature, initial funding has been provided for a series of "fast-track" capital projects to help provide meaningful water quality improvements.

The Lake Okeechobee Fast Track projects include the following components: an 800-plus acre expansion of the Nubbin Slough Stormwater Treatment Area (STA); the construction of a 4,000-acre reservoir in Taylor Creek; the construction of another 2,700-acre STA at Lakeside Ranch (south and west of Nubbin Slough); the re-routing of flows from the S-133 and S-191 basins to the Lakeside Ranch STA; and, potentially, the re-routing of flows from the S-154 basin also to the Lakeside Ranch STA. The combined storage and phosphorus reduction benefits are estimated at 48,000 acre feet and 65 to 75 metric tons, respectively.

In addition to the "turn-dirt" construction projects, several other far-reaching and innovative components – some which do not require large capital outlay – can also provide more immediate and measurable improvements in the condition of Lake Okeechobee, the St. Lucie Estuary and the Caloosahatchee Estuary. Combined, these initiatives comprise a bold and aggressive recovery plan.

To help achieve a better balance among management objectives – flood control, water supply and navigation, and the competing needs of the lake, estuaries and greater Everglades ecosystem – the U.S. Army Corps of Engineers (USACE) will revise the lake regulation schedule by December 2006. The ultimate goal is to achieve lower lake water levels and reduce high volume discharges to the estuaries. As structural components, such as the Acceler8 reservoirs are completed, the USACE will continue

to revise the schedule. The SFWMD and other supporting agencies will be working in tandem to revisit water supply demand estimates, supply side management strategies and the modeling and design of forward pumps.

Three ASR projects have been initiated under the LOER initiative including:

- Reactivation of the Taylor Creek ASR system (originally constructed and tested in the early 1990's);
- Partner with the Seminole Tribe to construct a pilot ASR system at the Brighton Indian Reservation; and
- Construct a 10-well ASR system somewhere in the vicinity of Lake Okeechobee.

## EMERGING ISSUES

For all ASR projects, performance factors relating to the aquifer itself are important. The aquifer hydrogeology will control the distribution of the injected water within the aquifer storage zone. The aquifer hydraulic conductivity controls the distribution of water level or potential (pressure) in the aquifer. Aquifers exhibiting low to moderate hydraulic conductivity may result in induced high water levels or aquifer pressure. Inordinately high water levels may result in surface flooding or loss of injected water through surface runoff. Inordinately high aquifer pressures may result in local hydraulic fracturing of the aquifer material itself or surrounding confining units in the case of a confined aquifer. Aquifers exhibiting low porosity or aquifers where fluid flow is concentrated in fractures rather than aquifer pore space, may increase groundwater flow velocity or may induce high rates of diffusion (Gale, 2002; Anderson and Lowry, 2004); either of these phenomena may reduce recoverability of injected water.

Besides the ASR performance factors related to hydrogeology, ASR performance can also be negatively affected by geochemical reactions that occur between the aquifer matrix and the injected water or the ambient groundwater and the injected water. Geochemical issues at multiple ASR sites around the globe have hindered further project development efforts and led to mistrust of the technology by regulators. Geochemical issues related to arsenic and pyrite have been very problematic in Florida.

Geochemical reactions around recharge and recovery wells have been reported for decades. ASR wells have only come into being in the early 1980s so that literature specific to ASR wells is relatively recent. Mirecki (2004) studied geochemical data at 11 ASR sites located in south Florida. Mirecki observed that several of the sites had elevated concentrations of arsenic, radionuclides, ammonia, and sulfate. Seven of eleven ASR sites studied reported arsenic results; only one of these sites reported arsenic concentrations in excess of water quality standards. Three of the eleven sites

reported ammonia concentrations in the recovered water in excess of Florida surface water regulatory standards.

Price and Pichler (2006) found that arsenic was present within the Suwannee Limestone portion of the Upper Floridan Aquifer System. Bulk arsenic concentrations ranged from a few part per million (ppm) to 11,200 ppm. The study also concluded that pyrite is “ubiquitous throughout the Suwannee Limestone” in the form of framboidal pyrite and is “most abundant in high porosity zones”. The study also discounts other potential arsenic sources such as hydrous ferric oxides, clay minerals, and apatite. The average As concentration for all 306 samples that were collected during the study was 3.5 ppm, which is higher than the global average for limestone (2.6 ppm).

Another emerging issue is competition between users of the FAS for water supply and ASR projects. The upper portion of the FAS is not only a good zone for ASR, but is also an excellent source of artesian water for agricultural and public water supply. In the past twenty years, water withdrawals from the FAS have doubled, from 2 billion gallons per day (BGD) to slightly more than 4 BGD. Through the alternative water supply grant program and other environmental initiatives designed to wean users off the Surficial Aquifer system, the SFWMD has encouraged users to exploit the FAS. As a result, the proximity between ASR projects and water supply (withdrawal) projects is diminishing, forcing the permitting agencies to utilize more complex analyses for determining permittable quantities and the most beneficial use of the resource.

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## ARTIFICIAL RECHARGE IN NEW ENGLAND

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### ABSTRACT

Artificial recharge to increase aquifer production is currently rare in New England. Only one utility (Dover, New Hampshire) currently uses it to enhance production from four of its wells, which are hydraulically isolated from nearby rivers. Two utilities in Maine have used artificial recharge only in emergency.

There is a newly perceived need to replenish streamflow by recycling wastewater in watersheds from which the water was originally withdrawn, rather than discharging it into the lower reaches of rivers or the ocean, as is still generally practiced. The new Massachusetts State policy is graced by the title "Keeping Water Local". As a result, several tens of sites for the recharge of treated wastewater have been developed in Massachusetts and Rhode Island.

Regulations governing in-stream flow protection is a current concern throughout the region, because of withdrawals either from surface water or by induced recharge, particularly in summer. This mismatch of summer season reduced availability and increased demand can result in fish kills or dry river beds, neither of which is culturally acceptable. Recently, most New England States have set limits on permitting new wells and cut previously permitted withdrawals from existing riverside wells due to these concerns.

Thus artificial recharge, if it can be practiced, has value in New England.

### BACKGROUND

Artificial recharge is practiced all around the globe, but little in New England. The reasons are several.

1. Confined aquifers suitable for aquifer storage and recovery are rare;
2. Most sand and gravel aquifers are unconfined and closely tied hydraulically to nearby streams to which they quickly release groundwater; and
3. There is a public distaste for the concept of recycling wastewater, even treated wastewater, through aquifers.

In New England, however, artificial recharge may be valuable under the following conditions:

- Aquifers with relatively poor hydraulic connections to surface water bodies;

- Seasonal withdrawals from rivers (somewhat similar to flood skimming from rivers to surface water storage reservoirs); and
- Discharge of treated wastewater in locations where the additional water will enhance well withdrawals or streamflow, without negatively impacting water quality.

## NEW ENGLAND EXPERIENCE IN ARTIFICIAL RECHARGE

Of the six New England States, the following artificial recharge projects have been allowed, permitted or are being planned:

**TABLE 1**

| State         | Drought<br>(Emergency) | Surface Water |         | Wastewater |         |
|---------------|------------------------|---------------|---------|------------|---------|
|               |                        | Permitted     | Planned | Permitted  | Planned |
| Connecticut   | -                      | -             | -       | -          | -       |
| Maine         | 2                      | -             | -       | -          | -       |
| Massachusetts | -                      | -             | -       | 25-30      | more    |
| New Hampshire | -                      | 2             | 2       | -          | -       |
| Rhode Island  | -                      | -             | -       | 5          | -       |
| Vermont       | -                      | -             | -       | -          | -       |

In Massachusetts, however, there are 25-30 facilities that discharge tertiary treated wastewater at sites within the Zone II wellhead protection areas around drinking water wells, and thus constitute artificial recharge to an aquifer. However, in Connecticut, the discharge of treated wastewater to a drinking water aquifer is prohibited.

This paper describes artificial recharge systems and investigations at three sites in New Hampshire where surface water is or will be used for the artificial recharge of glacial sand and gravel aquifers from which drinking water is withdrawn.

## INDUCED RECHARGE AND FLOOD SKIMMING

Many public supply wells in the region induce recharge from nearby surface water bodies such as rivers and lakes. If so, then they are “under the influence of surface water”. But if the riverbank filtration capacity of aquifer materials is sufficient, then the water withdrawn from wells passes the MPA (micro-particulate analysis) test, and is deemed not “under the direct influence of surface water” under Federal statutes and regulations. The water supply source will thus not require filtration under the Safe Drinking Water Act. One such water system is the Old Town Water District, in Old Town, Maine, the water supply wells are all located in a small aquifer close to the Stillwater River and can be shown to induce 90% of their recharge from

it. In the case of Old Town, the wells are located >150 feet from the nearest surface water, and are screened at >50 feet below ground surface and thus do not require either MPA testing or filtration according to Maine (and Massachusetts) guidelines.

Water is pumped from the Ipswich River during storm events (in fall, winter and spring) to fill the Salem-Beverly (Massachusetts) reservoirs. Large pumps (25 mgd) located on the river bank approximately in the middle reaches of the river, only operate when the river is in flood stage. This is flood skimming.

High river flows can also be used to replenish aquifers (as described in this paper). The process works best in situations in which there is a poor hydraulic connection to the nearest surface water body. Three examples are described below, in each of which artificial recharge is used to enhance yield from production wells.

## **DOVER, NEW HAMPSHIRE'S TWO ARTIFICIAL RECHARGE SYSTEMS**

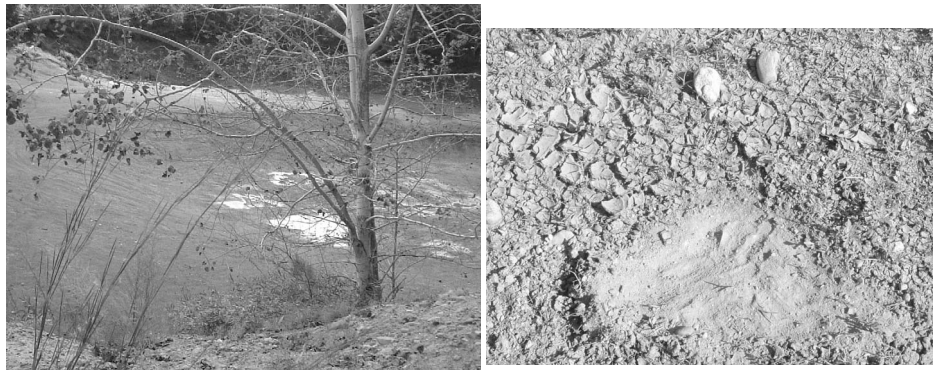
In 1954 the Dover Sand and Gravel Company began washing gravel for a concrete plant located on a parcel adjacent to the City of Dover's Ireland and Griffin wells (Figure 1). The Company obtained a permit from the Army Corps of Engineers in 1959 for withdrawal of an unspecified volume of water from Bellamy River. Gravel washing takes place on top of the hill at the plant site (at a current rate of 800 gallons per minute during construction season of April through November). The silty wash water is discharged on the ground where it disappears by infiltration into the aquifer. The City's Ireland and Griffin wells, located 1000 and 1200 feet away were and are the beneficiaries of the unintentional artificial recharge.



**FIGURE 1. Gravel washing in Dover, New Hampshire, for concrete production. Discharge of wash water provides an inadvertent source of artificial recharge to two nearby municipal wells.**

During the period 1980 through 1986, Dover Sand and Gravel was working at limited capacity, at which time the City noticed a drop in water level in the Ireland well. So the City installed its own intake and pumping station on the Bellamy and constructed a recharge basin in an active gravel pit.

In the 1980s, pumping of Dover's Calderwood and Campbell wells appeared capable of dewatering their aquifer located close to, but apparently not hydraulically connected to the Isinglass and Cocheco Rivers. So an artificial recharge system was designed and constructed by the City, with a 16-inch intake, a 75 HP 10-inch pump, pumping at 1,500 gpm when in operation, feeding 4,000 feet of 12-inch main, pumping overland to a recharge basin. The recharge basin is about 2 acres in size, and is located among active gravel pits (Figure 2A). The source of water chosen was the relatively pristine Isinglass River, rather than the Cocheco River that flows through the nearby City of Rochester, NH.



A.

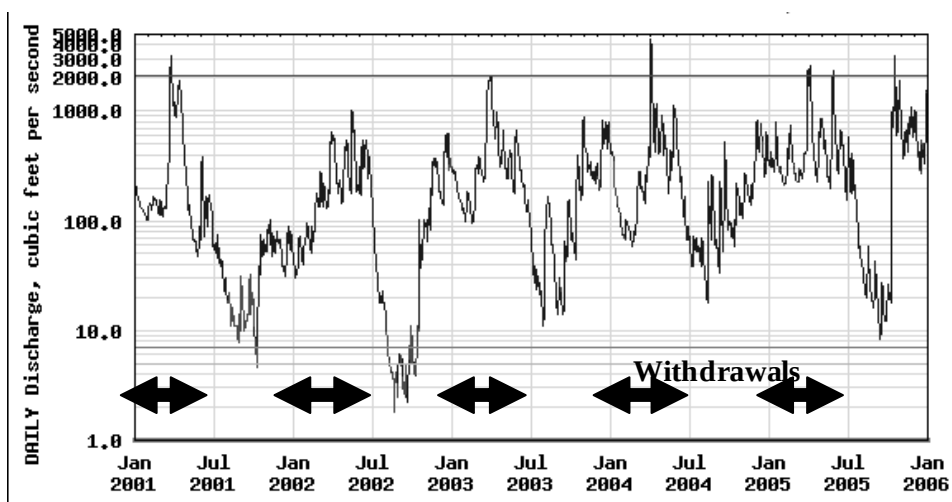
B.

**FIGURE 2. Artificial recharge that serves two wells located in Dover's Hoppers Aquifer is withdrawn from the Isinglass River during seasons of high flow. The recharge basin is a natural glacial "kettle hole" in an active sand and gravel pit. It is a pool, partially frozen over in winter, but dries up in summer (A). Every few years, during summer, the silt from the river water is scraped off to expose aquifer sands below (B).**

Recharge takes place over 8 months, from November through June (Figure 3). In 2005 the total volume pumped was 524 million gallons for an average of 2.2 million gallons per day. The pit requires maintenance every few years. This is done during the summer season, by scraping off the top 1/2 inch of sandy soils with accumulated silt and decayed organic matter pumped over to the pit from the river (Figure 2B). This material is used elsewhere as topsoil.

The geology of the recharge site is unknown in detail, except that layers of silt are known to exist among the sand and gravel. The silt presumably functions to hydraulically separate one portion of the aquifer from another and to isolate it from the river.





**FIGURE 3. Hydrograph typical of southeast New Hampshire (Lamprey River) showing seasonal variations in flow from a dry period (2001-02) through a wet period (2005-06). Withdrawals from the river for artificial recharge of the Hoppers aquifer take place during periods of high flow.**

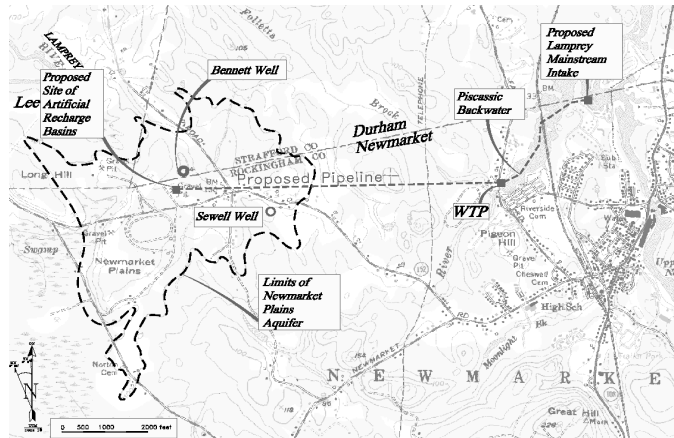
Since 1993 regulatory oversight from the New Hampshire Department of Environmental Services comes in the form of an application for underground injection registration, requiring that the river water be sampled and analyzed once at the beginning of each recharge event for VOCs and nitrate-nitrogen. Also that: “the facility owner shall be responsible for ensuring that no discharge from this activity shall contain a regulated contaminant as defined in New Hampshire Statute as “any physical, chemical, biological, radiological substance, or other matter, other than naturally occurring substances at naturally occurring levels in water, which adversely affects human health or the environment.”

Water quality in Dover’s four wells that receive artificial recharge has been excellent. The Calderwood, Campbell and Ireland wells require no treatment of any kind. The Griffin well does require treatment for iron, though the origin of the iron problem is unknown and may have nothing to do with the artificial recharge. River water quality is reasonably good in both Bellamy and Isinglass Rivers, with Dissolved Organic Carbon below 10 mg/L.

## **NEWMARKET, NEW HAMPSHIRE’S HISTORICAL ATTEMPTS AT ARTIFICIAL RECHARGE**

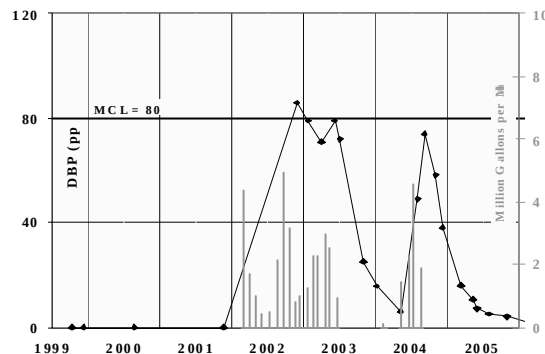
The Newmarket Plains aquifer is hydraulically isolated from the Lamprey River, with no point of contact between the aquifer and the river. The aquifer is located on high

ground in the town, and would normally discharge through springs to tributaries of the Lamprey if it were not tapped by two public supply wells. The two wells are the Bennett and the Sewall wells operated by the Newmarket Water Department. Both obtain their entire recharge through precipitation falling on portions of the aquifer.



**FIGURE 4.** Hydraulic isolation of the Newmarket Plains aquifer (dashed line to left) from the Lamprey River (to right) creates a favorable situation for artificial recharge. Proposed recharge pipeline extends from the Lamprey River main stem in Newmarket, over a distance of 2 miles.

Newmarket's Water Treatment Plant (WTP) was used to supplement water withdrawn from the wells. It treated Lamprey River water, but was difficult to operate because of manganese and naturally occurring organics in the raw water, resulting in high color. Difficulties in treatment led to high disinfection byproducts (DBPs) in the treated water. Experiments run in the early 2000s to test ways in which it might run better found it best to run the WTP at full capacity, full time. Because this provided more water than the town required, the extra water was pumped through the distribution system to a recharge gallery near the Bennett Well, in order to store it in the aquifer for later use and to replenish the aquifer after 2 years of drought. The gallery consisted of perforated pipes buried in a disused portion of an otherwise active gravel pit 300-400 feet from the well.



**FIGURE 5.** Concentrations of Disinfection Byproducts in Bennett well water (data diamonds) before and after artificial recharge using surplus treated surface water (grey bars).

High DBPs were noted in well water as a result of this artificial recharge of treated water (Figure 5), with concentrations occasionally exceeding the MCL of 80 ppb. Thus the plan was abandoned. So was the operation of the WTP.

## **INVESTIGATIONS FOR ARTIFICIAL RECHARGE USING UNTREATED RIVER WATER IN NEWMARKET**

In a project now under investigation, artificial recharge with Lamprey River water is being considered, with an open recharge basin near the Bennett well. The water quality issue of most concern was whether the high color and organic carbon content typical of the raw water at Newmarket's WTP derived from the Piscassic backwater would exceed the natural filtration capacity of the Newmarket Plains aquifer sands.

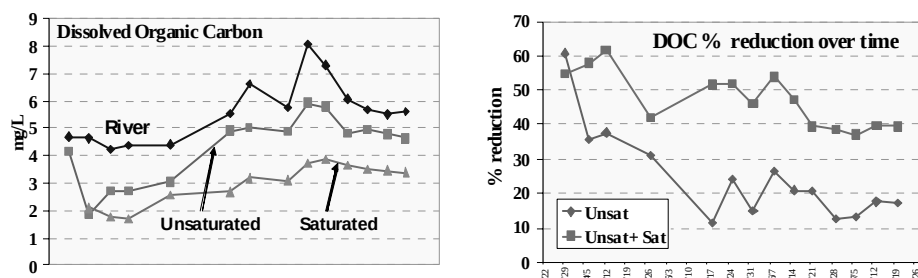
Bench scale testing mixing aquifer materials with river water from the WTP showed that organics (measured by DOC, color and UV-254 absorbance) were all reduced in concentration in the water in contact with aquifer sands. However, metal ions (iron, manganese and arsenic) all increased, some above the MCL, with concentrations of particulate iron and arsenic exceeding the respective dissolved form of the metal. The result for the metals was probably due to the mechanical shaking or tumbling, causing erosion of metal oxide and hydroxide coatings on sediment grains. We conclude that bench testing is not a useful evaluation technique for artificial recharge.

Column testing, passing Piscassic water through aquifer sands found indications of breakthrough of organic parameters, though in other respects filtration appeared effective. We concluded that the concentration of Dissolved Organic Carbon (DOC) in the water was too high (6-18 mg/L) to allow for sufficient filtration through aquifer sands.

A second phase of column testing used water from the main stem of the Lamprey River, a source that is at 4500 feet further from the recharge site, but is known to have DOC at 4-8 mg/L. The experiment passed river water through a 9-ft column of unsaturated aquifer material, followed by a second column also 9-ft in length. The experiment lasted 4 months and produced the following results:

1. pH remained approximately neutral throughout the experiment. In general passage of river water through the aquifer material tended to increase pH above 7.0 (natural groundwater at the Bennett Well has a pH in the range 7.1 to 7.6).
2. Dissolved oxygen (DO) and Redox potential were reduced through the columns, as expected, but not to the point where deleterious geochemical reactions might occur.

3. About 40% of the organic material was removed, and the results stabilized after about 3 months (Figure 6). There was a clear progression of removal as water passed through the saturated column such that estimates can be made regarding the length of saturated aquifer that will be required for nearly complete removal of DOC.



**FIGURE 6. Results of second phase of column testing of Lamprey River water. Dissolved Organic Carbon (DOC) is reduced considerably by passage through aquifer materials, and stabilizes at about 40% removal.**

4. Iron was not above MCL in river water, and was reduced mostly to below detection by passage through aquifer material. Manganese behaved similarly.

5. Tests were performed to determine if there would be any deleterious effects of mixing recharge water and Bennett well water. There were none.

6. A final dosing of the columns with E.coli and MS2 bacteriophage (a surrogate for viruses) found that the aquifer material was highly effective in removing both.

Our conclusion is that water from the Lamprey River, having lower concentrations of DOC, is suitable for artificial recharge, while water from the Piscassic backwater is not.

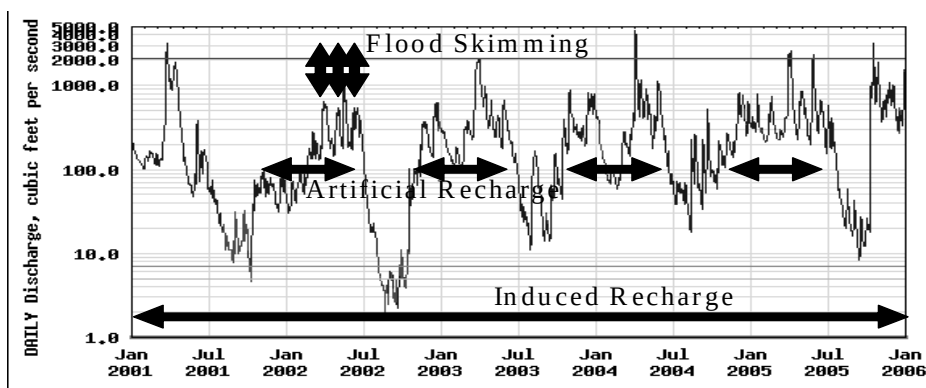
## ARTIFICIAL RECHARGE IN DROUGHT EMERGENCY

Artificial recharge can also be used in emergency, during drought periods, when natural recharge to the aquifer is insufficient to keep up with municipal withdrawals. That was the case for the Water Utilities in Pittsfield, Maine, and Alfred, Maine, both of whom artificially recharged aquifers close to their production wells during drought periods in 1991 and 2001. Pittsfield used water pumped from a well that had been discontinued due to high manganese concentrations. Alfred used water pumped from a nearby lake that had once been used as a water supply. No water quality problems were noted in either case. Artificial recharge was discontinued after new wells had been installed in nearby aquifers.

## WATER QUANTITY IN RIVERS

Precipitation varies little from season to season in New England. Yet all rivers in the region exhibit seasonally high flows in late fall, winter and spring seasons, reflecting seasonally low evapotranspiration rates, and snow melt at the end of the winter. Thus it makes sense to withdraw water for artificial recharge during those seasons (Figure 7). This scheme also avoids concern about taking water from rivers, either by direct withdrawals for surface water treatment, or by induced recharge to riverside wells, during summer low flow periods. Such a seasonal withdrawal scheme not only meets the needs of the public water suppliers whose demands are highest in summer, but also the needs of native fish and other flora and fauna, which tend to be stressed during periods of low flow. Withdrawing 1 cubic foot per second (cfs = 0.65 MGD) from a river flowing a 100 cfs or more will have little ecological effect compared to withdrawing the same volume from a river flowing at 10 cfs or less.

Summer, the season during which river withdrawals will not take place for artificial recharge, provides an opportunity for recharge basin maintenance.



**FIGURE 7. Typical hydrograph showing ideal periods of withdrawal for artificial recharge. These seasonal withdrawals are contrasted with withdrawals from wells located close to rivers, which induce their recharge directly from the river at all seasons and flood skimming for which water is withdrawn only during periods of extreme flow.**

## FLOOD SKIMMING

A few water utilities in New England utilize flood skimming as an alternative method of storing water during periods of high flow. For instance, the Beverly-Salem water system withdraws 25 million gallons per day for about 40 days per year to reservoir storage during times of flood flow only. This system is remarkable because the

Ipswich River has been classified by the Massachusetts Water Resources Commission as “highly stressed,” and was designated by American Rivers as the third most endangered river in the nation due to extremely low flows and extended periods of no flow along much of the upper watershed during droughts.

## **RECHARGE OF TREATED WASTEWATER**

There is a general antipathy by the New England public regarding the recycling of treated wastewater by artificial recharge to aquifers. This is so despite the fact that all of rural New England, and even many towns that provide municipal water supply, use septic drainfields for the disposal of wastewater. Septic drainfields are, of course, unsung artificial recharge systems. Some towns, particularly in Massachusetts, having installed a public sewer to replace septic system, have noted that the sewer, by removing water out of the drainage basin, robs that basin of recharge to serve its production wells.

As a counter to that concern, Massachusetts has instituted a “Keep Water Local” policy and has allowed the construction of between 25 and 30 systems for the discharge of treated wastewater into aquifers that are mapped as within the Zone II wellhead protection areas contributing groundwater to public supply wells. Tertiary treatment of wastewater is required for such systems.

## **SUMMARY**

1. Artificial recharge is rarely used in New England at present, mostly due to a lack of suitable aquifers, and a general distaste for using treated wastewater to recharge aquifers.
2. Dover, New Hampshire, has two artificial recharge systems, each of which takes water from rivers to recharge isolated aquifers, allowing for increased yields of existing wells. Both systems have been in use for 20 years, require little maintenance, and appear free of water quality problems. Minimal monitoring is required.
3. An attempt was made in Newmarket, New Hampshire, to use treated water high in DBPs for artificial recharge. However, aquifer materials did not filter out the DBPs, so the attempt was abandoned.
4. However, Newmarket recently performed column experiments on two different river sources for artificial recharge at the same site, and determined that the one with dissolved organic carbon = 6-18 mg/L is probably not suitable for artificial recharge, though the one with DOC = 4 - 8 mg/L appears to be suitable.
5. Artificial recharge of the type practiced by Dover and under study by Newmarket, New Hampshire, is only possible if there is sufficient aquifer storage. Such storage is

often difficult to find in aquifers in New England because almost all are unconfined and hydraulically connected to nearby surface waters.

6. Withdrawals of water from aquifers, rivers and lakes throughout New England have recently collided with a regional concern to maintain in-stream flows, especially during low-flow periods in summer when water usage rates are typically highest. Artificial recharge to aquifers during seasons of high river flow should alleviate such concerns while providing a reliable source of water in summer.

7. Artificial recharge of treated wastewater is a relatively recent development in New England, and is being actively encouraged by a 2004 Massachusetts policy called "Keeping Water Local".

## CHALLENGES IN OPTIMIZING A LARGE-SCALE MANAGED AQUIFER RECHARGE OPERATION IN AN URBANIZED AREA

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### ABSTRACT

Over the past 70 years the Orange County Water District (OCWD) has constructed a large, complex managed aquifer recharge (MAR) operation consisting of over two dozen recharge facilities that cover over 6.1 km<sup>2</sup>. These facilities have allowed OCWD to recharge an average of 287 million m<sup>3</sup>/yr into the groundwater basin, thereby effectively increasing the yield of the groundwater basin and allowing for groundwater withdrawals in the range of 400 to 430 million m<sup>3</sup>/yr.

Nevertheless, further enhancement of OCWD's MAR operations is needed to further increase the groundwater basin's sustainable yield and to optimize the water resources within the basin. Due to high property values, urbanization of the recharge area, and limited land availability, MAR enhancement is centered on maximizing the efficiency of existing facilities, including construction of recharge Basin Cleaning Vehicles (BCVs) that continuously remove the clogging layer that forms in the basins as well as development of a Long-Term Facilities Plan, which presents a sequential roll-out of recharge enhancement projects that will become cost-effective as the value of recharged water increases. Such projects include desilting the recharge water to reduce basin clogging with solids, installation of subsurface infiltration galleries under parking lots and parks, and the installation of vadose zone recharge wells.

### KEYWORDS

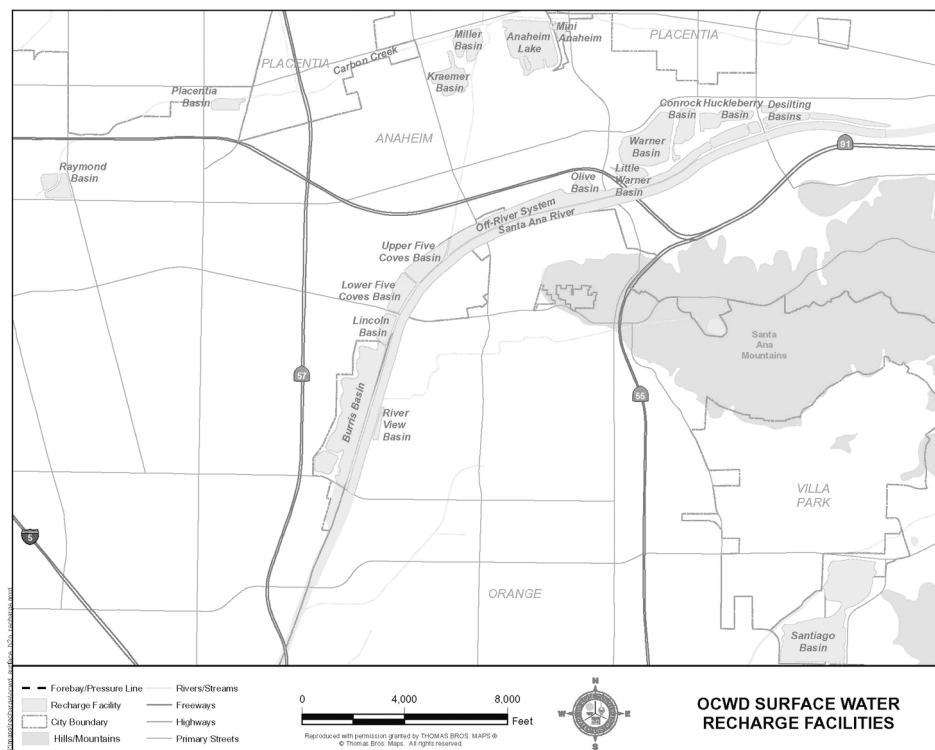
Surface water recharge, optimization, Orange County Water District

### INTRODUCTION

OCWD was formed in 1933 by a special act of the California state legislature to 1) Manage the Orange County groundwater basin; 2) Conserve the quantity and quality of groundwater in the basin; 3) Reclaim water for beneficial uses; and, 4) Conserve and control storm and floodwaters flowing into the basin. As part of meeting these objectives, OCWD has, over the course of over 70 years, constructed a large, complex MAR operation consisting of 26 facilities that cover over 6.1 km<sup>2</sup> (1,510 acres). These facilities include 10 km (6 miles) of the Santa Ana River (SAR) channel, deep recharge basins that are up to 45 m (150 ft) deep, shallow recharge basins up to 1 m (3 ft) deep, desilting basins, two rubber dams, many kilometers of pipelines, multiple pump stations, and a seawater intrusion barrier. Figure 1 shows the location of



OCWD's surface water MAR facilities, which are located in the cities of Anaheim and Orange.



**FIGURE 1: Location of OCWD's Surface Water MAR Facilities**

Over the past 18 years, OCWD's surface water MAR facilities have provided an average of 280 million m<sup>3</sup>/yr (227,000 acre-feet/yr [afy]) of recharge to the groundwater basin. This recharge has effectively increased the yield of the groundwater basin and has allowed for groundwater withdrawals in recent years in the range of 400 to 430 million m<sup>3</sup>/yr (325,000 to 350,000 afy). On average, surface water MAR provides approximately 75 percent of the volume of groundwater pumped each year. Nevertheless, further enhancement of OCWD's recharge operations are needed to further increase the groundwater basin's sustainable yield and to optimize the water resources within the basin.

## BACKGROUND

OCWD purchased its first parcel of land in 1936. This 0.11 km<sup>2</sup> (26 acre) parcel, which is part of the SAR channel, was purchased for \$722, or \$6,560/km<sup>2</sup> (\$27/acre). Since that time, OCWD has made over 125 acquisitions and now owns more than 6.1 km<sup>2</sup> (1,510 acres) of land. Land acquired for surface water MAR is located in the cities of Anaheim and Orange because the underlying deep Principal Aquifer used for groundwater production daylights at the ground surface in these cities. Other portions of the basin are not ideal for surface water recharge due to the presence of fine-grained sedimentary deposits that separate the shallow aquifer from the deeper Principal Aquifer. In recent years, the cities of Anaheim and Orange have become increasingly urbanized and there are few remnants remaining of the area's agricultural heritage. As a result, there are few large parcels of land left that can be converted to surface water MAR facilities. OCWD's most recent purchase of a 0.04 km<sup>2</sup> (10 acre) nursery that will be converted to a shallow recharge basin cost \$8.3 million, or \$205 million/km<sup>2</sup> (\$830,000/acre). Obviously, future expansion of OCWD's surface water MAR system through additional land purchases would be costly, forcing OCWD to focus on maximizing the efficiency of the existing surface water MAR system and exploring opportunities to recharge water under parking lots, in decorative lakes and other innovative recharge techniques that are compatible with an urbanized environment.

## MAR OPERATIONS OVERVIEW

The primary source of natural surface water to the groundwater basin is the SAR, which is the largest coastal stream in southern California with a length of 129 km (80 miles) and a drainage area of 6,400 km<sup>2</sup> (2,470 miles<sup>2</sup>) (Blomquist, 1988). Discharge varies from season to season because SAR flows are primarily linked to precipitation within the SAR watershed. In recent years, SAR baseflow has steadily increased due to increased discharges of treated wastewater as the upper reaches of the SAR watershed have become more developed. From the late 1940s to mid-1970s, SAR baseflows were generally less than 49.3 million m<sup>3</sup>/yr (40,000 afy). Baseflows have steadily increased since the mid-1970s and in recent years have averaged approximately 185 million m<sup>3</sup>/yr (150,000 afy). The maximum total SAR flow, including stormflow and baseflow, occurred in 2005, when over 786.8 million m<sup>3</sup> (638,000 af) flowed down the SAR.

Up until the late 1950s, the only surface water MAR facility operated by OCWD was the SAR channel. To increase OCWD's ability to recharge water, OCWD purchased 0.26 km<sup>2</sup> (64 acres) in Anaheim and excavated a deep recharge basin, which is now called Anaheim Lake. Up until the mid-1970s, water supplied to Anaheim Lake was

exclusively imported water from the Colorado River purchased from the Metropolitan Water District of Southern California (MWD). Anaheim Lake and the SAR channel were the primary artificial recharge facilities used to recharge the nearly 2,467 million m<sup>3</sup> (2,000,000 af) of imported water purchased in the 1950s and 1960s as part of an aggressive effort to refill the overdrafted groundwater basin.

In the 1970s and 80s, OCWD purchased additional land and began developing the ability to divert SAR water to the new recharge basins by constructing two rubber dams, over 6 km (10 miles) of pipelines, and six pump stations, including one that is capable of pumping 400 m<sup>3</sup>/min (235 cubic feet per second [cfs]). In addition, new pipelines were installed to convey imported MWD water to recharge facilities other than Anaheim Lake. Today, OCWD's surface water MAR system is capable of diverting up to 1,700 m<sup>3</sup>/min (1,000 cfs) from the SAR for short periods and can convey both SAR and imported water to multiple facilities depending on operational needs.

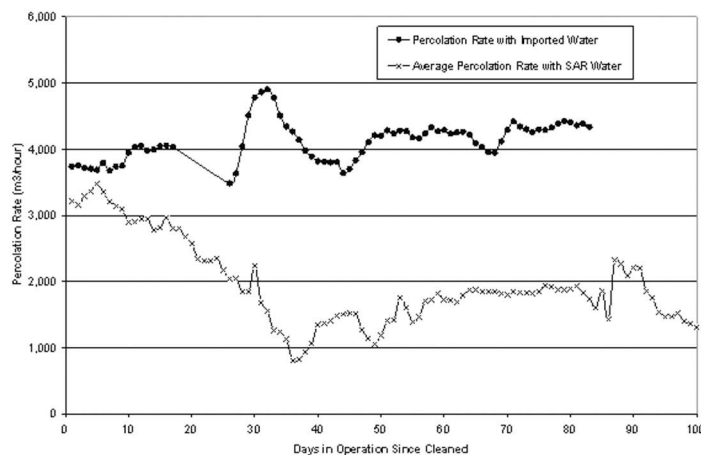
One of the main constraints to increasing OCWD's surface water MAR capacity is clogging. Clogging is caused primarily by the accumulation of suspended solids such as silts, clays, and a small fraction of organic material (20 percent) in the recharge basins. The total concentration of suspended solids (TSS) in SAR baseflow ranges from 15 to 400 mg/L and can be extremely high during stormflow conditions. Basin clogging is relatively rapid, generally causing percolation rates to decline in exponential fashion. To mitigate clogging and restore percolation rates, the basins are drained or pumped out and then allowed to dry. Once dried, the basin bottoms are scraped with large, twin-engine scrapers. The steep basin sidewalls (3:1 slopes) are disturbed with bulldozers, which break up the clogging layer but do little to remove it. Recent trenching work has revealed that some basin sidewalls are covered with a mantle up to 1 m (3 feet) thick in places of a mixture of clogging material and native sediments. There are a number of problems with the current basin cleaning method, including continued deepening of the basins, which results in increasingly poor drainage. Another problem is the inability of the bulldozers to remove the clogged material on the basin sidewalls. The current cleaning operation using scrapers and bulldozers is not sustainable.

## **RECENT FACILITY OPTIMIZATION**

### **Basin Cleaning Vehicles**

To address the issue of basin clogging, OCWD embarked on a research program to see if a device could be developed to remove the clogging layer while the basin remained in service. The objective was to maintain elevated percolation rates and limit the basin downtime required for cleanings, which can last up to three months in the larger basins. In 1993, a small prototype device that resembled a pool cleaner

was dragged across the bottom of Anaheim Lake using cables. When the basin was drained, a clean strip of sand showed that the device successfully removed the clogging layer. Based on this experiment, OCWD developed a large, fully submersible Basin Cleaning Vehicle (BCV) that was controlled by an on-board Global Positioning System (GPS). A patented hood was designed with an rotating auger to break up the clogged sediment and place the clogging material in suspension and an internal baffle that would return the heavier sand particles back to the basin while still removing the fine-grained clogging material via a slurry pump. Extensive testing of the BCV in 2001 showed that it was capable of sustaining elevated percolation rates for several months; however, due to leaks and harsh underwater conditions, extensive and costly maintenance was required to keep the BCV in operation. As a result, the BCV was redesigned using dredge barges to house the hydraulic and electrical components above water with the hood being the only underwater component. The dredge-type BCV uses an on-shore mounted cable/winch system to traverse the basin and can operate in basins up to 6 m (20 feet) deep. Four of these vehicles were installed in early 2004 in four separate basins. In three of the basins, the BCV has not performed as expected due to inability to keep up with the high solids load in these basins, damage to cleaning system from rocks, and poor hood contact with the uneven basin bottoms. However, in Miller Basin, which receives relatively low concentrations of clogging material and has a relatively smooth, sandy bottom, the BCV is able to meet expected performance targets, effectively maintaining a “clean” percolation rate in the half of the basin it cleans.



**FIGURE 2: Percolation Rate in Miller Basin with SAR Water and Imported Water**

Although the BCV in Miller Basin is able to meet performance targets, recent experiments in which basins were dedicated to the recharge of imported MWD water

for many months (i.e., no SAR water), show that clogging rates are extremely low when this clean source of water is used. The imported water is untreated water from the Colorado River or from northern California (State Water Project) and typically has TSS concentrations on the order of 1 to 2 mg/L. Figure 2 shows how percolation rates compare in Miller Basin using SAR water or imported MWD water. These experiments showed that removing the clogging material before it reaches the basins has the potential to significantly increase OCWD's recharge capacity.

### **Clogging Research**

To further assess how TSS concentrations affect basin clogging rates, OCWD conducted column experiments in 2006 to investigate the interaction between suspended solids and recharge basin sediments. These experiments resulted in the development of a parametric model of percolation decay that is being used to optimize basin operations (Phipps, *et al.*, 2007). In addition, the model provides a means to understand the trade off in costs to reduce TSS concentrations and increased recharge capacities and thus will be an important tool in evaluating the feasibility of alternatives to reduce TSS concentrations in SAR water prior to recharge.

### **Basin Cleaning Optimization**

Building on the percolation decay model, OCWD developed an economic model to calculate the optimum time to take recharge basins out of service for cleaning. The model incorporates the value of lost recharge due to basin downtime, pumping costs to dewater the basins, and the cost to clean the basins using heavy equipment, including equipment costs, fuel costs, labor costs, etc. Because percolation rates tend to decay rapidly, the model shows that maximizing basin recharge generally means cleaning the basins more frequently. One of the challenges OCWD faces in implementing more frequent basin cleanings is draining the deep basins, which can contain up to 3 million m<sup>3</sup> (2,000 af) of water. This is one of the reasons newer recharge basins are constructed so that they are shallow and thus can be quickly drained, dried, cleaned and placed back into service.

As mentioned above, the basin bottoms have historically been cleaned using large scrapers. These scrapers are incapable of selectively removing the thin clogging layer that accumulates in the recharge basins, which is typically 0.3 to 1.3 cm (0.125 to 0.5 inch) thick. As a result, the scrapers remove significant quantities of clean sand that underlie the clogging layer. The continual removal of clean sand from the basin bottoms is not only unsustainable, but has resulted in increasingly poor drainage of the basins, which requires the basins to be out of service longer so they can adequately dry prior to cleaning. To test alternative methods to increase the efficiency of clogging layer removal and reduce the amount of clean sand removed from the basins, OCWD rented a Cherrington Model 5000 Beachcleaner

([www.cherrington.net](http://www.cherrington.net)). The Cherrington Model 5000 is the world's largest self-contained, self-propelled beachcleaner (Figure 3). It is capable of cleaning 2 m (6 ft) wide swaths per pass and can clean to a maximum depth of 15 cm (6 inches). The screening system can be quickly and easily adapted to changing conditions including changing the screen sizes for better materials removal. An additional useful feature is the unit exerts very low ground pressure, which will minimize compaction of basin



**FIGURE 3: Picture of Cherrington Model 5000 Beachcleaner**

bottom sediments. This is important because increased sediment compaction has been related to reduced percolation rates. OCWD tested the beachcleaner in two recharge basins. The unit was able to move rapidly and efficiently removed the thin clogging layer. Grain size analyses of material removed using scrapers and the beachcleaner showed that the material removed by the beachcleaner contained 22 percent more silt and clay than material removed by the scrapers. This shows that the beachcleaner is significantly more effective than the scrapers in removing the clogging layer while removing less clean sand. OCWD also found the beachcleaner easy to operate and able to operate when the sand underlying the clogging layer was not completely dry. This is important because basin recharge capacities are increased the sooner the basin can be cleaned and placed back into service. Based on these test results, OCWD purchased one unit and now uses it to clean the small basins that are less than 0.02 km<sup>2</sup> (5 acres) in size. OCWD is exploring ways to make the unit efficient for cleaning the larger basins, including using a conveyor system to convey the removed material to a dump truck running along side the unit. In addition to the beachcleaner, OCWD is exploring ways to better clean basin sidewalls and possibly rehabilitate the older recharge basins by stripping off the clogged material, cleaning it by processing it in a sandwash plant, and then placing the cleaned sediment back into the basin. This would be an expensive, lengthy undertaking, so OCWD is carefully studying the cost and benefits of such a basin rehabilitation program.

## **FUTURE OPTIMIZATION**

In 2005, OCWD prepared a Long-term Facilities Plan (LTFP) (OCWD, 2005) to identify potential projects that OCWD could implement in the future to further increase the

capacity of the groundwater basin. A total of 50 projects were evaluated, including projects involving MAR, new water supplies, water quality, and basin management. One of the drivers behind preparation of the LTFP was to have a plan in place that sequentially rolls out recharge enhancement projects that will become cost-effective as the value of water increases in the future.

Some of the MAR enhancement projects identified in the LTFP include vadose zone recharge wells, multi-lateral recharge wells (aka Ranney Wells), subsurface infiltration galleries under parking lots and parks, and desilting SAR water. In the near future OCWD will be evaluating the feasibility of desilting SAR water. Alternatives that will be considered will range from conventional treatment using flocculants to constructing subsurface collection galleries under portions of the SAR channel that do not provide much recharge due to shallow groundwater levels. The construction of new recharge basins is also considered in the LTFP, but the prospects of obtaining land for new recharge basins are slim due to high land values, the low availability of large parcels of land in Anaheim and Orange, and political resistance from local leaders who do not want to lose tax revenue on land converted to recharge basins.

In fall 2007, OCWD will complete construction of a 265,000 m<sup>3</sup>/day (70 million gallons per day [mgd]) treatment plant that will purify secondary treated effluent to near distilled quality using microfiltration and reverse osmosis treatment followed by advanced oxidation for organic compound removal and disinfection. Approximately half of the flow from the plant will be used to supply the seawater intrusion barrier along the coast with the other half conveyed via a 21 km (13 mile) pipeline along the banks of the SAR to Miller and Kraemer Basins in Anaheim. Due to the high quality of the water and extremely low TSS concentration, this water source is ideal for use in infiltration galleries, vadose zone recharge wells, injection wells, and multi-lateral recharge wells. OCWD will be conducting pilot testing of infiltration galleries using this water source in fall 2007 in order to obtain performance and design information that can be used to justify construction of injection facilities near the pipeline.

One key issue with the recharge of recycled water is proving compliance with California's Department of Health Services (DHS) regulations governing recycled water recharge. Currently, the regulations require a six-month travel time between the recycled water source and the nearest groundwater production well. OCWD had to conduct extensive groundwater tracer testing using various isotopes and sulfur hexafluoride (SF<sub>6</sub>) to define the area downgradient of Miller and Kramer Basin within which the travel time was less than six months. If similar studies are required for planned injection projects along the pipeline reach, the projects may not be feasible simply due to the high cost of conducting the studies, which would likely require the installation of numerous monitoring wells and collecting numerous groundwater samples for an extended period of time.

## CONCLUSIONS

Over the past 70 years, OCWD has constructed a large, complex surface water MAR system. OCWD's surface water MAR capacity is built on a foundation of large amounts of land. Due to urbanization in areas where surface water MAR activities are most efficient due to the underlying geologic conditions, further expansion by purchasing additional land is now extremely costly and politically challenging. Future increases in MAR capacity must come from optimizing existing facilities through better basin cleaning methods or desilting SAR water prior to basin recharge. In addition, OCWD must look to find innovative ways to recharge water that are consistent with the new urban landscape, such as infiltration galleries under parking lots, vadose zone injection wells, and multi-lateral injection wells.

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## ASR WELL EQUIPMENT, ABOVE AND BELOW GROUND

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### ABSTRACT

The growing installed base of aquifer storage & recovery (ASR) well equipment is broadening and accelerating our accumulation of operational data. Based on the author's growing field experience, this paper summarizes "lessons learned" and "best practices," with emphasis on practical examples. Specifically, it outlines above-ground considerations for well head facilities. Below-ground considerations center on equipment choices for pumps, column piping, valving, sensing and controlling ASR wells. Aimed primarily at consulting engineers, this paper discusses how these equipment decisions, in conjunction with well construction decisions, shape the investment and operation costs of projects, and ultimately their failure or success.

### KEYWORDS

Well head, pump, column pipe, valve, construction, operation, lessons, best practices

### INTRODUCTION

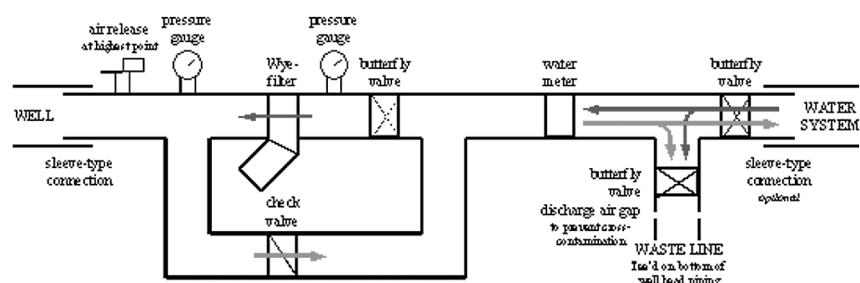
Over the past 25 years, the installed base of aquifer storage & recovery (ASR) well equipment has grown to more than 350 wells<sup>1</sup> in the USA. Typically, these either allow storage & recovery of fresh water "bubbles" in brackish, near-surface aquifers (e.g. Florida), or allow filling deeper aquifers that have minimal natural recharge. In 1992, the first downhole ASR Flow Control Valves (FCV) enabled access to these (500 feet and deeper) aquifers. The author has accumulated field experience with the majority of installations using downhole FCVs. Since 2000, the majority of ASR wells (approximately 150) had this type of valve installed. In the context of working with the consulting engineers, contractors, and operators of these installations, the author encountered designs, installations and operations that worked and didn't work<sup>2</sup>. Experiences ranged from delays due to basic failure to ensure that all materials and staff required for

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1. R. David Pyne: private communication, further to the preface of his book's Second Edition (Aquifer Storage Recovery – A Guide To Groundwater Recharge Through Wells, 2005).
  2. Hank Baski: Aquifer Storage and Recovery – We've learned a lot and now we're ready for ASR's growth (Jan 2007).

installation are on site, to errors in operation of established facilities. These experiences shaped the “lessons learned” and “best practices” presented below.

## ABOVE GROUND CONSIDERATIONS

From a design standpoint, the piping of wellhead facilities can be placed near or remote from the well, above ground, inside / outside a building, or in a vault. Vaults offer hidden piping with better protection from vandalism and freezing climate. The author prefers wells outside of buildings, making the well easy to service and the pump more easily accessible. Such designs are relatively easy for submersible pumps with a pitless unit. In warm climates, they are often equipped with a vertical turbine and above ground piping outside. The author has noticed that in a number of these locations the subsequent well work, such as pump removal and well redevelopment, can be difficult because of access to the well casing. The design should provide for access for a pump rig or crane. Make sure landscaping near the well will not inhibit removal of the pump and that vehicles have clear access by driveway, not over grass, flowers, bushes, etc. In addition, allow for use of column pipe wrenches with extensions up to 7 feet long. At 90-180° to the rig orientation, provide open space for “laying down” the pump column pipe. Wells inside of buildings make it more difficult and expensive to pull a pump or redevelop the well. However, you can plan to mitigate this. For example, make sure the building provides roof access over the pump. Some sites don’t – and have to cut roof access or even remove it entirely. Where a crane is used to service the well, enable the crane operator to have an unobstructed view of the well casing in order to allow a safe operation. This avoids reliance on hand signals by a third party, which is an added safety issue.



**FIGURE 1. Wellhead Piping (side view)**

Wellhead piping for ASR has evolved in recent years to simplify the configuration with the use of bi-directional flow meters and electronic valves. Figure 1 offers a simplified schematic side view of the well head piping, indicating the essential design

considerations. The valves channel metered injection and pumping flows between the water system and the well, providing for some filtering and line flushing operations. Generally speaking, the less-used line (injection or pumping) should follow the more circuitous route than the one used for the prevalent operation. Notably omitted from this schematic are a sediment sampling bypass line discussed below and surge relief valves.

### **Dresser-type Connections and Piping Size**

Where the pump column pipe meets the well head piping, a sleeve-type or telescoping (known in the trade as Dresser-type) connection facilitates disassembly and reassembly for ease of maintenance. The correct size of piping is the same size or one size larger than the pump column pipe (for example 8-inch or 10-inch well head piping for an 8-inch column pipe). All pressure ratings need to accommodate hydraulic hammer (pulses). Also, anticipate surges. For example, when power shuts off suddenly, the terrain may force the entire well field into reverse flow. The piping, valves, and fittings must have a pressure rating to handle the “shut-off head” of the well pump. If the “shut-off head” is very high, then a pressure relief valve may be needed in the piping system. The most common material for wellhead piping is ductile iron, steel, or cast iron.

### **Entrained Air and Dirty Flows**

An air release should be installed at the highest point near the well. A pump to waste line is essential to ensure that the first flush of dirty flow with sand, rust and similar debris can be discarded away from the system piping. The waste line should be attached to the bottom of the wellhead piping in form of a “Tee” with three valves and a water meter. In addition, a Wye-filter for injection flow should protect the downhole pump. The author learned of one instance where a foreign object of some sort made its way into a well and ruined a vertical turbine pump bowl assembly. Therefore, he recommends a Wye- or some other sort of filter for most installations. Injection water must be free of suspended solids. Otherwise, well plugging can occur - especially in sedimentary types of aquifers.

### **Magnetic Water Meter and Valves**

The main water meter is installed in such a way that it meters flows from the well, injection line, and well flushing (at a greater rate than injection) can be done with a second low cost meter. The main meter should be a magnetic type, which offers accurate, bi-directional measurement over a broad dynamic range. To ensure correct readings, adhere to manufacturer-recommended design rules for length and symmetry of piping to magnetic water meters. For shorter wellhead piping, straightening vanes may be used. Other meters are optional and prone to plugging with dirty water. Valves can be manually- or automatically-operated. Butterfly or

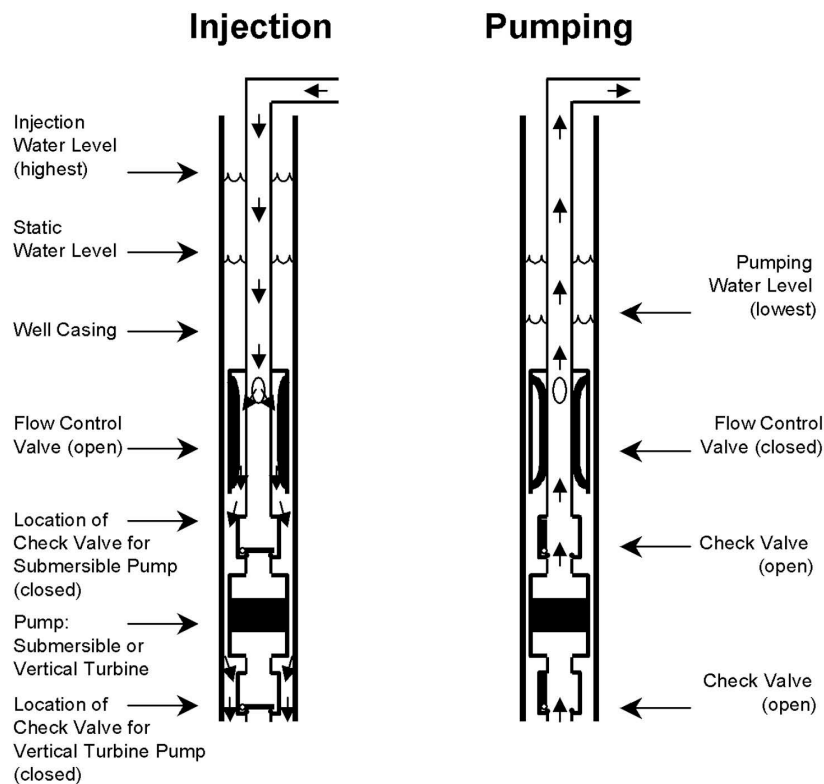
globe-style are the most common valves in this application. They are easy to operate and lend themselves to motorized operation.

### Sampling Ports and Optional Bypass Line

A smaller (for example, 1/2-inch) line with intakes from the center of the main well head piping and manual valves at either end is hooked up to bypass the wye-filter. The pressure drop across the filter drives a measured bypass flow through a small meter and a check valve sediment filter, to determine suspended solids.

## BELOW GROUND CONSIDERATIONS

Figure 2 shows the schematic operation of an ASR well equipped with a flow control valve. Note how during injection the well water level rises above the static water level and drops below it during pumping.



**FIGURE 2. Below-ground operation of a FCV-equipped ASR well (side view)**

## Pitless Units

A pitless unit replaces above-ground well housing and well pits, improving sanitation, convenience, frost protection, and vandalism security. Connected to the well casing underground, the pitless unit safely redirects water below the frost line. It can provide well completions that occupy little room in highly populated areas and, with a submersible pump, it is virtually noiseless. At some installations fake rocks cover the top of the pitless, making for an attractive completion in parks and public areas.

## Pumps

Table 1 illustrates important relationships of a pump and well casing size. Depending on well diameter and production (gallons per minute), the pump of choice may be a submersible or vertical turbine type. Vertical turbines operate at 1,800 RPM and 1,200 RPM for most ASR wells. They require a larger casing size than submersible pumps which typically operate at 3,600 RPM or 1,800 RPM. There are many other factors to consider when choosing a vertical turbine vs. a submersible pump. Vertical turbine pumps typically are not set deeper than 500 feet, but can be set as deep as 1,000 feet if necessary. Submersibles can be set as deep as 3,000 feet and deeper. With greater pumping depths, 3,600 RPM-submersibles require fewer stages. Oilfield submersibles are becoming commonplace in water wells, especially where higher power ratings are required (200 to 2,000 HP) for higher flow rates and deeper pumping levels. Both vertical turbines and submersibles at 1800 rpm are more efficient than 3600 rpm pumps.

Variable frequency drives (VFD) are useful to adjust pumping rates, both to start up and shut down well pumps, and to optimize their pumping rate during extraction. One must, however, avoid certain critical speeds which can result in vibration or unstable motor operation (check with the submersible pump manufacturer). Care must be taken at lower flow rates to ensure adequate flow past the submersible motor for proper cooling. Therefore, a flow-inducing shroud for the submersible motor may be needed, especially for deeper sets.

For submersible pump installations it is recommended that a heavy-duty ("armored") submersible electric power cable be used. This helps avoid damage to the cable. For both submersibles and turbines, a vibration analysis should be conducted. It helps to prevent excessive pump vibrations, which in some cases have worn holes through the well casing or screen. This vibration analysis also serves as a benchmark for vibration checks done on equipment that has been in use. If a vibration develops, it is important to pull the pump and fix the problem.

Use of oil-lubricated enclosed line shaft vertical turbine pumps can lead to oil collecting in the well. It is better to use a water-lubricated pump with an adjustable downhole flow control valve (FCV) and check valve. That allows for the vertical

turbine pump column pipe to be filled and its rubber bearings to be lubricated before it is turned on.

**Table 1. Casing I.D. vs. Pump Capacity**

| Nominal Pump Diameter (in) | Minimum Recommended Casing I.D. (in) | Maximum Pump Capacity (gpm) | Pump RPM                |
|----------------------------|--------------------------------------|-----------------------------|-------------------------|
| 4                          | 5 to 6                               | 100                         | 3,600                   |
| 5                          | 6 to 7                               | 250                         | 3,600                   |
| 6                          | 7 to 8                               | 400                         | 3,600                   |
| 7 ¼                        | 8 to 9                               | 700                         | 3,600                   |
| 8                          | 9 to 10                              | 1,000<br>600                | 3,600<br>1,800          |
| 10                         | 11 to 12                             | 2,500<br>1,500<br>700       | 3,600<br>1,800<br>1,200 |
| 11                         | 12 to 13                             | 4,000                       | 3,600                   |
| 12                         | 13 to 15                             | 2,500<br>1,500              | 1,800<br>1,200          |
| 14                         | 15 to 17                             | 3,500<br>2,500              | 1,800<br>1,200          |
| 16                         | 18 to 19                             | 5,000<br>3,500              | 1,800<br>1,200          |
| 20                         | 22 to 23                             | 7,000<br>5,000              | 1,200<br>900            |
| 24                         | 26 to 27                             | 9,000<br>7,000              | 1,200<br>900            |

For vertical turbine pump installations, it is important that the discharge head be installed with its axis parallel to the axis of the pump column pipe hanging in the well. That is, if the well casing is out of plumb the discharge head must also be out of plumb equal to the column pipe deviation, which is generally less than the casing deviation.

Some types of downhole flow control valves can shut-off drip-tight and thereby keep the column pipe filled. This helps to reduce corrosion and allows the column pipe to be filled with water before recharge / injection to prevent air from entering the aquifer. If injection water levels are at or near pipeline level, and the static water level is above 20 feet below pipeline level, downhole flow control devices are not needed. In any case, when using a submersible pump, a check valve should be installed to avoid reverse rotation of the submersible motor.

## Column Piping

Engineers designing the pump column, often choose the pipe diameter to reduce the flow velocity and head loss. It is important, however, to keep a minimum velocity of 3 ft per second in order to lift and pump out sand and sediment that the well may have produced. If the velocity is too low, the sand will collect above the bowl assembly, or check valve, and can form a plug, blocking off the flow of water.

The most common material for column pipe is mild steel. Fiberglass and PVC are seldom used, but can be used in shallow submersible applications. Epoxy-coated steel sounds good, but the author has found accelerated corrosion at “holidays” (breaks or damaged epoxy coating locations). This can be due to electrical current flow concentrated at small locations rather than potential current flow to a large area of non-insulated column pipe. In any case, stainless steel is probably the best, although expensive, material.

Thread galling can occur with steel pipe and is a more serious problem with stainless steel pipe threads. In the galling process metal transfers heavily from the male to the female connector and vice-versa. In the worst case, the threaded connection cannot be taken apart. Then, the coupling must be replaced and the pipe end rethreaded. Precautions to avoid galling include (1) accurately machining threads to a given specification and tolerance, (2) preventing damage to threads by proper handling of pipe and using thread protectors, (3) using a pipe joint compound with “anti-seize” qualities, (4) using an experienced pump crew for handling and for properly starting engagement of the threads. Furthermore, in the case of stainless steel threads, it is advisable to have a Higbee cut at the beginning of the thread. This eliminates a partial profile of a thread. Potential galling of stainless steel column piping is best avoided by using threaded couplings made from Nitronic-60 for threaded column, or using flanged column connections. A “spring loaded” hoisting block is also helpful. Some oil field submersible bowls have the impellers turn in the opposite direction of water well bowl assemblies; and therefore, the oil field motor turns in the opposite direction, such that the column pipe couplings are tightened.

When vertical turbine bowls are driven by a submersible motor, pipe couplings can unscrew themselves, due to torque of the motor. To prevent this from occurring, make sure joints are tightened properly to API specifications. The author does not recommend welding straps across couplings to avoid unscrewing of couplings. For tight vertical turbine pump installations, the author recommends use of over-the-coupling protectors to prevent damage to downhole control lines, pump cable, transducer cables, airlines, and any other downhole instrumentation cables.

Carefully consider joint strength for the pump column pipe whenever the pump is set deeper than 500 feet. The author does not recommend an eight sharp “V” thread or NPT thread, either tapered or straight, for such depths unless special precautions are taken to ensure adequate threaded joint strengths. For submersible pumps set

deeper than 500 feet, a good policy is to use oilfield casing or tubing for which joint strengths are published. The author is not aware of any official publications with joint strengths for NPT (National Pipe Taper) threaded connections. Therefore, he had an engineer at Baski Inc. calculate these<sup>1</sup>. For vertical turbine pumps set deeper than 500 feet, the author recommends that a higher strength steel column pipe be used along with an adequate wall thickness. In a discontinued recommendation, the American Water Well Association (AWWA) listed Schedule 30 pipe for vertical turbine pump column. Such a wall is too thin for depths greater than 500 feet, especially when using mild steel pipe with yield strength of 30,000 psi. Oilfield casing material has minimum yield strengths of from 55,000 to 110,000 psi. Keep in mind that to the author's knowledge, currently there are no standard specifications or tolerances for vertical turbine pump column threads. The author strongly feels this situation needs to be rectified.

The downhole check valve is important and needs to seal drip tight. This ensures that the pump column pipe stays full. There are some vertical turbine foot / check valves made which are designed to leak. Do not use these, as they can leak from 100 to 1,000gpm.

Sometimes a hole can develop in the pump column pipe. When this happens, the water jet from this hole can erode holes through the casing or well screen. Such holes result in gravel or formation entering the well and are expensive to fix. When an ASR well is new, it is useful to verify the tightness or potential leak capacity of the check valve, downhole Flow Control Valve and column pipe. This can be done by measuring the decline of the water level in the column pipe. It can also be measured using the bypass line on the above ground piping. Its small water meter can detect low flows caused by leakage.

Chloride corrosion has occurred in some well installations, due most likely to excess chlorine concentrations during installation of the well and of the pump. It is recommended that chlorine concentrations be less than 100 ppm. The author has encountered sites where the contractor was going to disinfect the pump and column pipe with a 10% chlorine solution. This is 100,000 ppm and a 1,000 times higher concentration than recommended. After the 100 ppm chlorination, the well should be pumped and a po water supply having a chlorine residual of 1-2 ppm be used to flush the annulus between the pump column and the casing with 50 to 100 gpm.

Sometimes, engineering solutions for perceived problems create real ones. In one case, fiberglass pump column nipples were specified above and below a Flow Control Valve. This electrically isolated the valve from the column pipe, apparently, in an effort to prevent corrosion between dissimilar metals of the stainless steel Flow

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1. Available via download of Catalog 6, p.18 from <http://www.baski.com>



Control Valve and the steel column pipe. This resulted in some chloride corrosion of downhole parts due to a high chlorine concentration and a naturally-occurring, spontaneous “potential voltage” within the well casing. To date, the author has never seen any corrosion problems between threaded stainless steel and steel connections. There have been, however, documented corrosion cases involving stainless steel screen connections to steel well casings. This is probably due to electrical currents with the earth acting as a battery and contacting the two different materials. This problem is avoided by use of a “dielectric” insulated connection. It apparently does not happen to pump column pipe, which is within the well casing and is surrounded by either air or water.

Finally, the author advocates applying a vacuum (approximately 1/3 psi absolute) to the well casing pump column annulus. This procedure is especially important for ASR wells. The vacuum in the unsaturated portion of the aquifer allows the water level to move up and down freely with water vapor in the unsaturated zone. Oxidation is decreased in the casing and aquifer as air is removed and replaced with water vapor. Biological growths can also be largely curtailed.

## **CONTROL AND OPERATION**

### **Sensing and Controlling Equipment**

Most ASR wells are controlled automatically rather than manually. The control system is typically a part of an existing Supervisory Control And Data Acquisition (SCADA) or Programmable Logic Control (PLC) system. The logic for controlling recharge and pumping can be “tricky”. The inputs to consider are flow rate, well water level, pipeline pressure, FCV throttling position, reservoir or storage tank level, and other variables that may be unique to the location / operation. At new locations, the author has invariably found that there are problems with the logic programming or choice of components. This applies not only to standard operating procedures, but even more so to emergency procedures and contingency planning for various failure modes.

In addition to the logic and programming for automatic operations of an ASR well, it is important to avoid problems by having alarms for the following conditions: (1) high injection well water level, (2) minimum pipeline pressure, (3) minimum tank or reservoir level, (4) minimum and maximum pumping flow rates, (5) minimum and maximum injection flow rates, (6) pump on with no water being pumped at the surface, as indicated by the water meter. During injection operations, it is important not to “overflow” or “flood” the well.

Manual operations in particular are subject to operator errors, such as the following incident during injection (refer Fig. 2). At a deep well with a static water level 1000 feet below the surface, the operator started injecting water through the 7-inch

column pipe at a high rate without closing the downhole Flow Control Valve. The air in the column pipe was compressed and pumped the well by air-lifting. Geyser-like, water shot out of the vent cap of the pitless unit. In addition air and water were forced into electrical conduits leading to various controls and a power supply transformer.

During pumping, a downhole Flow Control Valve must be in the closed or non-injection position. Occasionally, through various kinds of errors, a pump is operated with the Flow Control Valve open. This circulates and heats the water downhole. In one case, the water temperature rose to approximately 270 F (substantially above atmospheric boiling temperature!) and softened a PVC sounding tube resulting in a coiled and tangled mess. Fortunately, in this case, the downhole Flow Control Valve and vertical bowl assembly were not damaged.

The overall pumping efficiency of a new well should be determined. This ensures that a properly sized and efficient pump is used. In addition, it is a benchmark for determining pumping efficiencies at a later date. Poor pumping efficiencies will cause energy costs to be much higher than necessary. While there is currently no single-well method for determining well efficiency, the author also advocates determining well efficiencies (in addition to pumping efficiencies) as soon as such a single-well method can be established.

ASR wells can be prone to plugging by four factors: (1) suspended solids from injected water, (2) air binding due to introduction of air at the beginning or during an injection cycle, (3) biological fouling due to existing contamination in a well and/or the water being injected, (4) chemical precipitation within the aquifer due to chemical reactions between formation material of the aquifer, native and injected water. For example, under cold conditions such as of Northern Minnesota where the groundwater temperature is 40 F, there is less biological fouling as compared to warmer climates. However, Iron bacteria can still cause fouling. In the Phoenix area, where the groundwater temperature can be about 70 F, the author has encountered a much greater problem with biological fouling. While some preventive measures are design choices (e.g. filters to capture suspended solids), many are operational in nature. For instance, the introduction of air can be avoided by having the pump column filled with water before injection and keeping the pipeline pressure at the surface greater than 2-5 psi to avoid introduction of air. In this context, a downhole Flow Control Valve is key to avoid the introduction of air, to prevent cascading, and to suppress cavitation within the pump column.

Before each injection / pumping phase, first clear lines to waste, making sure that the flow rates are higher than during actual injection / pumping and thereby minimizing later movement of debris (into the well or water system). Remember that injection and pumping are mild forms of well development.

The author has seen installations where much thought was given to the components, instrumentation, and various equipment. Once the system is installed, it is important to also collect data, to analyze it, and to adjust operations accordingly. In addition, it is important to maintain the equipment in addition to maintaining the well. It is important to monitor the injection specific capacity. This information is useful to determine how often back-flushing or maintenance pumping needs to be done to keep the well in shape. The author has seen where maintenance pumping has been as frequent as three times a day to as infrequent as once every six months. Especially during such maintenance pumping, the pumping specific capacity should also be monitored.

In closing, the author gratefully acknowledges the careful review of this paper's manuscript by C. Hemenway and S. Orr, and the many expert comments and thoughtful suggestions derived from ASR experiences.

## AN OVERVIEW OF ARIZONA'S RECHARGE PROGRAM

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### ABSTRACT

In 1986, the Arizona Legislature established the Underground Water Storage and Recovery Program which is administered by the Arizona Department of Water Resources (ADWR). This program promotes the underground storage and use of the State's renewable water supplies, such as effluent, surface water, and Colorado River water, instead of non-renewable groundwater. There are two types of Underground Storage Facilities (USFs) permitted by ADWR; constructed and managed. A constructed facility is designed to store water underground by methods such as basins, trenches, vadose zone wells, and/or direct injection wells. A managed facility utilizes the channel of a stream to store water through the release and infiltration of renewable waters not part of the natural flow of the channel. Currently, 53 constructed and 6 managed USFs are permitted by ADWR. The annual permitted recharge volumes for these USFs range from 67 acre-feet (ac-ft) ( $8.3 \times 10^4 \text{ m}^3$ ) per year up to 200,000 ac-ft ( $2.5 \times 10^8 \text{ m}^3$ ) per year. ADWR has encountered unique and challenging issues when permitting each USF. This paper will present a sampling of Arizona's USFs, showcasing the methodologies used and the unique issues and challenges associated with each facility.

### KEYWORDS

Artificial Recharge, Arizona, Arizona Department of Water Resources, ASR, Basins, Vadose Zone Wells

### INTRODUCTION

For Arizona, artificial recharge is a very important water management tool especially for meeting the goals of the 1980 Arizona Groundwater Management Code. The Arizona Legislature enacted the Groundwater Code for several reasons – to help control the severe groundwater overdraft in certain areas of the State, called Active Management Areas or AMAs, to provide a framework for the allocation of the State's groundwater resources and to offset the use of groundwater and promote the use of renewable water resources. To accomplish these goals the legislature created the Arizona Department of Water Resources and, in later legislation, Arizona's recharge program in the form of the Underground Storage and Recovery Act. The recharge program was legislatively modified twice to the existing Underground Water Storage, Savings and Replenishment Program that was passed in 1994. The Arizona Department of Water Resources (ADWR) administers this program with consultation for water quality issues from the Arizona Department of Environmental Quality (ADEQ). Arizona's recharge program promotes the use of renewable water supplies,

particularly Arizona's entitlement to Colorado River water, instead of groundwater through a flexible and effective regulatory program for the underground storage, savings and replenishment of water. The goals of the recharge program are to promote the use of renewable supplies, to augment the water supply, to help reduce groundwater overdraft, to provide for the efficient use of all water resources by allowing water to be "transported" by storing water in one location, but recovering a like quantity elsewhere and to accommodate seasonal demand for water through recharge and recovery.

While the State actively encourages recharge, ADWR, through the recharge program, imposes administrative and hydrologic requirements on entities applying for and operating a USF. As part of the USF permit application process, applicants are required to submit extensive technical information to demonstrate that the full permitted volume for the proposed USF will be hydrologically feasible and will not cause unreasonable harm to current surrounding land and water users. Also part of the application process, the applicant must demonstrate that the proposed USF is sited and will be operated such that recharge is the principle purpose of the facility. Unlike incidental recharge from agricultural irrigation or leakage from manmade bodies of water, the primary purpose of any permitted recharge facility must be to add water to the underlying aquifer. Recharge program policy does allow facilities to have multiple uses but recharge must always be the primary purpose. Once an application is approved and a USF permit is issued, the permittee is required to schedule a pre-recharge inspection with the Department. As part of this requirement, the Department inspects the facility for compliance with the issued USF permit after all associated recharge and monitoring components have been built or installed and prior to recharge water being delivered to the facility. As soon as the Department has given the permittee approval for recharge to commence, the permittee must comply with a series of on-going operating, monitoring and reporting requirements which include submission of quarterly and annual facility reports to the Department. After the permittee's reporting has been reviewed for permit compliance, the Department issues long-term storage credits for the water that has been stored and is eligible to be recovered.

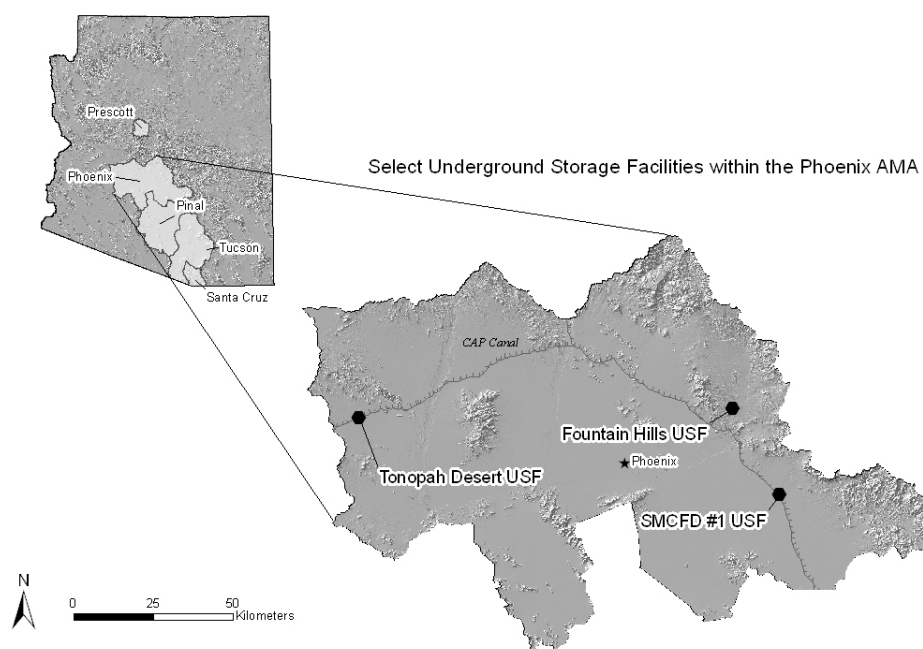
There are two types of Underground Storage Facilities permits issued by ADWR, constructed and managed. A constructed facility is designed and constructed to store water underground by various methods such as direct injection and/or vadose zone wells, trenches or basins. A managed facility is designed to utilize the natural channel of a stream to store water through the artificial and controlled releases of water other than naturally present surface water. Facilities can be permitted to recharge effluent, surface water, and/or CAP water which is the portion of Arizona's allocation of Colorado River water delivered through the Central Arizona Project (CAP) canal. Since the creation of the recharge program in 1986, ADWR has issued 170 USF permits including permit modifications throughout the state of Arizona. Currently under the recharge program, there are 59 active USFs, 53 constructed

facilities and 6 managed facilities, most of which are located within AMAs. Of the State's permitted constructed recharge facilities, 27 utilize basins for recharge, 16 utilize injection and/or vadose zone wells, 1 facility uses trenches and the remaining 15 facilities use a combination of these recharge methodologies. The permitted volumes for the active recharge facilities range from 67 ac-ft ( $8.3 \times 10^4 \text{ m}^3$ ) per year to 200,000 ac-ft ( $2.5 \times 10^8 \text{ m}^3$ ) per year. Of those 59 facilities, 32 are permitted to recharge effluent, 18 are permitted to recharge CAP water and 9 recharge a combination of source waters. To date, over two million ac-ft ( $2.5 \times 10^9 \text{ m}^3$ ) of water has been stored at permitted USF's statewide.

## DISCUSSION

Each recharge facility site has its own distinct hydrogeology, geomorphology and geochemistry. As a result, ADWR has encountered unique and sometimes challenging issues permitting and regulating each USF. The five facilities discussed below provide a representative cross-section of various facility types that ADWR regulates along with a small sampling of the issues that ADWR and permittees have tackled during the program's history.

Arizona Active Management Areas



**FIGURE 1. Shaded relief map displaying the location of the Phoenix AMA within the State and in proximity to three recharge facilities located within the Phoenix AMA.**

One of the newest and largest recharge projects recently permitted by the Department is the Tonopah Desert Recharge Project, which is located west of Phoenix approximately 40 miles (64 km), adjacent to the CAP canal (Figure 1). The Central Arizona Water Conservation District (CAWCD) is the permittee and operator for this project as well as 7 other large recharge facilities in the Phoenix and Tucson AMAs. Prior to building the Tonopah USF, CAWCD undertook a comprehensive study to identify and characterize locations within the region that were ideal for recharge. After three years of trenching, exploration borehole drilling, and infiltration testing, the Tonopah Desert location was ultimately selected for the District's next large scale recharge project (cap-az.com, 2007).

The USF consists of 19 recharge basins with over 200 acres ( $0.8 \text{ km}^2$ ) of recharge capacity (Figure 2) (cap-az.com, 2007). The Tonopah USF's permitted volume for the first 4 years of the facility's permit is 150,000 ac-ft ( $1.9 \times 10^8 \text{ m}^3$ ) per year. The annual permitted volume is then reduced to 87,500 ac-ft ( $1.1 \times 10^8 \text{ m}^3$ ) per year for the duration of the 20-year permit. Typically, recharge facilities are permitted to recharge a fixed annual volume for the entire duration of their permit. The "phased" permitted volume requirement at the Tonopah USF is primarily a result of the potential for greater source water availability during the first four years of USF operation.

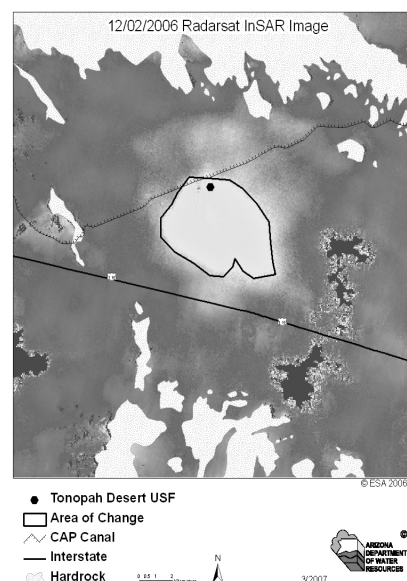


**FIGURE 2. CAWCD's Tonopah Desert Recharge Facility looking northeast. The CAP canal which delivers the recharge water to the facility is in the background (arrow). (Photo courtesy of CAWCD)**

In 2006, the facility's first year of operation, the permittee recharged 129,992 ac-ft ( $1.6 \times 10^8 \text{ m}^3$ ) of CAP water. The facility had a maximum infiltration rate of 12.5 feet per day (3.8 m per day) and an average infiltration rate of 5 feet (1.5 m) per day. As an added benefit of these exceedingly high infiltration rates, the facility reported a very small amount of evaporative loss, 675 ac-ft ( $8.3 \times 10^5 \text{ m}^3$ ) or 0.5% of the 2006 delivered volume (CAWCD, 2006). Based on the facility's performance during 2006, in future years the facility could potentially recharge its full

annual permitted volume within 6 months.

The facility has been so successful at recharging water that the groundwater mound created beneath the facility during its first year of operation was identified through the use of InSAR (Interferometric Synthetic Aperture Radar) images. InSAR is a satellite-based radar imaging system that uses interferometric analysis to detect and measure land surface deformation, such as subsidence or uplift, with a vertical centimeter-scale resolution. Figure 3 shows the lateral extent of land surface uplift in the area around the recharge facility outlined in black. This InSAR data also indicates that there has been a maximum of 1.6 cm of land surface uplift. One potential explanation for this observed land surface rise is the wetting of expanding soils in the area under and around the recharge facility (B. Conway, pers. comm., March 20, 2007).



**FIGURE 3. InSAR image showing the lateral extent of land surface rise due to recharge at the Tonopah Desert USF.**

The Tonopah USF encountered an unexpected water quality problem shortly after commencement of recharge at the facility. Nitrate concentrations in two monitor wells at the facility exceeded the State's Numeric Aquifer Water Quality Standards (NAWQS) for nitrate almost immediately after recharge operations began. This problem was unusual and unexpected because nitrate concentrations in the CAP source water as well as groundwater samples collected from the facility's monitor wells prior to recharge showed nitrate levels less than 2.0 mg/l.

CAWCD initiated recharge at Tonopah on January 24, 2006. Groundwater samples collected from two of the facility monitor wells in March, 2006 showed nitrate concentrations exceeding the nitrate NAWQS of 10mg/l; MW-1 had a concentration of 57 mg/l and MW-2 had a concentration of 25 mg/l. By that time the facility had recharged approximately 21,700 acre-feet ( $2.7 \times 10^7 \text{ m}^3$ ) of water which produced a water level rise of over 100 feet (31 m) in these two monitor wells.

Flushing of the vadose zone is very common at the start-up of a recharge facility, especially when the facility is located on land that supported previous uses, such as farming. However, the Tonopah facility was constructed on native desert. Based on the amount of water recharged and the lack of nitrate in the source water and groundwater prior to recharge, the permittee has suggested that the source of the nitrate was, most likely, from naturally occurring nitrate in the unsaturated zone



beneath the facility that was dissolved and carried to the regional aquifer (CAWCD, 2006).

In compliance with their USF permit, CAWCD notified ADWR of this exceedance, increased their water quality sampling frequency and kept ADWR updated on the status of the nitrate issue at the facility as well as their plan for and progress in addressing the situation. Because the Tonopah Recharge facility is not located near any water production wells or other water users, the permittee continued to recharge at the facility to flush the nitrate out of the vadose zone and dilute the existing nitrate in the groundwater. To accomplish this, CAWCD moved more recharge water into the basins nearest MW-1. And as a result, the April, 2006 sample showed a nitrate level in MW-1 of 6.6 mg/l, below the NAWQS. Unfortunately, the nitrate level in MW-2 increased to 84 mg/l rather than decreased in the April sampling. Recharge activities were then moved to the basins closest to MW-2 and since that time all monthly samples collected from MW-2 have shown a continued decrease in the nitrate level. By

November, 2006 the nitrate concentration in groundwater collected from MW-2 had dropped below the NAWQS (CAWCD, 2006).

The Superstition Mountains Community Facility District #1 Recharge Facility (SMCFD #1) is located approximately 36 miles (57.9 km) east of Phoenix (Figure 1). ADWR permitted the USF in 2005 with an annual recharge volume limit of 2,352 ac-ft ( $2.9 \times 10^4 \text{ m}^3$ ) per year for 20 years. The USF recharges treated effluent generated from the wastewater treatment plant adjacent to the recharge facility using basins and vadose zone wells located within the basins (Figure 4). There are 18 permitted USFs in Arizona that use vadose zone wells as a method of recharge. However, SMFCD #1 is one of the few permitted USFs utilizing this unique combination of vadose zone wells within basins. This methodology can increase recharge capacity at a facility as well as provide operational flexibility for permittee.

Initially, the facility operated with 3 recharge basins and 6 vadose zone wells (2 wells in each basin) but in late 2006, after inspection by ADWR, 7 additional basins and 14 additional vadose zone wells were put into operation. Unlike the first three basins, the last 7 basin/vadose zone well combinations were constructed so that recharge water is piped through a control valve located at the edge of each of the basins and then directly into the vadose zone wells located within each of the basins (Figure 5). This set-up gives the permittee even more operational flexibility. Flow can be controlled so that only the vadose zone wells are used for recharge. Or flow can be increased so that recharge water overfills the vadose zone wells and flows into basins thus utilizing both the basin and the vadose zone wells for recharge.



**FIGURE 4. Basin 2 at The Superstition Mountain Community Facility District #1 Recharge Facility. Vadose zone wells 3 and 4 are located within the basin in the foreground and background respectively. (Photo by ADWR, 2005)**



**FIGURE 5. Looking north into Basin 5 at the Superstition Mountain Community Facility District #1 Recharge Facility. The flow control valve in the foreground (arrow) controls flow into the vadose zone well seen within the basin in the background of the photo. (Photo by ADWR, 2006)**

The permittee modified the plumbing to the basins and vadose zone wells in response to lower than expected infiltration rates and algae development in the basins. Another advantage to operating the facility in this manner is reduced evaporative losses. In 2005, the District delivered 704 acre-feet ( $8.7 \times 10^5 \text{ m}^3$ ) of treated effluent to the recharge facility. Based on the estimated daily basin wetted area, the permittee calculated an evaporative loss of only 7.41 acre-feet ( $9.1 \times 10^3 \text{ m}^3$ ) or 0.14% for the year (SMCFD#1, 2005). In an effort to control the algae growth, the District is exploring other options such as frequent basin rotations and the potential use of algacide, but currently the most effective means of algae control within the basins is for the permittee is to restrict recharge to the vadose zone wells. Despite this restriction, use of the vadose zone wells will inevitably lead to clogging and lower recharge rates at the facility. This biological fouling or clogging is a common problem encountered by vadose zone wells. While there have been some techniques developed for rehabilitating vadose zone wells, the clogging will ultimately reduce long-term recharge rates.

The Fountain Hills Recharge Project is located approximately 32 miles (52 km) northeast of Phoenix (Figure 1) and is operated by the Fountain Hills Sanitary District (FHSD). The recharge project was designed utilizing Aquifer Storage and Recovery (ASR) wells (Figures 6 & 7). ASR wells are used to inject and store treated effluent underground with the ability to later use the same well for water recovery. FHSD uses the recovered water on the Town's parks, golf courses and turf areas. As a result of limited land availability, Fountain Hills needed a recharge methodology

such as vadose zone or direct injection wells that would not require large amounts of space. The ASR recharge methodology was ultimately selected because of an approximately 400 foot (122 m) thick clay-rich layer present beneath the facility. The ASR wells were constructed to a depth of 760 ft (232 m) below ground surface, penetrating through this clay-layer and into the regional aquifer below (HydroSystems, Inc., 2003).



**FIGURE 6. Fountain Hill's ASR well. View of well from inside the vault. (Photo courtesy of HydroSystems, Inc.)**



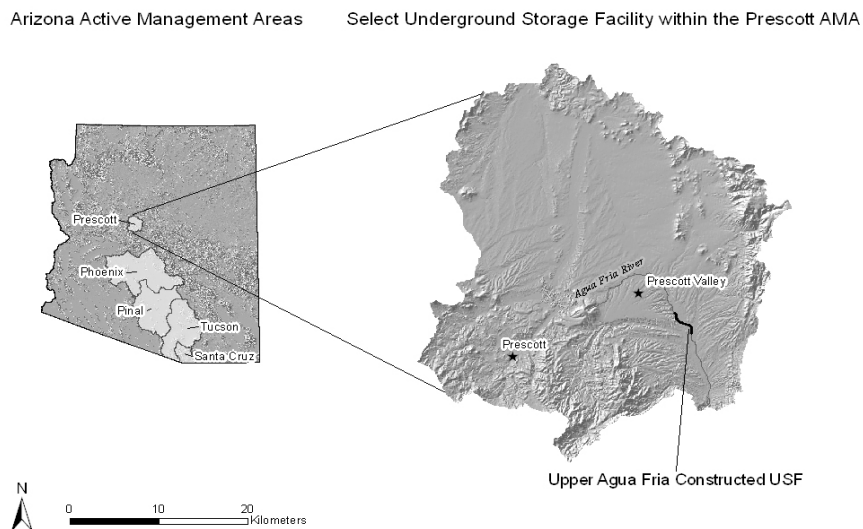
**FIGURE 7. Fountain Hill's ASR completed vault to well which is located underground. (Photo courtesy of HydroSystems, Inc.)**

To test the suitability of the recharge site and methodology, the facility was permitted in 2000 with a 2-year USF permit. The facility was originally permitted to recharge a maximum of 1,904 acre-feet ( $2.3 \times 10^6 \text{ m}^3$ ) per year. This short duration USF permit not only allowed the permittee the ability to test the effectiveness of the recharge methodology and location, but it helped demonstrate the hydrologic feasibility of the project and gave the applicant an opportunity to collect data to be used in support of a long-term, full-scale USF permit. Based on the permit-required reporting submitted by the permittee during this test phase, the FHSD consistently maintained injection rates of 200-500 gallons per minute ( $0.8\text{-}1.9 \text{ m}^3$  per minute) per well (HydroSystems, Inc., 2003). In 2004, the FHSD applied for and received a 20-year USF permit with an increased permitted volume of 2,241 ac-ft ( $2.8 \times 10^6 \text{ m}^3$ ) per year.

Maintenance of ASR wells is generally easier than with vadose zone recharge wells due to the ability of ASR wells to recover water and flush the wells. However, water quality issues may arise as a result of the treated effluent being directly injected into the aquifer. Because there is no soil aquifer treatment of the source water with ASR as there is with basin and vadose zone well recharge, water quality has to be scrutinized more so than with the other methods. ADEQ is responsible for reviewing and overseeing the water quality portion of facilities that recharge treated effluent

through an Aquifer Protection Permit (APP). The Fountain Hills facility had a reported APP exceedance for Total Trihalomethanes (TTHMs), a chlorine disinfection by-product, in one of the facility's water quality monitoring wells during the first and second quarters of 2005. In an effort to correct this problem, FHSD constructed a UV disinfection system to replace chlorination of the reclaimed water that is recharged. The UV system began operation in March, 2006 (HydroSystems, Inc., 2006).

In 2004, ADWR permitted the Prescott Valley Upper Agua Fria Recharge Project located 92 miles (148 km) north of Phoenix (Figure 8). The Town of Prescott Valley, as the permittee, converted approximately 4.5 miles (7.2 km) of an ephemeral reach of the Agua Fria River into a constructed in-channel recharge facility by building small dirt berms within the channel approximately every 235 feet (72 m) (Figure 9) (Montgomery, 2005). Treated effluent from the Town's wastewater treatment facility is discharged into the channel. The berms increase the sinuosity of the river channel which slightly pond and slow down the flow of recharge water resulting in increased infiltration.



**FIGURE 8. Shaded relief map displaying the location of the Prescott AMA within the State and in proximity to one of the recharge facilities located within the Prescott AMA.**



**FIGURE 9. Town of Prescott Valley Upper Agua Fria Constructed Recharge Facility. Looking downstream at a berm located in the constructed in-channel recharge facility. The arrow indicates where the end of the berm is located which allows water to flow past and on down to the next berm.**

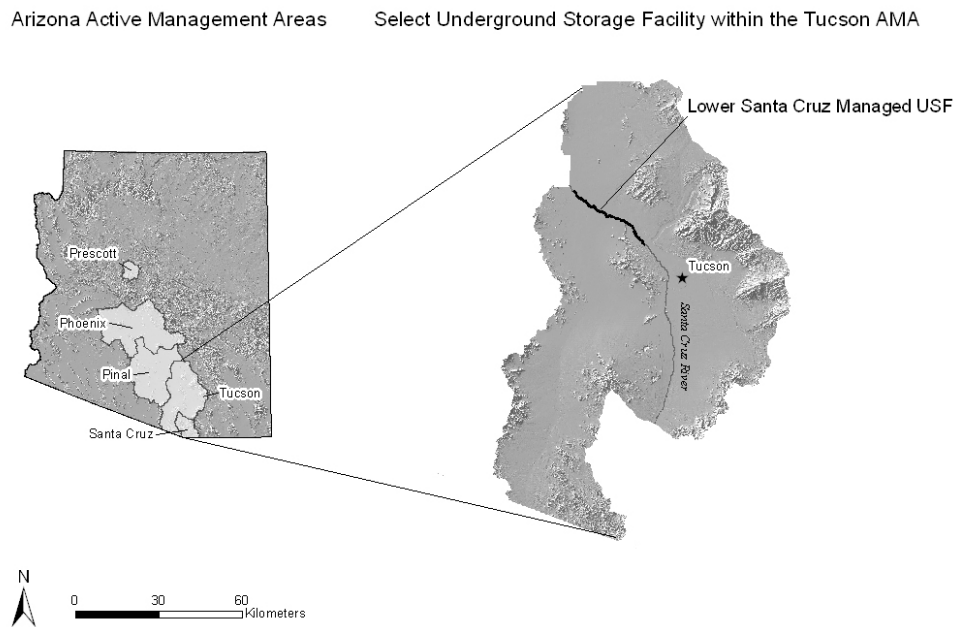
The operation and maintenance of this “in-channel” constructed USF has proven to be difficult for the permittee for several reasons. Because this facility is built in a river channel, the berms were designed and built to wash out when natural surface flow in the channel is greater than approximately  $200 \text{ ft}^3$  ( $5.7 \text{ m}^3$ ) per second (Montgomery, 2005). Relatively small precipitation events in this mountainous part of the state can generate significant flows in even minor washes. As a result, storm flows frequently wash out the berms within this USF. In addition, the incised river channel limits access to the facility for berm re-building equipment. During times when there are natural flows through the facility channel or berms have been washed out, the permittee

is non-compliant with their USF permit and is not eligible to earn long-term storage credits until natural flows through the facility cease and/or berms are rebuilt. Another operational challenge the permittee has had to deal with at this facility is the freezing of the recharge water in-channel. This facility is located at an elevation of approximately 4800 ft (1463 m) above mean sea level and during winter months it is not uncommon for temperatures to be below freezing. At those temperatures, some of the effluent in the channel freezes which greatly reduces the infiltration and often results in the recharge water flowing past the end of the facility. This also results in permit non-compliance and loss of credits for the permittee. During 2005, the facility was in operation, by permit standards, 217 days recharging a total of 1,918 ac-ft ( $2.4 \times 10^6 \text{ m}^3$ ) of their permitted annual volume of 5,150 ac-ft ( $6.4 \times 10^6 \text{ m}^3$ ) (Prescott Valley, 2005). In 2006, the facility operated 259 days and recharged 2,143 ac-ft ( $2.6 \times 10^6 \text{ m}^3$ ) for the year (Prescott Valley, 2006). In response to these maintenance and operational challenges, the permittee is currently looking at potential locations for off-channel recharge sites.

In contrast to constructed USFs, a managed USF is designed to utilize the natural channel of a stream to store water through the release and subsequent infiltration of treated effluent or other renewable water supplies that are not part of the natural flow of the stream or river. Water is released into a river channel or wash and allowed to infiltrate as it flows down the channel without the addition of any constructed components to increase that infiltration. One example of a managed USF

is the Lower Santa Cruz River Managed Recharge Facility. This facility is located in a 17.9 mile (29 km) stretch of the Santa Cruz River channel within the Tucson AMA (Figure 10) approximately 94 miles (151 km) southeast of Phoenix. Treated effluent from two Tucson-area wastewater treatment plants is discharged to the Santa Cruz River and allowed to infiltrate as the effluent flows down the channel. The facility is permitted to recharge 43,000 ac-ft ( $5.3 \times 10^7 \text{ m}^3$ ) per year with a maximum in storage limit of 680,000 ac-ft ( $8.4 \times 10^8 \text{ m}^3$ ).

Significant vegetation has developed along the river from and is sustained by the effluent flows (Figure 11) that these wastewater treatment plants have continuously discharged into the river channel since the early 1960's. As a result of this, during the application process the applicant used data from a USGS study of the project area to determine how much recharge water would be lost to transpiration and how those losses would be accounted for in the facility's required reporting. The Department and the applicant agreed on a fixed loss rate of 2.9726 ac-ft per day ( $3.7 \times 10^3 \text{ m}^3$ ) to be used to calculate the combined transpiration and evaporation losses for the facility (LSCMRP, 2001).



**FIGURE 10. Shaded relief map displaying the location of the Tucson AMA within the State and in proximity to one of the recharge facilities located within the Tucson AMA.**



**FIGURE 11. Looking downstream at a portion of the Lower Santa Cruz River Managed Recharge Facility.**

In 2005, 20,118 ac-ft ( $2.5 \times 10^7 \text{ m}^3$ ) was delivered to the facility and the permittee calculated 999 ac-ft ( $1.2 \times 10^6 \text{ m}^3$ ) of transpiration/evaporation losses or 5% of the discharged volume. The net volume recharged for 2005 was 19,119 ac-ft ( $2.4 \times 10^7 \text{ m}^3$ ). (LSCMRP, 2005) For 2006, the facility delivered 20,888 ac-ft ( $2.6 \times 10^7 \text{ m}^3$ ) with 1,043 ac-ft ( $1.3 \times 10^6 \text{ m}^3$ ) lost by transpiration/evaporation for a net recharged volume of 19,845 ac-ft ( $2.4 \times 10^7 \text{ m}^3$ ) (LSCMRP, 2004). That represents 5% of the delivered

volume lost by transpiration/evaporation.

The calculation and reporting of infiltration rates is a condition of all USF permits. Compilation and reporting of this data serves two purposes. First, it provides data that could potentially indicate when a facility requires maintenance, such as basin rotation, drying and discing or well rehabilitation, thus making a more effective and efficient USF. Second, it provides ADWR a means to evaluate the on-going hydrologic feasibility of the recharge project as well as an objective and analytical way to ensure that a recharge facility's primary purpose remains adding water to the underlying aquifer. But for managed facilities, on-going maintenance by the permittee can not occur and this can affect its ability to recharge water. However, the Lower Santa Cruz River Managed Recharge Facility has identified a natural mechanism for facility maintenance resulting from storm water periodically flowing through the facility. In 2002, the permittee documented an increase in the facility's infiltration rate during the second and third quarters that corresponded to an increase in storm activity and flows through the facility during those months. Conversely, the permittee observed a decrease in infiltration rates after the cessation of monsoon storms and the resulting storm flows (Thomure and Wilson, 2003). This correlation was also observed at this facility after a large storm flow event in August of 2005 (Thomure, 2007). According to the permittee, this correlation suggests that during these storm flow events, the river channel bottom is scoured by these high flows and this removes fine-grained sediments and organic matter that have settled out and serve as a barrier to recharge (Thomure and Wilson, 2003). While the recharge facility is not eligible for credits during the times that storm water is flowing through the facility, the scouring that occurs during storm flow events can be a valuable and effective means of facility maintenance at a managed USF.

## CONCLUSIONS

Every recharge methodology has advantages and disadvantages and every proposed USF location is unique. As illustrated by the previous five recharge facility examples, the effectiveness and efficiency of a USF depends largely on combining a properly located USF site suitable for recharge with the appropriate recharge methodology and the employment of proper facility operation and maintenance. But not every issue can be anticipated. For Arizona, the intensive USF permit application process incorporates the data collected for the proposed USF site by the applicant with the recharge program's experiences and requirements in order to resolve or at least minimize these issues before a facility begins recharging. Because of this, the recharge program has developed into a meaningful and vital water management tool for the State.

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## ASR PROGRAM KEY TO WATER RESOURCES MANAGEMENT

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### ABSTRACT

Chandler, Arizona is one of the fastest growing communities in the United States among cities with populations over 150,000. Location is a major factor in Chandler's prosperity, offering many advantages to existing and new businesses and industries, with a reputation as a global leader in technology. Projected buildout population estimates range from 280,000 to 290,000, which could occur in the next 15 years.

From a water resources stand point, Chandler manages 100 percent of its reclaimed water through exchange, recharge and recovery, and direct reuse. At build out Chandler is expected to produce 32,500 acre-feet/year (AF/yr) of reclaimed water, of which 15,700 AF/yr is dedicated to exchange and 16,800 AF/yr to direct reuse and recovery. To help manage its reclaimed water supply, Chandler began developing two Aquifer Storage and Recovery (ASR) project sites in 1996, namely the Tumbleweed Park and Ocotillo ASR sites. The ASR wells are intended to provide:

- Storage of excess reclaimed water during periods of low direct reuse
- Recovery of stored water during peak demand periods
- 100% back up recharge capacity for periods of no use or emergency conditions.

This paper will present the following regarding Chandler's recharge program:

1. Historical development and operation of the ASR wells
2. ASR well construction overview
3. Historical recharge rates/volumes
4. Future ASR well development
5. How site development and operational challenges were overcome.

The Tumbleweed Park recharge site currently has seven ASR wells each with recharge rates ranging from 2,270 - 6,056 L/min (600 - 1,600 gpm). The cumulative volume recharged at the site to date totals

33,866 acre-feet (AF). It is anticipated that ultimately the site will contain up to twelve recharge wells, with an average site recharge capacity of 37.9 ML/d (10 mgd), and a peak rate of 75.7 ML/d (20 mgd).

The Ocotillo recharge site currently has four operational ASR wells each with a recharge rate of 3,785 L/min (1,000 gpm). The cumulative volume recharged at the site to date totals 5,378 AF. It is anticipated that ultimately the site will contain up to eight recharge wells, with an average site recharge capacity of 37.9 ML/d (10 mgd), and a peak rate of 75.7 ML/d (20 mgd).

## KEYWORDS

ASR, Aquifer Storage, Recharge, Reuse

## INTRODUCTION

As one of the fastest growing cities in the United States among cities with population over 150,000, the City of Chandler, Arizona offers excellent quality of life amenities, superior schools, a rapidly expanding health care system, and a reputation as a global leader in technology. Buildout of the community is expected in the next 10 - 15 years, with a population totaling about 280,000 to 290,000. In reaching its goals, Chandler has had to ensure that plans were in place for sustainability of the community. Effective development and management of the City's water resources in a desert environment is certainly a key component to ensuring sustainability.

Chandler manages its water resources from an integrated standpoint; water, wastewater, and reclaimed water supply management. In 1980, the State of Arizona passed the Groundwater Management Act that requires Chandler, and its neighboring cities, to develop and use only sustainable water supplies. With increasing State imposed restrictions on the use of potable water it became cost effective to maximize the use of reclaimed water in the City's overall water resources portfolio. Chandler developed a multi-phase strategy to manage its reclaimed water supply, which includes direct injection, non-potable reuse, long-term storage, and exchanges delivery to nearby farms for crop irrigation.

Utilization of Chandler's reclaimed water resource was not without challenges. First of all, providing reclaimed water supplies to non-potable users required wastewater treatment facilities to produce a high quality, safe effluent, and a self-contained distribution system to deliver the supply. Second, regulatory permits were needed to protect the environment and provide for accounting of the reclaimed water use. Public safety was of a prime concern. And lastly, non-potable uses are subject to significant seasonal demand variations, while the wastewater flow is essentially constant. Therefore, a storage mechanism was necessary to balance the supply and demand. A traditional solution would be to construct large, aboveground storage reservoirs, which are expensive and land intensive, and subject to evaporative losses.

The solution to balancing the reclaimed water demand with supply proved to be Aquifer Storage and Recovery (ASR) wells. Chandler began implementation of this strategy in 1998 with planning the first ASR wells at Tumbleweed Park. A second ASR project was developed at the Ocotillo Water Reclamation Facility (WRF) site where four ASR wells were placed in operation in 2002. Recharge at both sites occurs in the Upper Alluvial Unit, a geologic formation that is an impaired aquifer and not suitable for potable uses without treatment.

## **HISTORICAL DEVELOPMENT OF ASR WELL PROGRAM**

Chandler has developed several recharge facilities to manage its seasonal distribution of reclaimed water supply and demand. These recharge facilities include the Tumbleweed Park site, the Ocotillo WRF site, and the Chandler Heights Recharge Project at Veterans Oasis Park. The Tumbleweed Park recharge site was first developed in 1996 using vadose zone recharge well technology. The vadose zone well recharge rates eventually declined to a point where they could no longer meet the design capacity of the site. The decline in recharge capacity was believed to be related to a combination of the impacts of biofouling, plugging, and air entrapment on the shallow fine-grained sediments. Therefore in 1999, Chandler initiated design and construction of its first ASR well at the Tumbleweed site, and to date has constructed a total of seven ASR wells. The wells are constructed to a depth of 107 m (350 feet), and recharge the Upper Alluvial Unit. They are equipped with down-hole flow valves to control recharge rates, and submersible pumps for recovery. During the recharge mode, the pumps are used to periodically purge the wells to maintain recharge capacity. Long term, Chandler plans to install up to 12 recharge wells at this site to maintain 37.9 ML/d (10 mgd) average and 75.7 ML/d (20 mgd) peak recharge capacity.

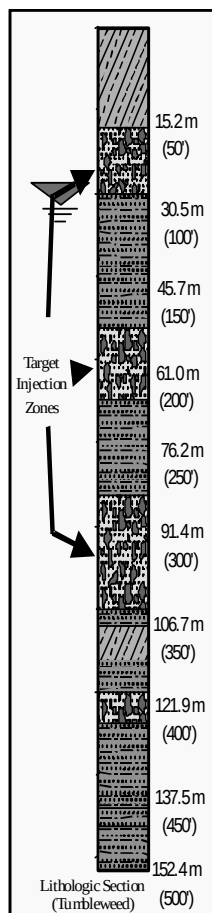
In 2001, Chandler initiated the development of the Ocotillo recharge facility located just south of the Ocotillo WRF. The initial development included construction of four ASR wells, with plans to install up to eight recharge wells to meet sustained as well as peak recharge needs. These wells are drilled to a depth of about 76.2 m (250 feet) and recharge the Upper Alluvial Unit. The design of these wells also includes down-hole flow control valves to control recharge rates, and vertical turbine or submersible pumps for recovery.

Each of these projects operates under approved water storage and aquifer protection permits issued by the Arizona Department of Water Resources (ADWR) and the Arizona Department of Environmental Quality (ADEQ).

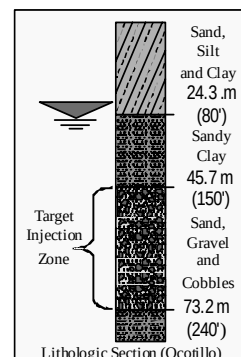
To date, Chandler has stored a total of 33,866 AF and 5378 AF at its Tumbleweed and Ocotillo recharge facilities respectively.

## ASR WELL DESIGN OVERVIEW

The Ocotillo and Tumbleweed ASR sites are located within the eastern Salt River Valley (SRV) along the edge of the ancestral Salt River channel. Shallow sediments of this large alluvial basin generally consist of interbedded deposits of silt, sand, gravel and cobbles (Laney & Hahn, 1986). These sediments are usually referred to as the Upper Alluvial Unit (UAU), where all of Chandler's recharge takes place. Deeper sediments of the Middle and Lower Alluvial Units (MAU & LAU) are generally finer grained with greater percentages of clay. The deepest sediments can also be highly cemented.

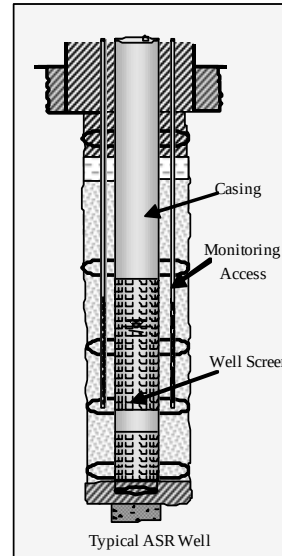


At Ocotillo, sandy clay and sand were encountered from land surface to about 45.7 m (150 feet) below land surface (bls). Gravel and cobbles were encountered from 45.7 to 73.2 m (150 to 240 feet) bls. Fine-grained sediments were encountered below this depth. Based on the site-specific hydrogeology, the target injection zone was the 27.4 m (90 feet) of coarse material from 45.7 to 73.2 m (150 to 240 feet) bls. Hydraulic conductivities within this zone ranged between 150 – 210 m/day (~500 and ~700 ft/day) and the resulting transmissivities exceeded 2,265 m<sup>2</sup>/day (80,000 ft<sup>2</sup>/day).



At Tumbleweed, the lithologic materials encountered were similar; however the injection zones were separated by fine-grained intervals. Also, cobbles were generally not present in the coarse grained zones and the resultant aquifer characteristics were somewhat less productive. Nonetheless, sufficient coarse-grained materials in the UAU were present to facilitate ASR. Hydraulic conductivities at Tumbleweed were about 30 m/day (100 ft/day) and transmissivities were calculated to be about ~700 m<sup>2</sup>/day (~25,000 ft<sup>2</sup>/day).

The Ocotillo and Tumbleweed ASR wells have utilized industry design standards similar to those used in the southwestern US for municipal supply wells. Because of hydrogeologic and water quality constraints, the wells are relatively shallow ranging between 76 and 107 m (250 and 350 feet) deep. Casing diameters are similar at 45.7 cm (18 inches) typically. Well screens are of the Full Flow louvered type having an average open area on the order of 1,100 cm<sup>2</sup> per lineal meter (50 in<sup>2</sup> per lineal foot) or 7 to 8%. Filter pack consists of 6 x 9 grade Colorado silica sand and the screen slot size is generally 1.5 mm (0.060 in). To ensure longevity of the ASR wells and to mitigate corrosion, the City chose 304 stainless steel (SS) for both blank and screened portions of all their ASR wells. While the capital cost for SS is significantly greater than that for low carbon steel or copper bearing steel, operation and maintenance costs have been significantly less, and recharge capacity declines due to mineralization or biofouling of the well screens have been reduced.



Prior to rapid population growth, the City of Chandler was a rural community dedicated to agriculture and dairy farms. These historical land uses, coupled with shallow depths to water (80 to 100 feet bls) and highly transmissive sediments, have resulted in degradation of the water quality in the UAU. Particularly, elevated concentrations of nitrate and total dissolved solids (TDS) are present in many areas. Based on pre-ASR water quality data, nitrate concentrations in excess of 20 milligrams per liter (mg/L) and TDS concentrations exceeding 2,000 mg/L are present. While this shallow impacted groundwater has been used for agricultural purposes, its presence has forced public supply wells to be completed in deeper portions of the regional aquifer (MAU and LAU). Nonetheless, the impacted UAU provides an ideal environment for injection and recovery of high quality reclaimed water. Since public water supply wells are completed in the MAU and LAU aquifers, ASR well permit limitations have confined recharge to the UAU (<107 m, 350 feet bls).

## RECHARGE RATES/STORAGE VOLUMES

Chandler initiated ASR recharge at its Tumbleweed and Ocotillo facilities in 2001 and 2002 respectively. Initially, Tumbleweed ASR Well No. 1 was put into operation in May 2001, followed by Well Nos. 2 and 3 in November 2001. Well Nos. 4 - 7 followed in 2004 and 2006. Ocotillo ASR Well Nos. 1 - 4 were put into operation in August 2001.

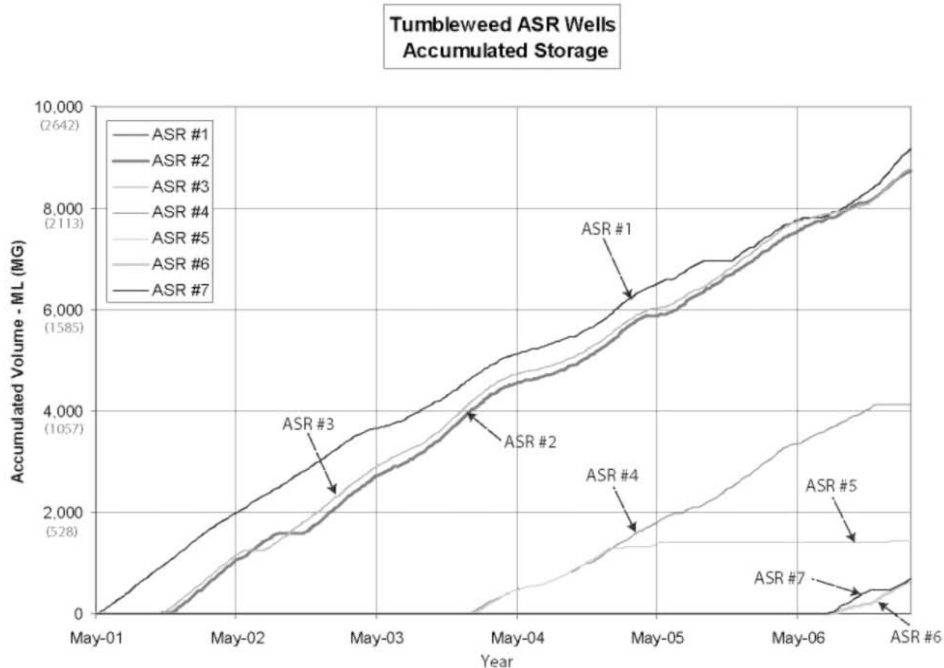
Table 1 summarizes the Tumbleweed ASR well capacities. Current recharge rates range from 2,270 - 6,056 L/min (600 - 1,600 gpm), while recovery rates range from

4,543 - 5,300 L/min (1,200 - 1,400 gpm). The total current recharge and recovery capacity of the Tumbleweed site is 29,900 L/min (7,900 gpm) (11.4 mgd) and 34,829 L/min (9,200 gpm) (14.1 mgd), respectively.

**TABLE 1. Tumbleweed ASR Wells**

| Well No.     | Date      | Current Recharge Capacity<br>L/min (gpm) | Current Pumping Rate<br>L/min (gpm) |
|--------------|-----------|------------------------------------------|-------------------------------------|
| ASR 1        | May 2001  | 6,056 (1,600)                            | 4,543 (1,200)                       |
| ASR 2        | Nov. 2001 | 4,542 (1,200)                            | 4,543 (1,200)                       |
| ASR 3        | Nov. 2001 | 4,542 (1,200)                            | 4,543 (1,200)                       |
| ASR 4        | Jan. 2004 | 4,920 (1,300)                            | 5,300 (1,400)                       |
| ASR 5        | Jan. 2004 | 2,291 (600)                              | 5,300 (1,400)                       |
| ASR 6        | Aug. 2006 | 3,406 (900)                              | 5,300 (1,400)                       |
| ASR 7        | Aug. 2006 | 4,164 (1,100)                            | 5,300 (1,400)                       |
| <b>TOTAL</b> |           | <b>29,900 (7,900)</b>                    | <b>34,829 (9,200)</b>               |

The Tumbleweed ASR wells have recharged a total of 33,600 ML (8,874.4 MG) (27,244 AF) over the life of the project. Figure 1 shows a graphical representation of the accumulated recharge volume of each of the Tumbleweed ASR Wells.

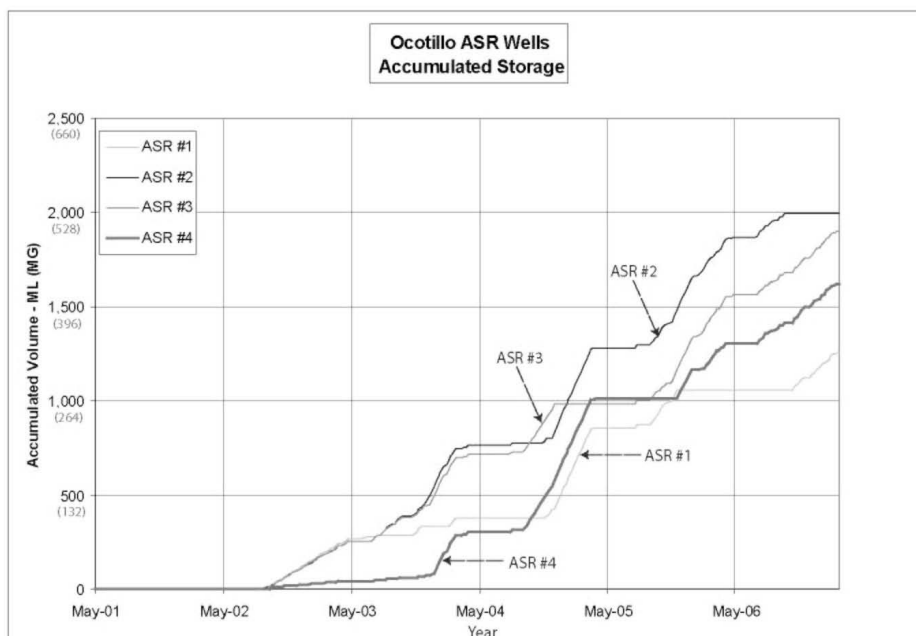


**FIGURE 1. Tumbleweed ASR Wells Accumulated Storage.**

Table 2 summarizes the Ocotillo ASR well capacities. Current recharge rates average about 3,028 L/min (800 gpm), while recovery rates are about 1,514 L/min (400 gpm). The current recharge capacity is limited by infrastructure restrictions that will be upgraded in the near future. The total current recharge and recovery capacity of the Ocotillo site is 12,112 L/min (3,200 gpm) (4.6 mgd) and 6,056 L/min (1,600 gpm) (2.3 mgd), respectively.

**TABLE 2. Ocotillo ASR Wells**

| Well No.     | Date      | Current Recharge Capacity<br>L/min (gpm) | Current Pumping Rate<br>L/min (gpm) |
|--------------|-----------|------------------------------------------|-------------------------------------|
| ASR 1        | Aug. 2001 | 3,028 (800)                              | 1,514 (400)                         |
| ASR 2        | Aug. 2001 | 3,028 (800)                              | 1,514 (400)                         |
| ASR 3        | Aug. 2001 | 3,028 (800)                              | 1,514 (400)                         |
| ASR 4        | Aug. 2001 | 3,028 (800)                              | 1,514 (400)                         |
| <b>TOTAL</b> |           | <b>12,112 (3,200)</b>                    | <b>6,056 (1,600)</b>                |



**FIGURE 2. Ocotillo ASR Wells Accumulated Storage.**

## **SITE DEVELOPMENT/OPERATIONAL CHALLENGES**

The ASR project sites have had several challenges to overcome. These challenges include:

- Site development/land use
- Flow control valve issues
- Check valve failure
- Biofouling
- Infrastructure limitations
- Low flow conditions.

The Tumbleweed project site has had several site development and operational challenges. First, the recharge site is located in a Public Park. This creates land use, site development, and access challenges for both the City's Municipal Utilities and Parks Departments. There is limited land available in the park for well sites. All but one well site is located below ground in order to reduce the impact of the wells on the park and to provide security. This has created operational challenges for well access, maintenance, and potential flooding from nearby irrigation.

Operational challenges have included excessive nitrogen use for flow control, check valve failures, and biofouling. As the reclaimed water distribution system pressure changes, the Baski flow control valve uses nitrogen to control flow by exerting more or less pressure on the valve to maintain a flow setpoint. In order to reduce nitrogen use and stabilize flow control, a different flow control strategy has been developed. The wells now operate with a nitrogen pressure setpoint rather than a flow setpoint. This has dramatically reduced the consumption of nitrogen for well flow control. Differences in the reclaimed system pressure cause slight variations in flow, but the Baski flow control valve does not react to them, conserving nitrogen.

Check valve failures are being addressed by installing more robust valves, reducing water hammer by pulsing open butterfly valves, and by moving the check valve further from the well pump to reduce turbulence.

Maintaining a chlorine residual in the reclaimed water, occasional superchlorination of wells, and consistent purging of the wells controls biofouling problems, and maintains the recharge rate.

The Ocotillo project site has a different set of site development and operational challenges. The Ocotillo ASR well site was initially designed and permitted to recharge 4.5 ML/d (1.2 mgd) based on pilot borings. Once the site was developed, it became clear that the wells were capable of recharging much more than previously thought. The site has been re-permitted to allow recharge up to 37.9 ML/d (10 mgd).



Due to infrastructure limitations related to pumping and distribution piping capacity, the well site is currently only capable of recharging about 11.4 ML/d (3 mgd). In addition, the infrastructure limitations are causing operational issues with the system such as frequent starting and stopping of supply pumps. The site is currently under construction to mitigate these infrastructure limitations.

Another operational challenge at the Ocotillo site is the low flow condition through the water reclamation facility during early morning hours. The ASR well site automatically shuts down if not enough water is available to meet well system pressure parameters. An operator, traveling from a different site, must then restart the well site. This problem is also being addressed in the infrastructure upgrades.

## **FUTURE ASR WELL DEVELOPMENT**

Chandler manages 100 percent of its reclaimed water supply through a combination of direct deliveries, recharge and recovery, and direct in-city reuse. At buildout, Chandler is expected to produce about 110 ML/d (29 mgd) of reclaimed water, of which 53 ML/d (14 mgd) will be delivered continuously to nearby farms, and 57 ML/d (15 mgd) reused within the city boundaries. Long term, the ASR well program is intended to provide:

1. Storage of excess reclaimed water during periods of low direct reuse
2. Recovery of stored water during peak demand periods
3. 100 percent back up recharge capacity for periods of no use or emergency conditions.

The deliveries to nearby farms are expected to be a constant 53 ML/d (14 mgd). However, there will be periods when these constant deliveries can be suspended for up to 30 days to allow maintenance of the farm delivery system. The in-city use demand ranges from a low of 7.6 ML/d (2 mgd) during winter to 102 ML/d (27 mgd) in the summer. During the winter low demand period, essentially all unused reclaimed water must be recharged for withdrawal during the summer peak demand period. The ASR well facilities must have the capacity to recharge 110 ML/d (29 mgd) for brief, low demand periods. To ensure Chandler has a reliable 110 ML/d (29 mgd) recharge capacity, 151 ML/d (40 mgd) of instantaneous recharge capacity is planned. Long term, Chandler plans to install up to 12 ASR wells at the Tumbleweed site to be able to provide a sustainable 37.9 ML/d (10 mgd) capacity, and an instantaneous recharge rate of 75.7 ML/d (20 mgd).

At the Ocotillo site, plans are to upgrade the existing infrastructure to match the existing ASR well recharge capability, and to add up to four additional wells to achieve the needed instantaneous rate during the low demand periods. Once

upgraded and expanded, the Ocotillo site will have a sustainable capacity of 37.9 ML/d (10 mgd), and an instantaneous recharge rate of 75.7 ML/d (20 mgd).

Future ASR well capital costs are expected to be about \$1.5 million dollars per well. Annual operation and maintenance costs are currently budgeted at about \$23,000 per well.

## **SUMMARY**

Chandler's ASR well program has proved to be an essential component of its reclaimed water program, and is expected to effectively manage and utilize the City's projected 32,500 AF/yr (29 mgd) reclaimed water supply. The ASR well program provides for:

1. Seasonal balancing of available reclaimed water supplies versus seasonal demands
2. A mechanism for aquifer storage during periods of low demand or during emergencies
3. A cost effective storage mechanism that meets aquifer water quality standards
4. Small land requirement compared to surface storage reservoirs that are land intensive in a high land cost market
5. Eliminates evaporation losses that occur with surface storage reservoirs
6. Capability to sustain well recharge capacities with a mechanism to rehabilitate declining recharge rates.

# PLANNING AND LIFE-CYCLE COST ANALYSIS OF AN AQUIFER STORAGE AND RECOVERY SYSTEM (WELL #299) IN THE NORTHEAST REGIONAL AQUIFER, CITY OF PHOENIX

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## ABSTRACT

Phoenix's objective is to manage aquifers to ensure the future availability of groundwater, sustain production capacity of our existing wells, and reduce land subsidence. The need to rebuild well capacity for drought, operational flexibility, and management of surface water supplies has been identified. A life-cycle cost analysis was conducted for an Aquifer Storage and Recovery (ASR) well to assess whether this system is economically and technically feasible. The cost-effective and technically feasible scenario is an ASR system that injects, stores, and recovers treated Central Arizona Project (CAP) water at an existing booster station. The financial and operational advantages of this system are listed below:

- Existing distribution and treatment systems can be utilized for the ASR system, hence, additional land acquisition is not required;
- From an operational perspective, utilizing injected treated CAP water will not be as problematic (i.e., well clogging) as other water sources (i.e., raw CAP and reclaimed water); and
- Due to system integration of the ASR well, operational flexibility and redundancy can be achieved.

Ultimately, we want to implement a joint management/planning strategy with City of Scottsdale, so that both cities can utilize ASR to manage and sustain groundwater levels and future water resource supplies.

## KEYWORDS

Aquifer Storage and Recovery, Life-Cycle Costs Analysis, Water Resource Planning

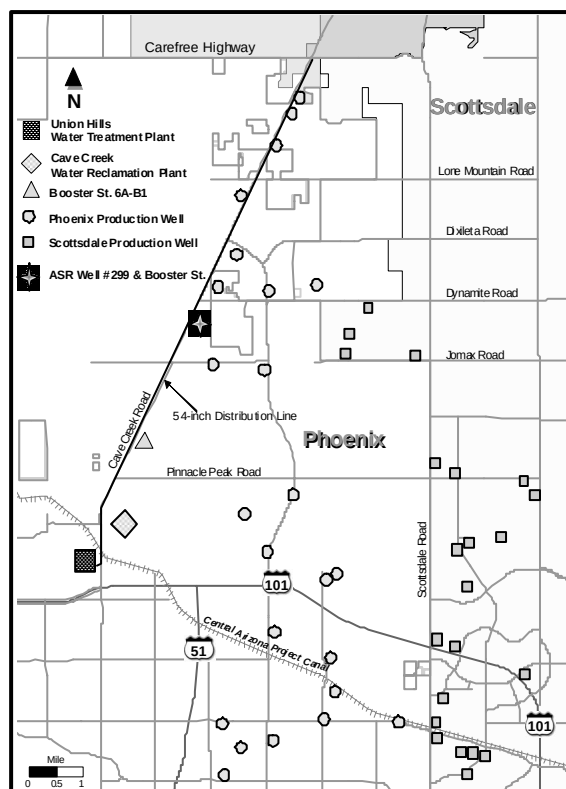
## BACKGROUND AND OBJECTIVE

In 2005, the cities of Phoenix and Scottsdale cooperated in a groundwater modeling study that evaluated the effects of current/future well production and recharge in the Northeast Aquifer, which is the aquifer that underlies the far northeastern region of Phoenix and northern Scottsdale (Figure 1). Previous groundwater modeling studies indicated that groundwater production in the Northeast Aquifer was not sustainable without supplemental recharge (AMEC, 2005). Based on these studies, the cities decided to assess current water use plans and develop management options that could be implemented to improve the long-term sustainability of the aquifer resource. The 2005 study indicated that the acquisition of new water sources and siting of new recharge facilities needed to be consistent with the aquifer management strategy in order to ensure the long-term sustainability of the northeast aquifer system.

To follow up on the 2005 study, this life-cycle cost analysis was developed to identify scenarios of an ASR well system that is economically and technically feasible. To evaluate the economic merits of an ASR system, we conducted a 40-year life-cycle costs analysis to assess both short and long term investments of such a system. From a technical perspective, we had to evaluate how the ASR system would be integrated and function with the existing City of Phoenix treatment and distribution system.

Four scenarios were created to compare various types of injected water sources ranging from treated CAP (potable water), raw CAP (non-potable water), and reclaimed water (Class A+). We included capital, treatment, operations and maintenance, electrical, and management costs associated with each scenario. The first three scenarios entailed utilizing these various source waters with ASR, while the fourth scenario looked at the delivery of raw CAP water to a surface reservoir. This reservoir scenario was selected since the operational and management objectives are similar to an ASR system. The economic and technical merits of each scenario were compared and contrasted to determine what type of water source or system would be most beneficial for water resources management.

Because groundwater resources are limited and the City is losing well capacity due to declining water levels, we had to investigate a new strategy for managing aquifers (Figure 2). We recognized that the ASR concept would be a viable water resource management tool since we have a large surface water allocation (CAP water) that can be utilized for storage during the winter months and recovered during the summer months. We are advancing our strategy with ASR to provide Phoenix with additional operational flexibility, redundancy for drought and emergency needs, and to facilitate the management of the northeast aquifer (maintain water levels, sustain production capacities for nearby city wells, and mitigate land subsidence), which is consistent with the recommendations detailed in the Water Resource Plan and Northeast Valley Aquifer Management Study (City of Phoenix, 2006 and AMEC, 2005).



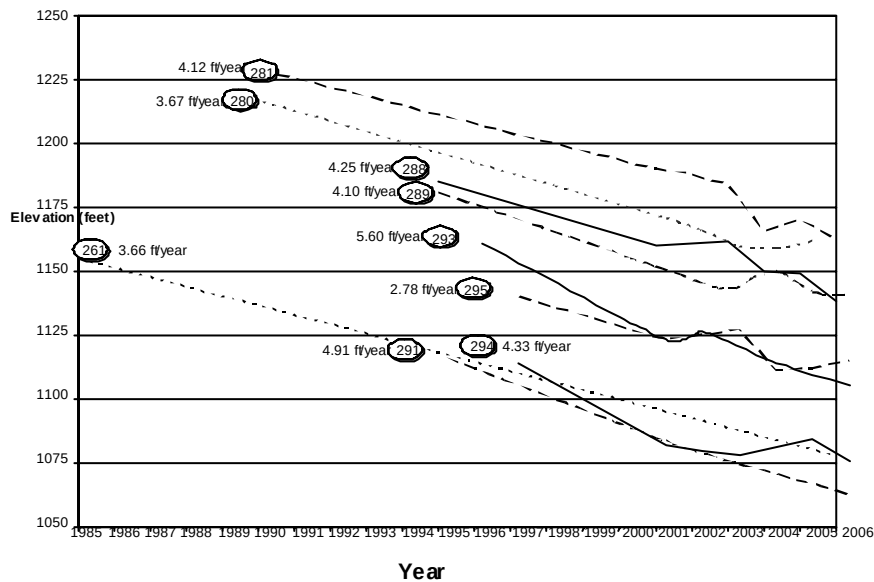
**FIGURE 1. Northeast Phoenix and Scottsdale.**

## HYDROGEOLOGY

The project area's main aquifer is within the alluvial basin fill deposits of the East Valley Salt River Sub-Basin (EVSRSB) (Laney and Hahn, 1986). The bounding mountain ranges are comprised of relatively impermeable crystalline rocks and are considered barrier boundaries. The Upper Alluvial Unit (UAU) is characterized by relatively high hydraulic conductivity, but is dewatered (unsaturated- 200 to 600 feet thick) in the study area. The Middle Alluvial Unit (MAU) is characterized as low to moderate hydraulic conductivity, and is unsaturated in the northern portion of the study area. The Lower Alluvial Unit (LAU) is saturated throughout the study area and is characterized by moderate to high hydraulic conductivity. As such, the LAU is the primary source of groundwater for wells in the study area.

Based on the Arizona Department of Water Resources Salt River Valley model (Corell and Corkhill, 1994), natural recharge (10,100 acre-feet/year) to the Northeast Aquifer occurs as mountain front from the McDowell Mountains, and as mountain front, stream channel, and underflow from Cave Creek. According to the City of Scottsdale Groundwater Model (Brown and Caldwell, 2003), the primary sources of discharge from the aquifer are groundwater withdrawals by wells and underflow to the larger portion of the EVSRB to the south.

Due to the minimal natural recharge and long-term pumping in the northeast aquifer and surrounding areas, static water levels collected from 9 City of Phoenix water supply wells indicate that the northeast aquifer has been declining since the mid-1980s (Figure 2). This phenomenon is also observed in the City of Scottsdale well-field that borders the City of Phoenix (AMEC, 2005). The average rate of water-level decline ranges from 3 to 5 feet per year (Figure 2). With water levels declining in the northeast aquifer, the vadose zone becomes thicker in the UAU and MAU. A thicker vadose zone allows a greater potential for ASR to be successful, since there is substantial capacity in the vadose zone for both storage and recovery.



**FIGURE 2. Hydrograph of 9 Water Supply Wells from 1985 to 2006, Northeast Phoenix.**

## **LIFE-CYCLE COSTS ANALYSIS**

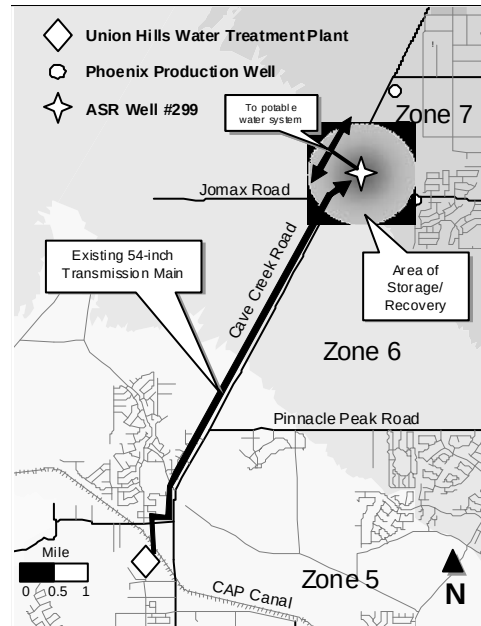
The purpose of this life-cycle costs analysis was to estimate the overall costs of ASR project alternatives and to select the design that ensured the facility would provide the lowest overall cost of ownership consistent with high standards of quality and function. The most challenging task of the life-cycle cost analysis was to determine the economic effects of alternative system designs and quantify those effects in dollar amounts. Four scenarios were created that could be easily integrated into the existing water distribution/treatment system and meet emergency events. An emergency was defined as an unplanned event where part of the system is stressed. An emergency might occur in summer months when the entire system is needed to meet peak demands, or in the off season when a specific part of the system is heavily loaded due to a facility being off-line for scheduled maintenance. The distribution/treatment system is detailed in the following section.

## **CITY OF PHOENIX WATER DISTRIBUTION SYSTEM**

In the northeastern portion of Phoenix's service area, the majority of potable water is provided by CAP water conveyed by the CAP canal to the Union Hills Water Treatment Plant (UHWTP) (Figure 1), which is then treated at the water treatment plant and distributed through the transmission network. The current operational capacity of this plant is 145 million gallons per day (MGD). Union Hills WTP is a direct filtration plant with treatment processes that include aeration, coagulation, flocculation, and filtration. Future improvements planned for the UHWTP include replacing the dual media filters to Granular Activated Carbon (GAC) filters (in 2010), ozonation, and ultraviolet disinfection. Free chlorine is utilized for both primary and secondary disinfection. Potable water is conveyed through a complex network of transmission mains, booster systems, and pressure reducing valves. The facilities and infrastructure included in this life-cycle cost analysis include the UHWTP, Cave Creek Water Reclamation Plant (CCWRP), 54-inch diameter transmission main along Cave Creek Road, and Booster Stations 6A-B1 and 7A-B1 (Figure 1).

## SCENARIO 1: TREATED CAP- ASR

This scenario entailed conveying treated CAP water from UHWTP, along Cave Creek Road to ASR Well #299 with an estimated volume of 1,887 acre-feet/year (615 MG) of injection and storage, which would be the equivalent volume of water pumped from 1.5 to 2 water supply wells (Figure 3). The injection rate in the ASR well was estimated at 2,000 gpm for 8-months of operations per year (including off and on-peak hours of operations). To off-set peak demands (summer months), an estimated 1,000 acre-feet/year would be recovered for four months, which would be a recovery rate of 1,887 gpm. We estimated 887 acre-feet/year of injected water to be stored in the aquifer. Extending the stored volume over a 40-year period would total about 35,000 acre-feet.



**FIGURE 3. - Scenario 1 Schematic.**

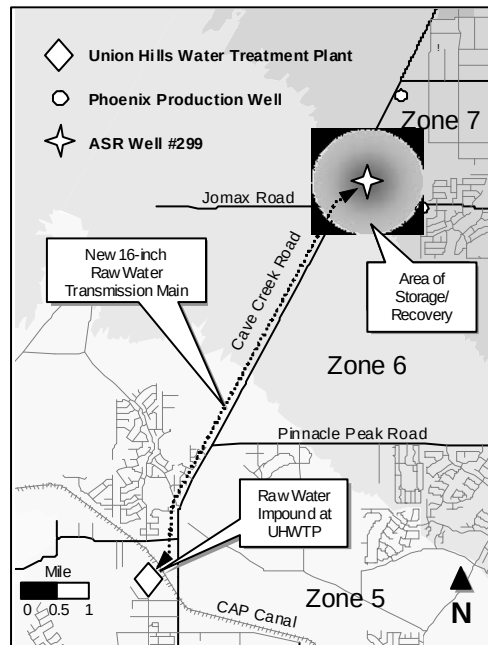
Capital costs entailed drilling and testing the ASR well, installing pump and motor, ancillary equipment, compliance monitor well, and chlorination system. Delivery and treatment costs entailed purchasing CAP water, power costs for delivery of water from UHWTP to the ASR well, treatment of the water at UHWTP before injection, and the energy costs related to pumping the recovered water. For operations and maintenance costs, we projected well rehabilitation (both mechanical and chemical treatment) to occur every ten years.

Scenario 1: Potable water is pumped from UHWTP to ASR well via existing 54-inch transmission main along Cave Creek Road. Recovered water supplies pressure zones 6 and 7.



## SCENARIO 2: RAW CAP- ASR

This scenario entailed delivering raw CAP water from UHWTP through a new dedicated transmission line accompanied by two booster stations that would inject raw CAP water into the ASR well (Figure 4). The injection and recovery systems were the same as detailed in Scenario 1. Before injection, water would be treated to remove suspended solids, mollusks, algae, and other biological organisms that would otherwise clog the well screens. Additional treatment of the recovery water would be required to meet Federal Drinking Water Standards. For this scenario, recovered water would be pumped back for treatment at the UHWTP. Water would then be conveyed through the Cave Creek potable water distribution system (Figure 4).



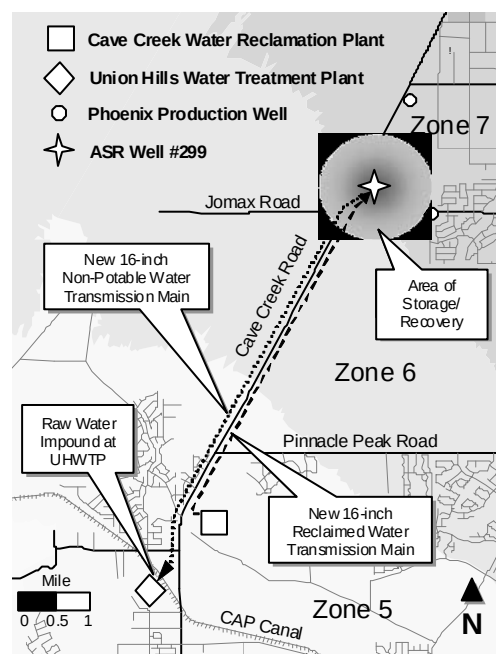
**FIGURE 4. Scenario 2 Schematic.**

Capital costs were the same as detailed in Scenario 1, but the design and construction of a new distribution line and booster pump system from UHWTP to the ASR well, and design and installation of a micro-filtration (MF) system before injecting the source water would be required. Delivery and treatment costs entailed the costs purchase of CAP water, electricity for delivery of water from UHWTP to the ASR well, MF treatment of the injected water at the wellhead, treatment of the recovered water at the UHWTP, and the energy costs for pumping the recovered water. For operations and maintenance, we included a well rehabilitation (both mechanical and chemical treatment) cycle every five years and increased rehabilitation costs, since there is a higher probability of clogging the well screen. In addition, we also had to include labor, maintenance, and management for the treatment systems.

Scenario 2: Raw water is pumped from UHWTP pre-treatment impound to ASR well via dedicated 16-inch transmission main along Cave Creek Road. Recovered water is pumped back to the raw water impound.

### SCENARIO 3: TREATED RECLAIMED WATER-ASR

This scenario entailed delivery of treated reclaimed water from CCWRP through a dedicated 16-inch diameter transmission line that conveys water utilizing a 3 MGD booster station to Well #299 (Figure 5). Reclaimed water would be injected in Well #299 for the same duration, volume, and rate detailed in Scenario 1. Pre-treatment (ozone treatment and micro-filtration) of the reclaimed water would occur at CCWRP to minimize biological and mechanical clogging of the ASR well. In addition, pre-treatment would reduce any potential contamination (emerging contaminants) to the aquifer. In this scenario, recovered water from the ASR well would be conveyed back to UHWTP through a 16-inch dedicated transmission main for additional treatment per State regulatory agency requirements.



**FIGURE 5. Scenario 3 Schematic.**

Pre-treatment (ultraviolet, ozone, and micro-filtration) at the well site was considered, but the footprint of the treatment system would exceed the existing size of the site and would be cost prohibitive. Based on this reasoning, the 16-inch diameter water line for treated reclaimed water to the well site was necessary and was relatively cheaper than pre-treatment at the well site.

Capital costs entailed the same details described in Scenarios 1 and 2, however a dedicated distribution line and booster system would need to be designed from CCWRP to the ASR well, the design and installation of an ozone/micro-filtration system before injecting the source water would be required, and the construction of a dedicated 16-inch diameter transmission main from the ASR well to UHWTP will be required to treat the recovered water. Delivery and treatment costs entailed power costs for the delivery of water from CCWRP to the ASR well, treatment of the water at CCWRP for the source water to be injected, the energy costs related to pumping the recovered water, and treatment of recovered water at UHWTP. For operations and maintenance, we included well rehabilitation (both mechanical and chemical treatment) costs every five years and increased rehabilitation costs, since there is a

higher probability of clogging the well screen. In addition, we also had to include labor, maintenance, and management for the treatment systems.

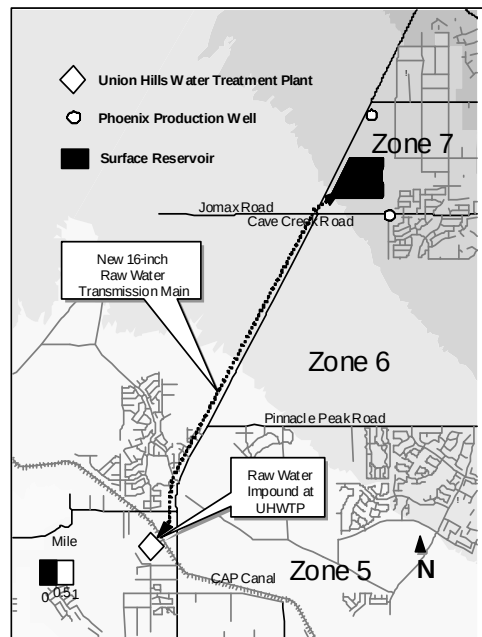
Scenario 3: Reclaimed water treated to potable water standards is pumped to ASR well via dedicated 16-inch transmission main along Cave Creek Road. Recovered water is pumped into a dedicated 16-inch diameter transmission main to UHWTP for additional treatment before delivery to the potable water distribution system.

### SCENARIO 4: RAW CAP WATER DELIVERED AND RECOVERED FROM A SURFACE RESERVOIR

This scenario entailed delivering raw CAP water from UHWTP to a surface reservoir at the same location as the ASR well (Figure 6). This reservoir would be lined and be utilized to store water for drought and emergency conditions in the northeast Phoenix area. The volume of storage for the reservoir was estimated to be 35,000 acre-feet, which is the estimated volume of water stored in Scenario 1. A dedicated distribution / booster pump system would have to be constructed between UHWTP and the reservoir. During recovery, water from the reservoir would have to be delivered back to UHWTP for final treatment.

Capital costs regarding this scenario included the design and construction of the surface reservoir (total footprint was estimated at 90 acres), design and construction of a dedicated distribution line with booster pump system, and preparation and submittal of an Environmental Impact Statement. Operations and maintenance costs included costs of CAP water, electricity for pumping water to the reservoir and then back to UHWTP for final treatment, treatment of raw CAP water at UHWTP, vector control, and security.

Scenario 4: Raw water is pumped from UHWTP pre-treatment impound to surface reservoir via dedicated 16-inch transmission main along Cave Creek Road. Stored water is pumped back to the raw water impound.



**FIGURE 6. Scenario 4 Schematic.**

## RESULTS OF THE ANALYSIS

Scenarios 2 through 4 yielded costs ranging from \$600 to \$26,877/AF (Table 1). These costs are significantly higher than Scenario 1 (treated CAP water- \$250/AF) (Table 1). Reasons for higher costs were the design and construction of new, dedicated infrastructure to support each system (transmission mains, booster pumps, treatment/filtration systems, new facilities/systems, and SCADA), operations and maintenance (potential clogging) of each system, electrical costs related to conveying water and treatment, real estate costs for additional land, dual treatment processes to meet Federal Drinking Water Standards, and capital and O&M costs related to water treatment to prevent clogging and water quality issues (Table 1).

## OTHER ISSUES RELATED TO SCENARIOS 2 THROUGH 4

Several important issues need to be considered when determining the type of water source to be utilized for ASR. The following section details some of the issues not delineated in the life-cycle cost analysis for Scenarios 2 and 3 (Table 1):

- Well clogging: Raw CAP or treated reclaimed water will clog the well screen more frequently than the injection of treated CAP water. Mechanical (total suspended solids) or biological (bacteria) clogging of the ASR well is costly and requires additional monitoring to ensure the system is operating properly. Reduction in injection capacity of the ASR well will decrease the storage capability and increase life-cycle costs. If clogging is extensive and the well can not be adequately developed, then the capital costs to fix the problem can range from \$1,000,000 to \$1,500,000. Costs would be related to abandonment of the existing well, drilling a replacement well and re-equipping the pump system.
- Water quality: Based on the nature of raw CAP and reclaimed water, there is a greater probability of negatively affecting the water quality of the aquifer. If contaminants (e.g., disinfection by products, Volatile Organic Compounds (VOCs)- perchlorate, endocrine disruptors, and pharmaceuticals) are not properly removed or treated before injection, the creation of a contamination plume in the aquifer would be costly to remediate.
- Lack of redundancy and flexibility: For Scenarios 2 and 3, if UHWTP was down for maintenance or emergency issues, water recovered from Well #299 could not be used for potable use since the recovered water must be sent back to UHWTP for additional treatment before delivery. Two levels of treatment are required for water sources utilized in Scenarios 2 and 3 since the injected water sources do not meet Federal Drinking Water Standards. From an operations perspective, dual treatment limits any flexibility for Scenarios 2 and 3 since the recovered water source must be delivered back to UHWTP for final treatment to produce potable water. Besides limited operational flexibility, dual treatment for

Scenarios 2 and 3 is expensive because of energy use, chemical use, maintenance, and management of these complex systems.

- Public perception: Utilization of reclaimed water for potable ASR would essentially be direct re-use, which is currently not practiced in the United States. Not only is direct re-use not practiced, the regulatory agencies currently do not issue permits for this type of application.
- Lack of available water resource: For Scenario 3, the CCWRP system has limitations as to how much reclaimed water would be available for recharge via injection throughout the year since the reclaimed system is driven by customer demand and seasonal variations (wet winter versus dry winter). In the first year of operation, 140 AF was recharged in the vadose zone, which was significantly less than the estimated volume to be injected in Well #299. In addition, this volume (140 AF) was not significant enough to impact water levels in the existing COP well field. If injection volumes were less than 1,887AF/year, then the economics of the ASR system would be more costly than the estimated \$661/AF (Table 1).
- Public relations: For Scenarios 2 and 3, there would be challenges regarding the concept of ASR and how water quality issues (endocrine disruptors, pathogens, and etc.) could potentially impact the public. Since these scenarios require dedicated infrastructure and facilities there would be considerable traffic control and construction in public right-of-way issues.

Issues related to the surface reservoir (Scenario 4) (Table 1):

- Evaporation: Based on the dimensions of the surface reservoir (70 acres) and an average evaporation rate of 6.2 feet/year, the total lost of water per year would be 434 acres/year. Extending the yearly water lost due to evaporation over 35 years would total 15,190 acres, which suggests a total of 70 years to meet the 35,000 acre-feet storage goal.
- Acquisition of land: Locating, purchasing, and obtaining land for the reservoir would be a long and challenging process, so construction costs could escalate to a level not economically feasible.
- Lack of redundancy and flexibility: If UHWTP was down for maintenance or emergency issues, the reservoir would be of no use since the water would have to be treated to meet Drinking Water Standards.
- Public relations: Since this scenario requires dedicated infrastructure and facilities there would be considerable traffic control and construction in public right-of-way issues.

**TABLE 1. Comparison of Water Sources Related to ASR, Groundwater Quality, Capital, Operational Considerations, and Assumptions to the Life-Cycle Analysis**

| Scenarios                                                                                                                                      | Groundwater Quality Impacts                                                                                                | Additional Capital Outlay                                                                                                                                                                                                                                                                                                                                                           |
|------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>1. ASR Utilizing Treated CAP Water:</b><br><br>Inject- 1,887 ac-ft/yr., Recover- 1,000 ac-ft/yr.,<br>Stored in aquifer- 887 ac-ft/yr.       | 1. Potential adjustment to metal constituents-arsenic                                                                      | Low- \$17.9 M, \$250/AF<br>1. Design, install new pump / motor and include other ancillary equipment.<br>2. Pumps, distribution, and treatment facilities exist.<br>3. Install compliance monitor well.                                                                                                                                                                             |
| <b>2. ASR Utilizing Raw CAP Water:</b><br><br>Inject- 1,887 ac-ft/yr., Recover- 1,000 ac-ft/yr.,<br>Stored in aquifer- 887 ac-ft/yr.           | 1. Potential pathogen and VOC introduction.<br>2. Increased TDS.<br>3. Potential adjustment to metal constituents-arsenic. | High- \$40.7M, \$600/AF<br>1. Dedicated pump and facilities would be constructed at UHWTP.<br>2. Dedicated pipeline/booster pump (3MGD) along Cave Creek Road.<br>3. Dedicated pre-treatment (micro-filtration) system required at UHWTP.<br>4. Install compliance monitor well.                                                                                                    |
| <b>3. ASR Utilizing Treated Reclaimed Water:</b><br><br>Inject- 1,887 ac-ft/yr., Recover- 1,000 ac-ft/yr.,<br>Stored in aquifer- 887 ac-ft/yr. | 1. Potential pathogen introduction.<br>2. Potential contaminate introduction -(e.g.,- EDCs, VOCs).<br>3. Increased TDS.    | High- \$44.9M, \$661/AF<br>1. Dedicated booster pump and facilities would be constructed at CCWRP.<br>2. Dedicated pipeline/booster pump (3MGD) along Cave Creek Road.<br>3. Dedicated pre-treatment system required at CCWRP (ozone and micro-filtration).<br>4. Install compliance monitor well.<br>5. Dedicated transmission main back to UHWTP for treatment of recovered water |
| <b>4. Raw CAP Water Reservoir:</b><br><br>Deliver: 1,887 ac- ft/yr.,<br>Recover- 1,000 ac-ft/yr.,<br>Stored volume- 887 ac-ft/yr.              | None.                                                                                                                      | Intensive- \$1.82B, \$26,877/AF<br>1. 90 acres of land required.<br>2. Dedicated pump facilities would be required at UHWTP.<br>3. Dedicated pipeline along Cave Creek Road.<br>4. 3 MGD booster pump system.                                                                                                                                                                       |

**TABLE (Continued) Comparison of Water Sources Related to ASR, Groundwater Quality, Capital, Operational Considerations, and Assumptions to the Life-Cycle Analysis**

| Scenarios                                                                                                                                      | Operational Considerations                                                                                                                                                                                                                                                                                                                                                                                                                                                       | Assumptions                                                                                                                                                                                               |
|------------------------------------------------------------------------------------------------------------------------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|-----------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>1. ASR Utilizing Treated CAP Water:</b><br><br>Inject- 1,887 ac-ft/yr., Recover- 1,000 ac-ft/yr.,<br>Stored in aquifer- 887 ac-ft/yr.       | Minor risks compared to Scenarios 2 through 4.<br>1. Well clogging- well injection system.<br>2. Well operations may be time consuming.<br>3. Water quality issues may occur.                                                                                                                                                                                                                                                                                                    | 1. 40-year life cycle.<br>2. \$164/MG- treatment of water.<br>3. 10-year maintenance cycle.                                                                                                               |
| <b>2. ASR Utilizing Raw CAP Water:</b><br><br>Inject- 1,887 ac-ft/yr., Recover- 1,000 ac-ft/yr.,<br>Stored in aquifer- 887 ac-ft/yr.           | 1. High probability of well clogging.<br>2. Lack of redundancy and flexibility.<br>3. Electrical costs related to treatment and distribution.<br>4. Extensive water quality sampling- Maricopa County.<br>5. Increase SCADA control.<br>6. Increased maintenance costs related to micro-filtration system.<br>7. Increase operational costs due to closed distribution system.                                                                                                   | 1. 40 year life cycle.<br>2. \$164/MG- treatment of water before injection.<br>3. \$270/MG- final treatment.<br>4. 5-year maintenance cycle.                                                              |
| <b>3. ASR Utilizing Treated Reclaimed Water:</b><br><br>Inject- 1,887 ac-ft/yr., Recover- 1,000 ac-ft/yr.,<br>Stored in aquifer- 887 ac-ft/yr. | 1. High probability of well clogging.<br>2. Electrical costs related to treatment and distribution.<br>3. Lack of available water resource.<br>4. Extensive water quality sampling- APP- ADEQ.<br>5. Increase SCADA control.<br>6. Increase maintenance costs related to treatment and filtration system.<br>7. Increase operational costs due to closed distribution system.<br>8. Lack of redundancy and flexibility.<br>9. Extensive water quality sampling- Maricopa County. | 1. 40 year life cycle.<br>2. \$603/MG- treatment of water before injection.<br>3. 5-year maintenance.<br>4. Aquifer Protection Permit required.<br>5. \$164/MG- treatment of recovered water at UHWTP.    |
| <b>4. Raw CAP Water Reservoir:</b><br><br>Deliver: 1,887 ac- ft/yr., Recover- 1,000 ac-ft/yr.,<br>Stored volume- 887 ac-ft/yr.                 | 1. Water loss due to evaporation.<br>2. Vector control / algae blooms.<br>3. Lack of redundancy and flexibility.<br>4. Electrical costs related to treatment and distribution.<br>5. Increase operational costs due to closed distribution system.<br>6. Potential endangered species mitigation.<br>7. Potential vulnerability issues                                                                                                                                           | 1. 40 year life cycle.<br>2. \$164/MG- treatment of recovered water.<br>3. 6.2 feet/year evaporation rate.<br>4. Potential loss due to evaporation- 434 acres/year.<br>5. \$1M/acre for land acquisition. |

## CONCLUSIONS

An ASR system with treated CAP water from UHWTP appears to be the most economically and technically feasible scenario based on the following findings:

- Existing distribution and treatment system can be utilized for the ASR system, hence, additional land acquisition is not required and construction costs are minimized.
- Utilizing other water sources (raw CAP and reclaimed water) would require a separate ancillary infrastructure system (pipelines, booster stations, water treatment process (ozone), filtration system, and other equipment), acquisition of additional land, dual treatment for injected and recovered water supplies, and a potentially long and exhaustive effort to coordinate with the public, stakeholders, and developers.
- The chemistry of raw CAP and reclaimed water means there is a higher probability of causing mechanical and biological clogging of the ASR well, which would require additional O&M practices and management. Injecting treated CAP water would be most cost effective because it would reduce biological/chemical clogging issues.
- If reclaimed water was to be utilized for ASR, the amount of water available for recharge would be limited by customer demands for irrigation and seasonal variations (dry winter versus wet winter).
- For Scenarios 2, 3, and 4, there would be a lack of operational flexibility (i.e., emergency shut down at Union Hills) since additional water treatment at Union Hills would be required after recovery.
- Since Scenarios 2, 3, and 4 were defined as closed well/distribution systems, additional operational staff would be required to manage and maintain the systems.

ASR is an effective water resource management tool that is being applied throughout the world to help manage declining groundwater-levels, provide emergency back-up of surface water supplies, and facilitate cost-effective long-term storage. The life-cycle cost analysis summarized in this document identified Scenario 1 to be the most practical application of ASR for this case study. By taking a proactive role in managing declining groundwater levels with ASR, we will be able to protect and sustain production capacity of our existing well-field and also reduce subsidence in the northeast region of Phoenix. Our initial objective for ASR was to develop a system that can achieve both emergency flexibility and long-term storage (drought), but this analysis, resulted in the realization that ASR may be capable of resolving peak demands in localized areas in our system throughout the year. Based on the various water resource management needs for our water system, we believe that ASR will become an important water resource management strategy for Phoenix's future.



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## MAINTENANCE TECHNIQUES UTILIZED BY CENTRAL ARIZONA PROJECT TO OPTIMIZE RECHARGE

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### ABSTRACT

The Central Arizona Project (CAP) operates six recharge projects in Maricopa and Pima Counties, Arizona. CAP began recharge operations in 1996 with the latest project coming on-line in 2006. Operation of the facilities includes remote data acquisition and project operations, basin maintenance to rejuvenate infiltration rates, weed control, and repairing storm and vandalism damage. Maintenance techniques have varied over the years as CAP has gained experience operating the individual projects.

The simplest way to restore infiltration rates is to dry the basin and allow the fine-grained material to dry and form desiccation cracks. When drying the basins is no longer effective, the basins are scarified using a spring-tooth harrow. Finally, the basins are scraped using a paddle wheel scraper, taking off the top 6 (15 cm) to 12 (30 cm) inches of soil.

Other maintenance activities include weed control, damage repair, and remote data acquisition systems that monitor flow and basin water levels which is relayed to CAP Headquarters and used to make operational decisions. In the past, weeds and tamarisks were sprayed with Rodeo. Today, the weed and tamarisk growth are controlled using goats that are brought in twice a year to eat the plants.

### KEYWORDS

Arizona, Artificial Recharge, Maintenance, Remote Data Acquisition, Water, Weed Control

### INTRODUCTION

CAP provides a renewable source of water for central Arizona's agricultural and municipal customers. The project delivers Colorado River water from Lake Havasu through Phoenix down to Tucson via a 336-mile (540 km) aqueduct, 14 pumping plants, and three tunnels. In 2006, CAP delivered 1.6 million acre-feet (1.97 billion cubic meters) of water to its agricultural, municipal, and Indian customers. In addition to direct deliveries, CAP has developed an extensive artificial groundwater recharge program (Figure 1).

CAP currently operates six recharge projects in Maricopa and Pima Counties with a combined size of approximately 430 acres (174 hectare) of recharge basins with the ability to recharge over 300,000 acre-feet/yr (370 million cubic meter/yr) (Table 1). A

seventh project is in the design phase and will add an additional 150 acres (61 hectare) of basins and approximately 56,500 acre-feet/yr (70 million cubic meter/yr) capacity. The first project, Avra Valley Recharge Project (AVRP), was brought on-line in 1996 with the most recent, Tonopah Desert Recharge Project (TDRP), beginning operations in 2006 (Figure 2).

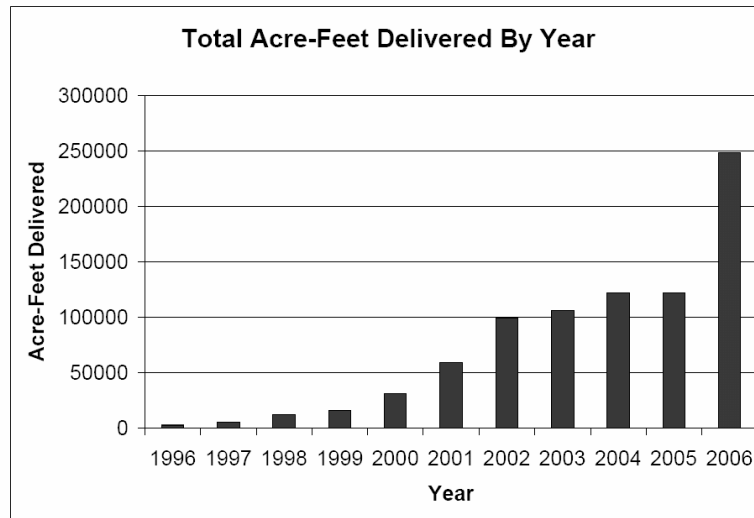
There are 235 miles (378 km) of canal separating TDRP to the west and Pima Mine Road Recharge Project (PMRRP) to the south. This presents challenges in keeping the projects properly managed and maintained. CAP staffs a fully equipped group that performs all the maintenance work for the recharge projects. The Maintenance Department includes maintenance engineers, planners, repair and fabrication shops, and field crews. Operation of the recharge facilities includes remote data acquisition and project operations, basin maintenance to rejuvenate infiltration rates, weed control, and repairing storm and vandalism damage.



**FIGURE 1. Location map of the CAP recharge projects.**

**TABLE 2. CAP's recharge projects.**

| <b>Project</b>                                 | <b>Year Operational</b> | <b>Acres of Basins</b> | <b>Annual Capacity</b> |
|------------------------------------------------|-------------------------|------------------------|------------------------|
| Avra Valley Recharge Project (AVRP)            | 1996                    | 11                     | 6,500 AF/YR            |
| Pima Mine Road Recharge Project (PMRRP)        | 1998                    | 37                     | 30,000 AF/YR           |
| Lower Santa Cruz Recharge Project (LSCR)       | 2000                    | 30                     | 50,000 AF/YR           |
| Agua Fria Recharge Project (AFRP)              | 2002                    | 93                     | 35,000 AF/YR           |
| Hieroglyphic Mountains Recharge Project (HMRP) | 2003                    | 38                     | 35,000 AF/YR           |
| Tonopah Desert Recharge Project (TDRP)         | 2006                    | 220                    | 150,000 AF/YR          |
| Superstition Mountains Recharge Project (SMRP) | 2009                    | 150                    | 56,500 AF/YR           |



**FIGURE 2. Total acre-feet delivered by year for all six projects. The large jump in 2006 is due to TDRP coming on-line.**

## REMOTE DATA ACQUISITION AND OPERATION

All CAP direct recharge projects have some level of data telemetry to facilitate operation of the project. CAP projects are permitted by the Arizona Department of Water Resources (ADWR) and have different monitoring, reporting, and operational rules that drive project operations. Permits generally require that project flows, basin water levels, monitoring well and piezometer water levels be recorded at regular intervals. CAP collects this data using a variety of equipment and stores all the data in a central database.

Each of the CAP projects has different levels of automation. Some projects meet the minimum in terms of data collection to operate within permit compliance, while others have a partial control component that allow for remote shutdown and, in some cases, start-up of the project. CAP's newest and largest direct recharge project, TDRP, was designed and constructed with a complete monitoring and operational control system that delivers water from the CAP canal to the constructed infiltration basins.

TDRP consists of 19 individual infiltration basins, and the required piping and associated equipment needed to distribute water across the project site. Each basin consists of an ultrasonic flow meter, water level indicators, a motor operated valve, and a local Programmable Logic Controller (PLC). These devices are used to regulate flow into each of the infiltration basins. All of the data collected at each of the basins is communicated over a local Ethernet to a control building on site. At the

control building there is a main PLC that manages and transmits project data from the site to the CAP control center, and the main PLC performs command and control functions for operating the turnout and the individual basins from CAP's central operations center.

Project automation allows integration of TDRP operations into the regular operation of the CAP canal system. This creates efficiency in utilizing CAP's human resources by not requiring additional people to perform operations. In addition, the ability to make remote real-time flow changes increases the project efficiency, which translates into more water being stored at the project with the use of fewer basins. The real-time telemetry allows the operator to monitor water levels and calculate real-time infiltration rates to support reliable operation and determine maintenance cycles. Telemetry and storage of data in a central database also allows for the automation of billing and reporting functions, further reducing the amount of human overhead to operate the project.

A less obvious benefit of project automation is the integration into the main operations of the CAP canal system. Because direct recharge is, by definition, storing water for use at a later date, the reliable timing and quantity of the delivery is insignificant when compared to a water user who is relying on the delivery of water for direct use. For example, a municipal treatment plant or an agricultural user requires that water be delivered at the right time and in the right amount. Direct recharge projects do not require this level of reliability. Deliveries can be reduced in times of high water demand and increased at times of low water demand, thereby creating peaking capacity within the existing delivery system. Project automation further enhances peaking capacity by making changes to the project deliveries available at the push of a button.

In the future, CAP plans to increase the remote data acquisition and project automation at all the recharge sites. The last CAP recharge project, Superstition Mountains Recharge Project (SMRP), is being designed to be fully automated, similar to TDRP.

The main function of the telemetry is to allow the operators at the CAP headquarters to make operational decisions that maximize recharge deliveries. Decisions on basin maintenance, delivery rates, and emergency responses are optimized through the use of the real time data.

## **INFILTRATION RATE FACTORS**

All of CAP's recharge facilities are direct surface infiltration systems consisting of constructed spreading basins or in-stream natural channel flow. CAP does not operate well injection facilities of any kind. Recharge rates vary from project to project and range from 2 ft/day (0.6 m/day) up to 8 ft/day (2.4 m/day). Algae and

minor amounts of wind blown silt and clay are the main contributors for reducing infiltration rates in the recharge projects. Sediment from the Colorado River settles out quickly and is generally not a factor at the recharge projects. Sediment does become an issue during storm events and during high releases from Alamo Dam (upstream of the intake at Lake Havasu on the Bill Williams River). Previous investigations have identified the algae as primarily diatoms, blue-green algae, unicellular green algae, filamentous algae, and recently chara (Figure 3).

In the past, techniques to rejuvenate infiltration rates included disking or ripping the basins. These techniques work in the short term, but because they churn the fine-grained material deeper into the soil, they reduce the infiltration rate in the long term. Today, CAP utilizes a series of techniques that begin at the least costly method and progresses to the more time consuming and higher cost options.



**FIGURE 3. Algae growth in the basins at AVR.**

## **BASIN MAINTENANCE TECHNIQUES**

The simplest way to restore infiltration rates is to dry the basin and allow the fine-grained material to form desiccation cracks (Figure 4). When drying the basins is no longer effective, the basins are scarified using a spring-tooth harrow that breaks up the soil in an upward motion rather than churning it deeper into the soil layer. Finally the basins are scraped using a paddle wheel scraper taking off the top 6 (15 cm) to 12 (30 cm) inches of soil (Figure 5). The material is removed and stored on nearby spoil piles so the fines will not be washed back into the basin area. After scraping the basin bottom, the soil is ripped 12 (30 cm) to 24 (61 cm) inches deep to loosen the compaction caused by the scraper.

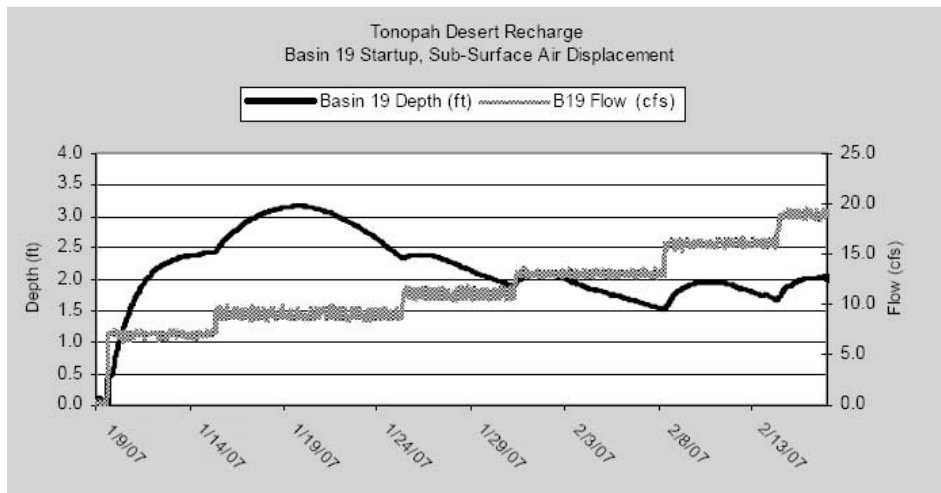
The basins generally need to be dried for at least two weeks before any equipment can enter to perform work. Once deliveries have resumed, the flow needs to be increased slowly because of air entrainment in the vadose zone. The water level in the basins will rise until air is vacated from the soil pores, then flow can be increased to the basin (Figure 6). The time it takes for the air entrainment to no longer affect the infiltration rates varies from project to project and ranges from a few days to six weeks.



**FIGURE 4. Desiccation cracks in the basins at HMRP.**



**FIGURE 5. 623 CAT elevator (paddle wheel) scraper in Basin 1 at LSCR.**



**FIGURE 6. Water level versus basin flows at TDRP. Note that at each rise in deliveries, the water level in the basin first rises then drops as the entrained air escapes from the vadose zone.**

## WEED CONTROL

A major aspect of maintaining the basins involves controlling the growth of weeds and woody plants. Tamarisk (salt cedar) is an evasive species that occurs in all the recharge facilities. It is nearly impossible to eradicate and is difficult to control. In the past, CAP used the herbicide Rodeo to control the plant growth. While the Rodeo was effective on many of the weeds and plants, it was not effective on the tamarisk. Also, Rodeo is the only herbicide that the ADWR has permitted for use on the recharge project and the cost to spray is considerable. As an alternative, CAP has been utilizing goats for weed control (Figure 8).



**FIGURE 8. Goats grazing at HMRP.**



The goats have been a success and, while they don't kill the tamarisk, they do help to keep the plants manageable. If the tamarisks are too large, or there are too many, they are removed manually. The removal method involves digging them out using a backhoe, piling up the tamarisk, and then either chipping or burning the plants (Figure 9).



**FIGURE 9. IT 14 loader with a brushhog attachment picking up loose tamarisk at LSCR (note the 310 G John Deere backhoe in the background).**

In 2006 at HMRP, there was an outbreak of the algae chara that had not previously been seen in the recharge basins. Fish have been successfully used in the CAP canal to control different types of algae. An attempt to stock the basin with grass carp, however, was unsuccessful. After releasing the fish into the basin, they were not seen again, and likely became a nearby great blue heron's dinner.

## CONCLUSIONS

The recharge program helps CAP meet its mission to be the steward of central Arizona's Colorado River entitlement and a collaborative leader in Arizona's water community. The maintenance program ensures that the recharge projects operate as efficiently as possible to optimize deliveries. Additional information about CAP and its recharge program can be found on the website at [www.cap-az.com](http://www.cap-az.com).

## FOUNTAIN HILLS SANITARY DISTRICT: ASR WELL REHAB — IT'S A MUST.

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### ABSTRACT

As Aquifer Storage and Recovery (ASR) wells mature, they exhibit a decrease in performance leaving rehabilitation a must. Sediment and/or biological materials deposited in the gravel pack and along the borehole wall generally causes the decrease in well performance. Some strategies of well rehabilitation with well development techniques include mechanical cleaning, chemical treatment and the Aqua Freed process. Mechanical cleaning by itself is generally not enough to make a significant improvement in the well's performance and therefore a combination of techniques are generally employed. The chemical treatment of ASR wells typically starts with using various acids to dissolve mineral precipitates like calcium carbonate, sulfate, magnesium hydroxide, iron and manganese oxides, phosphates, and silicates. The chemical treatment helps to terminate biological growth but does not remove the remains. Finally, the Aqua Freed process injects a gaseous carbon dioxide into the aquifer as a highly effective medium for removing a wide variety of plugging deposits from the well screen, gravel pack and/or rock surfaces in the water environment. Results have shown that if chlorine treatment is no longer successful, then a mechanical cleaning followed by the Aqua Freed process is the most cost effective with the highest rate of success.

### INTRODUCTION

#### Brief History of FHSD

Established in 1969, The Fountain Hills Sanitary District (FHSD) operates in accordance with Title 48 of the Arizona Revised Statutes. Its mission is to collect, treat, and dispose of wastewater and the byproducts produced within its boundaries. Governed by an elected five-member Board of Directors, FHSD serves approximately 12,700 residential units and 300 commercial units.

#### Overview of Treatment Process

Raw sewage is conveyed to the main wastewater treatment plant where tertiary treatment including denitrification on the wastewater prior to reuse on local golf

courses and parks. Any excess reclaimed water is microfiltered, disinfected by ultraviolet light, and distributed to the ASR wells for recharge.

## **Overview of Hydrogeology**

### **Geologic Conditions**

The geology of the Fountain Hills area is described in a report entitled “Geologic Map of the Fountain Hills/Mount McDowell Area Maricopa County, Arizona by Skotnicki (1995). Most of the Fountain Hills area is of Tertiary age (2 to 20 million years old) exposing deposits of sedimentary basin fill (younger) and of conglomerate, breccia, and sandstone (closer to 20 million years old). In the Fountain Hills area, these deposits occur within and recent alluvial material occurs between the ridges.

Near the Fountain Hills Recharge Facility, the land surface elevations generally range from 1,500 to 1,700 feet above mean sea level (amsl). The majority of the surface drainage within the study area is from the west to the Verde River system east of the Town of Fountain Hills. The one exception to this is in the southern area (south of Shea Boulevard) where the surface drainage is south to the Salt River. All of the surface water drains via ephemeral washes within the study area.

Red-bed deposits occur in the southeastern portion of the study area. The red-bed deposits consist of an upper fluvial unit and a lower red conglomerate breccia unit. A breccia rock is made of highly angular coarse fragments of various sizes. The upper unit consists of tilted, tan-colored, and well-bedded fluvial conglomerates and sandstones mostly of sub-angular to sub-rounded clasts of volcanic and metamorphic rocks. The breccia makes up most of the red-bed deposits. The breccia that makes up the lower unit of the red-bed deposits contains fragment sizes that range from silt to boulders (Skotnicki, 1995).

### **Hydrogeologic Conditions**

A geologic cross-section based on the limited data available for the study area indicates a confined aquifer system within the Fountain Hills area (Figure 1). A significant amount of clay occurs in the upper layers, which is approximately 400' in the area south of Fountain Lake. Below the clay unit occurs a conglomerate material identified by the well driller's logs. Figure 1 also shows the rise in water levels or potentiometric surface above the conglomerate unit that indicates confined aquifer conditions.

Nemecek and Briggs (1975) report transmissivity values derived from aquifer tests of wells in the Fountain Hills area range from 20,000 to 40,000 gpd/ft. Reported storage coefficients range from  $7.8 \times 10^{-4}$  to  $2.8 \times 10^{-3}$ . Nemecek recommends using an average storage value of  $3.0 \times 10^{-2}$  to determine the available groundwater in storage in the Fountain Hills area.

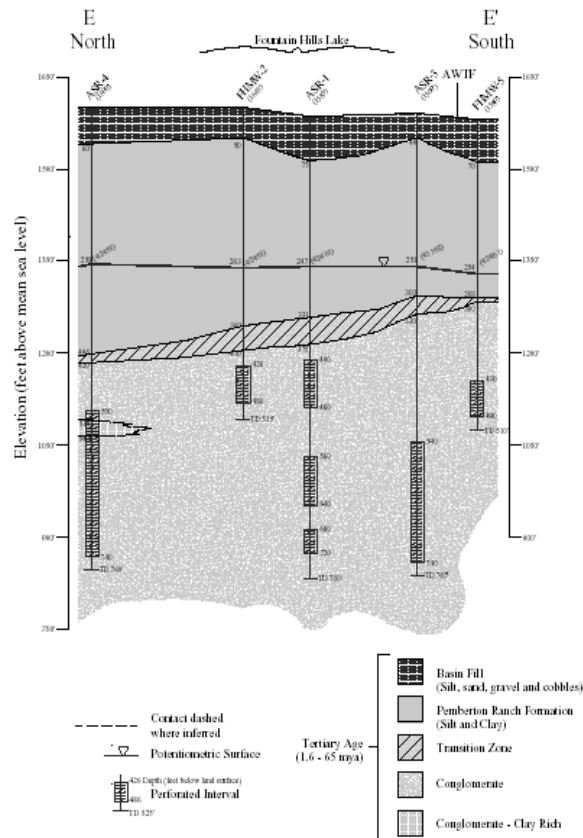
## Overview of ASR Recharge System

### Well Design

The ASR well designs have an average injection capacity of 400 gpm each. The wells are drilled to an average depth of 750 ft bls. The wellheads are below grade vault structures. The ASR wells are protected from surface contamination using a 30" conductor casing from land surface to 100 ft bls. The ASR wells are constructed with 12" in stainless steel blank casing from land surface to approximately 450 ft bls. Below 450ft, the louvered stainless steel well casing was staged according to well cutting and geophysics. The ASR wells are also equipped with a sounding tube, and sampling tube.

Source water conveys to the top of the injection well via an eight in diameter pipeline. The wells are equipped with an ultrasonic flow meter, a downhole flow control valve to regulate the volume of the water flowing into the well, and a water level transducer to measure the depth of the water within the injection well. In addition, the ASR wells connect to a SCADA system that transmits continual volumetric and instantaneous operational data to the District's control room.

**Vault completion of the ASR wells on land surface, the soil is excavated surrounding the wells to make room for the steel vaults (Photo 1). Each vault has a "dry" side and**



**FIGURE 1. Cross-section E – E'.**

“wet” side and work as one unit. The dry side of the vault houses all of the electrical controls, piping, metering equipment, compressed air tanks, and other equipment that must remain dry during the well’s operation. A concrete pad constructed at the bottom of the pit prior to vault placement serves as the bottom to only the dry side of the vault. The wet side houses the wellhead, piping and any other equipment that does not need to remain dry. Also included in the wet-side is a waterproof circuit breaker for the submersible pump. The wet side has a gravel bottom.

Traffic rated vault doors rest at grade. The wet side requires two 48 in by 36 in doors and the dry side one 36 in by 36 in door. The vault dimensions are 48 in by 72 in by 96 in deep each. *Photo 2* shows the vault with the dry side on the right and the wet side on the left. Descriptions of some of the equipment housed in the vaults and wells are in the following sections.



*Photo 1.* Well and concrete pad prior to vault placement.



*Photo 2.* Well and concrete pad housed inside vault.

## METHODS

### Well Rehab Techniques

As an ASR well matures, it experiences loss in well capacity. Sediment and/or biological materials deposit into the gravel pack materials and along the borehole wall, which generally leads to this loss. There are many strategies for well rehabilitation along with well development techniques in an attempt to restore the well's specific capacity. According to Pyne (2005), there are five main factors affecting plugging: 1) entrained air and gas binding, 2) deposition of total suspended solids (TSS), 3) biological growth, 4) geochemical reactions, and 5) particle rearrangement. At the FHSD, four of the five forms of well plugging may have occurred over time. TSS is not a factor due to the utilization of microfilters. Three well rehab techniques improved the well capacity of the ASR wells. These techniques include mechanical cleaning, chemical treatment, and the Aqua Freed process.

### Mechanical Cleaning

The mechanical process includes the typical wire brushing of the well screen section to remove any buildup on the inside of the casing that results in well blockage (*Photo 3*). After brushing the well, it is necessary to bail or airlift the loosened material, which is part of the well's development. Further well development is typically done with a test pump to surge and over pump the well. The mechanical process by itself is generally not enough to make much of an improvement in the well's capacity and therefore a combination of techniques are generally employed



*Photo 3. Wire brush*

### Chemical Treatment



*Photo 4. Mixing in chemicals*

The chemical treatment of ASR wells typically starts with various acids to dissolve mineral precipitates like calcium carbonate, sulfate, magnesium hydroxide, iron and manganese oxides, phosphates, and silicates (*Photo 4*). The most common and useful mineral acids used in water well rehabilitation projects are hydrochloric or muriatic acid, phosphoric acid and sulfamic acid (Schnieders, 2003).

The described chemical treatment is a four-step process. The first step is the evaluation of the water quality of the native groundwater, the injection source water, and the blended water pumped from the well. This data will provide a basis for the selection of the chemicals needed to treat the well. The second step is the neutralization of the chemicals that remain in the water after the treatment period has ended. The third step is the use of disinfectant chemicals (chlorine) to treat the well and to help prevent the rapid growth of bacteria not affected by the chemical treatment. The fourth and final step is the chlorine discharge neutralization associated with the final pumping of well prior to the injection mode.

### Aqua Freed

A third process that is becoming more popular in ASR well rehabilitation is the Aqua Freed process. This process injects a gaseous carbon dioxide into the aquifer as a highly effective medium for removing a wide variety of plugging deposits from the well screen, gravel pack and/or rock surfaces in the water environments.



*Photo 5. Aqua Freed setup*

The Aqua Freed process requires the placement of up to three injection lines at specific depths within the screen section of the well (*Photo 5*). The injection lines will direct the gaseous carbon dioxide to the designated treatment area creating a highly carbonic acid solution penetrating the aquifer formation. The carbon dioxide reacts to water exposure by rapidly expanding and producing incredible agitation. This process causes freezing of water within the formation, resulting in disinfection and dislodging of mineral encrustation. With the injection tubes removed and the Aqua Freed treatment completed, swabbing and airlifting techniques remove the newly freed particulate matter from the well and the aquifer. Further development of the well consists of installing a test pump for surging and over pumping prior to installing the permanent pumping equipment.

## RESULTS AND DISCUSSION

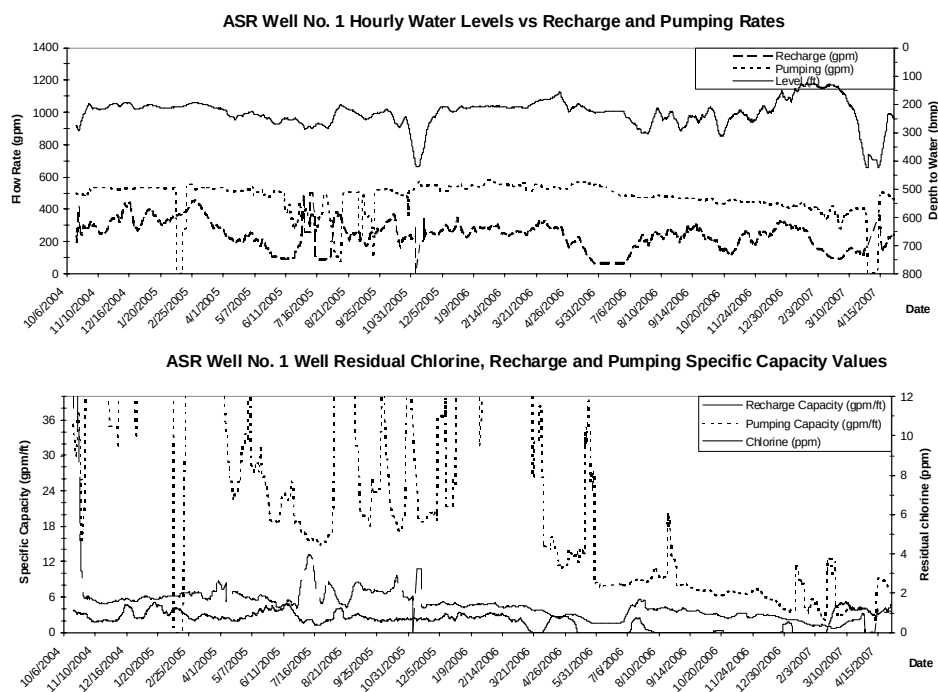
### Overview of ASR Well Performance

Since the construction of the ASR wells in 2001 to the present, operational data is continually collected. However, the operational data from 2002 and 2003 was lost due to SCADA system problems. In addition to operational data, residual chlorine data is available from October 2004 to the present. The time duration selection, October 2004 – present, contains a complete record of the pumping and recharge volumes, water levels, and residual chlorine information necessary to help identify the main factors affecting the well plugging and the performance of ASR wells. In March of 2005, the FHSD switched from using chlorine as the primary disinfection to ultra violet light due to the formation of trihalomethanes (THMs).

#### *Recharge and Pumping Specific capacity*

Figure 2 represents the operational data displayed in two separate graphs for ASR Well No. 1. The top graph presents the water level fluctuations during recharge (injection) and pumping with flow rates. The bottom graph shows residual chlorine with recharge and pumping well capacity. Recharge specific capacity (gpm/ft) is the rate of recharge that occurs for every one foot of water level rise whereas pumping specific capacity is the rate of pumping that occurs for every one foot of drawdown. The higher well performance is directly related to the presence of a chlorine residual in the source water. This is an indication that biological growth could be a major cause of well plugging.

The other operational data plots for ASR Well No. 2, ASR Well No. 3 and ASR Well No. 4 are very similar to Figure 2 with regard to the effect of chlorine on both the pumping and the recharge flow rates. However, because of paper length constraints, they are not included in this paper.



**FIGURE 2. ASR Well No. 1 Operational data.**

Pyne (2005), describes that accumulation of impermeable slimes, the development of a mat of dead cells, and the dispersion or alteration of colloidal particles in the aquifer matrix, are some of the components of biological growth leading to well plugging. Three months before residual chlorine was removed from the source water, the recharge capacity values were reasonably steady with an average value of 4.89 gallons per minute per foot (gpm/ft), 4.4 gpm/ft, 4.7 gpm/ft, and 3.43 gpm/ft, respectively, for ASR Well No. 1, ASR Well No. 2, ASR Well No. 3, and ASR Well No. 4. However, approximately one month after the disengagement of chlorine from the system, the recharge capacity dropped to 1.82 gpm/ft, 2.08 gpm/ft, 1.73 gpm/ft, and 1.28 gpm/ft, respectively for ASR Well No. 1, ASR Well No. 2, ASR Well No. 3, and ASR Well No. 4 (Table 1). The impact of removing the chlorine residual from the source water resulted in a decrease in recharge capacity in the range of 53%-64%.

Beginning of April 2006, a residual chlorine value of 0.84 ppm was sustained for approximately one month after being introduced back into the system.



The addition of the residual chlorine caused an immediate increase in the well performance reaching recharge capacity levels similar to the ones before the removal of chlorine from the system (Table 1).

**TABLE 1. Recharge Capacity value for FHSD ASR Wells with and without Chlorine.**

| Recharge Capacity Value (gpm/ft) |                             |                   |                        |
|----------------------------------|-----------------------------|-------------------|------------------------|
| ASR Wells                        | Average<br>(Jan-March 2006) | End<br>March 2006 | Percentage<br>Decrease |
| No. 1                            | 4.89                        | 1.82              | 63                     |
| No. 2                            | 4.4                         | 2.08              | 53                     |
| No. 3                            | 4.76                        | 1.73              | 64                     |
| No. 4                            | 3.43                        | 1.28              | 63                     |

The FHSD operation behaves more like a “Put and Take System: whereby its ASR wells operate in a recharge mode during the winter months and in pumping mode for the summer months. The operational data indicates that the highest well performances are directly related to the end of the pumping periods for both with and without chlorine residual in the system. This positive short-term impact on well performance is likely due to the removal of the dead bacteria, biomass, TSS, etc. being removed from the wells during the pumping period.

The operational data displays another important aspect of ASR well performance, a continued long-term decline in overall efficiency. Recharge and pumping specific capacities have decreased in the system to an average of 78% and 73% of their 2004 values to the present (Table 2).

**TABLE 2. Beginning and Present Specific Capacity Values for FHSD ASR Wells.**

|                          | Recharge Specific Capacity (gpm/ft) |                                                         |                             | Pumping Specific Capacity (gpm/ft) |                                                         |                             |    |
|--------------------------|-------------------------------------|---------------------------------------------------------|-----------------------------|------------------------------------|---------------------------------------------------------|-----------------------------|----|
| ASR Wells                | Beginning<br>(End 2004)             | Present Before<br>Rehabilitation<br>(End 2006-Beg 2007) | Percent-<br>age<br>Decrease | Beginning<br>(Middle 2005)         | Present Before<br>Rehabilitation<br>(End 2006-Beg 2007) | Percent-<br>age<br>Decrease |    |
| No. 1                    | 5.66                                | 0.62                                                    | 89                          | 16.00                              | 3.56                                                    | 78                          |    |
| No. 2                    | 5.53                                | 1.93                                                    | 65                          | 7.88                               | 3.35                                                    | 57                          |    |
| No. 3                    | 4.51                                | 0.34                                                    | 92                          | 4.80                               | 1.00                                                    | 79                          |    |
| No. 4                    | 4.31                                | 1.41                                                    | 67                          | 7.90                               | 1.78                                                    | 77                          |    |
| System Average Decrease: |                                     |                                                         | 78                          | System Average Decrease:           |                                                         |                             | 73 |

By the end of 2006, all of the ASR wells had reached their lowest historical recharge and pumping capacities. In order to improve well performance, well rehabilitation (mechanical, chemical and Aqua Freed) was performed.

The rehabilitation program consisted of mechanical cleaning and chemical treatment on ASR Well No. 4 and the mechanical cleaning and the Aqua Freed process on ASR Well No. 1, ASR Well No. 2 and ASR Well No. 3. This provided the basis for comparing the effectiveness of the chemical treatment versus the Aqua Freed process. ASR Well No. 1, ASR Well No. 2, and ASR Well No. 3 increased their recharge specific capacity

values after well rehabilitation from 74-101% as shown in Table 3. This is an average improvement of 83%. ASR Well No. 4, which received only the mechanical and chemical treatment, improved its recharge specific capacity by 77%, which is slightly better than the worst result of the Aqua Freed process, but 6% less than the average improvement. As for pumping capacity, the ASR wells had the same improvement response but not of the same magnitude as with of recharge capacity improvement. For instance, ASR Well No. 2 showed the highest improvement for recharge capacity (68%) but this improvement is lower as compared with the 101% improvement for recharge capacity improvement. Overall, pumping capacity improvement was 20% smaller than recharge capacity. It is not clear the recharge capacity reacts better to rehabilitation techniques and chlorine; however, further study and analysis will be required to gain a better understanding of this behavior.

**TABLE 3. Recharge and Pumping Specific Capacity Improvements for ASR wells after Rehabilitation.**

|                          | Recharge Specific Capacity (gpm/ft) |                                               |                             | Pumping Specific Capacity (gpm/ft) |                                               |                             |
|--------------------------|-------------------------------------|-----------------------------------------------|-----------------------------|------------------------------------|-----------------------------------------------|-----------------------------|
| ASR Wells                | Average<br>(Jan-March 2006)         | Present-After<br>Rehabilitation<br>(Beg 2007) | Percent-<br>age<br>Increase | Average<br>(End March 2006)        | Present-After<br>Rehabilitation<br>(Beg 2007) | Percent-<br>age<br>Increase |
| No. 1                    | 4.89                                | 4.05                                          | 83                          | 14.30                              | 8.70                                          | 61                          |
| No. 2                    | 4.40                                | 4.44                                          | 101                         | 8.61                               | 5.86                                          | 68                          |
| No. 3                    | 4.76                                | 3.53                                          | 74                          | 4.72                               | 2.82                                          | 60                          |
| No. 4                    | 3.43                                | 2.63                                          | 77                          | 5.57                               | 3.66                                          | 66                          |
| System Average Increase: |                                     |                                               | 83.8                        | System Average Increase:           |                                               | 63.8                        |

Another comparison was the amount of time required to complete the mechanical cleaning and Aqua Freed process as compared to the mechanical cleaning and chemical treatment. The mechanical treatment usually required an average of one week to perform including the bailing and surging of the well. The Aqua Freed process required approximately two additional weeks to complete the process including the air lifting and pumping the well clean. The chemical treatment was the most time consuming taking approximately four weeks to work the chemical into the perforated interval of the well, allow the chemical to work, airlift, and pump the well clean. Therefore, while it only took three weeks to rehabilitate a well using mechanical cleaning and Aqua Freed, it required five weeks to perform a well rehabilitation using mechanical cleaning and chemical treatment. This is an approximate increase of 67% in time to perform the mechanical cleaning and the chemical treatment as compared to the mechanical cleaning and Aqua Freed process.

## Cost

The cost for the mechanical cleaning, the chemical treatment, and the Aqua Freed process was tracked on a well-

**TABLE 4. Summary of Well Rehabilitation Costs.**

| ASR Wells      | Mechanical Cleaning | Chemical Treatment | Aqua Freed  | Total Costs  |
|----------------|---------------------|--------------------|-------------|--------------|
| ASR Well No. 1 | \$60,644.00         | N/A                | \$49,644.00 | \$110,288.00 |
| ASR Well No. 2 | \$49,406.00         | N/A                | \$51,694.00 | \$101,100.00 |
| ASR Well No. 3 | \$47,953.00         | N/A                | \$49,644.00 | \$97,597.00  |
| ASR Well No. 4 | \$30,980.00         | \$54,034.00        | N/A         | \$85,014.00  |

by-well basis as shown in Table 4. The costs for mechanical cleaning varied from well to well depending on the amount of time that was required to clean the inside of the well casing and remove the loosened material from the well. Two chemical treatments were performed on ASR Well No. 4 where an acid application was worked into the well throughout the perforated interval using a double rubber swab. The first chemical treatment used a combination of Aqua Clear AE and some mild hydrochloric acid. The well sat idle for 24 hours before pumping. There was only slight improvement in the well's performance. A second chemical treatment was conducted using a higher concentration of acid leading to a significant improvement in the well's performance. The Aqua Freed process utilized the same 20-ton volume of CO<sub>2</sub> at each of the well sites with a small discount after the first well, therefore the costs per well are very similar.

The average overall costs for the mechanical cleaning and Aqua Freed process was approximately \$103,000 per well as compared to the mechanical cleaning and acid treatment at \$85,000. This is a cost difference of approximately 17%.

## CONCLUSIONS

The FHSD operates four state-of-the-art ASR wells drilled to an average depth of 750 feet and are completed in a wet/dry partitioned steel vault. The wells operated remotely as part of the overall SCADA system. Water level transducers and flow meters allow the SCADA system to operate the wells in an automated mode whereby controlling recharge and pumping as a specific total volume or as specific flow rate. Should the water level move outside the normal operating range, the SCADA system will alarm prior to flow modification or shut down.

These wells have operated for several years with only minor plugging until the removal of the chlorine residual from the source water. Without the chlorine residual, all of the wells experienced biological plugging during the injection of microfiltered effluent. Recharge specific capacities have declined from 65-92% and pumping specific capacities also declined from 57-79%.

Three well rehabilitation techniques were used to help restore the specific capacities in the wells. These techniques include mechanical cleaning, chemical treatment and the Aqua Freed process. The well rehabilitation processes were used in dual combination not individually. Recharge specific capacities showed an increase after well rehabilitation of 74-101%. The Aqua Freed process showed the most overall improvement.

The costs associated with well rehabilitation are significant and average approximately \$100,000 per well. This level of maintenance is generally required every three to five years. Minor maintenance is required on an annual basis.

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# UNCERTAINTY ANALYSIS OF MOUND MONITORING FOR RECHARGED WATER FROM SURFACE SPREADING BASINS

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## ABSTRACT

This study has conceptually reviewed issues related to implementation of a groundwater mound monitoring well (GMMW) for monitoring recharged water from a surface spreading basin with emphasis on uncertain hydrogeological conditions. For this study, a site was selected in the City of Mesa, Arizona, that is characterized with near surface clay lenses of low permeability. A geostatistical simulation technique was used for generating hydrogeological fields under the recharge basin, using soil boring logs and historical hydrological data. More than 50 hydrogeological fields were generated and used for modeling. Five scenarios were formulated with varying parameter values and different initial and boundary conditions and each scenario was evaluated with the 50 hydrogeological fields generated. Results of this study indicate that travel times to the mound may vary by over one order of magnitude and the use of GMMW will only be practical for regulatory compliance in a homogeneous system. Multiple depth monitoring in the saturated zone downgradient from the recharge basins is recommended to provide samples representative of the potential impact on aquifer water quality.

## KEYWORDS

Groundwater mound monitoring well (GMMW), surface recharge basin, uncertainty analysis

## INTRODUCTION

A growing number of communities have been motivated in the last few decades to implement water reuse. The need for additional water resources in some urban areas has made water reclamation for non-potable reuse more feasible than developing

new sources of fresh water. This trend extends to indirect portable reuse in some arid urban areas.

There are many issues to review in indirect potable reuses, due to concerns with possible chemical and microbial contamination. Even if analytical tests and toxicological and epidemiological studies show no significant health risks, (NRC, 1998) these concerns create uncertainties regarding the potential health risks of drinking reclaimed water. Therefore, the NRC (1998) viewed planned use of reclaimed water to augment potable water supplies as a solution of last resort, and recommended safety measures such as the use of multiple barriers with strict performance evaluation and monitoring.

In a discussion about monitoring (Crook et al., 2000), the use of a groundwater mound monitoring well (GMMW) for surface recharge basins was addressed as follows: "Using the new sampling technology, it is possible to collect water quality samples from the mound above the point at which it mingles with the native groundwater." (The point is defined as the groundwater mound monitoring point, GMMP, for convenience.) There will be many GMMPs in the mound and some of them will be selected as locations for GMMWs. The concept was to allow for compliance with water quality standards just before recharged water mixes with the native groundwater.

The GMMW should be located to sample the earliest waves of infiltration under various operational conditions of surface recharge basins, to take samples from each wet/dry cycle before infiltrating water mixes with the native groundwater. It is quite possible that infiltrating water paths vary between cycles as initial moisture conditions change. McCord et. al (1997) reported that even where application of water to the ground surface is relatively spatially uniform, unsaturated fluid fluxes in the underlying soils are often highly spatially variable since material parameters usually display spatial variations within a geologic formation. The current practices for determining infiltration, however, rely on limited data to cover such geologic variations and thus the estimation of the paths inevitably bears a high degree of uncertainty.

The technical issues regarding the field application of the GMMW concept are as follows: Identifying the earliest wave of infiltration from a surface basin to the GMMPs for each cycle of operation; and locating one or multiple GMMPs as GMMWs which will catch the earliest waves with a given probability. To effectively deal with the two technical issues, we need to estimate the uncertainties in flowpaths in relation to heterogeneous hydrogeologic conditions as well as other operational conditions. This study was initiated to estimate the uncertainty of these issues for the application of the GMMW concept. The uncertainty was analyzed with emphasis on heterogeneous hydrogeologic conditions below a surface recharge basin, recognizing there are uncertainties from other sources.

## METHODS

The methodology that this study adopts to estimate the uncertainties discussed above is briefly introduced as follows:

2. Simulation of infiltration waves for a solute under with various hydrogeologic conditions that were generated with geostatistical simulation techniques using real field data.
3. For each simulation, travel time estimations were made for a solute to points just above native groundwater. Selection of the points was based compliance with the GMMW concept (Crook et al., 2000).
4. A point was selected with the shortest travel time as a GMMP for the realization.
5. Hundreds of simulations were done to locate an array of GMMPs and select some of them as GMMWs with a certain level of probability of sampling the earliest infiltration waves.
6. What-if scenarios were done to gain insight into related issues. For example, evaluation of hydro-geologic conditions, of initial and boundary conditions, and of modeling parameters for the location of GMMW and sampling requirements (for estimating soil and other hydrogeologic conditions). In addition, cost-effectiveness of field application of the GMMW concept was considered.

**Geostatistical Simulation:** To generate hydrogeologic conditions from limited data, geostatistical simulation techniques were used in this study. The geostatistical methods selected for this study are as follows:

**Variogram:** Typical methods for describing spatial variation (or continuity) of geo-environmental data include variograms. A variogram is a plot of the average squared differences between paired data values as a function of separation distance.

**Kriging:** The word “kriging” is synonymous with “optimal prediction” and kriging is a weighted moving average technique for interpolating unknown values from data observed at known adjacent locations. There are many kriging techniques and based the quantity and quality of data, simple and ordinary kriging was used with software developed by Deutsch and Journel (1998) for this research.

The geostatistical simulation used was a Monte Carlo simulation for generating spatial distribution of a property under study, based on a variogram model chosen to represent a probability distribution of the property. Different from geostatistical estimation, geostatistical simulation can not only reproduce an accurate spatial distribution but also generate many equally probable alternative distributions (Desbarats, 1996).

Sequential Gaussian Simulation was used in this research following the procedure described by Desbarats, 1996. For multiple simulations, this procedure was repeated. From the results of multiple simulations, the uncertainty was estimated. The software package GSLIB developed by Deutsch and Journel (1998) was used for this simulation

**Solute Transport Modeling** VS2DI: VS2DI from USGS (U. G. Geological Survey) consists was used to simulate solute transport modeling during this research.

## RESULTS AND DISCUSSION

A recharge facility at the Northwest Water Reclamation Plant (NWWRP) operated by the City of Mesa, Arizona was selected as an example site for this research. Data necessary for this research were obtained from previous studies that include the works of Johnson (2000), Schönheinz and Drewes (2000) and the NCSWS (1999).

For the objectives of this study, i.e., reviewing conceptually the applicability of the GMMW concept, the modeling and analysis approaches are simplified as follows:

- Modeling the site in 2 dimension using only x and z directions. (A-A in Figure 3), including only a mound under the basins.
- Considering only hydrogeological conditions represented by spatial distribution of horizontal and vertical hydrologic conductivities

The effects of initial moisture content and surface loading conditions on the GMMW concept was reviewed with the sensitivity analysis.

**Site Description:** The site located in Mesa, Arizona, recharges groundwater to the East Salt River Valley (ESRV) subbasin of the Salt River Valley (SRV) groundwater basin (Johnson, M. J., 2000). Of the 27 acres of recharge basins, a cross-section with a width of 1,818m and a depth of 24m was modeled. The groundwater table was 13m deep below the basin surface and therefore, 13m was considered for the unsaturated flow modeling and a layer of 11.5m was included as the saturated zone. Using intensive piezometric data collected during July- August, 1999 and March 2000 and FEMWATER model calibration results, Johnson (2000) generated a spatial distribution of the hydraulic conductivity, including low-permeability layers. Five layers of low permeability were identified and the remaining layers had a vertical conductivity of 1.5 m/day and an anisotropic ratio of 10 throughout.

**Modeling Initiation:** Initial modeling of the site for this study was done as follows:

**Groundwater Table Static:** The duration of infiltration in the mound on which this study focuses was only a couple of days or less. Fluctuation of the native groundwater table were therefore assumed negligible.



**Initial Moisture Content:** Two extreme conditions were assumed for analysis and discussion. The first one was a dry condition that might occur after an extended drying cycle. A moisture content,  $\theta$ , of 0.23 was used from the surface to the capillary fringe zone (i.e., the tension-saturated zone) located just above the groundwater table. The second one was a wet condition in which the vadose zone was almost saturated (i.e.,  $\theta \approx 0.4$ ). Based on field data, the wet condition was most representative of operating conditions.

**Surface Recharge Rates:** The simulated recharge rates developed by Johnson (2000) were used in this study.

**Wet/Dry Cycles:** Many wet/dry cycle variations are conceivable, however, only two cases were simulated to provide a basis for discussion on implementation of the GMMW concept. The first case assumes that all the basins begin to recharge at the same time for periods of 40 and 64 days, with no dry cycle. This was to generate the earliest waves of infiltration if all the basins recharge continuously and simultaneously. The other case used a cycle of 3 days wetting and 5 days drying for each basin for a period of 40 and 64 days with a recharging order of Basins 3, 1, 4 and 2, a day apart from each other. This order was obtained from the July, 1999 field observations. (NCSWS, 1999; Johnson, 2000)

**Development of Variogram:** A variogram of hydraulic conductivity was developed from a set of field data to generate the probability of its spatial distribution. The field data was obtained from the boring log data and other hydrogeological parameters used in previous studies. For this research, all these data in 3-dimension were transformed into 2-dimension (2-D). From the data in 2-D, the variogram model generated a spatial distributions of hydraulic conductivity. Vertical hydraulic conductivities were assigned to each soil types to conduct geostatistical simulation.

**Generation of Hydrogeologic Conditions Using Geostatistical Simulation:** Geostatistical simulation was done using the variogram developed above and a program called “sgsim” in GSLIB (Deutsch and Journel, 1998). The program “sgsim” conducted a sequential Gaussian simulation. The program was run 50 times each with simple kriging and ordinary kriging. One-hundred realizations of the spatial distribution of the vertical hydraulic conductivity were generated with an anisotropic ratio of 10. Averages and standard deviations of the 100 realizations were 10.87 and 0.21, respectively, for simple kriging and 11.30 and 0.5 for ordinary kriging. The difference between the averages was not significant even when comparing different kriging methods.

**Model Calibration:** To adjust and determine model parameter values and initial and boundary conditions, the VS2D model was calibrated using the distribution of hydraulic conductivity and the piezometer data obtained during a wetting cycle of July, 1999 (Johnson, 2000). The two initial water content conditions, dry and wet, were used as discussed previously. The final results which compared total heads (a)

field measured, (b) simulated in 3-D using FEMWATER (Johnson, 2000) and (c) simulated in 2-D using VS2D. There was excellent agreement among the total heads for all piezometer points where data was collected.

**Five Modeling Scenarios for Uncertainty Evaluation:** For reviewing issues with implementation of the GMMW concept in relation to hydrogeological uncertainty, scenarios with different initial and boundary conditions and different kriging methods were developed for sensitivity analysis. Using six different conditions, five scenarios were developed. As shown in Table 1, Scenario 1 is with a combination of continuous recharge, dry initial moisture content and simple kriging. Each scenario was simulated with each of the 50 realizations of the hydrogeologic conditions.

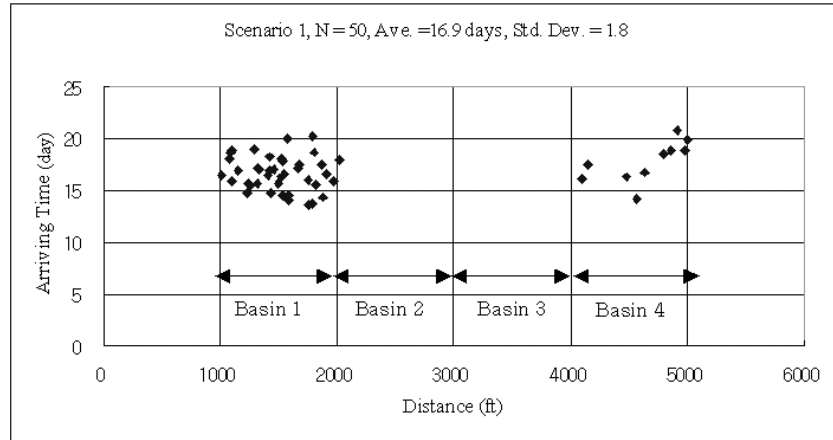
**Table 1. Five Scenarios with Different Modeling Conditions.**

|                           |                      | Simple Kriging        |            | Ordinary Kriging      |     |
|---------------------------|----------------------|-----------------------|------------|-----------------------|-----|
|                           |                      | Initial Water Content |            | Initial Water Content |     |
|                           |                      | Dry                   | Wet        | Dry                   | Wet |
| <b>Recharge Condition</b> | <b>Continuous</b>    | Scenario 1            | Scenario 2 | Scenario 5            | -   |
|                           | <b>Wet-Dry Cycle</b> | Scenario 3            | Scenario 4 | -                     | -   |

**Identifying the Earliest Wave of Infiltration with VS2D Modeling:** VS2DI is not capable of particle tracking and cannot directly identify the earliest wave of infiltration. To determine the earliest wave, observation wells were placed at certain grids and parameter values such as water content, pressure head and concentration were recorded at every iteration. In this way, the GMMP for the earliest wave was determined without tracing out the pathway of the earliest wave from surface. A certain concentration of tracer was assigned to the recharged water assuming no concentration existed in water before the start of recharge.

**Modeling Results of Five Scenarios** Scenario 1 with a combined condition of continuous recharge, dry initial moisture content and simple kriging was simulated for a period of 40 days with each of the 50 realizations of hydrogeologic conditions. Figure 1 shows the spatial distribution of the GMMPs of the 50 simulations. As shown in Figure 1, the locations of the GMMP are different for all the 50 simulations except at the eight observation wells (# 74, 88, 95, 102, 103, 106, 117 and 120) having the

earliest wave twice. As shown, the 40 observation wells are located underneath Basin 1 and the remaining 10 underneath Basin 4.



**FIGURE 1. Distribution of GMMPs catching Earliest Waves of 50 Simulations (Scenario 1).**

Table 2 shows a summary of the simulation results of the five Scenarios and Table 3 presents the observation wells having the earliest wave more than once in the 250 simulations. From these results, we can discuss the following issues:

1. **Travel time of the earliest wave of infiltration:** The travel times of the earliest waves of infiltration from the surface basin to the GMMP are around 1 day under the wet initial water content condition (in Scenarios 2 and 4) and about 17 and 30 days under the dry condition (in Scenarios 1 and 5, and 3, respectively). In fact, the real initial water content condition under the normal and continuous wet/dry operational cycles is closer to the wet initial water content condition. This indicates that the actual travel time of the earliest wave under the real operation of the surface recharge basin will be around one day.
2. **Location of the GMMPs:** From all 250 simulations, there was no observation well that had the earliest wave of infiltration more than seven times. As shown in Table 3, there was only one well having it seven times, three wells six times and four wells five times. This indicates that the location of a well to catch the earliest wave of infiltration depends largely on the hydrogeologic and operating conditions. It means that we can seldom locate a GMMP or an array of GMMPs to catch most of the earliest waves under normal field conditions of operation and hydrogeology. If we place a GMMW at a GMMP, its highest probability to catch the earliest wave is only 7 times out of the 250 simulations, i.e., only 2.8%.

3. **Insensitivity of data to kriging method:** Between the results of Scenarios 1 and 5, noticeable differences were not significant enough to change the above statements. Therefore, other kriging methods were not tested. For this research, the cokriging method with two variables, soil type and hydraulic conductivity, as Benson and Rashad [1996] was considered. However, the quantity and quality of the data available for this study were not sufficient for more sophisticated kriging method. Therefore the use of simple and ordinary kriging methods was adequate and more sophisticated methods would not provide additional information.

**Table 2. A Summary of Simulation Results of Five Scenarios.**

|                                                                      | <b>Scenario 1</b>                                        | <b>Scenario 2</b>                                        | <b>Scenario 3</b>                                     | <b>Scenario 4</b>                                     | <b>Scenario 5</b>                                        |
|----------------------------------------------------------------------|----------------------------------------------------------|----------------------------------------------------------|-------------------------------------------------------|-------------------------------------------------------|----------------------------------------------------------|
|                                                                      | <b>Continuous Dry Initial <math>\theta</math> Simple</b> | <b>Continuous Wet Initial <math>\theta</math> Simple</b> | <b>Wet Dry Dry Initial <math>\theta</math> Simple</b> | <b>Wet Dry Wet Initial <math>\theta</math> Simple</b> | <b>Continuous Dry Initial <math>\theta</math> Simple</b> |
| <b>Number of observation Wells with the Earliest Wave</b>            |                                                          |                                                          |                                                       |                                                       |                                                          |
| Once                                                                 | 42                                                       | 42                                                       | 41                                                    | 39                                                    | 41                                                       |
| Twice                                                                | 8                                                        | 7                                                        | 8                                                     | 8                                                     | 9                                                        |
| Three Times                                                          | -                                                        | 1                                                        | 1                                                     | 3                                                     | -                                                        |
| <b>Number of observation Wells with the Earliest Wave Underneath</b> |                                                          |                                                          |                                                       |                                                       |                                                          |
| Basin 1                                                              | 40                                                       | 7                                                        | 45                                                    | 1                                                     | 41                                                       |
| Basin 2                                                              | -                                                        | 10                                                       | -                                                     | 1                                                     | 1                                                        |
| Basin 3                                                              | -                                                        | 1                                                        | -                                                     | 48                                                    | -                                                        |
| Basin 4                                                              | 10                                                       | 32                                                       | 5                                                     | -                                                     | 8                                                        |
| <b>Travel Time of the Earliest Wave (in days)</b>                    |                                                          |                                                          |                                                       |                                                       |                                                          |
| Average                                                              | 16.89                                                    | 0.88                                                     | 30.35                                                 | 1.57                                                  | 16.16                                                    |
| Std. Dev.                                                            | 1.78                                                     | 0.13                                                     | 3.40                                                  | 0.49                                                  | 3.10                                                     |

**Table 3. Observation Wells having Earliest Waves More Than Once in 250 Simulations.**

| Frequency of having the Earliest Wave                                                                                                                                    | Total Number | Well Numbers                                                                                                                                     |
|--------------------------------------------------------------------------------------------------------------------------------------------------------------------------|--------------|--------------------------------------------------------------------------------------------------------------------------------------------------|
| Twice                                                                                                                                                                    | 31           | 73, 81, 83, 84, 87, 92, 93, 97, 98, 100, 104, 121, 125, 128, 132, 134, 135, 211, 218, 234, 235, 253, 255, 264, 265, 275, 301, 302, 317, 320, 325 |
| Three Times                                                                                                                                                              | 19           | 68, 72, 77, 82, 89, 94, 107, 111, 119, 122, 124, 204, 240, 242, 244, 277, 286, 322, 332                                                          |
| Four Times                                                                                                                                                               | 6            | 95, 103, 112, 117, 299, 305                                                                                                                      |
| Five Times                                                                                                                                                               | 4            | 96, 106, 126, 273                                                                                                                                |
| Six Times                                                                                                                                                                | 3            | 74, 88, 120                                                                                                                                      |
| Seven Times                                                                                                                                                              | 1            | 102                                                                                                                                              |
| Sum                                                                                                                                                                      | 64           |                                                                                                                                                  |
| Note: Well # 67 ~ 133 are underneath Basin 1; Well # 134 ~ 200 are underneath Basin 2; Well # 201 ~ 267 are underneath Basin 3; Well # 268 ~ 340 are underneath Basin 4. |              |                                                                                                                                                  |

## CONCLUSIONS

From the modeling results and discussions stated above, the applicability of the GMMW concept is summarized as follows:

- The travel time of the earliest wave was approximately one day. This alone demonstrates that a GMMW is not appropriate as a point of compliance. Monitoring recharged water at the surface basin combined with multiple depth downgradient monitoring wells will provide more accurate information on water quality.
- The depth of a vadose zone could make installation of the GMMW logical. This depends on many factors, especially related to the physical, chemical and biological treatment potentials in the vadose zone. Nevertheless, sub-surface horizontal transport to a down -gradient well will provide more representative samples of product water.
- It is very difficult to locate the GMMW or the GMMWs to catch the earliest wave of infiltration with more than a certain level of likelihood. As discussed previously, the best probability of a GMMW catching the earliest wave was 2.8%. The number of GMMWs to catch the earliest wave with a probability of greater than 50% is at least 20 GMMWs and such large number is not practical.
- Sampling requirements for hydrogeologic conditions must also be considered. For the simulations in this research, we have assumed and used 2000 data points.

Collecting sufficient data on soil type, hydraulic conductivity would not be practical to reduce the uncertainty in locating GMMWs.

- The geostatistical simulation, VS2DI modeling and methods for arranging the modeling results, all of which are described in the previous chapters, are recommended as an estimation procedure for future field-application of the GMMW concept, if necessary.
- In conclusion, this research states that based on all the discussions and simulation results above, implementing the GMMW concept as mentioned has serious practical limitations.

## ACKNOWLEDGEMENTS

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## DEVELOPMENT OF A PERCOLATION DECAY MODEL TO GUIDE FUTURE OPTIMIZATION OF SURFACE WATER RECHARGE BASINS

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### ABSTRACT

Reductions in percolation over time due to foulant accumulation are often a significant problem with surface water recharge basins. The ability to predict the impact of foulant accumulation on basin percolation facilitates evaluation of the potential effectiveness of cleaning strategies or water pretreatment schemes in improving percolation efficiency. In this study, both laboratory and field data were used to develop a relatively simple mathematical model capable of describing basin percolation kinetics. This model suggests that initial percolation rate, foulant loading and the interaction between foulant and sediment at or near the sediment/water interface can describe the decay of percolation over time. Simulating performance of a recharge basin indicates that the interaction of foulant with sediment and reduction of overall foulant loading is perhaps the most effective method of improving recharge basin efficiency.

### KEYWORDS

Fouling, model, percolation reduction, recharge, suspended solids

### INTRODUCTION

In an average year, the Orange County Water District (OCWD) diverts 247 million m<sup>3</sup> (200,000 acre-feet, af) of Santa Ana River (SAR) water for recharge into the Orange County groundwater basin. OCWD's recharge facilities include over two-dozen surface recharge basins covering a total of 6.1 km<sup>2</sup> (1,510 acres). SAR base flow, which is comprised primarily of tertiary-treated effluent, contains approximately 25 to 400 mg/L of organic and inorganic total suspended solids (TSS). During percolation, a fraction of these suspended solids in SAR water act as foulants, accumulating in OCWD's recharge basins and causing percolation rates to rapidly decay.

It was hypothesized that the time-dependent loss of percolation rate could be related to the mass of foulants accumulated at or near the sediment/water interface using a relatively simple log-decay kinetic approach. This hypothesis was tested using both laboratory analogs of recharge basins (sediment columns) and actual field data obtained during recharge basin operations. Manipulation of the inputs to this model was used to determine the relative contribution of the principal input variables to percolation decay, and to suggest means to improve recharge basin operations.

## **METHODS**

### **Fouling Materials Used for Study**

#### **Material Recovered from the Bottom of a District Recharge Basin**

Fouling material used for the study was derived from solids recovered from the bottom of Kraemer Basin, a  $\sim 0.12 \text{ km}^2$  ( $\sim 30$ -acre) terminal recharge basin operated by OCWD in Anaheim, CA. This basin can receive either SAR water or imported Colorado River water from the Metropolitan Water District (MWD), which has relatively low TSS concentrations (typically  $< 2 \text{ mg/L}$ ). When Kraemer Basin was drained and dried, foulant that had accumulated at the sediment/water interface during percolation dehydrated and formed “chips” on the basin bottom that were from one to several millimeters thick. These chips were manually harvested from the basin bottom. Foulants recovered in this fashion could be stored in the dehydrated “chip” state indefinitely.

#### **Material Recovered from the Basin Cleaning Vehicle (BCV)**

Additional fouling material for the study was recovered from the waste stream of an *in situ* underwater dredge designed by OCWD to remove foulants from operational recharge basins (the Basin Cleaning Vehicle, BCV; Hutchinson, 2007, submitted).

#### **Water Used for Column Studies**

Source water used for experiments was derived from a shallow extraction well near the SAR. This well-derived SAR (W-SAR) water was chemically identical to river water except that the suspended solids content was extremely low and contributed negligibly to sediment fouling.

#### **Preparation of Fouling Material for Experimental Studies**

Fouling material for experimentation was prepared by first disaggregating the large chunks using a Waring blender and suspending in W-SAR at  $\sim 50 \text{ mg/L}$  (w/v). This suspension was allowed to settle two minutes to remove any clumped material and



large particles, and then decanted into a stock container and the TSS determined. This stock was further diluted in 76L (20 gallons) of W-SAR water in a 208L (55-gallon) barrel to make up test loads of 21, 51, 55, 92 and 377 mg/L TSS. The suspension was allowed to mix in the barrel overnight before use, and the TSS concentration was checked for stability prior to experimentation.

### **Comparison of Particle Size Distribution of Experimental Foulant Material to SAR Suspended Solids**

The particle size distribution of test foulant derived from Kraemer Basin chips was compared to SAR water suspended solids using a Coulter Multisizer II particle size analyzer (Beckman Coulter, Hialeah, FL) equipped with 20mm and 100 mm counting orifices. Overlapping the counting ranges of these orifices allowed determination of the volume of suspended particles in water samples that ranged from 0.5mm to 57.2mm apparent diameters. The particle size distribution in this range of Kraemer Basin chips processed as described above compared favorably with that of particles derived from SAR water (data not shown).

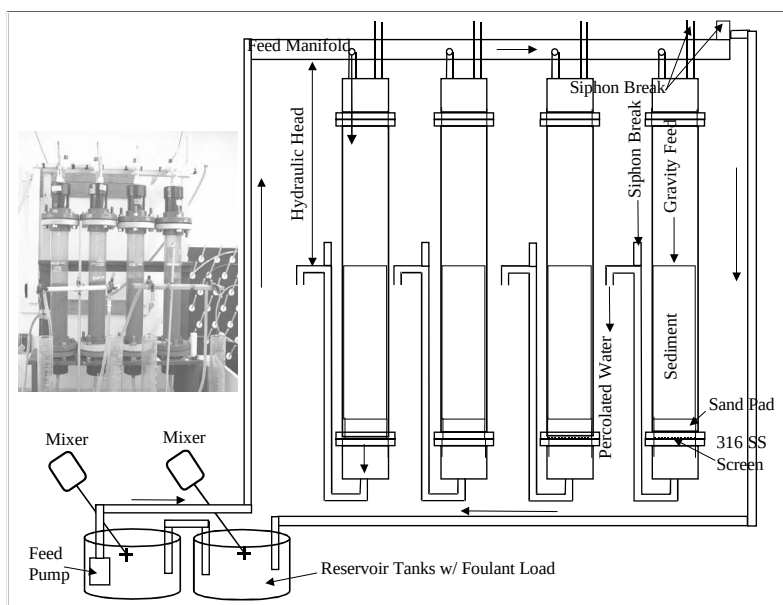
### **Collection and Preparation of Sediment /Used for the Study**

Sediment used for the column experiments was native sandy sediment obtained from the bottom and sidewalls of Anaheim Lake, another terminal deep recharge basin operated by OCWD, when it was dried for cleaning. Sediment obtained in this fashion contained mainly sand particles with sizes ranging from 2.4 mm (8 mesh) to 75mm (200 mesh), with an average size of ~600mm (~30 mesh). This sediment was dried, homogenized and stored in plastic buckets. The sediment was put through a 6.4 mm (#4 mesh) sieve to remove pebbles and other debris before use.

### **Sediment Column Experiments**

#### **Design of the Sediment Column Apparatus**

Figure 1 shows the column apparatus used in laboratory experiments.



**FIGURE 1. Laboratory sediment column apparatus.**

Four columns 82 cm (32 inches) long by 7.5 cm (2.96 inches) internal diameter (I.D.) (surface area =  $4.42 \times 10^{-4} \text{ m}^2$ ;  $0.0478 \text{ ft}^2$ ) were fabricated from 7.62 cm (3-inch) clear PVC pipe stock and fitted at the ends with bolted-on PVC flanges for easy filling access. A 316 stainless steel screen placed in the lower flange and covered with 1 cm (0.39-inch) pad of 1.2mm (16 mesh) sand captured the solid contents of the column during experiments. Each column was connected to a 2.54 cm (1-inch) feed manifold via a 1.3 cm (0.5-inch) diameter 90-degree right angle pipe emerging horizontally from the manifold and penetrating the upper flange of the column. This configuration tended to ensure that suspended solids were transported to the sediment in the columns nearly entirely by advection. A second vertical tube connected to the flange and rising above the manifold acted as a siphon break and allowed equilibration of internal pressure with the manifold, so that the height of the manifold above the column could be used to determine the head pressure and each of the four columns could operate at different flow rates without affecting the pressure of water in the manifold. A flexible tube attached to the lower flange allowed water to drain from the column; this tube was fitted with a siphon break at its distal end. By manipulating the difference in height between the between the feed manifold and the siphon break on the drain tube, the pressure difference across the column could be regulated. This configuration provided a nearly constant head of 0.78 m (1.6 feet) of water.

A recirculation feed pump was submerged in 76L (20 gallons) of W-SAR water in one of two 208L (55-gallon) water reservoir tanks equipped with propeller agitators and connected to the feed manifold with flexible tubing. A siphon break and riser at the distal end of the feed manifold prevented gravity-induced siphoning of water from the manifold, and ensured that fluid levels and pressures were maintained at nearly constant levels in the manifold at the water circulation velocity employed during experiments.

### **Sediment Column Experimental Protocol**

All experiments were performed in duplicate. Approximately  $1,700 \text{ cm}^3$  ( $0.06 \text{ ft}^3$ ) of sediment was poured into each test column, resulting in a sediment depth of 30.5 cm (1 foot). The sediment was slightly dampened to prevent fractionation of the fine matter during pouring. In order to remove voids, the sediment was compacted by a piston fitting the internal bore of the column and impacted twice with a 1 kg (2.2-pound) weight dropped from a distance of 50 cm (19.7 inches).

Packed sediment was hydrated by first introducing W-SAR water in an up-flow mode through the bottom of the column with a peristaltic pump so as to exclude air from the sediment. Once the sediment was completely hydrated, the drain tube was connected and then the siphon break was set level with the sediment/water interface. The column was connected to the manifold and hence operated in a down-flow mode by gravity.

The rate of water flow through the sediment columns was determined volumetrically using a graduated cylinder and a stopwatch. Flow was monitored during the initial percolation of W-SAR water until it reached a steady state, which typically required from several hours to more than 24 hours. Once the columns reached a steady percolation rate, experimental foulant material prepared as described above was introduced by quickly transferring the feed pump from the W-SAR water barrel to the barrel containing the propeller agitator stirred suspended foulant material. Volunteered flow measurements were initially made frequently (3 ñ 10 minute intervals) to better define fouling kinetics; later the time between measurements was extended. Measurements were continued until percolation declined to approximately 10 ñ 20 percent of its initial value; depending on the foulant loading that took from several hours to 24 hours.

### **Recovery of sediment columns by removal of the fouling layer**

Following experimentation, the outflow tube was clamped to halt flow through the columns, the flow manifold disconnected and the upper column flange carefully removed. The layer of foulant accumulated at the sediment/water interface was removed using a glass tube with gentle suction to aspirate the upper 5 cm (2 inches) of sediment and foulant. The sediment removed was replaced with a layer of fresh

material and the column re-hydrated as described above. Sediment columns treated in this fashion were observed to recover their initial percolation rates and therefore could be used for further experimentation.

### **Fitting a Mathematical Model to Sediment Column Data**

The total foulant deposited at the sediment/water interface was estimated as a function of time by the product of the foulant concentration and the total volume of water percolated through the column. A log-decay expression was fitted to these data using method of Marquardt nonlinear regression (Statgraphics, Centurion XV, Statpoint Incorporated, Herndon, VA).

### **Kraemer Basin Field Percolation Data**

Kraemer Basin field data pertaining to the percolation of SAR water was obtained for the periods December 1997 to May 1998, November 2004 to February 2005 and from May 2005 to August 2005. Data for the percolation of MWD imported water was obtained for the period August 2000 to November 2000. Specific average percolation was calculated from daily logs of total inflow, the change observed in basin volume and total basin wetted area.

## **RESULTS AND DISCUSSION**

### **Modeling Sediment Column Percolation Decay**

The results of non-linear regression analysis showed that the relationship between accumulated foulant and percolation could be described very well by the simple log-decay expression:

$$Q=Q_0e^{-rL} \quad (\text{EQ 1})$$

Where  $Q_0$ = the initial percolation rate,  $L$  = the total foulant/unit area deposited at the sediment/water interface and  $Q$  = percolation observed at  $L$  foulant loading. The value of  $r$  represents a sediment/foulant interaction coefficient presumed to be unique to the nature of the sediment and foulant.

Percolation decay was modeled by integration of this expression over time using an incremental approach with a spreadsheet calculator (Excel, Microsoft Corp., Redmond, WA). During this integration, foulant deposited during a particular time increment ( $dL/dt$ ) was determined using the previous time increment percolation rate and the suspended solids concentration derived during the increment (keeping  $dt$  small compared to the overall time elapsed prevented serious overestimation of

dL). The accumulated solids for the increment  $dt$  was totalized with all previous increments to determine the total solids load, and a new percolation rate determined using the relationship defining percolation and total solids loading. This operation was iterated until the end of the desired time was reached.

Sediment/foulant interaction coefficients resulting from fitting this expression to column data are shown in Table 1. In a majority of cases, the adjusted  $R^2$  indicated this simple approach was capable of describing greater than 90 percent of the observed data variability.

**TABLE 1. Determination of sediment/foulant interaction coefficients ( $r$ ) for laboratory sediment column experiments.**

| Foulant Type | Avg. Loading (mg/L) | Avg. Adj. $R^2$ | Sediment/Foulant Interaction Coefficient ( $r$ )   |                                                    |                                         | 95% CL Diff.? |
|--------------|---------------------|-----------------|----------------------------------------------------|----------------------------------------------------|-----------------------------------------|---------------|
|              |                     |                 | Average (n=2)<br>$M^2/mg$<br>( $ft^2/mg$ )         | S.D.                                               | p-Value (Comparison to To 21 mg/L Load) |               |
| BCV Waste    | 400                 | 88.17%          | $0.67 \times 10^{-6}$<br>( $0.72 \times 10^{-5}$ ) | $0.14 \times 10^{-6}$<br>( $0.15 \times 10^{-5}$ ) | 0.0211                                  | Yes           |
| KB Chips     | 377                 | 99.80%          | $21.8 \times 10^{-6}$<br>( $23.5 \times 10^{-5}$ ) | $1.05 \times 10^{-6}$<br>( $1.13 \times 10^{-5}$ ) | 0.0177                                  | Yes           |
| KB Chips     | 92                  | 97.45%          | $3.34 \times 10^{-6}$<br>( $3.60 \times 10^{-5}$ ) | $0.53 \times 10^{-6}$<br>( $0.57 \times 10^{-5}$ ) | 0.4136                                  | No            |
| KB Chips     | 51                  | 93.68%          | $2.60 \times 10^{-6}$<br>( $2.80 \times 10^{-5}$ ) | $0.13 \times 10^{-6}$<br>( $0.14 \times 10^{-5}$ ) | 0.3501                                  | No            |
| KB Chips     | 55                  | 97.94%          | $1.81 \times 10^{-6}$<br>( $1.95 \times 10^{-5}$ ) | $0.33 \times 10^{-6}$<br>( $0.35 \times 10^{-5}$ ) | 0.0747                                  | No            |
| KB Chips     | 21                  | 92.15%          | $2.88 \times 10^{-6}$<br>( $3.10 \times 10^{-5}$ ) | $0.26 \times 10^{-6}$<br>( $0.28 \times 10^{-5}$ ) | na                                      | na            |

Foulant accumulation was an excellent predictor of percolation decay. Although this sort of empirical modeling approach precludes by itself establishment of a mechanism, it seems probable from the differences in mean particle size of the foulant materials and of the sediment that a mechanism by which foulant enters and fills voids between sediment particles is indicated. Further research examining the ability of foulant particles of differing size and composition to reduce percolation in sediments of defined particle sizes may shed more light on this hypothesis.

## **Relationship Between Foulant Composition, Loading Rate and the Sediment/Foulant Interaction Coefficient**

### **Effect of Foulant Composition on the Sediment/foulant Interaction Coefficient**

The sensitivity of the sediment/foulant interaction coefficient to the nature of the fouling material was tested by comparing coefficients derived from sediment columns receiving similar loads of Kraemer Basin fouling material (from chips) and waste material dredged from Miller Basin (a recharge basin adjacent to Kraemer Basin) by OCWD's Basin Cleaning Vehicle (BCV). The sediment/foulant interaction coefficient of BCV waste material was 32 times smaller than that of Kraemer Basin fouling material, indicating it was substantially less effective in reducing percolation (Table 1). Particle analysis indicated a higher proportion of coarser particulates in the BCV waste material (data not shown), which could have reduced its ability to pack into sediment interstices.

### **Effect of Foulant Loading on the Sediment/foulant Interaction Coefficient**

The sediment/foulant interaction coefficient likely represents a composite of many specific interactions between foulant matter and sediment; therefore, it is improbable that it would be a universal constant. However, it was anticipated that for a given sediment and foulant, there would be a range of foulant concentrations over which the coefficient would be nearly a constant. In this range, variations in the foulant concentration would not affect the relationship between percolation and the total mass of foulant accumulated at the sediment/water interface (L).

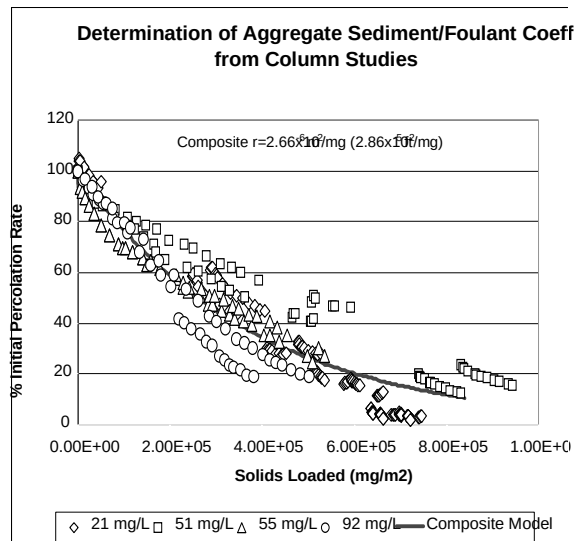
In order to investigate this, percolation fouling kinetics were determined using W-SAR water containing TSS concentrations of 21, 51, 55, 92 and 377 mg/L of Kraemer basin chip material. For TSS <92 mg/L, the values obtained for the sediment/foulant interaction coefficient were statistically similar by Student's t test ( $p > 0.05$ ). For TSS concentrations less than 92 mg/L, percolation was nearly a constant function of foulant accumulation at the sediment/water interface. This consequence is useful as it allows for compression of the time frame for sediment fouling experiments by employing concentrations of foulant several times greater than that found in actual recharge waters. Thus, fouling that normally takes months can be simulated over a few days, or perhaps hours.

However, for very high TSS concentrations (377 mg/L), the sediment/foulant interaction coefficient was statistically different ( $p = 0.0177$ ) and nearly an order of magnitude greater (Table 1). This may suggest a physically different fouling mechanism may operate at the higher foulant loading rates. In this instance, more rapid introduction of foulants may more rapidly reduce water flow, and decreased hydraulic shear may promote more extensive bridging of the sediment. Alternatively, more rapid generation of the fouling layer may result in increased compression and

more rapid water flux reduction through the fouling layer than occurs when deposition is slower.

### Generating an Aggregate Sediment/Foulant Interaction Coefficient

That foulant loading up to 92 mg/L resulted in statistically similar values of the sediment/foulant interaction coefficient justified combining data from all four experiments in that foulant loading range in order to generate a composite coefficient. Figure 2 shows the results of combining all eight experiments (4 [foulants] x 2 replicates) and determining a best-fit log-decay curve using nonlinear regression. This approach, incorporating all the experimental noise in the calculations, gave a value of  $2.66 \times 10^{-6} \text{ m}^2/\text{mg}$  ( $2.86 \times 10^{-5} \text{ ft}^2/\text{mg}$ ) for the interaction coefficient, and explained 90 percent of the variability in the data.



**FIGURE 2. Determination of an aggregate sediment/foulant interaction coefficient (r) from laboratory sediment column data.**

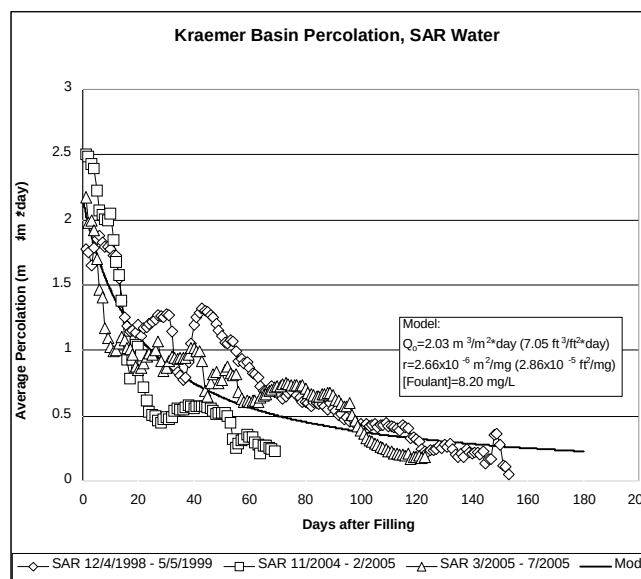
### Testing Ability of the Laboratory Model to Describe Decay Kinetics of a Percolation Basin

The ability of the laboratory model to describe recharge basin behavior was investigated by attempting to describe Kraemer Basin percolation decay using the aggregate sediment/foulant interaction coefficient of  $2.66 \times 10^{-6} \text{ m}^2/\text{mg}$  ( $2.86 \times 10^{-5} \text{ ft}^2/\text{mg}$ ) derived from the column studies. Although the foulant loading of the sediment columns was known, comparable data were not available for the Kraemer Basin field

measurements. In order to construct models for Kraemer Basin percolation, the foulant load value was varied in order to obtain a best fit between predicted basin performance and the observed field data.

### Percolation of SAR Water

A general model for the percolation decay kinetics of SAR water by Kraemer Basin was obtained by first modeling the kinetics of each individual percolation event observed, and then averaging the  $Q_0$  and foulant load to produce an aggregated value. Results of this analysis are shown in Figure 3. The model was able to describe the overall kinetics of percolation decay quite well, suggesting general similarity between the behavior of the laboratory sediment columns and the basin. The average foulant load predicted by this model was 8.20 mg/L. This value was somewhat lower than the 25 mg/L TSS that has been observed for SAR water; however, it is possible that foulant material recovered from Kraemer Basin represents only a fraction of the total TSS load observed in SAR water.

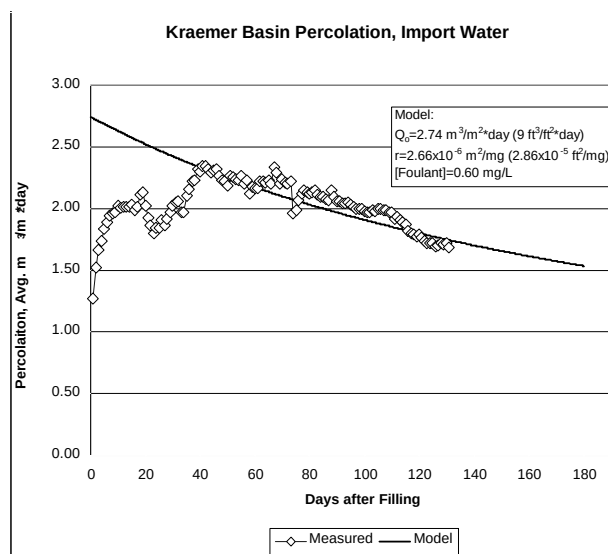


**FIGURE 3. Modeling Kraemer Basin percolation kinetics with SAR water.**

### Percolation of Import Water

A similar approach to fitting percolation kinetics of import water in Kraemer Basin is shown in Figure 4.





**FIGURE 4. Modeling Kraemer Basin percolation kinetics with imported MWD water.**

Again the sediment/foulant interaction coefficient defined by laboratory experiments was used to construct a model involving integration with time of total accumulated solids, relating this to percolation via the log-decay relationship. Although there was no general guarantee that the foulants in SAR water and import water were similar, this approach allowed a point of departure for predictions. In this case, field data did not show evidence of percolation decay until after approximately the first 45 days of basin operation. It is unclear why percolation was slow to start (perhaps an artifact of how the basin filled; this would be expected if the bottom percolated less efficiently than the side walls), but for modeling purposes only data showing percolation decay was used. Curve fitting was performed manually; values for  $Q_0$  and foulant concentration were varied until a best fit was obtained for the data in the region of 45-130 days of basin operation.

A best-fit model was obtained using a  $Q_0$  of 2.74  $\text{m}^3/\text{m}^2\cdot\text{day}$  ( $9 \text{ ft}^3/\text{ft}^2\cdot\text{day}$ ) at a foulant load of 0.60 mg/L. This value of  $Q_0$  is within a reasonable range for OCWD recharge basins, and import water TSS concentrations are typically less than 2 mg/L. Once again, the TSS concentrations in the field might somewhat overestimate the actual foulant load. It is interesting to note, though, that the models suggest a 14-fold difference in predicted foulant load between SAR water and import water, and that is close to the difference in TSS concentrations typically observed comparing SAR base flow water to import water.

### Effects of Varying Model Inputs on Water Production by a Simulated Recharge Basin

A model was constructed simulating percolation of a 0.12 km<sup>2</sup> (30 acre) recharge basin based on field parameters observed for Kraemer Basin recharging SAR water:  $Q_0=2.03 \text{ m}^3/\text{m}^2\cdot\text{day}$  ( $7 \text{ ft}^3/\text{ft}^2\cdot\text{day}$ ); [TSS] effective as foulant = 8 mg/L, and the laboratory sediment/foulant interaction coefficient  $r=2.66\times 10^{-6} \text{ m}^2/\text{mg}$  ( $2.86\times 10^{-5} \text{ ft}^2/\text{mg}$ ). These input parameters were then varied and the effect on percolation analyzed in terms of integrated water production over 180 days (Table 2). The simulated basin produced 12.7 million m<sup>3</sup> (10,269 af) in 180 days, which is reasonable for operation of an OCWD recharge facility such as Kraemer Basin.

**TABLE 2. Water production by a simulated 0.12 km<sup>2</sup> (30 acre) recharge basin @ 180 days assuming  $Q_0=2.03 \text{ m}^3/\text{m}^2\cdot\text{day}$  ( $7 \text{ ft}^3/\text{ft}^2\cdot\text{day}$ ),  $r=2.66\times 10^{-6} \text{ m}^2/\text{mg}$  ( $2.86\times 10^{-5} \text{ ft}^2/\text{mg}$ ) and [TSS] =8 mg/L. Nominal input values are italicized.**

| 1) Constant $r$ , [TSS]                                                                           |                                | 2) Constant $Q_0$ , [TSS]                                   |                                | 3) Constant $Q_0$ , $r$ |                                |
|---------------------------------------------------------------------------------------------------|--------------------------------|-------------------------------------------------------------|--------------------------------|-------------------------|--------------------------------|
| $Q_0$ ,<br>$\text{m}^3/\text{m}^2\cdot\text{day}$<br>( $\text{ft}^3/\text{ft}^2\cdot\text{day}$ ) | Million m <sup>3</sup><br>(AF) | $R$ , $\text{m}^2/\text{mg}$<br>( $\text{ft}^2/\text{mg}$ ) | Million m <sup>3</sup><br>(AF) | [TSS], mg/L             | Million m <sup>3</sup><br>(AF) |
| 1.52<br>(5.0)                                                                                     | 11.0<br>(8,902)                | $2.66\times 10^{-6}$<br>( $2.86\times 10^{-5}$ )            | 12.7<br>(10,264)               | 15                      | 8.5<br>(6,898)                 |
| 2.13<br>(7.0)                                                                                     | 12.7<br>(10,264)               | $1.39\times 10^{-6}$<br>( $1.5\times 10^{-5}$ )             | 18.1<br>(14,701)               | 8                       | 12.7<br>(10,264)               |
| 3.05<br>(10)                                                                                      | 14.5<br>(11,762)               | $0.65\times 10^{-6}$<br>( $0.70\times 10^{-5}$ )            | 25.6<br>(20,780)               | 4                       | 18.6<br>(15,063)               |
| 4.57<br>(15)                                                                                      | 16.7<br>(13,518)               | $0.46\times 10^{-6}$<br>( $0.50\times 10^{-5}$ )            | 29.0<br>(23,494)               | 1                       | 32.1<br>(26,061)               |
| 6.10<br>(20)                                                                                      | 18.2<br>(14,788)               | $0.19\times 10^{-6}$<br>( $0.20\times 10^{-5}$ )            | 36.9<br>(29,911)               | 0                       | 46.6<br>(37,800)               |

#### Effect of Varying Initial Percolation Rate ( $Q_0$ )

The effect of varying the initial percolation rate ( $Q_0$ ) and holding  $r$  and TSS constant on total water production over 180 days by the simulated basin is shown Table 2 (1). Total water production was directly proportional to initial percolation rate, indicating that basin performance is limited by this parameter. However, increasing  $Q_0$  only produces modest gains in water production since increasing the initial percolation rate simultaneously increases the rate of foulant transport. In this

example, a 50 percent increase in total water production requires a nearly 3-fold increase initial percolation rate. Increasing percolation rate of a recharge basin this much is unlikely, as percolation is ultimately limited by the hydraulic conductivity of the larger volume of sediment or aquifer underlying the recharge basin.

### **Effect of Varying the Sediment/Foulant Interaction Coefficient (r)**

The effect of varying the sediment/foulant interaction coefficient (r) and holding  $Q_o$  and TSS constant on total water production over 180 days by the simulated basin is shown in Table 2(2). Decreasing r increases water production. Unlike with  $Q_o$ , factors responsible for the sediment/foulant interaction coefficient may be manipulated. One of these is the mean sediment grain size at the sediment/water interface. Laboratory investigations by OCWD (data not shown) indicate that increasing mean sediment grain size to 1.2mm (16 mesh) decreases the sediment/foulant interaction coefficient to as low as  $0.46 \times 10^{-6} \text{ m}^2/\text{mg}$  ( $0.5 \times 10^{-5} \text{ ft}^2/\text{mg}$ ), which in this case would increase water production from 12.7 million  $\text{m}^3$  (10,264 af) to 28.9 million  $\text{m}^3$  (23,494 af), a 2.3-fold improvement in percolation efficiency. In the field, altering the mean sediment grain size at the sediment/water interface could be achieved by application of a relative thin layer of groomed material on top of basin sediments. Cleaning this material could be performed *in situ* using a submerged sand-washing dredge such as OCWD's BCV, which is capable of separating foulant material from sand while the basin is in operation.

### **Effect of Varying the Foulant Concentration**

The effect of varying foulant concentration (TSS concentration) and holding  $Q_o$  and r constant on total water production for the simulated basin operated for 180 days is shown in Table 2(3). For this model,  $Q_o$  is set at  $2.03 \text{ m}^3/\text{m}^2 \cdot \text{day}$  ( $7.05 \text{ ft}^3/\text{ft}^2 \cdot \text{day}$ ) and the sediment/foulant interaction coefficient is  $2.66 \times 10^{-6} \text{ m}^2/\text{mg}$  ( $2.86 \times 10^{-5} \text{ ft}^2/\text{mg}$ ). Decreasing the foulant concentration decreases the foulant accumulation on the sediment (L) with time, and this has a profound effect on water production. Reducing foulant concentration from 8 mg/L down to 1 mg/L (the range of foulant concentration in import water) increases basin total water percolation over a 180-day period by nearly 20 million  $\text{m}^3$  (>16,000 af). Even a modest reduction in foulant concentration has significant impact on water production. Reducing foulant concentration (TSS reduction) may be the most direct method by which recharge may be improved. However, it must be considered that the foulant particles are predominantly small (<10mm, and many less than 1-2mm), and if kept in suspension by slight water currents or Brownian motion, may be difficult to remove by gravity settling alone. Use of chemical or mechanical flocculation methods may improve removal. Media filtration may be useful for small recharge operations, but processing on the order of 10,000  $\text{m}^3/\text{hour}$  (100 CFS) would require a sizable filtration plant.

Furthermore, not all of the foulant material reaching a basin bottom may originate in the influent water; primary productivity (algal growth) may account for a fraction of this material, especially in summer months. The degree to which biomass accumulation in the recharge basin water influences total particle deposition during percolation is currently not well understood.

## CONCLUSIONS

Foulant accumulation at the sediment/water interface appears to be the predominant mechanism responsible for percolation decay in recharge basins operated by OCWD. The kinetics of percolation decay may be adequately modeled by integration with time of a log decay function requiring three input parameters: initial percolation rate, foulant concentration and a sediment/foulant interaction coefficient that may be obtained by laboratory determination or from historic field performance data. This percolation model may be easily implemented using a spreadsheet, and optimization routines readily performed. Finally, data generated from the model may be easily integrated into cost-benefit models, making it possible to predict the most cost-effective cleaning strategies, best pre-treatment strategies, etc., to maximize basin water production and minimize basin operating costs.

Widespread successful application of the model will be dependent on several factors. Although the mass loading of suspended solids into OCWD's recharge facilities is reasonably well documented, what fraction of this material contributes to basin fouling and the nature and stability of fouling material is virtually unknown. The degree to which local particle production in recharge basins (primary biological productivity) alters foulant composition seasonally also remains to be determined. The relationship between foulant mean particle size and fouling capability is only poorly understood. Calculation of the sediment/foulant coefficient with foulants of defined particle structure is needed to elucidate this relationship. More research is also needed to quantify the relationship between sediment particle size and the sediment/foulant interaction coefficient, which must be understood if grooming the sediment/water interface of a basin to maximize water production is to be attempted. Moreover, the relationship between sediment particle size and depth of foulant penetration must be quantified in order to determine the extent to which bottom-cleaning strategies need to be altered to operate groomed basins sustainably, and to balance increased bottom-cleaning costs against improvements in water production.

Once these data are obtained and the model is better honed, it should be a highly useful tool for designing and optimizing surface water recharge facilities.

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## BASIN GROUNDWATER RECHARGE AND DISCHARGE AT THE GULLY EROSION – PRONE AREAS OF ANAMBRA STATE, NIGERIA

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### ABSTRACT

Anambra state of Nigeria is ravaged by ecological disasters caused by frequent flooding, extensive soil and gully erosion and monumental landslides with consequent destructive implications on both surface and subsurface water resources, supplies and management. The erosion prone areas are concentrated on both sides of the cuetas of: Awka-Nanka-Umuchu-Orlu area, and Amawbia – Enugu-Ukwu – Obosi area. These cuetas with highest topographic elevations in the crest areas have steep scarp slope towards the east and gentle dip slope towards the west. Heavy rainfalls, averaging about 2000mm annually occur. Huge incidences of floods and consequent heavy runoff, soil and gully erosion and landslides are usually recorded. The geomorphic characteristic of a typical cuesta/escarpment for which the area is noted is a major controlling factor in flood flow, groundwater recharge and discharge within the drainage basins. The major drainage basins are the Odo/Mamu River, the Idemili River, Nkisi and Anambra Rivers. These rivers, their tributaries and distributaries wreak tremendous hydrologic havoc within the basins. The geologic Formations comprise Nanka Sands and Imo Shale. Huge watertable and confined aquifers with high permeability, hydraulic conductivity, transmissivity and storativity are found in the Nanka Sands. Heavy aquitards and thin lenses of confined aquitards may be found in the Imo shale. The undulating geomorphic features typified by undulating cuetas ensure high surface and subsurface hydraulic gradients, fast flow rates and groundwater recharge/discharge. In as much as flooding, soil and gully erosion occur; high soil water infiltration and groundwater percolation into the aquifers occur. The infiltration and percolation causes high rise in aquifer water levels, increase the hydraulic gradients and porewater pressures. These hydraulic phenomena cause high seepage force and phases, wet conditions, high porewater pressures, effluent seepages and groundwater discharge emerging springs/streams flows. Other hydrogeotechnical implications involve mild and severe ground movement, piping, quick conditions, sudden slope instability and failure, gully development and landslides. Cost for control measures are enormous and beyond the available resources of the government and the people. International funding to augment local resource are pleaded for. A well designed Catchments Management Water Strategy (CMWS) that is holistic in approach in containing both ecological hazards and harnessing of available surface water resources and groundwater recharge and storage are recommended. A people and community participation activity during planning, design, execution and management of water projects is also strongly advised.

### INTRODUCTION

Changes in the physical landscape of the erosion prone areas of Anambra State in the form of erosion occurrence and development started with the invitation of narrow channels which rapidly widened by erosion into major gullies that have now approached canyon proportions in many parts of the State. The gully system within

the area bounded by Agulu, Nanka and Oko cover an area of about 1100km<sup>2</sup> (Nwajide & Hoque, 1979). This has, now, expanded to more than one and half (1<sup>1</sup>/<sub>2</sub>) of the initial area coverage. The major towns along the cuetas that are heavily eroded and gullied include Agulu, Nanka, Ekwulobia, Uga, Umuchu, Alor, Oraukwu, Abagana, Umuoji, Obosi etc. The elevated areas are capped by cohesionless medium to coarse sand/lateritic covers, which are fairly porous and highly permeable. They are also described as part of the “acid sands of eastern Nigeria”. The lowlands are capped by fine to silt sands and clay materials that may be highly porous but aquitard impermeable. In some locality the sand/laterite and clays may be over 20m thick, whereas in some areas, the watertable outcrops on the surface.

The gully menace has attracted considerable concern and extensive control programmes were established by the British Colonial Office (Grove, 1951).

Development of the gullies has caused extensive damage to the environment. Earlier studies have attributed their genesis and growth to geographic factors and qualitative/semi-qualitative methods employed to produce suggestions for solving the problem. However, research by Egboka & Nwankwo (1982) showed that the primary causes of gully and its development lie in the hydrogeological and geotechnical properties of the complex aquifer system underlying the affected areas. The high hydrostatic pressure in the aquifer produces a reduction in the effective strength of the unconsolidated coarse sand in the walls of the gullies leading to intense erosion. The erosion is followed by mass movement and sediment removal by flood flow. This is most pronounced during the rainy season. Water from the effluent seepages that feed streams at the bottom of the gully is slightly acidic. The concrete dams built at several locations to control erosion by the Colonial Office in 1945 could not withstand the weathering effects of the slightly acidic groundwater and were weakened and destroyed. The undulating geomorphic features of the area ensure high surface and subsurface gradient, fast flow rates and groundwater recharge and discharge.

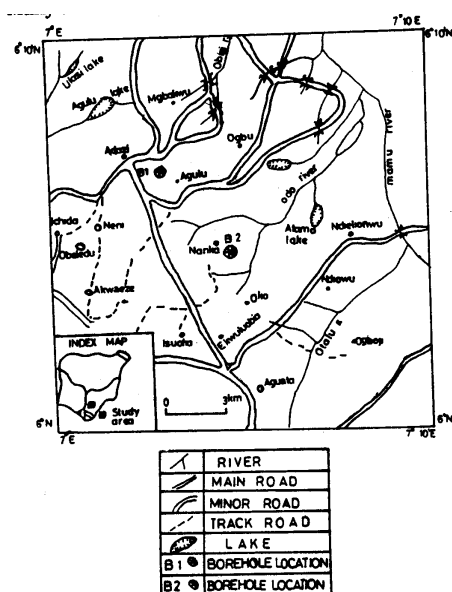
## METHOD OF STUDY

Field and desk studies were used. A review of earlier work was carried out. The roles of agricultural and human activities in gully formation were described by Grove (1951) and by Simpson (1954), Floyd (1965), Ofomata (1965), Ogbukagu (1976), Nwajide and Hoque (1979) and Egboka and Nwankwo (1982). Water budget analysis of part of the gully prone areas was studied by Egboka et al. (2006). Onwuemesi et al. (1991) studied the implication of hydrogeophysical investigation of Agulu-Nanka gully areas. The field study covered both the rainy and dry seasons. As a result of the instability of the gully slopes and margins, investigations were limited to the gully floors and sidewalls where groundwater flow patterns, effluent seepage and channel flow were observed and described.

## CLIMATE AND PHYSIOGRAPHY

The gully erosion prone areas of Anambra State form part of the humid tropical rainforest belt of Nigeria. The climatic and physiographic features are major factors in gully origin and development. The study area which used to form part of the thick rainforest belt of equatorial west Africa has been deforested to over 80%, thereby, exposing the hydrologic and pedologic environment to the inclement weather.

It is bounded in the south by rainforest belt, now, heavily deforested with remnant vegetation found as small patches along river valleys, and in the north by savannah grassland that is rapidly moving southwards. The gully areas are gradually evolving into a savannah type – vegetation as a result of deforestation.



**FIGURE 1. Map showing location and accessibility of study area.**

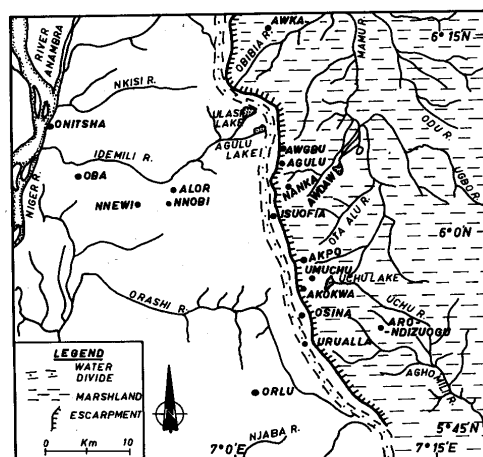
Two main climatic periods prevail in Nigeria, the dry season which starts in October and ends in March and the rainy season (April to September). The dry season is characterized by extreme aridity, dusty atmosphere lowering of surface water and shallow groundwater levels, and large-scale leaf fall. The annual rainfall in the area is about 2000mm. The rainfall occurs as violent downpours accompanied by thunderstorm, heavy flooding, soil leaching and extensive sheet outwash. It facilitates groundwater recharge and fast flow velocity (Egboka and Okpoko, 1984). The north-south trending Awka – Umuchu- Orlu cuesta or escarpment facilitates surface and subsurface flows and enhanced gully erosion (Fig 1).

## GEOLOGY OF THE STUDY AREA

The dominant geologic Formation in the gully prone areas of Agulu, Nanka, Alor, Oraukwu, Oko, Ekwulobia etc. is Nanka Formation (Eocene). It is underlain by the thick Imo Shale (Palaeocene). The gullied part of the Nanka Formation shows a sequence of unconsolidated, unstable, loose, friable and poorly-cemented sands with shale layers, 305m thick (Reyment, 1965; Ogbukagu 1976: and Nwajide 1977). Lithologically, the Nanka Sands consists of distinct units of sand, shale-siltstone and

finely laminated shale. A poorly-sorted, cross-bedded medium to coarse – sand approximately 30m thick, forms the lowermost unit. This is overlain by a thin bed of partially indurated shale- siltstone – fine- sand unit. The deposit exhibits a systematic pattern of alternating cross-bedded sands and thick dark grey shales. The aquiferous Formation has high porewater pressures exhibiting quick conditions. The sandy horizons are consistently thicker than those of shale- siltstone. The Nanka Formation forms the north – south trending ridge or cuesta (Fig 2). The hilly cuesta/ escarpment is gradually and steadily being lowered by soil and gully erosion with Imo Shale forming a quasi-stable platform. The beds and laminae of the sands/shale units exhibit a wide variety of colors, often arranged in bands. The strata have a low angle of dip up to  $6^{\circ}$ .

The Imo Shale is predominantly shale and occupies the stable lowlands east of the Nanka Formation. The Formation consists of dark-grey to blush grey shales, siltstone, mudstone, ironstone and sandstone lenses. In some places, the Ebenebe Sandstone outcrops as a distinct member of the Imo Shale. The shale members are highly- fractured and fissile. Whereas the Nanka Sands members are eroded, the Imo Shale forms the stable, limiting bottom boundary or base level at which gully erosion stops. It also serve as the depositional base for the eroded sediments creating extensive alluvial material on flood plains. The many tributaries of the Odo River and consequently the Mamu River flow and meander sometimes, on the clayey surface of the Imo Shale.

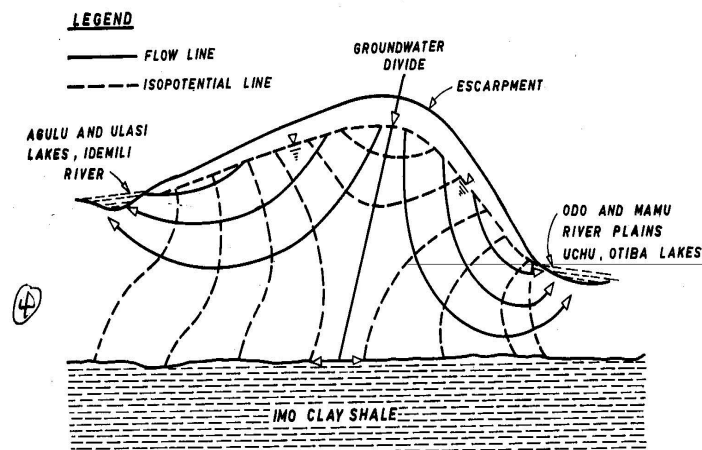


**FIGURE 2. The Agulu — Nanka — Oraukwu gully complex.**



## HYDROGEOLOGICAL AND HYDROGEOTECHNICAL CHARACTERISTICS OF THE STUDY AREA

The gully erosion prone areas are characterized by a series of aquifers, separated by aquitards. These aquifer-aquitard units form a multiaquifer system. The area is part of the Idemili River and Odo River drainage basin. The rivers either originate from the lakes or from the groundwater reservoirs beneath the gully complex through effluent seepages. A regional surface water divide running north-south occurs, with streams and rivers flowing away from the divide on both flanks (Fig 3). This indicates that a regional groundwater divide occurs at some depth below the surface water divide. While the tributaries of Idemili River drainage basin flows west of the divide, those of the Odo River drainage basin flow eastwards, implying the existence of a major surface and groundwater divide.

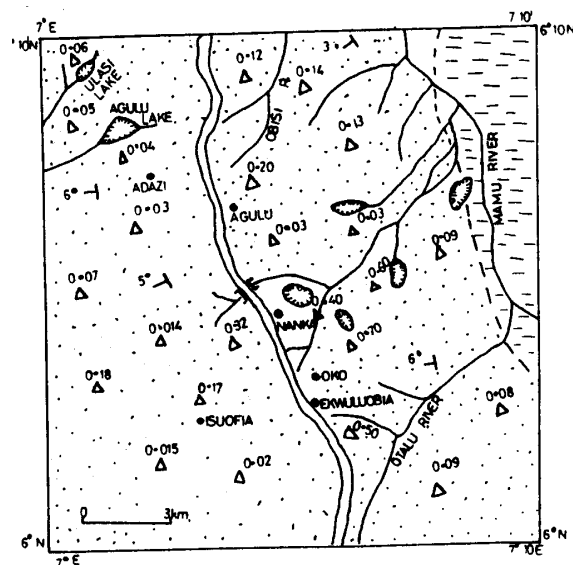


**FIGURE 3. Idealized flow net in a two-dimensional cross section through the Nanka escarpment (based on Egboka & Nwankwor, 1982).**

The area receives torrential downpours of rain during the rainy season when the watertable rises and high groundwater flow rates occur. The watertable becomes shallower towards the water courses and at the edges of numerous water bodies. During the dry seasons, watertable falls as a result of hydraulic head decay. A considerable rise in the watertable occurs during the rainy seasons, despite the thick unsaturated zone. This is due to the high vertical hydraulic conductivity of the freely

draining medium-to coarse-grained soil overlying the multiaquifer system. The calculated values of hydraulic properties are  $8.17 \times 10^{-3} \text{cms}^{-1}$ ,  $82 \text{m year}^{-1}$ ,  $2.96 \times 10^6 \text{m}^3 \text{year}^{-1}$ , and  $5.92 \times 10^5 \text{m}^2 \text{year}^{-1}$  for hydraulic conductivity, groundwater velocity, groundwater discharge and transmissivity (Egboka & Nwankwo, 1982) respectively.

The gully complexes cut into a series of aquifers separated by aquitards that combine to form a multiaquifer system of about 230m thick. This was confirmed by Onwuemesi et al. (1991) through geophysical study of the area. The sandy/shale units are of high geotechnical interest because it is the layer through which several springs and seepage zones occur during the rainy season. These springs and seepage faces are widespread on the gully banks at Agulu, Oko, Nanka and Ekwulobia. A close study of (Fig. 4) reveals that high values of hydraulic gradient correspond to areas that are devastated by gully erosion. During the rainy season, a fast downward flow of water occurs when the groundwater is being recharged. The rise in watertable increases the hydraulic gradient which in turn increases the groundwater velocities hence drastic erosion and gulying that take place in the loose and friable Nanka Sand through high groundwater fluxes. The high fluxes, thereby, excavate the sands, exposing pockets of aquifer to form the numerous springs, lakes and streams that have originated all over the area. The expansion of the gully complex results from the high porewater pressures in the aquifer complexes, mostly during the rainy season at the peak of groundwater recharge. These high porewater pressures reduce the effective stress of the unconsolidated coarse sands along the seepage faces of the gully walls, hence, the geotechnical failures that regularly occur.



**FIGURE 4. Map of the hydraulic gradient values.**

## SOIL ZONE HYDROLOGY

The type and characteristics of soils in parts of southeastern Nigeria, particularly, in Anambra State seem to have been so emplaced to encourage gully erosion and landslides. They are described as the “acid soils” of eastern Nigeria. In parts, they are lateritic and as thick as over 12m; in other areas they are clean sands that may be silty grained/ medium found at various levels and depths. These sandy laterites occur in the elevated areas of the Awka – Umuchu – Orlu cuesta, whereas, in the lowlands or along river valleys, silty and clayey soils are common place. They are described as “acid soils” because in some places the  $P^H$  is as low as 4 to 5. The soils on the elevated areas are very porous and permeable with porosity ranging from 0.3 to 0.4. The vertical permeability is so high that infiltration and percolation into watertable aquifer after a rainfall event are fast. This results in a fast groundwater recharge, a quick rise in groundwater level, increase in hydraulic gradient that results in effluent groundwater flow development along the slopes of the gullies in form of springs, piping, slope failure, and landslides. The soils are also carried by flood and deposited on the flood plains, stream channels and wetlands, thereby, destroying agricultural activities. During the rainy season, groundwater recharge through the soil builds upwards from the groundwater aquifer and transcends height such that in some places, the ground-water in water table aquifers outcrops in form of springs. During such periods, the lands are weak and easily amenable to soil and gully erosion and landslides because the ingress of fast recharge water increases soil porewater pressure. This over-comes the effective stress, thereby, overriding the total stress of the soil as shown in the equation below:

$$\sigma_T = \sigma_e + P_w \quad (\text{EQ 1})$$

Where  $\sigma_T$  is the total stress

$\sigma_e$  is the effective stress

$P_w$  is the porewater pressure

When this happens the porewater pressure becomes prevalent and dominant thereby overcoming the effective stress. This porewater pressure cannot carry any serious stress imposed on the area and, hence, failures in form of gullies or landslides.

This hydrodynamic situation is prevalent in the gully erosion prone area so that slopes collapse easily and the wetlands cannot carry agricultural activities (Fig 6). During dry season, the inverse relationship occurs. Lack of recharge with attendant drastic fall in water levels widens the pore spaces making the soil skeleton very porous. This also contributes to the occurrence of landslide even during the dry

season. The acid nature of the soil equally assists in the breaking down of the soil sediments, thereby, facilitating gully erosion development and growth.

## **ENVIRONMENTAL GEOLOGY**

Every development in terms of infrastructure must consider the effects of the Geology of the environment. The sign of heavy fracturing in sandstones displayed at the bottom of the gully in between which springs come out has great implications. The implications of the Geology also include the formation of a cuetas with a steep slope and scarp slope. The tributaries of Odo rivers take off along the west of the cueta and a join the northward flowing Odo river that empties into Mamu River and later joins Anambra river while the tributaries of Idemili River all emerge and join the Idemili River which flows east and suddenly turns to join River Niger. The orientation of the tributaries and the rivers shows that they are fracture/joint controlled. On the east and west of the cueta, lakes such as Atana in lake and Agulu Lake occur respectively, exhibiting fracture-oriented characteristics from the aquifer system. The hydrogeological implications is that the gully prone areas of Agulu, Alor, Oraukwu, Nanka etc. are perching on unstable platforms and fractured geologic units and geologic aquifers hence, the fast development of gully system in these areas.

## **IMPACTS OF GULLY EROSION**

Rampant removal of vegetation cover, unplanned urbanization with attendant use of corrugated iron sheets in buildings, the introduction of concrete flooring in residential places without adequate containment measures of the consequences etc. initiate and promote gully erosion growth and development. As a result, the water which initially infiltrated into the soil, now, moves on the surface as runoff. This produces floods and leads to sheet, rill, and channels erosion. The high rainfall intensity adds to its impacts on the soil, dislodging them and initiating erosion. Broken down drainage, unlined drainages and drainage that do not deliver flood to safe levels are commonly obtainable. Numerous gullies end, along the steep slope and scarp slopes. They form gully lines from the gullies to the slopes, lengthening and expanding the existing gullies. Streams, lakes and rivers serve as receptacles of sediments brought upland, by the devastating effects of the floods. They are heavily-polluted with sediments and organic materials transported from the urban and agricultural lands. The impacts are equally felt during the drying seasons when rainfall is minimal. Eutrophication of the lakes is commonplace. The lakes are becoming shallow, both vertically and horizontally. As a result, many springs and streams are lost. Enormous environmental pollution/contamination by floodwater is common. Borehole schemes have been destroyed. Surface reservoirs have been polluted or silted up. Roads, bridges, building, schools, churches, open spaces and other infrastructure have been destroyed. Many people's lives have been lost Agricultural lands and products have been lost. Industrial and commercial centers

have been damaged. The pollution of the water bodies has resulted in many aquatic resources as witnessed in the weathering of raffia palms, loss of crocodiles and poor fish catch. Domestic water supply is endangered resulting in public health implications and breakdown of waterborne diseases.

## SUGGESTIONS/RECOMMENDATIONS

There is need to consider both the hydrogeological and geological implications of natural and anthropogenic activities in the gully prone areas. The engineering and agricultural practices in the areas should possess anti erosion potentials. The drainages should be wide with thick edges, deep and terminate at safe levels in the river valley. Adequate planning involving experts from relevant fields should be carried out before embarking on any urban/estate development project. Integrated approach in water and environmental management in reservoir construction for optimum benefit is essential. A well designed Catchments Management Water Strategy (CMWS) that is holistic in approach in containing both ecological hazards and harnessing of available surface water resources and groundwater recharge and storage are recommended. A people and community participation activity during planning, design, execution and management of water project is also strongly advised.

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## USING MULTIPLE TRACER METHODS TO INVESTIGATE GROUNDWATER RESIDENCE TIMES AT BANK FILTRATION SITES IN BERLIN, GERMANY

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### ABSTRACT

Berlin abstracts bank filtrate from a broad-scale, lake-type surface water system for its drinking water production. Because treated sewage is released into the surface water, wastewater residues are present in surface water and groundwater. Various environmental tracers including tritium/helium-3 ( $^3\text{H}/^3\text{He}$ ), stable isotopes ( $\delta^{18}\text{O}$  and  $\delta^2\text{H}$ ) and a number of persistent sewage indicators such as chloride (Cl) and boron (B) were used to estimate travel times from the surface water to individual production and observation wells at 2 sites. The study revealed a strong vertical age stratification throughout the upper aquifer at all sites with travel times of the bank filtrate varying from a few months to several decades in greater depth. While the shallow bank filtrate is characterized by the reflection of transient tracer input curves and young  $^3\text{H}/^3\text{He}$  ages, the deeper, older bank filtrate displays no tracer seasonality,  $^3\text{H}/^3\text{He}$  ages of a few years to decades and strongly deviating concentrations of several pharmaceutical residues, reflecting changing concentrations in the source surface water over time. Bank filtration is only possible at the sandy lake shores, while impermeable organic rich muds inhibit infiltration further into the lakes. The older bank filtrate originates from shores hundreds of meters away from the wells. This paper illustrates the importance of using multiple rather than single tracer methods to gain a comprehensive understanding of time scales and infiltration characteristics in a bank filtration system used for drinking water abstraction.

### KEYWORDS

Bank filtration, travel times, wastewater indicators, AMDOPH, Gadolinium

## INTRODUCTION

Bank filtration is induced by placing production wells close to natural rivers, streams and lakes. The main objective of this practice is pre-treat of river water during passage through the river or lake bottom and the aquifer sediments (Bouwer, 2002) and to save limited groundwater resources. Beneficial effective attenuation processes of this practise include the elimination of suspended solids, biodegradable compounds, bacteria, viruses and parasites and the partial elimination of adsorbable compounds (Hiscock and Grischek, 2002). Prior to the interpretation of hydrochemical data in a bank filtration system travel times have to be evaluated. This is important because (i) the threat of contamination with pathogens decreases with increasing residence time within the subsurface and (ii) only with sufficient knowledge of the time scales is a subsequent estimation of degradation rates of biodegradable compounds.

The time difference between a tracer signal in the surface water and in the well water ("lag time"), can be used to estimate travel times (e.g. Hunt et al., 2005). However, depending on local dispersion, this method is limited to travel times of a few years at maximum. The tritium/helium age dating method allows to date water with an age of years to decades and was first suggested by Tolstikhin and Kamenskiy (1969). It is based on the analysis of tritium combined with its decay product, the lighter and rarer 3-helium isotope ( $^3\text{He}$ ).

In Berlin, water is recycled in a semi-closed urban water cycle. The surface water system is shaped by lake-type widened stream courses (Spree, Dahme, and Havel Rivers). The surface water discharge is low as is the hydraulic gradient of the rivers (Jahn, 1998). As a result of induced bank filtration, the surface water infiltrates into the ground, where it mixes with ambient groundwater. After withdrawal, only minimal treatment is required in the water works to ensure potable water quality (aeration and rapid sand filtration; BWB, 2006). Following use, the water is processed in the sewage treatment plants and released back into Berlin's surface water system. Hence, concentrations of a number of sewage-derived water constituents are elevated in comparison to the natural background. The amount of treated sewage in the surface water varies largely, depending on space (i.e., distance from the discharge points) and time (e.g. Massmann et al., 2004). The share of the abstracted groundwater originating from bank filtration and, to a much lesser extent, artificial recharge via ponds, is estimated to be more than 70 % (Pekdeger and Sommer- von Jarmersted, 1998).

The use of anthropogenic Gadolinium ( $\text{Gd}_{\text{excess}}$ ) as a tracer in Berlin was suggested by Knappe et al. (2005). The source for the Gd is Gd-DTPA, a contrast medium in magnetic resonance imaging. In a column study, the stable complex did not show any retardation due to sorption but a removal was observed with a half-life of about 130 days probably due to biological degradation (Holzbecher et al., 2005). 1-acetyl-1-

methyl-2-dimethyloxamoyl-2-phenylhydrazide (AMDOPH) is an oxidative product of dimethylaminophenazone (Reddersen et al., 2002; Zühlke et al., 2004a). AMDOPH has been described as the most persistent and undegradable metabolite of phenazone-type PhACs and has been detected in observation and drinking water wells throughout Berlin, where drinking-water treatment does not remove it efficiently in the presence of O<sub>2</sub> and an adapted microbial community (Reddersen et al., 2002). In addition to the input path via the sewage treatment plants, a former pharmaceutical plant north-west of Berlin is suspected to have released larger quantities of production residues directly into the river Havel in the past (Reddersen et al., 2002).

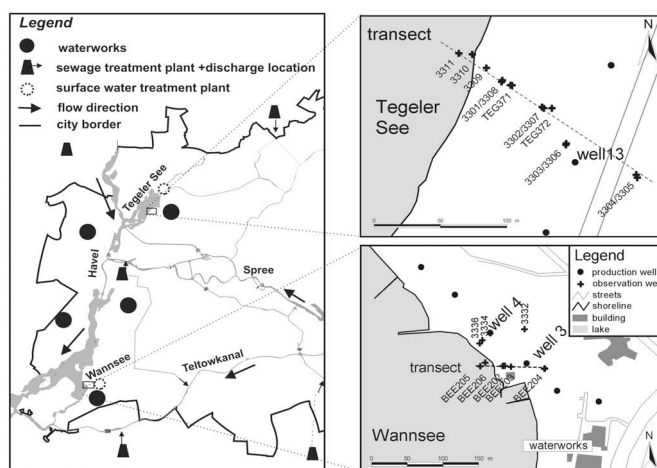
The objectives of this study are to apply a variety of tracers to (i) determine travel times from the surface water to individual wells, (ii) test the application of the persistent sewage indicators AMDOPH and Gd<sub>excess</sub> as additional time markers which may be applicable in similar aquatic environments in the future and (iii) develop a hydrodynamic concept of the lake bank filtration sites. Because the local hydraulic regime is highly complicated due to transient pumping performances of the production wells, hydraulic modelling is complicated and insufficient without accompanying tracer analysis, which serves as a basis for subsequent modelling. This work provides insights into comparable hydrogeological settings where lake-type bank filtration rather than bank filtration at fast flowing, permeable rivers is used for potable water production.

## METHODS

### Site Instrumentation and Geology

The two research transects (Figure 1) discussed were built in front of production well galleries of the Tegel and Beelitzhof waterworks. One is located at Lake Wannsee, in the south-west of Berlin and one is situated at Lake Tegel in the north-west of Berlin. Each transect consists of several observation wells screened in various depths and oriented in flow direction between the lake and a production well. Some observation wells were placed inland of the production wells to investigate mixing with ambient groundwater. Previous investigations at the sites include (Grünheid et al, 2005, Massmann et al., in press a, b).





**FIGURE 1. Outline of the western part of the city of Berlin with the surface water system (grey shading) as well as operating waterworks, sewage treatment plants and surface water treatment plants in the area (left). Surface water flow is from the east and north towards the south-west. Enlargements of the two field sites with production wells and research transects are shown to the right (Massmann et al., in press a).**

At both sites, the Pleistocene sediments of the uppermost aquifer containing bank filtrate are porous, glaciofluvial and fluvial mainly fine- to medium sized sands of the Saale and Weichsel glaciation (Hannappel and Asbrand, 2002). They may locally be divided by an aquitard, a glacial till of the Saale glaciation. The maximum depths of Lake Wannsee and Lake Tegel are 9.8 and 15.5 m respectively. Originally, the glacial valley below the lakes was much deeper, but it was subsequently filled with very fine-grained, organic-rich sediments (lacustrine sapropel)

which presently reach a thickness of tens of meters. These sediments have a low hydraulic conductivity, thereby restricting the infiltration capacity (Fritz et al., 2002). Only the lake margins, where the water depth is less than a few meters, the lake is underlain by sands (Massmann et al., in press b).

### Sampling and Analysis

Regular groundwater sampling was conducted monthly between May 2002 and August 2004. Measurements for redox potential ( $E_h$ ), pH,  $O_2$ , temperature (T) and electric conductivity (EC) were carried out in the field in a flow cell. Filtration with 0.45  $\mu m$  membrane filters was done immediately after sample retrieval for analyses of cations and anions. Samples for cation analysis were preserved with concentrated nitric acid ( $HNO_3$ ). Tritium samples were collected in glass bottles that were carefully filled without any air entrapment. Samples were stored at 4 °C, and full water analysis was generally performed one day after sampling. Samples for He-isotope analysis were filled in 40 ml copper tubes and pinched off by clamps to prevent gas exchange with the atmosphere.

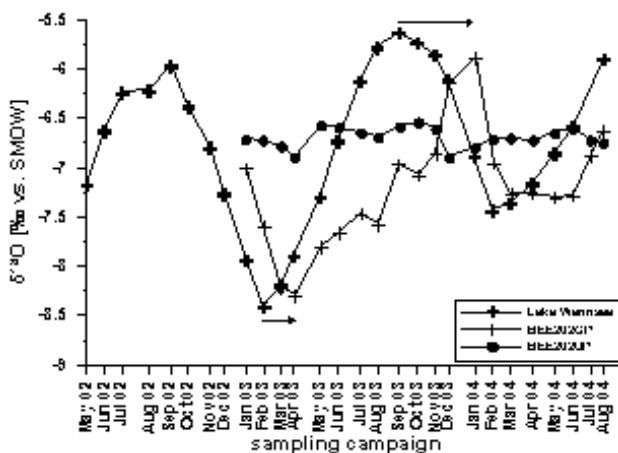
AMDOPH was detected using a method based on a solid phase extraction of the analytes on RP-C18 materials in an automated extraction system (Autotrace SPE Workstation from Zymark) developed by Zühlke et al. (2004b). Liquid chromatographic separation coupled with mass spectrometry was used to detect the analytes. To obtain both high sensitivity and selectivity, tandem mass spectrometry was applied, recording multiple reaction monitoring chromatograms. Anthropogenic gadolinium ( $Gd_{\text{excess}}$ ) measurements were carried out with an ELAN 5000 Perkin Elmer ICP-MS at the GeoForschungsZentrum Potsdam. The analytical procedure and preparation technique are described in Dulski (1994) and Knappe et al. (2005). The calculation of  $Gd_{\text{excess}}$  ( $Gd_{\text{excess}} = \text{measured Gd} - \text{geogenic Gd}$ ) is described in detail in Knappe et al. (2005). The stable isotope measurements of hydrogen and oxygen were carried out at the Alfred Wegener Institute, Research Unit Potsdam with a  $H_2$  equilibration method for  $^2H/^1H$  ratios and a  $CO_2$  equilibration method for  $^{18}O/^{16}O$  ratios with a Finnigan MAT Delta-S mass spectrometer. Details on the instrumentation, calibration and the measurement routine are described in Meyer et al. (2000). Analysis of tritium, helium isotopes (He) and neon (Ne) for  $^3H/^3He$  dating was conducted at the noble gas laboratory of the Institute of Environmental Physics, University of Bremen. Details of the sampling, analysis and data evaluation of the method are given in Sültenfuß et al. (2006).

## RESULTS AND DISCUSSION

### Travel Time Estimation with Tracer Breakthrough Curves and $^3H/^3He$ Dating

Ratios of  $\delta^{18}O$  and  $\delta^2H$  display the typical seasonality (Clark and Fritz, 1997) in the surface water of Berlin with more negative values in winter (Figure 2). Samples taken from different surface water sampling locations in Berlin look basically similar but may deviate slightly because of the variable proportion of treated wastewater in the surface water which displays a more negative isotopic signal due to the higher content of older background groundwater (Massmann et al., 2004a). Most reliable travel time estimates are obtained from phase shifts of the stable isotopes but conservative sewage indicators (e.g. B, Cl<sup>-</sup>) also work. The shallower observation wells between the lake and the production well at both sites studied generally reflect the surface water variations with travel times in the order of a few months, depending on the well (e.g. BEE202OP in Figure 2). In contrast, all observation wells screened at the bottom of the upper aquifer show no or very little seasonality making it impossible to estimate travel times. As an example, BEE202UP (deepest well screened in the upper aquifer at Wannsee) is shown in Figure 2. Despite a distance of less than 30 meters to the lake, the observation well does not reflect the transient tracer signals of the lake. This indicates that the travel time to BEE202UP has to be

much longer than to BEE202OP and that the age stratification within the uppermost unconfined, glacio-fluvial aquifer is large.



**FIGURE 2. Time series of  $\delta^{18}\text{O}$  values in Lake Wannsee and observation wells BEE202OP (shallow) and BEE202UP (deep). Clearly, the BEE202OP reflects the seasonal variations of the lake whereas BEE202UP does not. The arrows indicate the approximate phase-shift, i.e. travel time of 2-4 months from the lake to BEE202OP.**

Samples of the shallow observation wells have a  $^3\text{H}/^3\text{He}$  age of  $< 0.5$  years. They contain less than 0.5 TU of tritiogenic  $^3\text{He}_{\text{tri}}$ , and  $^3\text{H}$  concentrations reflect present concentrations of the lake. The  $^3\text{H}/^3\text{He}$  age dating results are consistent with the time-series analysis (Massmann et al., in press a).

The  $^3\text{H}/^3\text{He}$  ages of the deepest observation wells in the upper aquifer (TEG374 at Tegel and BEE202UP at Wannsee – both just above aquifer base) are 25 and 10 years,

respectively. These observation wells are those lacking seasonality of the stable isotopes and wastewater.

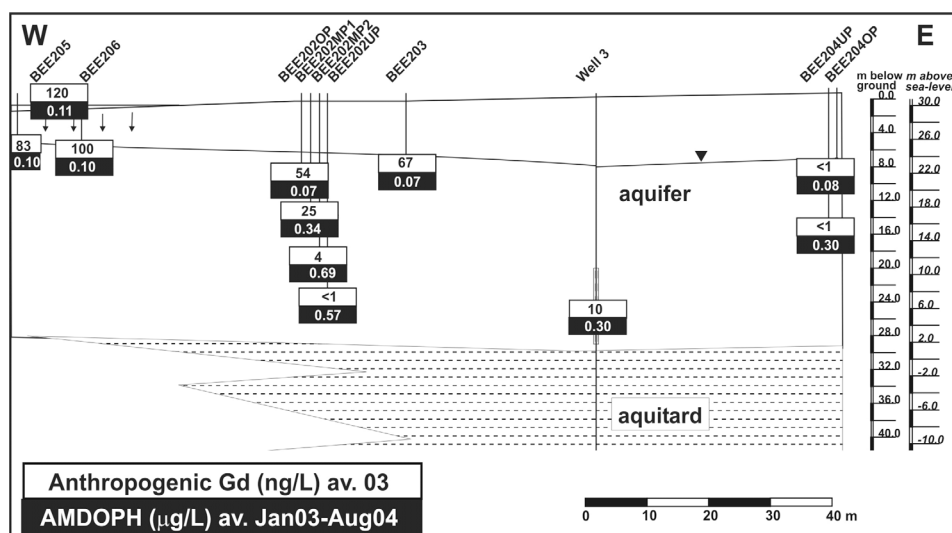
The production wells themselves show a strongly dampened tracer signal due to mixing of bank filtrate with ambient groundwater from both inland and from deeper aquifers (at Wannsee only) and due to the long individual filter screens. However, a slight seasonal signal was temporarily visible at Tegel and correlated to a dominant travel time of 4-5 months. The  $^3\text{H}/^3\text{He}$  ages of the production wells are not equivalent to travel times due to the mixing with both ambient groundwater from inland ( $^3\text{H}/^3\text{He}$  age of  $>10$  years) and, at Wannsee, deeper aquifers (almost  $^3\text{H}$  free and  $>> 50$  years old).

### Using Persistent Wastewater Residues as Additional Age Markers

During the investigations, it became evident that some of the persistent wastewater residues can be used as additional age indicators. Despite the use of, for example B or  $\text{Cl}^-$ , as time-variant tracers as discussed above, the presence of a number of substances in the groundwater appears to reflect their surface water concentration over time, which has not been constant over the past decades. Whereas the

concentrations of, for example, AMDOPH strongly increases with depth, reaching levels clearly above the average surface water concentrations within the monitoring period, the concentrations of  $Gd_{\text{excess}}$  decreases to concentrations in the range of geogenic background concentrations ( $Gd$ ). Figure 3 shows average  $Gd_{\text{excess}}$  and AMDOPH concentrations for the Wannsee 2 transect as an example.  $Gd_{\text{excess}}$  was only detected in samples with an effective  $^3\text{H}/^3\text{He}$  age of a few years (infiltration year 2000 or later), even though  $Gd_{\text{excess}}$  has been present in the surface water before. However, concentrations are unknown and information about the consumption and input into the surface water within the last two decades is sparse. The absence of  $Gd_{\text{excess}}$  in the bank filtrate from the 1990s is likely to be the results of slow biodegradation. It does not persist in an aquifer environment for longer than a few years. Given the surface water concentrations presently encountered in Berlin and a half-life of 130 days (Holzbecher et al., 2005),  $Gd_{\text{excess}}$  can not be expected in bank filtrates older than ~4 years.

The concentrations of phenazone-type pharmaceuticals and their residues including AMDOPH (Figure 3) increase with depth and likewise with the age of the bank filtrate (Massmann et al., in press a). The elevated concentration of AMDOPH in the older bank filtrate can only be explained by formerly elevated surface water concentrations. Any degradation or sorption would have resulted in lower concentrations, thus leading to the conclusion that original surface water concentrations were probably even higher than indicated by the groundwater concentrations. From the data alone, the origin of AMDOPH in the surface water, either from releases of plant production residues directly into the Havel (Reddersen et al., 2002) or from treated sewage cannot be determined. But whatever the source, these substances must have originated from the surface water, thereby proving that the deeper, decade-old groundwater in the upper aquifer is definitely bank filtrate. This shows that in the aquifers along the river Havel in Berlin, AMDOPH, if detected in elevated concentrations, can be applied as a qualitative indicator of old bank filtrate. In addition, the elevated concentrations indicate that it may persist over decades in an aquifer environment. So far, to our knowledge, long term investigations have never been conducted. By contrast, the ambient groundwater does not contain any wastewater residues.



**FIGURE 3. Average AMDOPH [ $\mu\text{g/L}$ ] and Gd [ $\text{ng/L}$ ] concentrations in observation wells of transect Wannsee 2. Note that production well 3 has also got filter screens in the deeper aquifers. AMDOPH concentrations from January 2003-August 2004, Gd<sub>excess</sub> concentrations from January 2003-December 2003 were used.**

## SUMMARY AND CONCLUSIONS

The bank filtrate in the upper aquifer between the lake and the adjacent production well is a mixture of two distinguishable types of bank filtrate:

1. Young shallow bank filtrate characterized by a  $^3\text{H}/^3\text{He}$  age < 0.5 years and a reflection of the transient input curve of the time-variant tracers which allows travel time estimations. The average concentrations of persistent wastewater residues are similar or lower than presently observed in the lake
2. Older bank filtrate with a  $^3\text{H}/^3\text{He}$  age of more than 10 years. It does not reflect the seasonality of the time-variant tracers in the lakes. The concentration of a number of minor water constituents differs largely from the present concentration in the lake (e.g. higher AMDOPH; lower Gd<sub>excess</sub>). Major water components and the stable isotopes (not discussed in detail) tend to present concentrations close to the mean concentrations of today's surface water, but with the lack of the seasonal variability.

Both types of bank filtrate differ from the ambient groundwater (inland and deeper aquifers) which is generally free of pharmaceutical residues. Water in the production

wells is a mixture of old and young bank filtrate with the ambient groundwater from inland and deeper aquifers.

While the young bank filtrate (type 1) originates from the nearest shores, the decade-old bank filtrate (type 2) is thought to infiltrate at opposite shores of Lake Wannsee (no production wells present) and Lake Tegel. This bank filtrate (type 2) then flows beneath the lakes towards the eastern lake shores. This is in agreement with the general flow direction but does, however, show that the surface water bodies are no flow divides as we previously assumed. This hydraulic situation is possible because the lake bottoms are sealed with lacustrine sapropels, enabling infiltration only along the lake shores (Fritz et al., 2002; Massmann et al., in press b). These findings have an impact on the outline and boundary conditions chosen for future hydraulic models at the sites which are needed to define precise infiltration areas.

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## WHAT IS THE APPROPRIATE SITE FOR RBF?

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### ABSTRACT

Many practicing engineers are hesitant to recommend river bank filtration (RBF) for consideration for a water system simply because they are not familiar with how to properly evaluate potential RBF extraction sites commensurate with achieving sustainable water production and treatment goals. Although local siting variables such as governmental regulations, land use, and costs should not be overlooked, the universal factors that should be evaluated at all potential RBF sites include aquifer hydrogeology, river hydrology, river morphology and river bed composition, and surface and groundwater quality and interactions. The paper provides a brief overview of these various factors that should be evaluated and assessed for optimal RBF siting while considering various and sometimes competing water quantity and quality objectives.

### KEYWORDS

clogging, river bank filtration, site, TOC

### INTRODUCTION

Aquifer recharge through induced infiltration of surface water in rivers, channels and lakes is an important process which is used to provide raw water for drinking water production and industrial water use. Large cities and industrial centres often developed at locations where surface water was available for water supply and transport. If groundwater resources near the cities are insufficient, aquifer recharge may become an important alternative for ensuring stable drinking water supply. This paper will focus on the appropriate siting conditions for induced river bank filtration



(RBF) comprising infiltration of river water through the river banks and/or the river bed.

RBF is used as a means of primary and pre-treatment by water utilities throughout the world, especially in Europe and North America. At many sites in Europe, RBF schemes have been efficiently operated for several decades. Most of these are well investigated and monitored for optimisation of processes in the river bed and in the aquifer. Some inefficient schemes have been closed due to clogging or water quality concerns resulting from inadequate siting or operation. Although in Europe new RBF site developments are rare due to decreasing water demand, there is a renaissance of RBF technology in the US, India and other countries. Currently guidelines do not exist to support engineers in identifying an appropriate site for RBF. A complex and comprehensive assessment of hydrological, hydrogeological, hydrochemical aspects together with state specific regulations, land use, cost and other issues is necessary for optimal RBF siting. It is important to have a clear understanding of how RBF will be utilized (e.g., as a means of pre-treatment or as a primary treatment) prior to commencing site location planning studies. The following questions should be answered before planning (Grischek et al., 2003):

4. What amount of water has to be produced?
5. Which length of the river or catchment area along the river can be used for water abstraction?
6. Which drawdown is acceptable depending on aquifer thickness and land use?
7. What amount of water can be continuously withdrawn from the river without ecological conflicts?
8. Which proportion of riverbank filtrate in the pumped raw water should be achieved from water quality reasons?
9. Which advantage of riverbank filtration is considered to be the most important?

RBF systems located on an island or at the inner bends of a meandering river are preferred for situations where: (1) RBF is seen as an alternative to direct water abstraction from the river to gain removal of particles, organic compounds, and pathogens, or (2) the proportion of pumped bank filtrate needs to be as high as possible to prevent abstraction of contaminated groundwater. If the equilibration of water temperature and water constituent concentrations or the removal of organic compounds and pathogens is the major objective, long flow paths and travel times are advantageous.

Scientific design approaches, together with evaluation of case studies, are combined to identify parameters and to propose feasible values for site selection through a RBF design manual, coordinated by the Water Treatment Technology Assistance Center at the University of New Hampshire and supported by the US EPA. A short overview

of universal siting factors is presented below to provide a basis for further discussion.

## HYDROGEOLOGY

The interaction between river water and groundwater is a precondition to abstract river bank filtrate. Factors that influence the yield and quality of water produced by RBF facilities include the size of the river and the underlying aquifer, hydrogeologic boundary conditions, the composition of the aquifer material, and the properties of the river bed and bank or so-called hyporheic zone. RBF facilities are commonly located in alluvial sand and gravel aquifers. In general, the thickness of the aquifer should be more than 10 meters (m) and the hydraulic conductivity of the aquifer should be in the range of  $10^{-2}$  to  $10^{-4}$  meters per second (m/s). But this does not mean that RBF located at sites with lower aquifer thickness or lower hydraulic conductivity is infeasible. At these marginal RBF sites the capacity of the facilities will be lower or specific techniques as drain pipes along the river instead of wells are required. Table 1 is a compilation of selected hydrogeologic information for RBF sites. The hydraulic conductivity of the river bed together with the thickness of the aquifer strongly influences the possible abstraction rate along a river if vertical wells are installed. Otherwise, aquifer thickness is less important in influencing abstraction rates if laterals of collector wells are installed directly beneath the river bed. Thus, two types of RBF settings can be distinguished: (i) water abstraction beneath the river bed and (ii) water abstraction along a river bed. The production capacity of type (i) is typically much higher than that of type (ii) if river bed clogging can be avoided, but a host of advantages of RBF (e.g., mixing and equilibration) processes are widely missed. For type (i), river bed hydraulic conductivity, river bed hydraulics (erosion, deposition), river channel morphology, floods and uses for shipping and dredging are the key aspects for planning. These factors can make water quality for type (i) RBF systems more variable than type (ii) RBF systems.

Aquifers in alluvial plains are often overlain by silty sediments or a clay layer. Such a layer acts as protective cover during floods. Whereas in most cases RBF is used in unconfined aquifers, during floods the higher river water level may cause confined conditions affecting flow regime and travel times. There is no general preference for siting RBF schemes with unconfined or confined conditions. One aspect of unconfined conditions is the increase in infiltration area at the river bank during floods. Infiltration of river water through the unsaturated part of the flood plain can cause breakthrough of pathogens.

For a river, the interaction between surface water and groundwater is often described in terms of gaining or losing conditions. Rivers can exhibit both gaining and losing conditions in different reaches or at different times of the year within the same reach. Losing conditions often cause a long term development of redox zones

and a higher microbial activity than under gaining conditions, where nutrient-poor groundwater enters the river. The river bed sediments and the hyporheic zone provide a food source to a wide variety of microorganisms which play an important role in water quality changes of the infiltrating water. Thus, RBF siting at river reaches with losing conditions will result in a shorter time to achieve stable well water quality and effective removal of organic compounds.

**TABLE 1: Aquifer thickness and hydraulic conductivity of the aquifer material for selected alluvial sand and gravel aquifers.**

| Bank Filtration System               | River System | Aquifer Thickness(m) | Aquifer Hydraulic Conductivity (m/s)      |
|--------------------------------------|--------------|----------------------|-------------------------------------------|
| Henry, Illinois, USA                 | Illinois     | 15 – 20              | $2 \times 10^{-3} - 3 \times 10^{-3}$     |
| Jacksonville, Illinois, USA          | Illinois     | 25 – 27              | $2 \times 10^{-3} - 3 \times 10^{-3}$     |
| Lincoln, Nebraska, USA               | Platte       | 23 – 25              | $1.4 \times 10^{-3}$                      |
| Boardman, Oregon, USA                | Columbia     | 13                   | $3.7 \times 10^{-3}$                      |
| Casper, Wyoming, USA                 | North Platte | 3 – 12               | $9 \times 10^{-4} - 3 \times 10^{-3}$     |
| Cedar Rapids, Iowa, USA              | Cedar        | 15 – 20              | $7.5 \times 10^{-5} - 2 \times 10^{-3}$   |
| Cincinnati, Ohio, USA                | Great Miami  | ~ 30                 | $8.8 \times 10^{-4} - 1.5 \times 10^{-3}$ |
| Louisville, Kentucky, USA            | Ohio         | 21                   | $6 \times 10^{-4} - 1.5 \times 10^{-3}$   |
| SCWA, California, USA                | Russian      | 14 – 30              | $2.4 \times 10^{-4} - 9.9 \times 10^{-3}$ |
| Dresden-Tolkewitz, Germany           | Elbe         | 10 – 13              | $1 \times 10^{-3} - 2 \times 10^{-3}$     |
| Torgau-Ost, Germany                  | Elbe         | 40 – 55              | $6 \times 10^{-4} - 2 \times 10^{-3}$     |
| Auf dem Grind, Düsseldorf, Germany   | Rhine        | 25 – 30              | $1 \times 10^{-3} - 2 \times 10^{-2}$     |
| Flehe, Düsseldorf, Germany           | Rhine        | 10 – 12              | $3 \times 10^{-3} - 6 \times 10^{-3}$     |
| Böckingen, Germany                   | Neckar       | 3 – 5                | $1 \times 10^{-2}$                        |
| Szentendre Island, Budapest, Hungary | Danube       | 3 – 30               | $6 \times 10^{-4} - 2 \times 10^{-3}$     |
| Csepel Island, Budapest, Hungary     | Danube       | 3 – 15               | $6 \times 10^{-4} - 2 \times 10^{-3}$     |
| Koppanymonostor, Komarom, Hungary    | Danube       | 5 – 10               | $6 \times 10^{-4} - 2 \times 10^{-3}$     |
| Linsenthal, Switzerland              | Töss         | 20 – 25              | $1 \times 10^{-5} - 1 \times 10^{-2}$     |
| Maribor, Slovenia                    | Drava        | 14                   | $2 \times 10^{-3} - 4 \times 10^{-3}$     |
| Karany, Czech Republic               | Jizera       | 12                   | $4 \times 10^{-4}$                        |
| Samorin, Slovak Republic             | Danube       | 90 – 120             | $3 \times 10^{-3} - 8 \times 10^{-3}$     |
| Cornella, Spain                      | Llobregat    | 30 – 45              | $5 \times 10^{-3}$                        |
| Seoul, South Korea                   | Tan          | 5 – 8                | $4.9 \times 10^{-5}$                      |
| Haridwar, India                      | Ganga        | 15 – 20              | $2 \times 10^{-4} - 5 \times 10^{-4}$     |

## **RIVER HYDROLOGY**

The aerial extent, structural diversity, and altitudes in the headwater regions of the drainage basin associated with the regional and seasonal distribution of precipitation create the runoff regime of natural rivers. The runoff regime is characterized by amount, frequency, length, time and rate of change of runoff conditions. Runoff dynamics and runoff volumes are important factors when looking at the river and channel morphology, the transport of solids, the modification of the river bed and the interactions with the groundwater. Periodic high river flow events can cause river bed scour and the removal of the clogging layer.

The runoff regime of a river may initially be characterized by the average monthly discharge, developed from daily data over several decades. Averaging discharge or level data is helpful to understand the basic properties of the runoff regime. But only unaltered original data give insight into the runoff dynamics of a river.

Human impact on streams (e.g., for flood control, hydropower generation and improved navigation) changes the natural runoff regime. Dams dramatically alter the flow characteristics of rivers; particularly, the transport and deposition of solids, the erosion of the river bed, and the interactions with the groundwater may be significantly affected by dams.

## **RIVER MORPHOLOGY AND RIVER BED COMPOSITION**

When considering potential sites for RBF systems, the river should have stable river banks and a natural sand river bed cutting into an alluvial sand and gravel aquifer. River instability can result in lateral migration of the channel route. RBF facilities are often located along river bends which can be prone to channel migration, dramatically changing the location and extent of surface water recharge to the underlying aquifer.

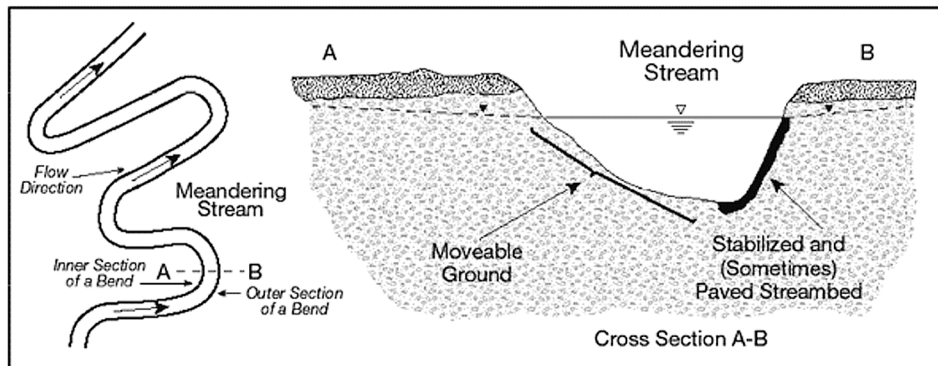
Key factors to consider to minimize river bed clogging are the slope (gradient) of the river, the stream velocity and the composition of the river bed. The profile of a river is continually exposed to the stream processes. Erosion is the dominating process in the headwater regions. Sediments are transported downstream and are deposited when the stream velocity falls below their settling velocities. In the mid-reach section of a river deposition and erosion may be regarded as alternating processes due to the runoff dynamics. Where a river approaches base level, flow velocity slows down and deposition is the dominating process, which creates depositional landforms, such as deltas. The general shape of the profile of a river shows a high gradient in the headwater region, a gradually waning gradient in the mid-reaches and a small gradient approaching base level (e.g. the mouth of the river). Depending on the geology and geomorphology, natural rivers may have several base levels.

The local gradient of the river level contains key information for the selection of RBF sites. The gradient of the river level is not only connected with the range of the shear stress, together with the depth of the river, but also with the average grain size of bed load material. The average grain size of bed load material again may be regarded as a clue for the average grain size of the river bed and together with additional information allow a first assessment of the hydraulic conductivity of the aquifer beneath and adjacent to the river bed.

It is important to understand how the local flow conditions (velocities) can affect the composition of the river bed (Hubbs, 2006). Caldwell (2005) documented slopes for several streams at RBF sites in the US and Europe, indicating that most surface slopes on un-impounded rivers ranged from 0.2 to 0.8 meters per kilometre (m/km). About 90 % of the RBF plants along the German part of the Rhine River are situated in the Lower Rhine region with a hydraulic gradient of the river level of 0,23 – 0,17 m/km, average shear stress on the river bed of 12 – 8 Newton per square meter ( $\text{N/m}^2$ ), average grain size of bed load of 13 – 8 millimeters (mm) and average grain size of the river bed of 32 mm at km 660 location to 14 mm at km 785 location. At many RBF sites in Europe the average flow velocity of the river is more than 1 m/s and the average shear stress is higher than 5  $\text{N/m}^2$ .

RBF systems are typically located in the mid-reaches of the river. Aquifers in the upper reach are often of limited extent and low thickness and river bed sediments are too coarse. Fine sediments are deposited limiting river bed permeability in the lower reach or delta.

The development of meanders in flat country regions is a natural phenomenon of river channels caused by the inertia of flow. In meanders or bends the maximum flow velocity occurs towards the outer section of the bend. This part of the river bed is washed out by erosion and the river bed is deepened, creating an asymmetric cross section (Fig. 1). In this dynamic process, the larger fraction of the river bed material that resists erosion (e.g. pebbles and cobbles) accumulates and may cause a paved bed along the outer section of the bend. The resulting river bed is immobile even during the passage of flood waves (Schubert, 2002). If the river bed is paved or armoured by pebbles and cobbles, induced infiltration of river water can cause filling of the space between the cobbles with very fine material (suspended solids) which cannot be removed during floods. In such cases the permeability of the river bed decreases dramatically. In regions near the inner section of the bend the flow velocity is lower and alternating deposition and erosion may occur, the river bed material remains movable. Meanders of the river channel are preferred sites for RBF plants if the proportion of bank-filtered water in the extracted well water should be high. Inside the loop of a meander the natural cross flow between the upstream and the downstream side of the loop augments the proportion of bank-filtered water.



**FIGURE 1: Meandering river with paved river bed at the outer section of the bend and movable ground at the inner section of the bend (Schubert, 2002, modified).**

The composition of river bed materials controls the permeability. It is likely to be comprised of relatively coarse-grained material along the river bed coincident with the Thalweg channel (exhibiting higher flow velocities). The coarse material is more conducive to infiltration of surface water than finer-grained material deposited along the lower velocity portions of the river.

Other important factors to prevent clogging are the infiltration rate determined by the pumping rate and well location along the bank as well as river water turbidity and particulate organic matter. From long term experiences average infiltration rates of less than  $0.2 \text{ m}^3 / (\text{m}^2 \cdot \text{day})$  over the river bed ensured stable infiltration conditions.

No general RBF siting advice can be given just based solely on a sieve analysis of river bed material. The grain size distribution of the river bed material has to be evaluated together with river hydrology data and shear stress data. A useful tool is the Hjulström diagram which comprises the conditions for erosion, transport and deposition (Hjulström, 1935). Uniform grain size of river bed material is inappropriate (e.g. armoured river bed) for RBF schemes. If the grain size is mixed, the tendency for bed material to be moved will increase.

## RIVER WATER QUALITY

The temperature of river water influences the infiltration rate and flow velocity in the aquifer as the viscosity of water is directly depending on temperature. Warmer water will exhibit higher infiltration rates than colder water. The variation of infiltration rates due to temperature changes can be observed on a seasonal basis as well as a daily basis. Furthermore, processes such as biodegradation, adsorption and gas

dissolution in the river bed and the aquifer are related to temperature. Higher river water temperatures support the degradation of organic compounds in the river bed but at the same time they are often connected with algae growth in the river and lower oxygen contents especially at night time. Sufficient dissolved oxygen in the river water is necessary to prevent anoxic conditions in the aquifer which may cause iron, manganese or arsenic dissolution and thus further treatment demands. A constant river water temperature throughout the year in the same range as the preferred drinking water temperature (e.g. 10 °C) would be the ideal case. Whereas a constant temperature is a more theoretical case, rivers with a moderate seasonal change in water temperature may be preferred compared to strong temperature changes which can cause gas formation and precipitation processes which affect clogging of river beds. If there are strong water temperature changes ( $\Delta T > 20$  Kelvin), additional negative effects such as changing redox conditions or breakthrough of easily biodegradable compounds may occur. Such effects are reduced at RBF sites with a longer distance between the river and the wells or sites with a thick aquifer due to equilibration of water temperature in the aquifer and mixing processes

A main advantage of RBF is the removal of organic compounds by adsorption and biodegradation along the flowpath. If the aim of RBF is to make use of this advantage, the organic carbon concentration in the river water is a key parameter. If no data for biodegradable dissolved organic carbon (BDOC) are available, as a first estimate the river water total organic carbon (TOC) and dissolved organic carbon (DOC) values should be less than 8 mg/L and 6 mg/L, respectively. At many sites in Europe and the US a DOC removal of about 50% or more is observed. If the biodegradable portion of TOC in river water is relatively high, biodegradation processes can consume all dissolved oxygen and often even nitrate. This again can cause anoxic environments in the aquifer (especially in aquifers that exhibit low hydraulic conductivity and/or confined conditions) and dissolution of iron and manganese, which is a disadvantage of RBF. A simple DOC analysis is not sufficient because particulate organic matter (difference between TOC and DOC) often accumulates in the river bed and consumes oxygen and nitrate.

High concentrations of particulate organic matter in the river water (e.g. algae, flocs from domestic effluents, waste water input from pulp and paper mills and sugar production) can also cause bioclogging of the river bed and stick sediment particles together. For these conditions, a higher shear stress is required to remove the clogging layer.

The ammonia concentration in river water should be low to prevent additional oxygen consumption along the flow path in the aquifer. If the river is polluted with effluents, more efforts have to be made to compare the resulting oxygen demand and the available dissolved oxygen and nitrate concentration, especially if iron, manganese or arsenic dissolution under anoxic conditions in the aquifer can occur.

## SUMMARY

RBF has long been utilized both as a means of pre-treatment and primary treatment to provide a reliable and high quality supply of potable water. The keys to a successful RBF installation are (1) a clear understanding of the quantity and quality of water to be provided by RBF, and (2) a comprehensive understanding of the river and aquifer systems under which the RBF facilities will operate. Site factors that should be evaluated at potential RBF locations should include aquifer hydrogeology, river hydrology, river morphology and river bed composition, and surface and groundwater quality and interactions. Several conclusions can be drawn from a review of numerous RBF operations and site characteristics. In essence, RBF systems are typically located in the mid-reaches of the river. Locating wells in the inner bend of a meander is often of advantage. A flow velocity in the river of more than 1 m/s and a shear stress higher than 5 N/m<sup>2</sup> reduce the risk of river bed clogging. The thickness of the aquifer is typically more than 10 m, the hydraulic conductivity of the aquifer is usually in the range of 10<sup>-2</sup> to 10<sup>-4</sup> m/s. Average infiltration rates of less than 0.2 m<sup>3</sup>/(m<sup>2</sup> \* day) are preferred. These parameters should be taken as general guidelines because RBF can be used in a wide variety of site conditions if the objectives of the application are met. If the biodegradable fraction of particulate and dissolved organic compounds in river water causes an oxygen demand higher than the available dissolved oxygen concentration in river water, anoxic conditions in the aquifer can develop resulting in dissolution of iron and manganese.

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## RESERVOIR INFILTRATION TO FRACTURED SANDSTONE AT SAND HOLLOW, SOUTHWESTERN UTAH, U.S.A.

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### ABSTRACT

As alluvial aquifers in arid climates become increasingly utilized, fractured bedrock is being targeted for water-resources development and artificial recharge. The Navajo Sandstone of the southwestern United States is one such target aquifer. This paper describes studies at Sand Hollow Reservoir in southern Utah, conjunctively managed for both surface-water storage and aquifer recharge. This large and deep surface-water reservoir (2.5-square mile area and about 90 feet deep) is underlain by fractured sandstone, either exposed or covered by a veneer of soils. A water-budget approach, including estimated surface evaporation rates calculated with the Jensen-Haise Method, was used to determine net recharge. Since the completion of the reservoir in 2002, estimated evaporation rates have ranged from about 0.05 to 1.0 feet per month and estimated recharge rates to the underlying sandstone have varied from about 0.002 to 0.01 feet per day. Total ground-water recharge from March 2002 through August 2006 is estimated to be about 51,000 acre-feet. With removal of the dynamic viscosity effects caused by large seasonal water temperature variations, the intrinsic permeability indicates a large seasonal variation in clogging, with larger winter permeability likely caused by a combination of both decreased biofilms and the reduced volume of trapped gas bubbles.

### KEYWORDS

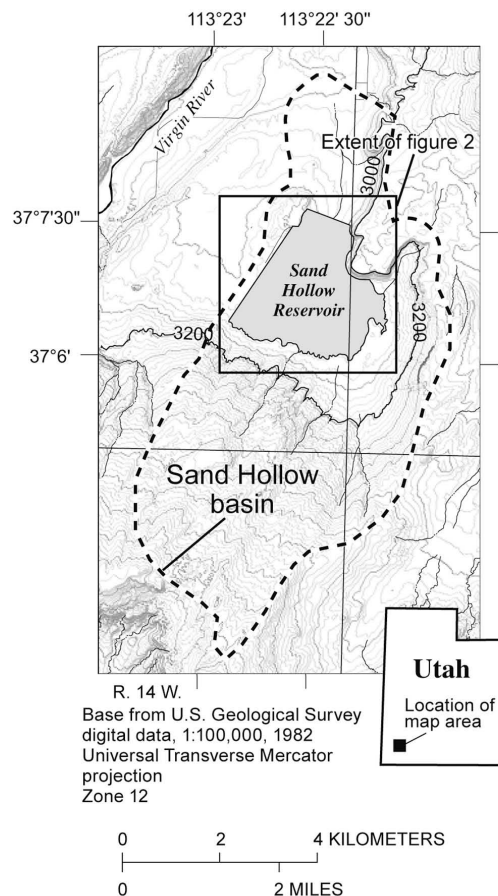
Artificial recharge, fractured sandstone, intrinsic permeability

### INTRODUCTION

Sand Hollow is a 20-square-mile (mi<sup>2</sup>) basin located in the southeastern part of Washington County, Utah, about 10 mile (mi) northeast of St. George (fig. 1) and in the northeastern corner of the Mojave Desert. It is part of the Virgin River drainage basin of the Lower Colorado River Basin. Altitudes at Sand Hollow range from about 3,000 to 4,200 feet (ft) and annual precipitation is about 8 inches (in). Sand Hollow is underlain primarily by Navajo Sandstone that is either exposed at the surface or covered by a veneer of soil or surface-flood basalts (Hurlow, 1998). Although the total stratigraphic thickness of the Navajo Sandstone in this region is more than 2,000 ft, erosion within the study area has resulted in sandstone thickness ranging from a few hundred to more than 1,200 ft. The Navajo Sandstone is a regionally important bedrock aquifer. It is part of the Dakota-Glen Canyon aquifer system,

which consists of permeable sedimentary formations ranging in age from Early Jurassic to Late Cretaceous. The Dakota-Glen Canyon aquifer is the principal source of ground water in the Colorado Plateau region (Robson and Banta, 1995). This aquifer system covers an area of more than 75,000 mi<sup>2</sup> in Utah, Arizona, Colorado, and New Mexico. Many municipalities in this region, including most cities and towns in Washington County, derive the majority of their municipal water from the Navajo Sandstone.

The population of Washington County doubled from 48,000 in 1990 to 90,000 in 2000 (U.S. Census Bureau, 1992, 2003), and is expected to increase to nearly 230,000 by 2020. To help meet the demand for additional water resources, Sand Hollow Reservoir was constructed in 2002 to provide both off-stream surface-water storage and artificial recharge to the underlying Navajo Sandstone. The maximum acreage and storage volume of the surface-water reservoir, when full, is 1,300 acres and 50,000 acre-ft, respectively. A large amount of precipitation occurred during 2004 and 2005, allowing the Washington County Water Conservancy District (WCWCD) to divert large quantities of surface water from the Virgin River, nearly filling the reservoir to capacity in February 2006. Previous reports (Heilweil et al., 2005, 2007) document both pre-reservoir ground-water conditions (prior to March 2002) and post-reservoir ground-water conditions and water budgets (2002-06).

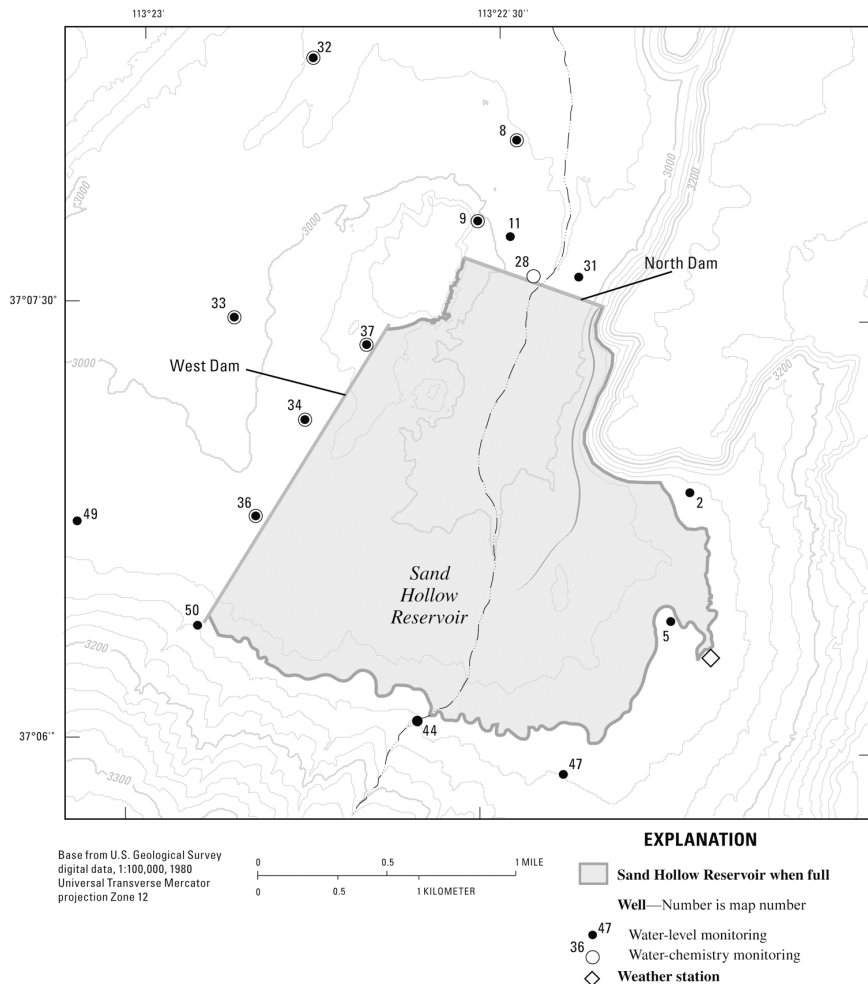


**FIGURE 1. Location of the Sand Hollow study area, Washington County, Utah.**

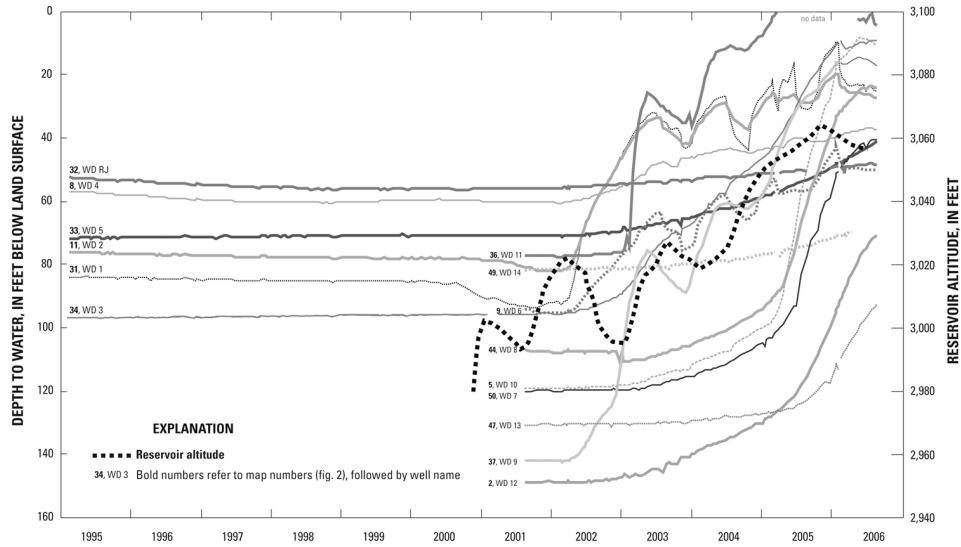
## METHODS

Data collection techniques and methods are described in Heilweil et al. (2005, 2007) and briefly summarized here. Water levels have been measured monthly at a

network of monitoring wells (fig. 2, 3) surrounding Sand Hollow Reservoir since 1995. Surface-water inflow to and outflow from the reservoir using in-line totalizing flow meters have been recorded daily. Reservoir stage has been measured daily with a pressure transducer and compared to weekly staff-gauge check measurements. Meteorology data have been collected continuously at a weather station in Sand Hollow since 1998 for evaluating evaporation and precipitation. Continuous water-temperature measurements were made in Sand Hollow Reservoir for evaluating effects of water viscosity changes on seepage rates beneath the reservoir. A string of five thermistors was installed in the deepest part of Sand Hollow Reservoir, about 300 ft from the North Dam. The thermistors were hung from a floating buoy at depths of 0.3, 3, 10, 16, and 33 ft or bottom of the reservoir, if shallower.



**FIGURE 2. Location of wells and the weather station in Sandy Hollow, Utah.**



**FIGURE 3. Relation between water level in selected wells and reservoir altitude, Sand Hollow, Utah.**

Ground-water recharge to the Navajo aquifer underlying Sand Hollow Reservoir is calculated with the following water-budget equation:

$$R = I_{SW} - O_{SW} \pm \epsilon S - E \quad (\text{EQ 1})$$

where  $R$  is recharge,  $I_{SW}$  is surface-water inflow,  $O_{SW}$  is surface-water outflow,  $\epsilon S$  is change in surface-water storage, and  $E$  is evaporation. Surface-water inflow and outflow to Sand Hollow Reservoir, along with reservoir-stage measurements, were recorded daily by the WCWCD. Changes in surface-water storage were calculated from reservoir-stage measurements and stage-volume relations for the reservoir.

On the basis of an earlier comparison of different methods (Heilweil et al., 2005), the McGuinness and Bordne (1971) version of the Jensen-Haise method was determined to be an accurate and appropriate method for calculating evaporation from Sand Hollow Reservoir and was used during this study. The method is based on the relation (McGuinness and Bordne, 1971):

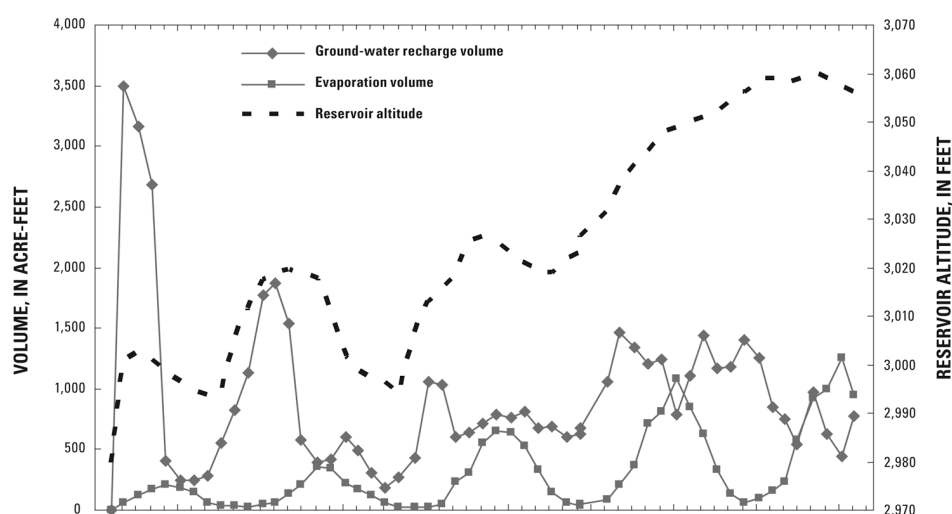
$$PET = \{[(0.01T_a) - 0.37](Q_s)\}0.000673\}2.54 \quad (\text{EQ 2})$$

Where  $PET$  is potential evaporation, in centimeters per day,  $T_a$  is air temperature, in degrees Fahrenheit, and  $Q_s$  is solar radiation, in calories per square centimeter per day. The units for  $PET$  can be converted to feet per day by multiplying by 0.0328.

## RESULTS AND DISCUSSION

Current (2006) ground-water levels in the basin range from about 5 to 90 ft below land surface. Since the inception of the reservoir, water levels have risen as much as 100 feet. Differences in hydraulic head between the reservoir and the 15 monitoring wells in Sand Hollow currently range from about 5 to 150 ft. From 1998 through 2006, daily average air temperature ranged from -1 to 37 C. Daily average solar radiation ranged from 34 to 840 calories per square centimeter per day. Monthly precipitation ranged from 0 to almost 4 inches. Water temperature from January 2003 through May 2006 at the shallowest and deepest thermistors has ranged from 3 to 32°C and 4 to 25°C, respectively. Net monthly total surface-water inflow (inflow minus outflow) has ranged from -3,000 to about 6,400 acre-ft. Surface-water storage in the reservoir has ranged from about 11,000 acre-ft to about 51,000 acre-ft.

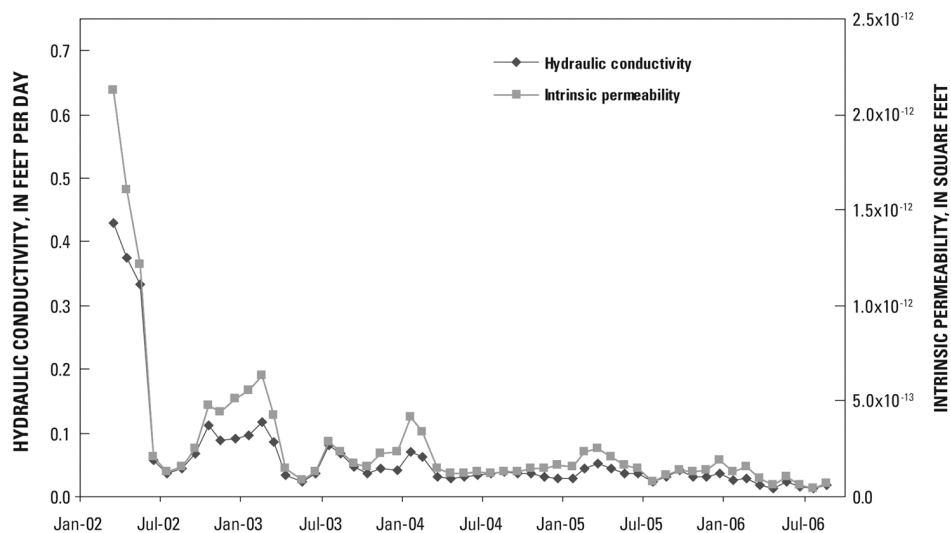
Using air temperature and solar radiation from the weather station in equation 2, calculated monthly evaporation rates ranged from 0.05 to 0.97 ft during 2002-06. Since 2002, the reservoir surface-water area has fluctuated, but gradually increased to about 1,300 acres in 2006. Multiplying evaporation rates by average reservoir surface area yields monthly evaporation losses ranging from 20 to 1,200 acre-ft. Total evaporative loss since 2002 has been about 17,000 acre-ft. Based on equation 1, monthly ground-water recharge has ranged from about 200 acre-ft to 3,500 acre-ft (fig. 4), with a cumulative recharge since 2002 of about 51,000 acre-ft. Net annual surface-water inflow has ranged from about 8,000 to 52,000 acre-ft, annual estimated evaporation has ranged from about 1,000 to 5,000 acre-ft, and annual ground-water recharge has ranged from about 9,000 to 14,000 acre-ft (not including 2006, for which data are incomplete).



**FIGURE 4. Monthly estimated evaporation, ground-water recharge, and reservoir altitude, Sand Hollow, Utah, 2002-2006.**

Equation 1 does not account for precipitation falling directly on the surface of the reservoir or runoff to the reservoir. Because of high evaporation rates and permeable surficial soils, precipitation events seldom produce runoff that reaches the lower part of Sand Hollow basin. On the basis of monthly total precipitation data from the weather station and reservoir surface areas, the largest amount of monthly precipitation falling directly on the reservoir surface is only 180 acre-ft. This amount is small relative to the average monthly inflow of more than 2,000 acre-ft and, therefore, was not considered during the recharge calculations.

The monthly recharge volume was divided by the surface area of the reservoir, resulting in monthly average recharge rates. Assuming a unit vertical hydraulic gradient of 1ft/ft, monthly average hydraulic conductivity values range from 0.01 to 0.43 feet per day (ft/d; fig. 5). Monthly average hydraulic conductivity values from 2002 through 2006 indicate a general downward trend in hydraulic conductivity, even as the reservoir altitude (driving head) increased. The general decline in hydraulic conductivity during the 4-year period since initial filling is a typical pattern observed at most artificial recharge facilities. This decline is likely caused by a combination of (1) a lower horizontal hydraulic gradient once the ground-water table connected with the base of the surface-water reservoir, and (2) clogging (siltation, biofilm development, gas generation) of the sediments beneath the reservoir. Superimposed on this is a second-order effect of winter peaks for recharge rates.



**FIGURE 5. Average monthly hydraulic conductivity and intrinsic permeability, Sand Hollow, Utah, 2002-2006.**

In order to more closely examine clogging at Sand Hollow, specific changes in aquifer properties can be separated from changes in water properties by calculating

intrinsic permeability. Hydraulic conductivity is a function of both aquifer and fluid properties. Intrinsic permeability is the aquifer-properties component of hydraulic conductivity. Intrinsic permeability, therefore, removes the seasonal changes caused by variations in temperature-dependent viscosity of water. Intrinsic permeability is defined as (Freeze and Cherry, 1979):

$$\kappa = K \frac{\mu}{\rho g} \quad (\text{EQ 3})$$

Where  $\kappa$  is intrinsic permeability in square feet ( $\text{ft}^2$ ),  $K$  is hydraulic conductivity (Darcy velocity) in feet per second ( $\text{ft/s}$ ),  $\mu$  is dynamic viscosity in pound per foot-second ( $\text{lb/ft-s}$ , where 1  $\text{lb/ft-s}$  equals 1,488 centipoise or 1.49 kilogram per meter-second),  $\rho$  is the density of water of 62.5 pounds per cubic foot ( $\text{lb/ft}^3$ ), and  $g$  is the acceleration of gravity of 32.2 feet per square second ( $\text{ft/s}^2$ ). This equation yields intrinsic permeability in  $\text{ft}^2$ , which can be converted to square meters by multiplying by 0.093.

Of the fluid properties, the density of water does not vary substantially within the normal range of surface-water temperatures and the acceleration of gravity is not temperature-dependent. Dynamic viscosity, however, is very temperature dependent. In order to calculate intrinsic permeability, therefore, temperature data from the bottom thermistor in the reservoir was used to calculate dynamic viscosity. On the basis of monthly average measured water temperatures, the calculated dynamic viscosity of water in the reservoir has varied by about a factor of two, from 0.0010  $\text{lb/ft-s}$  (1.5 centipoise) at 5°C to 0.00059  $\text{lb/ft-s}$  (0.88 centipoise) at 24 C.

With equation 3, average monthly intrinsic permeability beneath the reservoir has ranged from  $4.5 \times 10^{-14}$  to  $1.2 \times 10^{-12} \text{ ft}^2$  from 2002 through 2006. There is a large seasonal variation in intrinsic permeability, which increases in the winter (fig. 5). This increase may be caused by either the dissipation of biofilms and algal mats (decreased biological activity) or the reduction in the size of trapped gas bubbles in the aquifer. As an example, there would be an approximate 10-percent decrease in the volume of trapped gas bubbles in sediments beneath the reservoir as bottom water temperatures cool from 24°C in the summer to 5°C in the winter. It was shown earlier (Heilweil and others, 2004) that small changes in the volume of trapped gas bubbles can cause substantial effects on the hydraulic conductivity of the sandstone.

## CONCLUSIONS

This study evaluated of ground-water recharge beneath Sand Hollow Reservoir from 2002 through 2006. To estimate ground-water recharge beneath Sand Hollow Reservoir, a water-budget approach was used. Components of the water budget

include surface-water inflow and outflow to and from the reservoir, evaporation, and changes in surface-water storage. Estimated evaporation rates since the inception of the reservoir, based on the Jensen-Haise method, have varied from 0.05 to 0.97 ft per month, resulting in evaporation losses of 20 to 1,200 acre-ft per month. Total evaporation from March 2002 through August 2006 is estimated to be about 17,000 acre-ft. Estimated recharge rates have ranged from 0.01 to 0.43 ft/d. Since the inception of the reservoir, monthly recharge volumes have ranged from about 200 to 3,500 acre-ft. Total recharge is estimated to be about 51,000 acre-ft, with annual recharge ranging from about 9,000 to 14,000 acre-ft. There has been a general downward trend in recharge rates from 2002 through 2006, even as the reservoir altitude (driving head) increased. This is likely caused by a combination of lower hydraulic gradients once the ground-water table connected with the surface-water reservoir, and clogging (siltation, biofilm development, trapped air, gas generation). The combination of generally declining recharge rates and generally increasing reservoir altitude and area explains the trend of an increasing ratio of evaporation to recharge volume over time, with the total volume of water lost through evaporation nearly as large as the volume of ground-water recharge during the first 8 months of 2006.

In order to more closely examine clogging at Sand Hollow, intrinsic permeability (representing the aquifer-properties component of hydraulic conductivity) was calculated in order to remove seasonal changes caused by variations in temperature-dependent viscosity of water. Average monthly intrinsic permeability beneath the reservoir has ranged from  $4.5 \times 10^{-14}$  to  $1.2 \times 10^{-12}$  ft<sup>2</sup> from 2002 through 2006 and shows a large seasonal variation, with generally higher values in the winter. This increase may be caused by either the dissipation of biofilms and algal mats (decreased biological activity) or the reduction in the size of trapped gas bubbles in the aquifer.

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# A QUANTITATIVE EVALUATION OF THE IMPACTS OF ARTIFICIAL RECHARGE TO THE MORNAG AQUIFER SYSTEM OF NORTHERN TUNISIA

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## ABSTRACT

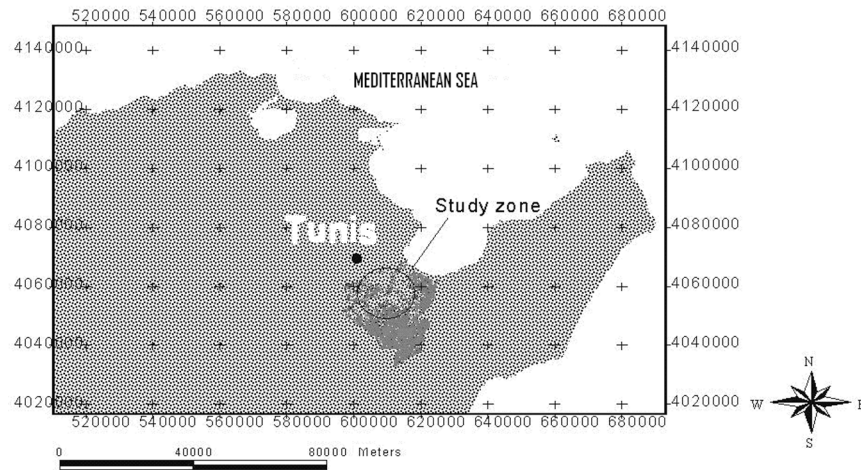
This paper presents results from a pilot study of artificial recharge to the Mornag aquifer, a primary source of water for northern Tunisia. The Mornag aquifer system is complex, consisting of both Oligocene sandstone and Quaternary alluvial deposits. Ground-water development of the Mornag aquifer system, predominantly for irrigation, has caused water-level declines of up to 15 meters. To counteract these declining water levels, artificial recharge of surface water from the Medjerda-CapBon Canal to an abandoned sandstone quarry began in 1991. Infiltration rates of up to 750 litres per second have been attained. Significant volumes (5.2 million cubic meters) of water were recharged in the late 1990s wet period, causing a substantial rise in water levels in a surrounding network of more than 40 wells and piezometers. This successful pilot project has shown the utility of conjunctively managed surface-water and ground-water resources and resulted in the current consideration of other potential artificial recharge sites within the Plain of Mornag.

## KEYWORDS

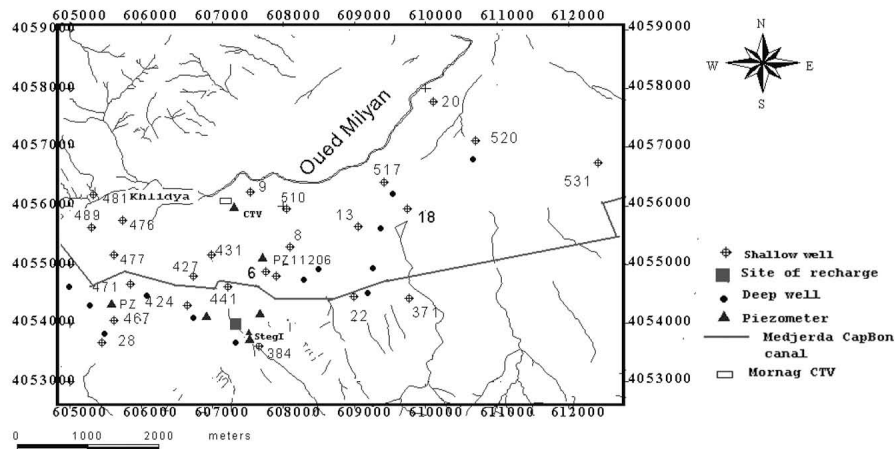
Artificial recharge, Oligocene sandstone, Mornag aquifer, Tunisia

## INTRODUCTION

The Plain of Mornag is 20 km southeast of the city of Tunis and within a kilometre (km) from the Mediterranean Sea (Fig.1). This plain encompasses an area of 200 km and is crossed by the Milyan River (Oued Milyan, Fig. 2). The climate is characterized by warm, dry summers and mild winters with variable rainfall. Average annual precipitation is about 480 mm. Average annual temperature is 18 C. Potential average annual evaporation is approximately 1,500 mm. The Mornag aquifer system is complex, containing both a shallow unconfined aquifer in recent Quaternary alluvium and a deep aquifer in older Miocene and Oligocene sandstone formations (Scholler, 1932). Ground-water storage within the Mornag aquifer system is estimated to be approximately  $1.3 \times 10^9 \text{ m}^3$ , assuming an aquifer volume of  $1.61 \times 10^{10} \text{ m}^3$  and a storage coefficient of 0.08 (Ennabli, 1980).



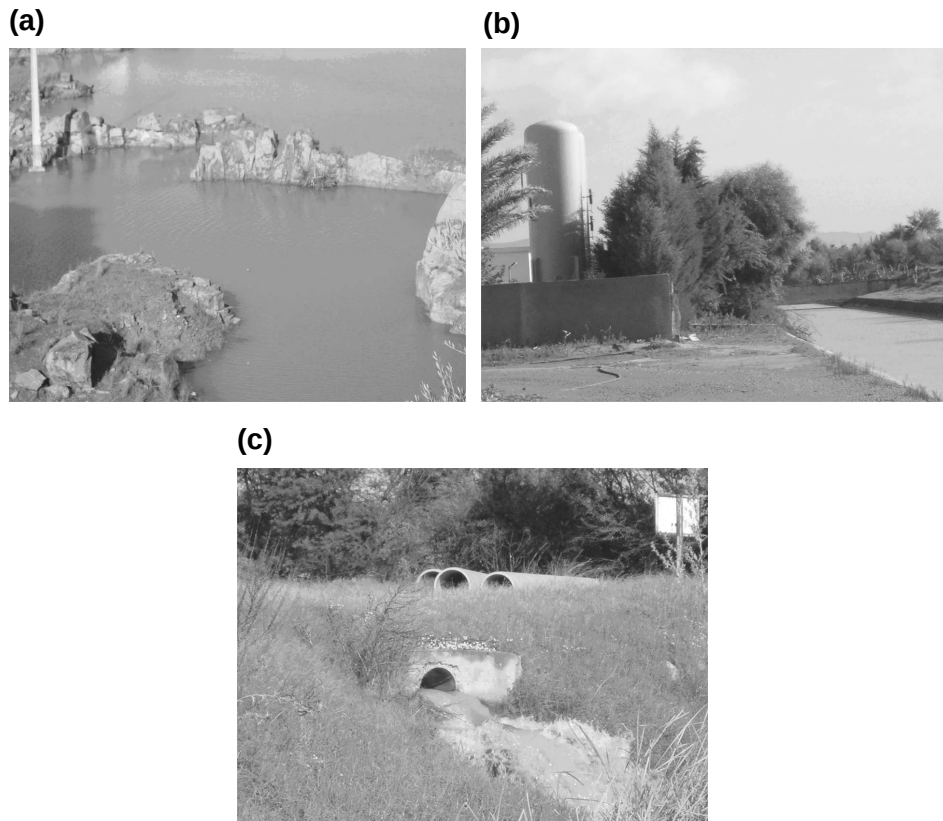
**FIGURE 1. Location of the Mornag aquifer study area, northern Tunisia.**



**FIGURE 2. Location of piezometers, production wells, the Mornag CTV precipitation station, and the Khlidya recharge site, Mornag aquifer study area, Tunisia (from Mhamdi, 2004b).**

Ground-water development of the Mornag aquifer system has caused widespread water-level declines. Ground-water levels are affected to a smaller extent by the variation in precipitation. To counteract these water-level declines, an artificial recharge pilot projects have been implemented (Mhamdi, 2004a). The Khlidya

artificial recharge site (Fig. 2) is an abandoned Oligocene sandstone quarry situated adjacent to the training center of the Tunisian Society of Electricity and Gas. This artificial recharge, utilizing surface water from the nearby Medjerda-CapBon Canal (Fig. 3b), began in 1991. The water is pumped from the canal through an asbestos cement pipe (Fig. 3c) to the quarry. The surface-water reservoir (Fig. 3a), when full, has a triangular shape with an area of 1,250 m<sup>2</sup> and a depth ranging from 3.6 to 11 m.



**FIGURE 3. (a) The Khlidya quarry recharge site, (b) the Medjerda-CapBon Canal, and (c) the asbestos cement pipe transporting water from the Medjerda-CapBon Canal to the Khlidya recharge site.**

This paper presents a case study of artificial recharge to the Mornag aquifer system at the Khlidya site. The objective of this work is to evaluate the relative effects of three factors on ground-water levels: artificial recharge, natural infiltration of precipitation, and water withdrawals from wells.

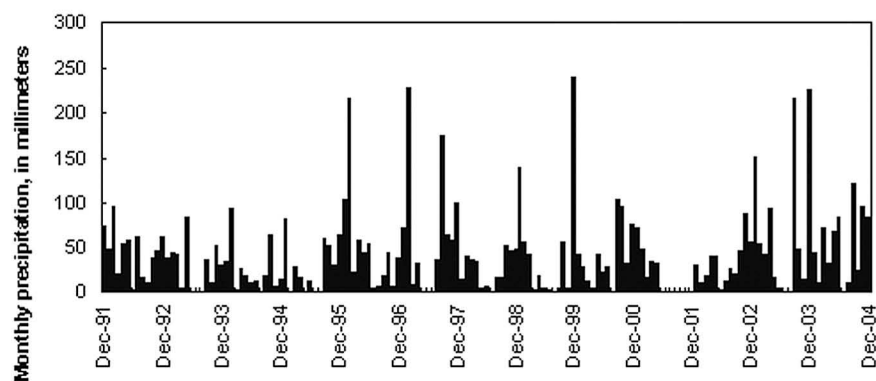
## METHODS

Daily precipitation was measured at the Mornag CTV precipitation station (Fig. 2) from 1991 through 2004. Monthly diversions from the Medjerda-CapBon Canal to the Khlidya artificial recharge site were measured and recorded using a flow meter. Evaporation was estimated by multiplying the average annual pan evaporation rate (1,500 mm) by the surface area of reservoir at the Khlidya site (1,250 m<sup>2</sup>, when full). The salinity of water diverted for artificial recharge was measured at the Foundek Ejedid's and Ghdir Golla stations located 30 km upstream and downstream of, respectively, the Medjerda-CapBon Canal pumping station. Salinity was also measured in water samples collected from 17 production wells completed in the deep aquifer (Oligocene sandstone).

To evaluate the impact of artificial recharge on ground-water levels, temporal and spatial changes to the potentiometric surface of the shallow aquifer were investigated by measuring water levels in the piezometers in both shallow (<50 m) and deep (>50 m) wells. A previous study showed that water levels in wells and piezometers located near the Khlidya site had risen during 1992-93, as a result of artificial recharge (Labben, 1994).

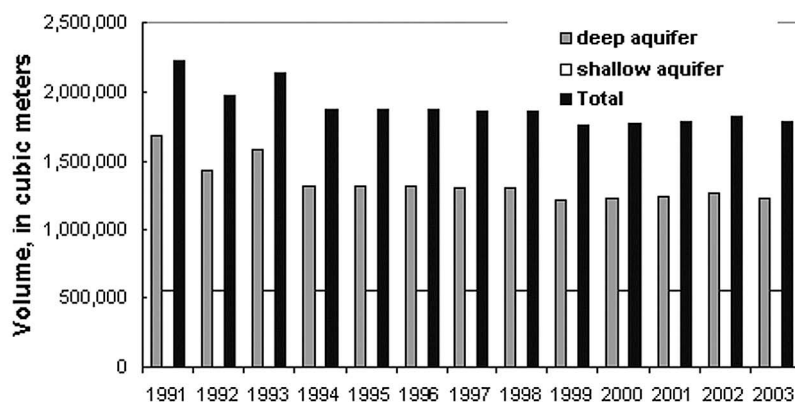
## RESULTS AND DISCUSSION

Monthly precipitation during 1991-2004 at the Mornag CTV precipitation station ranged from 0 to 250 mm (Fig. 4). Total annual precipitation varied substantially, ranging from about 200 mm in 2001 to almost 800 mm in 1997. Most of the rainfall within the study area occurs during the winter months.



**FIGURE 4. Monthly precipitation at the Mornag CTV station, 1991-2004.**

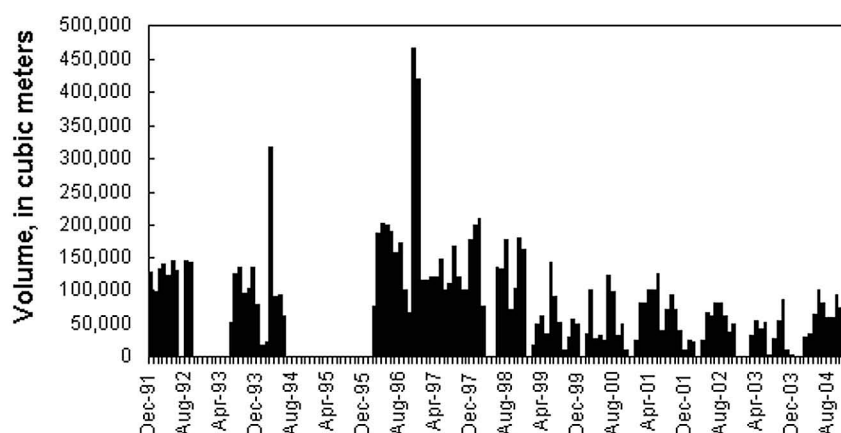
Average total annual withdrawal from wells in the Mornag aquifer between 1991 and 2003 is estimated to be about 1,800,000 m<sup>3</sup> and ranged from about 1,700,000 to 2,200,000 m<sup>3</sup> (Fig. 5). Approximately two-thirds of the water withdrawn (1,200,000 m<sup>3</sup>) comes from the deep aquifer and the other one-third (600,000 m<sup>3</sup>) comes from the shallow aquifer. Dividing total withdrawals by the 17 production wells yields an estimated pumping rate of about 300 m<sup>3</sup>/d per well.



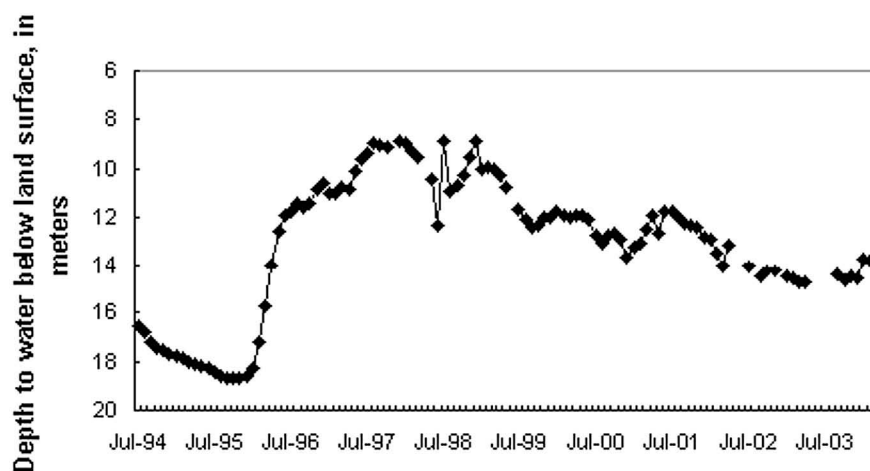
**FIGURE 5. Historical well withdrawals from the Mornag aquifer, 1991-2003.**

The monthly amount of water diverted from the Medjerda-CapBon Canal to the Khlidya artificial recharge site ranged from 0 to about 460,000 m<sup>3</sup> (fig. 6). Infiltration rates of up to 750 litres per second (L/s) have been attained (Mhamdi, 1998). A total of about 10,717,000 m<sup>3</sup> of water was diverted to the recharge site between 1991 and 2004. Total evaporative loss between 1991 and 2004 is estimated to be about 17,000 m<sup>3</sup>. Accounting for evaporative loss, about 10,700,000 m<sup>3</sup> of artificial recharge is estimated to have entered the shallow aquifer during this period. The most significant artificial recharge occurred during 1996-98. During these three years, it is estimated that about 5,173,000 m<sup>3</sup> of recharge occurred.

Salinity of the artificial recharge water, measured at the Foundek Ejedid's and Ghdir Golla stations along the Medjerda-CapBon Canal, ranged from 500 to 1,000 mg/L in the winter, and increased to 2,000 mg/L in the summer. Before initiation of artificial recharge at the Khlidya site, the salinity of ground water in wells varied from 700 to 1,700 mg/L. Ground-water salinity in the aquifer increased towards the Milyan River (Mhamdi, 1998).



**FIGURE 6. Monthly volume of water diverted to the Khlidya recharge site, 1991-2004.**

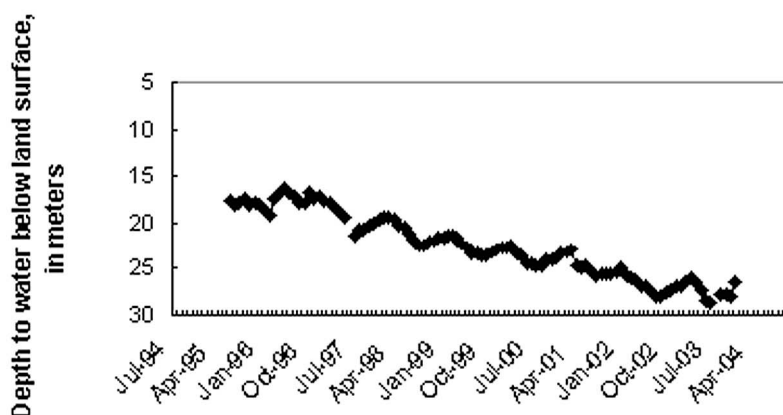


**FIGURE 7. Hydrograph of water levels at the Steg I piezometer, located adjacent to the Khlidya recharge site, 1994-2004.**

The hydrograph of water levels at the Steg I piezometer is characteristic of water levels near the Khlidya recharge site (Fig. 7). The hydrograph shows that water levels declined during 1994 and 1995, likely a result of the cessation of the artificial recharge during that period. Water levels then rose from 1995 through 1997, caused by the combined effect of natural infiltration of precipitation during these wet years (Fig. 4) and the large volume ( $5.2 \text{ Mm}^3$ ) of artificial recharge. Between 1999 and 2003,

the volume of artificial recharge was much smaller, and water levels generally declined. In early 2004, however, a small water-level rise occurred in the Steg I piezometer, possibly caused by natural infiltration associated with the above-average precipitation in September and December 2003.

In the center of the Plain of Mornag about 2 km north of the Khlidya recharge site, water levels at the CTV piezometer declined about 15 m between 1996 and 2003 (Fig. 8). This decline can be explained by the long-term impact of withdrawal from nearby wells. Superimposed on this decline are seasonal fluctuations. The water-level declines in the center of the plain indicate that the effects of artificial recharge at the Khlidya site are local and do not cause a substantial rise in water levels throughout the basin. The slight inter-annual fluctuations are caused by seasonal variations in the infiltration of precipitation and ground-water pumping.



**FIGURE 8. Hydrograph of water levels at the CTV piezometer located about 2 km north of the Khlidya recharge site, 1995-2004.**

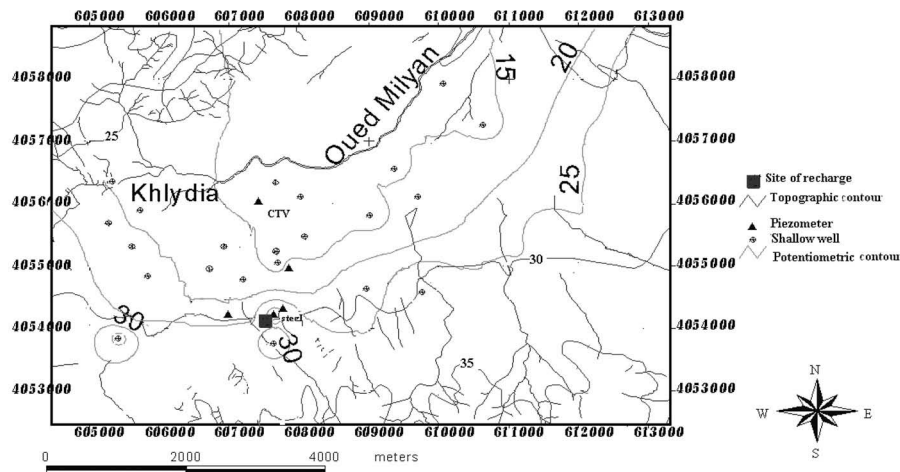
Mass water-level measurements were made at selected wells in the shallow aquifer of the Mornag aquifer system during January 1996, January 1999, and January 2004. The potentiometric surface map of January 1996 shows a decrease in hydraulic head towards the north, where ground water discharges to pumping wells and the Milyan River (Fig. 9). The effects of pumping are indicated by a depression in the potentiometric surface in the center of the plain, likely the result of over-pumping.

The potentiometric surface map of January 1999 shows a similar water-level pattern as in 1996, with a general hydraulic gradient towards the Milyan River (Fig. 10). However, a significant rise in the potentiometric surface occurred in the vicinity of the Khlidya artificial recharge site, associated with the large amount of artificial recharge during 1996-99. Water levels also rose in other parts of the study area away from the large production wells, caused either by artificial recharge or from

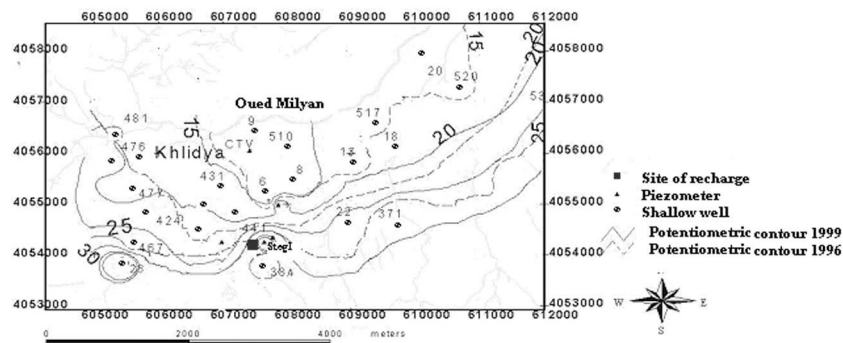


increased infiltration associated with above-average precipitation during 1996-1999. There was no substantial decrease in pumping during these years (Fig. 5).

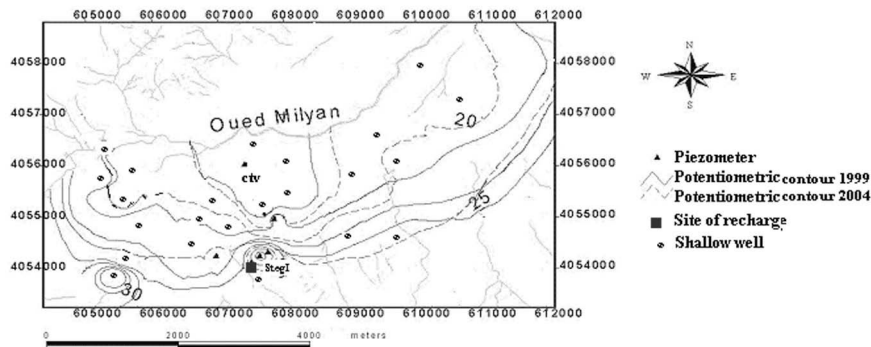
The potentiometric surface map of January 2004 (fig. 11) shows the same depression in the potentiometric surface the center of the Plain of Mornag, and a slight decline in water levels from January 1999, even though there was an increase in precipitation during 2002-2004. This water-level decline is likely caused by extensive ground-water pumping.



**FIGURE 9. Potentiometric surface of the shallow aquifer of the Mornag aquifer system, January 1996 (from Mhamdi, 2004b).**



**FIGURE 10. Potentiometric surface of the shallow aquifer of the Mornag aquifer system, January 1996 and January 1999 (from Mhamdi, 2004b).**



**FIGURE 11. Potentiometric surface of the shallow aquifer of the Mornag aquifer system, January 1999 and January 2004 (from Mhamdi, 2004b).**

## CONCLUSIONS

Between 1991 and 2004, it is estimated that about 10,700,000 m<sup>3</sup> of artificial recharge entered the shallow aquifer at the Khlidya artificial recharge site. The most significant volume of recharge, about 5,200,000 m<sup>3</sup>, occurred between 1996 and 1998. The increase in artificial recharge caused a rise in the shallow ground-water levels, especially near the recharge site. Water levels at the Steg I piezometer, adjacent to the site, rose by about 10 m. Water levels also increased to a lesser extent at other locations within the study area during 1996 to 1999, caused either by artificial recharge or increased infiltration associated with above-average precipitation during this period. More recently, extensive ground-water pumping in the Plain of Mornag has caused large water-level declines away from the recharge site, as evidenced by a nearly 15-m decline in the potentiometric surface from 1996 to 2003 at the CTV piezometer.

The positive impact of the recharge in and around the Khlidya site indicates that this type of artificial recharge facility can be used as a sustainable water-resources management strategy for minimizing ground-water declines associated with pumping of the Mornag aquifer system. Additional potential recharge sites along the exposed Oligocene sandstone are currently being considered for increasing artificial recharge volumes. The extensive pumping of ground-water for agriculture could also be mitigated by offering subsidies to farmers for decreasing withdrawals from wells.

## ACKNOWLEDGMENTS

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## THE CITY OF BEAVERTONS BASALT-HOSTED ASR PROJECT: A SUCCESSFUL CASE STUDY

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### ABSTRACT

Despite western Oregon's reputation for being "wet," many cities find it increasingly difficult to meet peak water supply demand during the dry summer months. The reasons: minimum stream flow requirements, difficulty in finding suitable aboveground reservoir sites in an urbanized environment, and over-drafting of aquifers. An increasing influx of new residents and businesses has added to increased pressures on peak demand. The City of Beaverton (City), Oregon, recognized these supply hurdles in the early 1990s and opted to evaluate aquifer storage and recovery (ASR) to help offset peaking demand. Since 1999, the City has installed three ASR wells hosted in a basalt aquifer. Currently, the City stores approximately 170 cubic meters (450 million gallons) of treated drinking water annually with its ASR wells. The three wells can provide up to about 22,700 cubic meters per day (6 million gallons per day) of peaking capacity, which is equivalent to 35 percent of the City's summer peak day demand. Favorable hydrogeologic response and significant economic savings have made the City's ASR system immensely successful. Key lessons learned include: storage in basalt is highly successful; aquifer clogging is a concern, but can be managed; well design is important; stored water adsorbs radon quickly; ASR is a cost-effective peak water management technique.

### KEYWORDS

Aquifer storage and recovery, ASR, water storage, Columbia River Basalt Group, CRBG, radon

### INTRODUCTION

Oregon has a reputation for being wet, but many cities find it increasingly difficult to meet peak water supply demand during the dry summer months. The reasons: minimum stream flow requirements, difficulty in finding suitable above-ground reservoir sites in an increasingly urbanized environment, over-drafting of aquifers, state designation of critical groundwater areas, and an increasing influx of new residents and businesses. Each of these elements has added to increased pressures on peak demand for many key west-side municipalities in Oregon. The City of Beaverton (City), Oregon recognized these supply hurdles in the early 1990s and

opted to evaluate and test aquifer storage and recovery (ASR) to help offset peaking demands.

Beaverton's ASR program involves injecting treated drinking water from the Joint Water Commission (JWC) water treatment plant into a basalt aquifer that is closed to native groundwater withdrawals. The JWC receives its water from the Trask and Tualatin rivers via Henry Hagg Lake (also known as Scoggins Reservoir; see Figure 1) and Barney Reservoir (coastal impoundments operated by the U.S. Bureau of Reclamation).



**FIGURE 1. Scoggins Reservoir at 12 percent fill capacity in October 2001.**

Specifically, stored water is released based on Beaverton's daily needs during the summer peak season to maintain stream flows in the Trask and Tualatin rivers, where it is eventually pumped to the JWC treatment plant to produce finished drinking water before it is piped to Beaverton. The City has water rights ownership in the reservoirs. During drought years, the amount of water available from the reservoirs is uncertain. In addition, Beaverton's transmission line capacity from the JWC treatment plant is limited to 53,000 cubic meters per day ( $\text{m}^3/\text{day}$ ) (14 million gallons per day [mgd]). Upsizing the transmission line is costly. For these primary reasons, the City developed ASR as a way to help meet peak demands, and, in part, to help offset water supply issues.

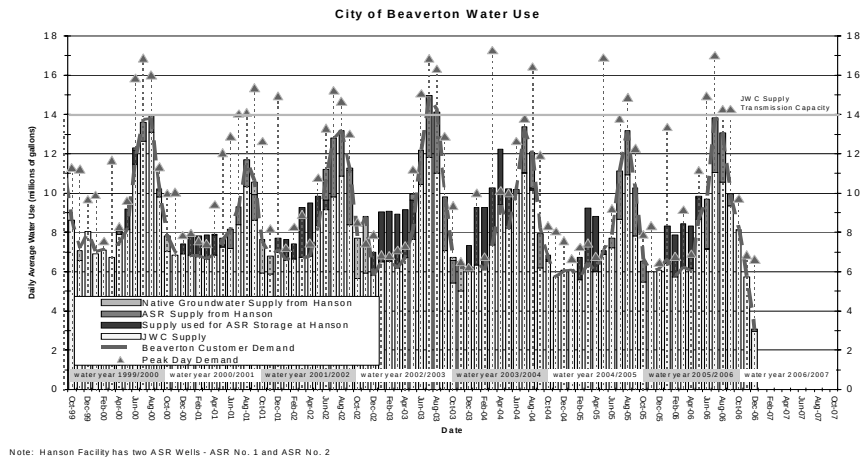
This paper will briefly describe the City's current ASR system, the hydrogeologic setting, and lessons learned during development and operation of the system during the past 8 years, and will present a brief summary of a cost/benefit analysis the City completed to evaluate its ASR system against more conventional options to meet peak water demand.

## **BEAVERTON'S ASR SYSTEM AT A GLANCE**

The City's current average daily demand is approximately  $34,000 \text{ m}^3/\text{day}$  (9 mgd), with peak water demands approaching nearly  $68,100 \text{ m}^3/\text{day}$  (18 mgd) (see Figure 2). Annually, drinking water supplied by the City is approaching nearly 13.2 million cubic meters per year (3.5 billion gallons per year). Demand for water continues as the City grows, and by 2020, the average daily demand is expected to be about  $45,400 \text{ m}^3/\text{day}$  (12 mgd), with peak water demands expected to reach  $90,850 \text{ m}^3/\text{day}$  (24 mgd), even after taking into account aggressive conservation and water management

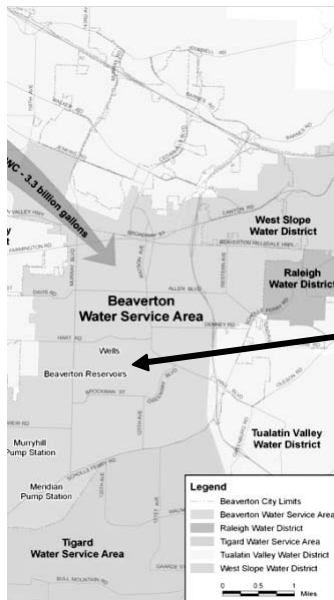
measures. As presented, the City's current transmission capacity from the JWC is limited to 53,000 m<sup>3</sup>/day (14 mgd).

The City's current ASR system consists of three operational ASR wells: ASR No. 1, ASR No. 2, and ASR No. 4, with a total peak delivery capacity of 22,700 m<sup>3</sup>/day (6 mgd). ASR No. 3, which is located in the southern part of the City, is expected to be online in 2008, and will add 2,850 m<sup>3</sup>/day (0.75 mgd) of peak capacity. The City is exploring additional sites to expand its ASR system, both in terms of peak delivery and storage volume. The three primary ASR wells are located in the central part of the City's distribution system (Sorrento Water Works facility) and were developed around an existing infrastructure system that includes source piping, storage reservoirs, and pump-to-waste capacity. Figure 2 illustrates how ASR is allowing the City to meet peak demands that are beyond its supply transmission capacity of 53,000 m<sup>3</sup>/day (14 mgd) and shows how ASR is "shaving off" peak summer-time demands.

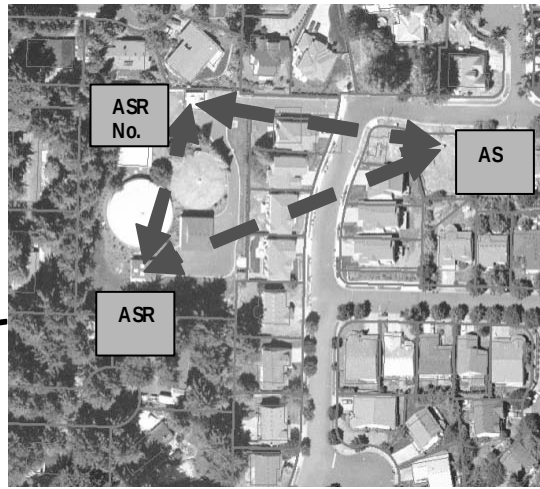


**FIGURE 2. City of Beaverton monthly water supply and demand, including monthly peak day demand with ASR 1 and ASR 2.**

Figure 3 shows the relative position of the City's ASR wells within the City's distribution system and Figure 4 shows the three primary ASR wells at the Sorrento Water Works facility in Beaverton. As shown in Figure 4, the ASR wells were developed in a highly urbanized environment centered on City land with existing infrastructure, initially built in the mid-1940s and consisting of one groundwater supply well, reservoirs, a pump station, and a water softening facility. Although the ASR wells are located in close proximity to each other, the hydrogeologic conditions at the site support the large injection and recovery rates, as discussed in the next section.



**FIGURE 3. Beaverton's service area.**



**FIGURE 4. ASR wells Sorrento Water Works Facility.**

Overall, the ASR wellfield at the Sorrento Water Works facility was developed in steps, beginning with development of ASR No. 1 in 1998 and bringing ASR No. 1 online in 1999 (see Figure 4). Feasibility and design studies were undertaken and included evaluation of groundwater users in the area, hydraulic design, hydrogeologic assessments, aquifer performance testing, and water quality compatibility studies, followed by pilot testing of the well(s). Initial work also included developing a numeric groundwater model of the site to support a wellhead protection plan for the site. Preliminary work at ASR No. 1 included rehabilitating and retrofitting the original well that was drilled in 1945, most likely using cable tool techniques. Because of well bore alignment issues and diameter constraints, it was not possible to install a well liner in ASR No. 1 and a downhole control valve even though static water level in ASR No. 1 is nearly 61 meters (200 feet) below ground surface (bgs). Additionally, because of size constraints within the existing building for ASR No. 1, the injection and recovery piping from the well occurs through the same pipe using a bi-directional clay-valve.

Based on successful pilot testing of ASR No. 1, installation of a second ASR well at the site was evaluated and determined to be feasible. ASR No. 2 was drilled using reverse circulation drilling techniques; lined with stainless steel and low carbon steel casing and well screens; and equipped with a more robust telemetry system, including a downhole control valve and a variable-frequency-drive turbine pump. ASR No. 2 was

brought online in 2001. The ASR No. 2 well house was acoustically upgraded after its completion because of noise generated by the variable-frequency-drive motor control, which was a nuisance to the neighbors. Even though ASR No. 2 penetrated the same primary interflow zones present in ASR No. 1, its yield is about 7,600 m<sup>3</sup>/day (2 mgd), whereas the yield for ASR No. 1 is about 3,800-m<sup>3</sup>/day (1-mgd). In addition, ASR No. 1 has to be routinely back-flushed to control head buildup, whereas ASR No. 2 requires much less maintenance. The difference in production between the ASR wells could in part be related to the difference used to drill the wells, how they were completed (lined versus unlined), the difference in diameters, and lastly, the downhole control valve installed in ASR 2 may also help in maintaining higher production rates at ASR 2.

Favorable ASR pilot testing results at ASR No. 1 and ASR No. 2 led to the development of a third ASR well (called ASR No. 4) at the Sorrento Water Works site. ASR No. 4 will be used to recover stored water during the 2007 ASR season. Overall, the plan is to use ASR No. 1 and ASR No. 2 to recharge the aquifer, and to use ASR No. 1, ASR No. 2, and ASR No. 4 to recover stored water. However, ASR No. 4 also is equipped to inject source water and was completed with a downhole control valve to limit the potential for air entrainment and to provide operational flexibility during recharge events. Lessons learned at ASR No. 1 and ASR No. 2 resulted in design changes to ASR No. 4 that included installing a submersible pump (compared to a line-shaft turbine pump) to reduce motor noise to nearby houses. ASR No. 4 also was upgraded to pump at about 11,400 m<sup>3</sup>/day (3 mgd) because aquifer testing showed that the well was capable of this yield for a short period of time. All of the ASR wells are connected to the City's supervisory control and data acquisition (SCADA) system. The ASR 3 site was drilled in the southern part of the City and will be developed in the future as a 2,850 m<sup>3</sup>/day (0.75 mgd) ASR well.

Since initial testing started in 1999, more than 7.6 million m<sup>3</sup> (2 billion gallons) of source water have been injected and recovered using the existing ASR wells. In addition, because the City has native groundwater rights, the ASR wells also have been used to pump native groundwater (more than 1.14 million m<sup>3</sup> [300 million gallons] to date) after the ASR storage account is depleted. In addition, the Beaverton area is closed to any new groundwater rights since the early 1974, when it was designated by the state as a Critical Groundwater Area, which is another key factor that makes ASR attractive in this area.

The City's ASR wells are operated under a limited license permit issued by the Oregon Water Resources Department (OWRD) as the lead agency, with support from the Oregon Department of Environmental Quality and the Oregon Department of Human Service Drinking Water Division. The City's ASR limited license is based on a 5-year testing period with the option to renew, which the City has done. After testing is complete, the City can apply to the OWRD for a full-scale operational permit. The key



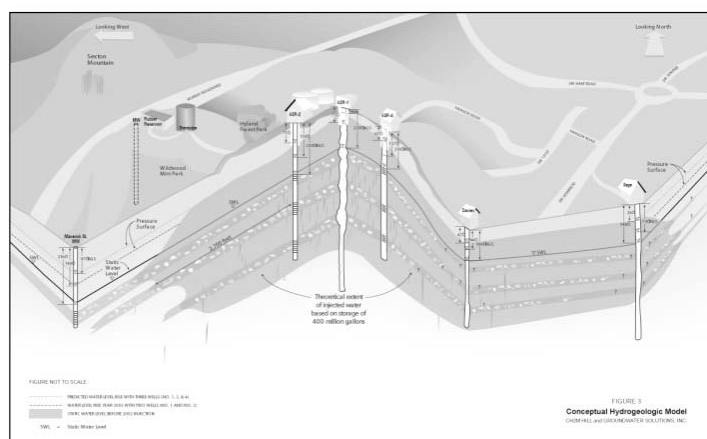
regulatory point that has helped spur the interest of ASR in Oregon is that state rules allow an exemption to the antidegradation regulations for ASR and permit chlorinated water to be injected and recovered. Specifically, ASR rules in Oregon allow disinfection byproducts (DBPs) in source water and recovered water up to the federal maximum contaminant level (MCL), and other constituents, as regulated by the Safe Drinking Water Act, are allowed at concentrations up to 50 percent of the federal MCLs. The DBP allowance in the ASR rules reduces the need to dechlorinate the source water, which helps reduce ASR capital, as well as, operation and maintenance costs. Per the ASR state-issued license, water recovered from the wells first must be debited against the ASR account. Overall, ASR has permanently transformed Beavertons utilization of the JWC supply system resulting in a dramatic flattening of previous peaks and valleys of water supply from the JWC.

## HYDROGEOLOGY

The ASR wells at the Sorrento facility are completed in basalts that are part of the Columbia River Basalt Group (CRBG). The CRBG ranges in age from 17 to 6 million years old as measured by K-Ar and 40Ar-39Ar ages (Long and Duncan, 1983; McKee et al., 1977; Swanson et al., 1979; Tolan et al., 1989). The flows of the CRBG were erupted from long linear vent systems in northeastern Oregon, southeastern Washington, and western Idaho. The CRBG has a volume of about 174,000 cubic kilometers (41,745 cubic miles) and covers about 164,000 square kilometers (63,321 square miles) (Tolan et al., 1989). These figures have been revised downward from previous estimates. Most of the CRBG was emplaced during a 1.5-million-year period between about 17 and 15.5 Ma and is represented by the Grande Ronde Basalt (Mangan et al., 1986; Reidel et al., 1989; Tolan, et al., 1989). Waning CRBG volcanism produced the volumetrically smaller Wanapum Basalt (about 15.5 to 14.5 Ma) and the Saddle Mountains Basalt (about 14 to 6 Ma) (Swanson et al., 1979a; Camp, 1981; Beeson et al., 1985). The CRBG section in the Beaverton area consists primarily of Grande Ronde Basalt, which is more than 305 meters (1,000 feet) thick. Individual CRBG flows typically range from 30.5 meters (100 feet) to more than 61 meters (200 feet) thick and are separated by interflow zones, which are highly permeable and the primary water-bearing zones in the basalt sections. The basalt colonnade and entablature (flow interiors) between interflow zones are much less permeable by orders of magnitude; however, in the Sorrento area, it is believed that secondary fracturing caused by uplifting may have increased the hydraulic connection between interflow zones.

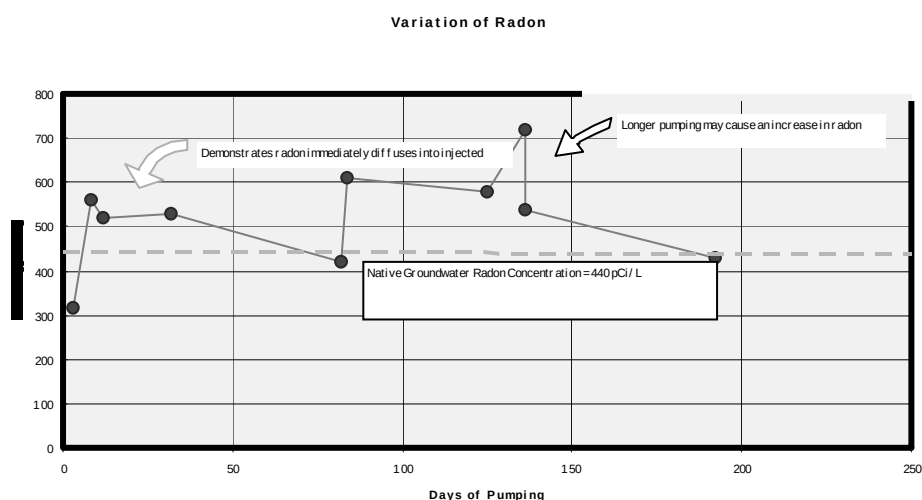
At the Sorrento facility (ASR No. 1, ASR No. 2, and ASR No. 4), three primary interflow zones are present that range in thickness from 3 to 12 meters (10 to 40 feet). The CRBG aquifer is semiconfined and has an average transmissivity of about 1000 m<sup>2</sup>/day (80,000 gallons per day per foot) based on late-time data and a storage coefficient of  $1 \times 10^{-4}$ . Faults are present in the area and either act as no-flow

boundaries or leaky boundaries. For example, the Mattel site, located approximately 2.5 kilometers (1.5 miles) east of the ASR site, has a network of monitoring wells installed within the CRBG aquifer to track groundwater contamination. The Mattel site is a state regulated clean up site with groundwater impacted by chlorinated solvents. No response has been observed in the Mattel monitoring wells since the start of ASR operation. In contrast, a measurable response from ASR activities at the Sorrento facility is observed at an historic seep more than 2.0 kilometers (1.3 miles) northwest of the site and at a future City ASR well (ASR No. 3) located over 4 kilometers (2.5 miles) south of the site. In addition, a state observation well has shown a response to ASR activities over 2.5 kilometers (1.5 miles) south of the site. The latter well is unique in that it is completed in basin sediments that abut the basalt section, which shows that a hydraulic response occurs across the basalt-basin sediment geologic boundary; something that has not been observed elsewhere in similar geologic settings, according to state hydrogeologists. Overall, it is believed that many of the faults in the CRBG beneath Beaverton are leaky, which is in part displayed in pump test data because the drawdown and recovery curves do not match an expected theoretical confined aquifer (Theis) response. Specifically, the response is less than that predicted by the Theis-type curve in early-time data (1,000 minutes or less) and the response is greater than the Theis-type curve in late-time aquifer test data. Figure 5 presents a generalized hydrogeologic conceptual cross section through the Sorrento facility, host to the City's three ASR wells. The source water bubble (i.e., treated drinking water that displaces native groundwater) may extend about 610 meters (2,000 feet) laterally from the site, based on a theoretical calculation assuming highly permeable interflow zones.



**FIGURE 5. Conceptual hydrogeologic model of Beaverton ASR system.**

Source water from the JWC and native groundwater are geochemically compatible based on mixing studies, as well as operational water quality testing data that has shown no adverse reactions are occurring. During recovery, low-level concentrations of iron, manganese, and DBPs were observed, but at concentrations well below regulatory screening levels (LJHB Partners, in press). Total haloacetic acids (HAAs) were reduced to non-detectable levels, while the total trihalomethanes (THMs) appear to have been reduced as a result of mixing patterns (LJHB Partners, in press). HAAs may have been reduced because of biodegradation at the Beaverton site (LJHB Partners, in press). Overall, water quality data during ASR operation shows a mixing pattern between source water and native groundwater. Radon, the only constituent to show a unique pattern, is present in native groundwater in the CRBG in the Beaverton area at concentrations ranging from 100 to 900 picocuries per liter (pCi/L). Radon typically is not present in basalts, since it is a decay product of Uranium 238, and thus the source may be from the underlying marine sediments. Work by Richard Glanzman (2006) supports that the marine sediments are the sources of the radon because the ratio of daughter products does not match what would be expected if the CRBG were the radon source. Regardless of the source, radon has been shown to adsorb quickly into source water and is the only constituent to be found at concentrations in early-time recovered water at levels similar to what is found in native groundwater (see Figure 6). Specifically, all other constituents monitored (cations and anions) do not approach native groundwater concentrations in recovered water until after 100 to 150 days of pumping, whereas radon is detected in recovered water at native groundwater concentrations within 2 weeks of pumping. These data show that radon diffuses quickly from the native groundwater to source water. Depending on future radon regulations, treatment for radon may be needed at the Beaverton ASR facility as well as at other ASR systems in the Portland area.



**FIGURE 6. Radon concentrations in recovered water, based on days of pumping.**

## ECONOMICS

*[Note: this section is presented in English units followed by SI units in brackets, where appropriate.]*

To evaluate ASR, the City completed a cost/benefit analysis comparing ASR to more conventional source water upgrades to meet demands. ASR projects, we believe, have to meet three components to be successful: (1) the hydrogeology and geochemistry have to be favorable, (2) infrastructure must be compatible with an ASR system, and (3) the project needs to be cost effective. As previously discussed in this paper, the hydrogeology and geochemistry are suitable for ASR, and the Sorrento site, where the City's ASR system was developed, has a robust infrastructure system that supports ASR development. As discussed in this section, the project also is cost effective.

**TABLE 1. Storage Cost Comparison – ASR vs. Conventional Storage Option.**

| Facility                           | Year of Cost | Cost (original) | 2006 Cost    | Storage Volume (MG) | Cost/MG     |
|------------------------------------|--------------|-----------------|--------------|---------------------|-------------|
| <b>ASR Wells</b>                   |              |                 |              |                     |             |
| ASR No. 1                          | 1997         | \$724,320       | \$924,051    | 100                 | \$9,241     |
| ASR No. 2                          | 2001         | \$1,228,881     | \$1,428,291  | 150                 | \$9,522     |
| ASR No. 3                          | 2001         | \$1,032,504     | \$1,199,259  | 50                  | \$23,985    |
| ASR No. 4                          | 2004-2006    | \$2,159,833     | \$2,273,511  | 200                 | \$11,368    |
| <b>Conventional Storage Option</b> |              |                 |              |                     |             |
| Sexton Mountain                    | 1995         | \$7,289,000     | \$10,596,284 | 15                  | \$706,419   |
| JWC No. 2 + Piping                 | 2005-2006    | \$25,210,247    | \$25,210,247 | 20                  | \$1,260,512 |

ASR = aquifer storage and recovery

JWC = Joint Water Commission

MG = million gallons

Table 1 compares the relative cost of ASR per cubic meters (million gallons) of storage to the cost of conventional storage (i.e., reservoirs) in the Beaverton area. From a storage perspective, ASR is cost effective. However, this type of comparison, in our opinion, is an oversimplification of the “true” cost comparison between ASR and conventional water supply options. To complete a more rigorous cost/benefit analysis, the following parameters were used to evaluate the cost to meet future demands using ASR or conventional water supply upgrades (e.g., increasing

transmission size capacity, reservoir upgrades, treatment capacity upgrades, and operation and maintenance costs):

- An increase of 500 MG (1.9 million m<sup>3</sup>) of storage capacity
- An increase of 5 mgd (about 19,000 m<sup>3</sup>/day) of supply
- Annualization of capital costs
- Life cycle of the systems
- Depreciation and return on investment
- Annual operation and maintenance costs

**TABLE 2. Overall Cost Comparison for 5 mgd (19,000 m<sup>3</sup>/day) of Flow Capacity and 500 Million Gallons (1.9 million m<sup>3</sup>) of Storage.**

| <b>ASR Supply</b>                 | <b>ASR Annualized Cost of 500 MG</b> | <b>Total ASR Unit Cost (\$/CCF)</b> |
|-----------------------------------|--------------------------------------|-------------------------------------|
| Total ASR Annualized Capital Cost | \$209,749                            | \$0.314                             |
| Total ASR O&M                     | \$417,303                            | \$0.624                             |
| <b>Total</b>                      | <b>\$627,051</b>                     | <b>\$0.94</b>                       |
| <b>JWC Conventional Supply</b>    | <b>Annualized Cost of 500 MG</b>     | <b>Total JWC Unit Cost (\$/CCF)</b> |
| Total JWC Annualized Capital Cost | \$565,516                            | \$0.846                             |
| Total JWC O&M                     | \$224,635                            | \$0.336                             |
| <b>Total</b>                      | <b>\$790,151</b>                     | <b>\$1.18</b>                       |

ASR = aquifer storage and recovery

ccf = 100 cubic feet

JWC = Joint Water Commission

MG = million gallons

O&M = operations and maintenance

Using capital costs to develop the City's existing ASR system, the cost to develop a new 5-mgd (19,000-m<sup>3</sup>/day) ASR source is about \$2,950,000 per mgd (\$800 per m<sup>3</sup>/day). In comparison, the capital cost to develop an additional 5 mgd (19,000 m<sup>3</sup>/day) of capacity for Beaverton, assuming a conventional source, is about \$6,000,000 per mgd (\$1,600 per m<sup>3</sup>/day). These options take into account raw water storage costs, water treatment capacity upgrades (JWC), finished water storage costs, and transmission capacity upgrades. Considering operation and maintenance (O&M) costs, which ASR is more costly when compared to a more conventional source, and

annualizing the capital costs, shows that ASR has a benefit/cost ratio of 1.26 over a conventional source (see Table 2). Specifically, the cost per ccf (100 cubic feet of water or 748 gallons), which is the typical unit of measurement used by municipalities, demonstrates that ASR is \$0.94 per ccf compared to \$1.18 per ccf for a conventional supply, assuming 5 mgd (19,000 m<sup>3</sup>/day) of flow and 500 MG (1.9 million m<sup>3</sup>) of storage capacity. Overall, ASR is more cost effective when compared to conventional water supply alternatives.

## CONCLUSIONS

Based on more than 8 years of ASR testing and operational data from the Beaverton ASR project, the following conclusions are presented:

- The CRBG aquifer in the Beaverton area is a good host for ASR. Specifically, the basalt interflow zones are very permeable and many of the faults in the area act as leaky boundaries rather than no-flow boundaries, which facilitates storing substantial quantities of source water.
- It has been shown that proper design of ASR wells in the CRBG is important. Larger diameters and lining the borehole helps increase well efficiencies. Downhole control valves are valuable in reducing the likelihood of introducing entrained air into the interflow zones, given the depth to the static water level, and provide operational flexibility during ASR injection.
- To track the dynamic response of the basalt aquifer caused by ASR activities, a network of monitoring wells is critical. This information has been helpful in assessing ASR expansion and evaluating the potential for development of seeps as a result of ASR activities.
- Overall, the increase in aquifer potentiometric head associated with ASR injection has not resulted in a loss of water from the natural system. In fact, the regional aquifer has shown a positive water level response because: 1) small amounts of injected water often are left in the aquifer annually due to ASR, 2) precipitation trends have been normal for the past several years, 3) ASR has reduced the City's dependency on native groundwater, and 4) many domestic and irrigation wells have been abandoned due to urbanization of the area.
- The pressure response in the CRBG resulting from ASR activities propagates radially for a distance of several kilometers (miles) from the ASR wells. If a preferential pathway exists, seeps can be generated or reactivated.
- Clogging of wells because of particulate matter from source water is a concern, but has been managed during the Beaverton ASR project by routinely backflushing the ASR well. Clogging resulting from mineral precipitation does not appear to be occurring at Beaverton because source water and groundwater have been shown to be compatible.

- Radon, which typically is not present in basalt aquifers, is present in the groundwater in the Beaverton area. The source of radon most likely is the underlying marine sediments. Analysis of recovered water indicates that radon diffuses very rapidly to stored water.
- Overall, the City of Beaverton has not received any customer complaints regarding water aesthetics since the start of ASR. In the past, the City received numerous complaints whenever native groundwater was used.
- Finally, in the Beaverton case, ASR is a cost-effective alternative to meet peak demands when compared to conventional water supply alternatives, such as increasing aboveground storage and/or increasing transmission capacity.

## ACKNOWLEDGEMENTS

The authors would like to thank the following individuals for their contributions to the success of this project. Jason Melady, GSI hydrogeologist, has been a key technical support on the project, as well as a great assistant project manager. Jeff Barry, also with GSI, has provided very valuable senior support and was instrumental in helping craft the state ASR rules. CH2M Hill has been the lead design and engineering firm since the start of the project. Rick Weaver, Tim Tilander, and Beth Doan, all with the City's operational department, have been the internal champions and the day-to-day operators of the ASR project. Without their support, the Beaverton project would not have been successful. Lastly, we would like to acknowledge Tom Ramish, former City Engineer, and the Honorable Mayor Rob Drake; both provided leadership and the financial support to make Beaverton's ASR project one of the top ASR projects in the Pacific Northwest.

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## ESTIMATING THE TARGET STORAGE VOLUME FOR ASR WELLS

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### ABSTRACT

The Target Storage Volume (TSV) for an ASR well is the sum of the stored water volume that is required for recovery, plus the buffer zone volume that separates the stored water volume from the surrounding ambient groundwater. Forming and maintaining a buffer zone facilitates recovery of water with suitable quality, whether the ambient groundwater quality is fresh, brackish, saline or contains other unwanted constituents. Estimating the TSV for a new ASR well can be based upon operating experience at existing wellfields. ASR performance at five Florida wellfields is evaluated to estimate the TSV required to achieve suitable recovered water quality. This experience may be applied at other sites. For the sites investigated, the buffer zone volume is approximately half of the TSV.

### KEYWORDS

Aquifer storage recovery, artificial recharge, groundwater storage, managed aquifer recharge, target storage volume, wells

### INTRODUCTION:

For almost all ASR wells, a difference exists between recharge water quality and ambient groundwater quality in the storage aquifer. Where it is fresh, the storage aquifer often contains water with concentrations of iron, manganese, fluoride, other metals, hydrogen sulfide, nitrate, radium or other constituents that are not wanted in the recovered water. Alternatively the storage aquifer may be brackish, as at approximately a third of all ASR sites, or even saline, with total dissolved solids concentrations of up to 20,000 mg/l (Englewood, Florida) or higher. Since 1968 in the United States, ASR wells have been storing drinking water, reclaimed water, treated stormwater and groundwater in confined, semi-confined and unconfined storage aquifers and have been recovering water of suitable quality and with acceptable recovery efficiency. A recent inventory indicated 77 operating ASR wellfields throughout the United States (Pyne et al, 2007) with recovery capacities up to 157 million gallons per day (594 MI/D).

A few ASR wellfields are storing drinking water in freshwater aquifers with no significant differences in water quality and no significant geochemical issues. For these few sites a primary incentive for ASR storage is to restore groundwater levels in depleted aquifers, utilizing the storage volumes created through decades of groundwater production at rates exceeding natural recharge. At some ASR sites an important difference in water quality may be the oxidation reduction potential (ORP), which is often high in the recharge water and low in the storage zone. This difference may cause subsurface geochemical reactions close to the ASR well that release unwanted constituents such as arsenic into the groundwater, and may stimulate subsurface microbial activity.

Successfully storing and recovering fresh water in such aquifers requires formation of a buffer zone separating the ambient groundwater from the fresh stored water needed for recovery. The buffer zone is analogous to the walls of a storage tank, a dam, or the embankment around a surface reservoir. The buffer zone is formed by recharging a given volume of the stored water and maintaining it, displacing the surrounding ambient groundwater. As long as the buffer zone is maintained, recovered water quality remains acceptable. If recovery continues, water quality deteriorates.

The Target Storage Volume (TSV) is the sum of the stored water volume that is desired for recovery, plus the buffer zone volume. The TSV is scale-dependent; for any given storage aquifer as the stored water volume increases, the buffer zone volume also increases. Achieving the TSV as early as possible during development of a new ASR well enables achieving full recovery efficiency for that well, usually close to 100%. If the TSV has not been achieved, less than full recovery efficiency may be expected.

The TSV concept was developed based upon operating experience at early ASR wellfields in brackish aquifers, for which recovery efficiency usually started relatively low in early test cycles and then steadily climbed with successive cycles until 100% recovery efficiency was achieved. For each cycle approximately the same volume of water would be recharged, and recovery would continue until a preset value for TDS or chloride was reached. Eventually it was realized that this time-consuming approach could be shortened by forming a buffer zone at one time instead of over a period of several cycles. The savings in time and cost was substantial. Prediction of the TSV for conservative constituents, and achieving a better understanding of the factors that influence it, then became a new ASR goal. More recently the TSV concept has been extended to achieving acceptable recovery efficiency in aquifers where non-conservative constituents are of concern. Specifically this has included arsenic.

In some situations potential ultimate recovery efficiency is less than 100%, even if the TSV has been formed. This may occur in unconfined aquifer storage zones where lateral velocity carries water away from the ASR well a sufficient distance that it cannot be recovered. Reduced ultimate recovery efficiency may also occur in situations where the duration of storage in a confined or semi-confined aquifer is

sufficiently long that lateral movement of the “bubble” becomes a significant operational factor. It may also occur in more saline aquifers, exceeding about 5,000 mg/l total dissolved solids (TDS), the concentration above which density stratification tends to cause a loss in recovery efficiency.

## METHODS

The use of water quality simulation models to estimate the TSV is invariably constrained by the lack of field data to support estimates of dispersivity for the storage aquifer. Furthermore, it is probable that dispersivity is variable, depending upon the volume of water being stored and recovered. As a result it is necessary to try to bracket a reasonable range of dispersivity estimates, such as within an order of magnitude. The resulting projections of water quality during ASR recovery for conservative constituents such as chloride or TDS are of interest, however they tend to vary quite widely, rendering such an approach of limited predictive value.

Operating results from the first ASR well in a proposed wellfield are typically utilized to refine such design criteria as well spacing, flow rates, and storage volumes. This “test well” approach is fine so long as the test well is a full size ASR well, operating with expected rates and volumes for storage and recovery. Since results are scale-dependent, any smaller size well, or lesser rates and volumes, will tend to yield incorrect results.

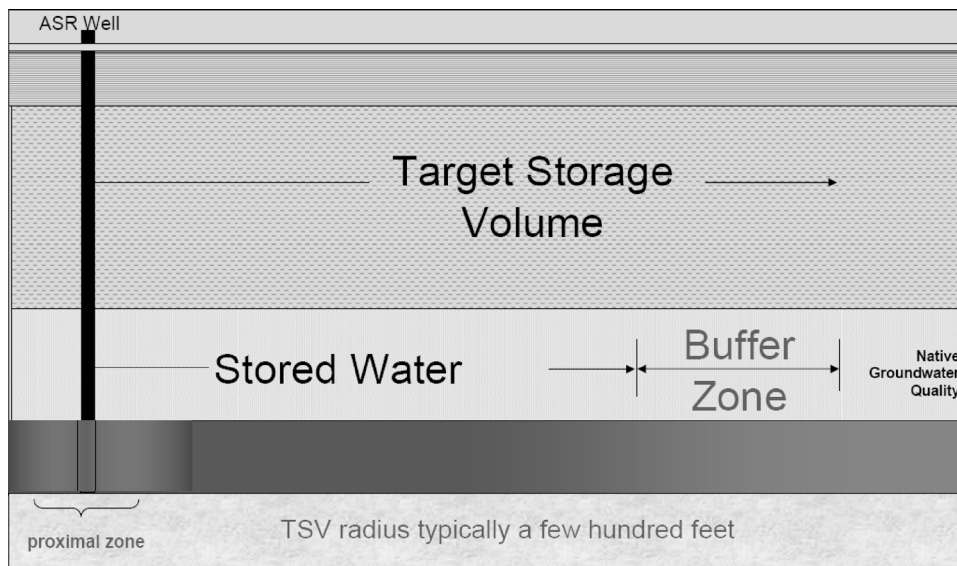
A different approach is to evaluate actual performance from several existing ASR wellfields, providing a basis of experience with which to plan and operate ASR wellfields, not only for conservative constituents but also for some non-conservative constituents such as arsenic. Data from five ASR wellfields in brackish aquifers have been evaluated to provide a basis for conclusions regarding the TSV and the buffer zone volume for ASR wells. Development of new ASR wells can then be planned in such a way as to quickly and cost-effectively achieve ultimate recovery efficiency and, in some cases, to overcome more complex geochemical issues. The five ASR wellfields are in brackish aquifers, therefore readily providing data on recovery efficiency. This then serves as a useful point of reference for application to other areas where storage aquifers are fresh or for which the constituents of concern are not conservative.

- City of Bradenton, Florida
- City of Tampa, Florida
- Peace River/Manasota Regional Water Supply Authority, Florida
- Miami-Dade Water and Sewer Authority, Florida
- City of Delray Beach, Florida

## RESULTS AND DISCUSSION:

### ASR Conceptual Model

Figure 1 shows a conceptual model for ASR storage in a semi-confined, artesian aquifer. During recharge, water passes initially through the proximal zone immediately adjacent to the ASR well, probably within a radius of a few meters. Substantial physical, geochemical and microbial gradients occur in this interval, causing changes in water quality. The recharge water then moves into the stored water interval, within which water quality is relatively stable after a few operating cycles. The stored water interval may extend to a radius of typically 100 to 200 m. At the outer edge of the stored water is the buffer zone, separating the stored water from the surrounding ambient groundwater. Width of the buffer zone is variable, depending upon aquifer hydrogeologic and water quality properties and other factors, however a typical width would be about 50 to 100 m. During recovery the stored water moves back through the proximal zone to the ASR well. Conservative constituents such as chloride will remain similar to that of the recharge water until near the end of the stored water recovery, when it will start to increase as the buffer zone is reached and will climb steadily, approaching ambient groundwater concentrations after perhaps 200% recovery or more. However for non-conservative constituents such as arsenic, a more complex set of subsurface geochemical and microbial reactions occurs, many of which are not well understood. For arsenic, if over-recovery occurs and the buffer zone is pulled back through the proximal zone, desorption of arsenic from the aquifer matrix or from ferric hydroxide floc is postulated to occur. (ASR Systems, 2007).



**FIGURE 1. ASR Conceptual Model.**

## **Factors Affecting the TSV**

The volume of the TSV depends upon several considerations. Hydrogeologic factors associated with the storage zone include aquifer thickness; hydraulic characteristics such as transmissivity, vertical confinement, effective porosity and potential for vertical flow between multiple producing intervals; lithology and groundwater quality. Hydrologic factors include recovered water storage volume needs which, in turn, are based upon variability in supply, demand and quality of the source water. Hydrologic factors also include duration of storage and the associated potential for loss of stored water (Pyne, 2005).

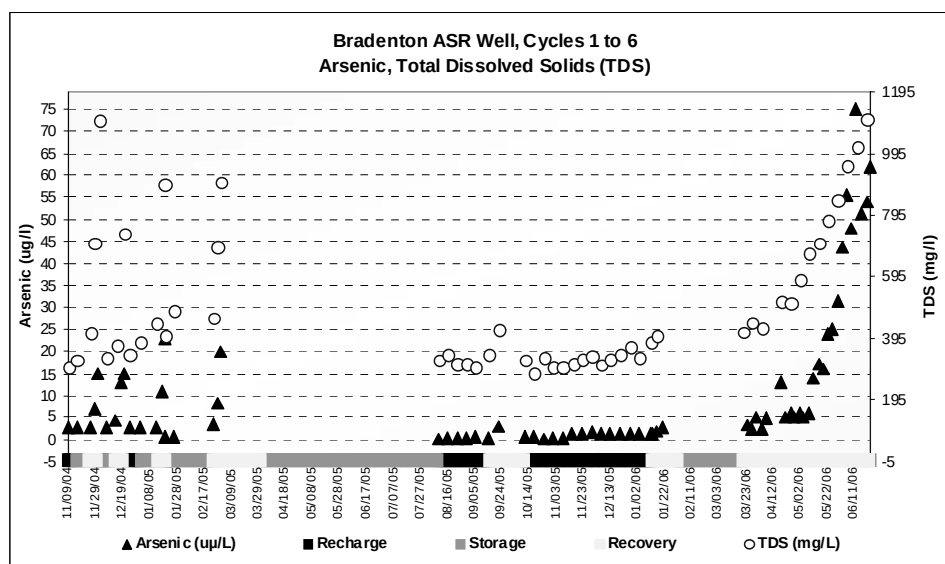
The TSV is measured in days, or more specifically in terms of million gallons per MGD of recovery capacity, or cubic meters per CMD. Typical TSV values range between 50 and 350 days. The low end of the range is associated with storage of water from a reliable treated drinking water source to meet seasonal peak demands, for a few days or weeks, with water being stored in a relatively thin, sandstone aquifer that is quite brackish but tightly confined. The high end of this range is associated with meeting seasonal peak demands for several months from a highly unreliable surface water source that is routinely unavailable for several months each year, with water being stored in a semi-confined karst limestone aquifer containing slightly brackish groundwater. Thin aquifers are preferable to thick aquifers if a substantial difference exists between recharge and ambient groundwater quality. Similarly mixing tends to be minimized when the effective porosity of the storage zone is associated with the primary porosity of a hard rock aquifer such as sandstone or limestone as opposed to secondary porosity associated with solution features or fractures. Tightly confined aquifers are also preferable when water quality differences are substantial since they inhibit contributions of poor quality water from adjacent aquifers during recovery periods. For any ASR well, larger volumes needed for recovery require larger buffer zones.

Over-estimating the TSV tends to have little penalty unless the cost of the recharge water utilized to form the buffer zone is very high. Under-estimating the TSV tends to lead to recovered water quality problems since the buffer zone is too close to the ASR well. Where no significant difference exists between recharge water quality and ambient groundwater quality, and no potential exists for geochemical reactions, then no buffer zone is required. The cumulative stored water volume then should equal the cumulative recovered water volume unless an objective is to raise local and regional groundwater levels.

## **City of Bradenton**

A single ASR well, storage zone monitor well and a second monitor well in an overlying aquifer were constructed and tested during 2003 to 2006. The storage aquifer is a semi-confined, artesian, karst limestone aquifer with an ambient

groundwater TDS of 1,100 mg/l. The storage aquifer extends from a depth of 415 to 505 feet (126 to 154 m) and the wells are open hole. Aquifer hydraulic characteristics are as follows: transmissivity 2,100 ft<sup>2</sup>/day (195 m<sup>2</sup>/day); storativity  $1.1 \times 10^{-4}$ ; leakance  $5.1 \times 10^{-3}$ /day. The well produces water at rates up to 2.6 MGD (9.8 MI/D) and recharges water at variable rates between 0.3 and 1.5 MGD (1.1 and 5.7 MI/D). Six test cycles were conducted, the first four of which included 10 MG (38 MCM) recharged, stored for a week, and then recovered. During the fifth cycle 50 MG (189 MCM) were recharged and 10 MG (38 MCM) recovered. During the sixth cycle the TSV of 150 MG (567 MCM) was finally achieved. Recovery then commenced, initially planned for 60 MG (227 MCM) but was subsequently reduced to 10 MG (38 MCM). Upon completion of Cycle 6 recovery, and after a two month waiting period, pumping continued until 120 MG (454 MCM) had been recovered. Figure 2 shows the variation in TDS and arsenic concentration in the recovered water during this period.



**FIGURE 2. Bradenton Cycle Test Data, November 2004 to June 2006.**

During the first four cycles, with no formation of a buffer zone, arsenic concentrations were below the drinking water standard of 10 µg/l for the first half of recovery and then climbed to between 15 and 23 µg/l at the end of recovery. During Cycle 5, with formation of one-third of the TSV, arsenic concentration during recovery of 10 MG was below 10 µg/l. During Cycle 6 recovery arsenic concentrations were acceptable during recovery of the planned stored water volume. However with continued recovery, arsenic concentrations increased, reaching as high as 75 µg/l.

Had the buffer zone not been recovered, elevated arsenic concentrations would not have occurred at this site, either in the recovered water or in the aquifer.

The storage zone monitor well at this site is at a radius of 231 feet (70 m). No elevated arsenic concentrations were ever observed at this monitor well during ASR recharge, even though the stored water volume would theoretically have extended about 600 ft (183 m) from the ASR well. However during extended ASR recovery after completion of Cycle 6 elevated arsenic concentrations were noted at the monitor well, reaching as high as 20 µg/l. A geochemical model is postulated in which arsenic is mobilized during recharge, due to oxidation of pyrite contained in the limestone matrix. The mobilized arsenic adsorbs on ferric hydroxide floc and moves through the pores and conduits in the karst limestone until it is physically trapped, but does not move more than about 200 feet (61 m) from the ASR well. During recovery of stored fresh water, the adsorbed arsenic remains in place. However with continued recovery of the brackish water in the buffer zone, constituents in the recovered water preferentially displace the adsorbed arsenic from the ferric hydroxide floc, releasing it into the recovered water. This is similar to brine regeneration of filters designed to remove iron and manganese from water. Bicarbonate has been identified (Appelo et al, 2002) as a possible constituent that may cause such a geochemical reaction since it preferentially sorbs onto ferric hydroxide, displacing arsenic. Bicarbonate in the recharge water at Bradenton ranges between 48 and 73 mg/l while in the ambient groundwater it is estimated at 110 mg/l.

For this site, the TSV is about 60 days, capable of supplying 75 MG (284 MI) to meet peak or emergency water needs. The buffer zone comprises about half of the TSV, whether for TDS or for arsenic. Forming and maintaining this buffer zone would have achieved drinking water standards in the recovered water for arsenic. At a typical recharge rate of 1.5 MGD (5.7 MI/D), this would have required less than two months of continuous recharge to form the buffer zone. If formed during offpeak periods, the marginal cost of this water would be due only to power, chemicals and residuals disposal, cost of which is relatively small. This would be a one-time investment immediately following well construction and prior to full operation of the well.

### **Miami-Dade Water and Sewer Authority**

Three ASR wells were constructed and tested during 1996 to 2000, each storing seasonally available fresh water from an overlying surficial aquifer. The storage aquifer is a semi-confined, karst limestone artesian aquifer, generally from a depth of about 850 to 1,250 ft. (259 to 381 m). The wells are constructed open hole and are in a straight line, 1,000 ft (305 m) apart. Aquifer hydraulic characteristics are as follows: transmissivity 15,775 ft<sup>2</sup>/day (1,465 m<sup>2</sup>/day); storativity  $3.6 \times 10^{-4}$ ; leakance  $4.1 \times 10^{-4}$ /day Ambient groundwater is brackish with a TDS of 4,000 to 6,000 mg/l. Design production rate from each well is 5 MGD (19 MI/D). During the cycle testing program

at two of the three wells, results indicated the need for a TSV of 200 days in order to sustain recovery operations for periods up to 90 days.

### **Peace River/Manasota Regional Water Supply Authority**

With 21 ASR wells, this is the largest ASR wellfield in the eastern United States. Recovery capacity is 18 MGD (68 MI/D). The wellfield began operations in 1985 and has been expanded in several phases. Nine of the wells are in Wellfield One surrounding the 24 MGD water treatment plant. Recovery capacity of these wells is about 6 MGD (23 MI/D). Twelve wells are in Wellfield Two, located one mile (1.5 km) away with a recovery capacity of about 12 MGD (45 MI/D). Most of the ASR wells are in a production interval at the top of the Suwannee aquifer at depths of about 600 to 700 feet (183 to 213 m) although the aquifer thickness is about 320 feet (98 m). All wells are open hole construction. The storage aquifer hydraulic characteristics at the two wellfields are as follows: transmissivity 5,000 to 6,000 ft<sup>2</sup>/day (464 to 557 m<sup>2</sup>/day); storativity  $1 \times 10^{-4}$  to  $3.1 \times 10^{-4}$ ; leakance  $5.1 \times 10^{-3}$ /day. Almost all of the leakance is associated with the underlying confining layer. The storage aquifer is slightly brackish with an ambient TDS of about 700 to 1,070 mg/l, however it is underlain by a very productive brackish aquifer at a depth of about 1,200 feet with a TDS concentration of about 2,800 mg/l. Typical well yields in the ASR storage aquifer are 1 MGD (1.5 MI/D). Regional water demands supplied by the Authority have climbed steadily since 1985, reaching an average of 20 MGD (76 MI/D) during 2007.

Based upon initial operations at Wellfield One between 1985 and 1997, the TSV was estimated at 350 days, primarily because the Peace River at this location can experience periods as long as about six months when no diversions are available from the river due to regulatory low flow restrictions. Typical no-flow periods each year are two to three months' duration. ASR storage volume of treated drinking water in Wellfield One increased steadily to a maximum of about 2.4 billion gallons (BG) (9.1 million cubic meters, MCM) between 1987 and 1999. It was drawn down to 0.5 BG (1.9 MCM) by 2002 and then rebuilt to 2.3 BG (8.7 MCM) by 2005. By comparison, an existing offstream surface storage reservoir at this site stores about 0.6 BG (2.3 MCM) of untreated surface water.

For Wellfield Two, construction of which was completed in 2002, operations to date have not developed any significant cumulative storage volume, partly due to regulatory requirements to recover almost all of the stored water each year. In addition, increasing water demand has steadily reduced the opportunity for water storage and increased the volume of water recovered each year. Combined cumulative storage volume for all 12 wells reached 1.5 BG (5.7 MCM) in 2005, following which it was drawn down to 0.2 BG (0.8 MCM) due to an extended dry period in early 2006.



For Wellfield One, the TSV is about 2.1 BG (8.0 MCM), approximately equaling the volume stored in 2005. For Wellfield Two, the TSV is about 4.2 BG (15.9 MCM). If a combined TSV of 6.3 BG (23.9 MCM) were in place, recovered water quality during an extended six-month drought should achieve ASR goals for this facility. In fact, all stored and buffer zone water in Wellfield Two has been fully recovered in 2007 while in Wellfield One it has been drawn down substantially. As a result, recovered water quality is brackish, similar to ambient groundwater quality in the storage zone. The wellfield is currently being operated essentially as a brackish raw water source, with water being blended with surface water from the reservoir. Construction of a new 6 BG (23,000 CM) surface reservoir and 24 MGD (91 MI/D) water treatment plant expansion has begun. Within about 3 years it should be possible to start rebuilding the ASR wellfield TSV and expanding the wellfield, reflecting the low cost of water storage with ASR compared to water storage in surface reservoirs, and also reflecting the adverse environmental impact of surface reservoir construction.

The estimated TSV of 350 days for the Peace River wellfields was based upon considerations of conservative constituents, particularly TDS and sulfates. These are both secondary (aesthetic) drinking water standards. Arsenic in the recovered water from the ASR wells in Wellfield Two has been a regulatory issue during the past few years. Arsenic is a primary (health-related) drinking water standard. It is pertinent to consider arsenic concentrations at Well S-16 in Wellfield Two, which has yielded the highest arsenic concentration in this wellfield. Sixty to 99 MG (227 to 375 MI) was stored in each of four cycles and was then recovered. During the four ASR cycles the cumulative storage volume for this well ranged from zero to 28% of the TSV. No significant buffer zone was formed. Nevertheless during the first three cycles the cumulative storage volume at the end of recovery increased from -8 to 14 to 39 MG while the peak arsenic concentration declined from 138 to 52 to 31  $\mu\text{g/l}$ . During Cycle 4 the cumulative storage volume at the end of recovery declined to 2 MG (7.6 MI) and the arsenic concentration increased to 260 $\mu\text{g/l}$ . The small buffer zone that had been formed during the first three cycles was insufficient to sustain extended recovery with acceptable arsenic concentrations during the fourth cycle. In contrast, Well S-17 yielded among the lowest peak arsenic concentrations in Wellfield Two, declining from 60 to 16  $\mu\text{g/l}$  in four cycles as the cumulative storage volume increased from zero to 77 MG (291 MI) at the end of recovery for the fourth cycle. During the four cycles the cumulative storage volume ranged from 21 to 53% of the TSV. By comparison, during the same period peak arsenic concentrations for the original wells in Wellfield One ranged from 2 to 10  $\mu\text{g/l}$ , plus a single value of 13  $\mu\text{g/l}$  at one well that was sandwiched between two values of 1  $\mu\text{g/l}$ . As described previously, the TSV was achieved in Wellfield One. It seems clear from analysis of the Peace River ASR data that forming and maintaining the TSV for conservative water quality constituents at this site is effective at also achieving compliance with arsenic standards. If the TSV is not formed and maintained and arsenic is present in minerals such as pyrite in the storage aquifer, it is probable that arsenic concentrations may exceed standards in the recovered water.

### City of Delray Beach, Florida

A single ASR well was constructed in 2000, storing drinking water in a semi-confined, karst limestone artesian aquifer with an open borehole from 1,016 to 1,200 feet (310 – 366 m). Due to the absence of a storage zone monitor well, aquifer hydraulic testing has not been conducted. Storage zone TDS is 4,200 mg/l. Recharge and recovery flow rates are about 2.5 and 2.8 MGD (9.5 and 10.6 MI/D), respectively. The desired recovery volume is about 50 MG (189 MI). A TSV of 250 MG was estimated based upon prior experience at other south Florida ASR wells and was initially formed in order to achieve a recovery TDS concentration not exceeding 550 mg/l. Based upon encouraging initial results the TDS goal was reduced to 450 mg/l and the TSV was increased to 320 MG (1,211 MI). Full recovery efficiency was achieved in subsequent cycles. The TSV at this site is 114 days. Arsenic has not been an operational issue at this site so TSV is based upon conservative constituents such as chloride and TDS.

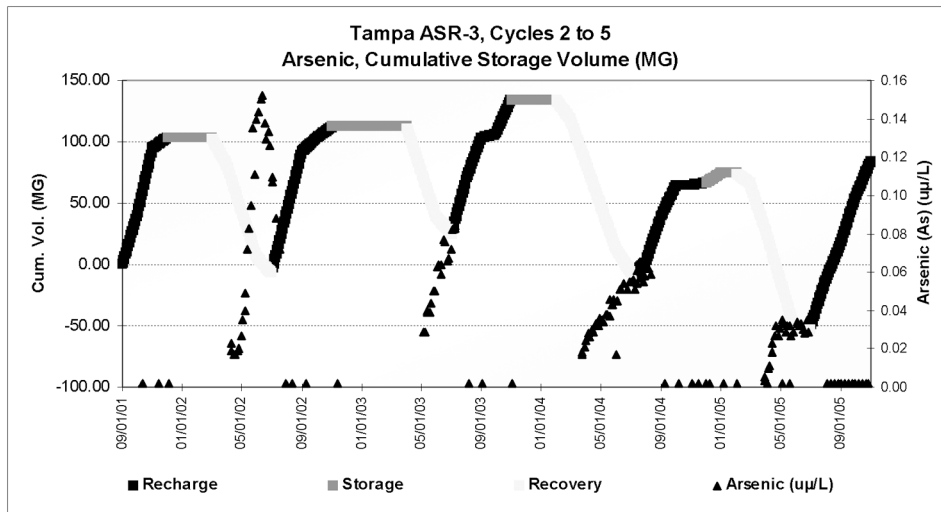
### City of Tampa, Florida

Eight ASR wells, five storage zone monitor wells and several other monitor wells were constructed and tested between 1996 and 2002, storing treated drinking water in a semi-confined, karst limestone artesian aquifer underlying a City park within a residential area of Tampa. The aquifer is fresh, however ambient groundwater quality has a high odor content (70 T.O.N.) due to the presence of hydrogen sulfide. Wells are open hole between about 300 and 400 feet (91 – 122 m). Aquifer hydraulic characteristics are as follows: transmissivity 20,300 ft<sup>2</sup>/day (1,886 m<sup>2</sup>/d); storativity  $9.8 \times 10^{-5}$ ; leakance  $5.2 \times 10^{-4}$ /day. TDS concentrations in the aquifer storage zone range from 210 to 500 mg/l. Wellfield capacity is 10 MGD (38 MI/D).

No attempt has been made to form and maintain a buffer zone at this site. Each year since 2002 approximately one billion gallons (3.8 MCM) has been stored during wet months and then recovered during dry months. However some wells have achieved small buffer zones while others have recovered more water than has been stored. Wells ASR-2 and ASR-8 maintained generally positive but small cumulative storage volumes, ranging from +191 MG to -23 MG (+723 to -87 MI). Arsenic concentrations in these wells declined to peak concentrations of 11 and 22 µg/l, respectively, during the fifth cycle. Wells ASR-1 and ASR-5 maintained generally negative cumulative storage volumes, ranging from +74 MG to -281 MG (+280 MI to -1,064 MI), respectively, at the end of Cycle 5. Peak arsenic concentrations during Cycle 5 were 20 and 48 µg/l, respectively. However ASR-1 was the first ASR well in the wellfield and has been in operation since 1996. Consequently to the extent that cyclic operation of the ASR well contributes toward purging arsenic from the aquifer then arsenic concentrations would tend to be reduced for this well compared to other wells completed more recently. Figure 3 shows the variation in arsenic concentrations at well ASR-3 during the recovery periods for Cycles 2 through 5. Peak arsenic concentrations declined steadily with successive cycles. To a greater extent than for

the other wells, the cumulative storage volume at the beginning of each recovery period remained relatively steady from one cycle to the next. Average arsenic concentration declined from 80 µg/l in Cycle 2 to 20 µg/l in Cycle 5, or about 27 µg/l in each cycle. It is probable that a buffer zone is slowly being formed at this well due to mixing of recharge water in each cycle with residual water from the previous cycle, due to dispersion within the aquifer.

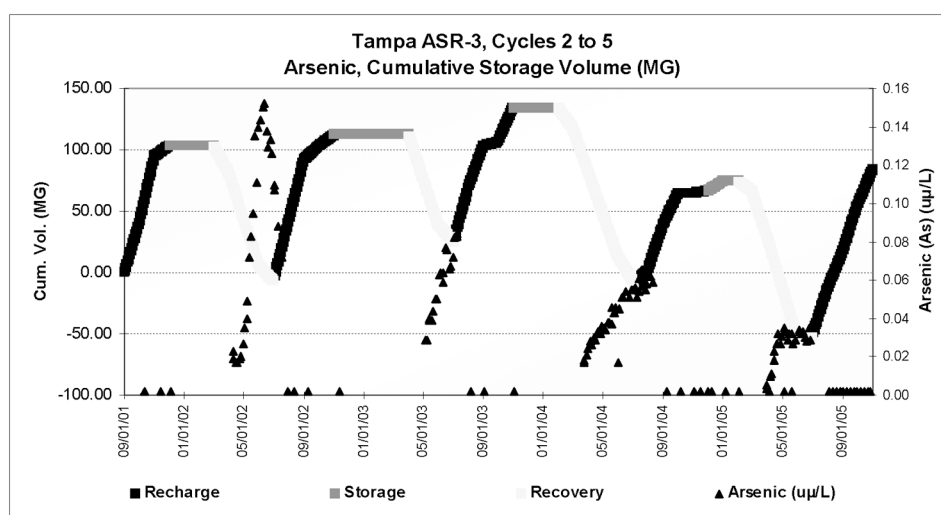
This process could be accelerated by initial formation of the buffer zone, thereby meeting the 10 µg/l water quality standard for arsenic in probably one or two years instead of more than five years. The TSV has been estimated at about 135 MG (511 MI) per well, or a total of about 1.1 BG (4.2 MCM). This is equivalent to a TSV of 110 days. However the wholesale purchase cost for the water required for buffer zone formation prior to initial operations would have been quite expensive for this site relative to achieving the same objective over a period of several years with no supplemental water purchases. At most sites the TSV is formed with relatively low cost water available during offpeak months, however for the City of Tampa this approach is not an option.



### 3. Tampa Well ASR-3, Arsenic Concentrations During ASR Recovery, Cycles 2 to 5.

Formation and maintenance of a buffer zone is essential to the successful operation of almost all ASR wells, separating higher quality recharge water from lower quality ambient groundwater. Based upon experience from the ASR wellfields considered for this analysis the TSV ranges typically from 50 to 350 days of recovery at the design recovery rate. Where the constituents constituting the difference between stored and ambient groundwater quality are conservative, such as chloride and TDS, the

buffer zone volume as a percentage of the TSV may vary, depending upon several hydrogeologic, water quality and operational factors. Maintaining a buffer zone volume comprising 50% of the TSV has been found sufficient at several Florida ASR wellfields to ensure that recovered water quality is satisfactory and that arsenic is not recovered from an ASR well. Forming the TSV is a one-time procedure, best conducted immediately after well construction and prior to ASR operations. If the buffer zone is subsequently recovered from the ASR well, water quality may be expected to decline, possibly requiring recruitment to maintain recovered water quality standards. Operational and regulatory measures to form and maintain the buffer zone are therefore important to successful ASR operations.



**FIGURE 3. Tampa Well ASR-3, Arsenic Concentrations During ASR Recovery, Cycles 2 to 5.**

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## WATER QUALITY ASPECTS OF ARTIFICIAL GROUNDWATER RECHARGE AT WINDHOEK

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### ABSTRACT

Windhoek is situated in the central highlands of Namibia in a semiarid region subject to large fluctuations in the annual rainfall. Local surface water resources are very limited while extended droughts aggravate the situation. This lead to direct recycling of reclaimed wastewater and implementation of water demand management. Groundwater is the only reliable local source but yields on average only 9% of the present demand of about 21 Mm<sup>3</sup>/a, and is subjected to serious over-abstraction during drought periods. The groundwater level decline obligated consideration of water banking and over the past decade pilot tests proved the hydraulic feasibility of injecting surface water into the aquifer and full scale injection started in 2006. The success of managed aquifer recharge also depends on a thorough understanding of the ambient water quality in the aquifer together with potential water-rock interactions. The objective of this study was to develop water quality guidelines with specific management options for the sustainable artificial recharge of the Windhoek Aquifer. Water-rock interactions that are expected to occur in the longer term when the unsaturated zone is largely saturated due to injection has been inferred from the hydrochemistry of the saturated zone and its evolution in the aquifer.

### KEYWORDS:

Hydrochemistry, injection, iron, Namibia, reclaimed wastewater, sulfate

### INTRODUCTION

Namibia, an arid country, located in the south-western part of southern Africa, extends over 825 000 km<sup>2</sup> but has a population of only 2.0 million (2007). Windhoek, the capital, is located in the central highlands and has approximately 270 000 inhabitants (2007). Local water resources are very limited and most of the city's water supply is obtained from surface impoundments located up to 250 km away.

This is because Windhoek is situated in the central highlands of Namibia and river systems, originating in the Auas and Eros Mountain ranges, drain away from the city in all directions. Due to the aridity of the region it experiences large fluctuations in its average annual rainfall of 360 mm and periodic droughts aggravate the water supply situation. Already in 1968 the lack of water prompted direct reclamation of potable water from treated wastewater and this is still being practised. In addition, water demand management has been introduced and has been in force for nearly two decades. The only reliable local source of water is the Windhoek Aquifer found in the Auas Formation quartzite and other quartzitic formations located to the south of the city. However, periods of drought have led to overexploitation and water levels in some parts of the aquifer are presently tens of metres lower than initially when the boreholes were drilled. The existing infrastructure can provide approximately nine per cent of the present annual demand of about 21 Mm<sup>3</sup>/a on a sustainable basis from the aquifer. Artificial groundwater recharge, or managed aquifer recharge (MAR), provides an attractive option for augmenting the natural groundwater recharge and through subsurface storage of surplus surface water. Four pilot scale injection studies between 1997 and 1999 proved the feasibility of borehole injection (Murray and Tredoux, 2002) while subsequent studies (Sweco, International 2002) confirmed the viability of artificial recharge of the Windhoek Aquifer. NamWater (Central Area JV Consultants, 2004) undertook a more detailed analysis for supply augmentation to the central area of Namibia from the Tsumeb and Karst III Aquifers in the north of the country with optimisation of the different available sources using the Central Area Model, (CA-Model). The study concluded that the Windhoek MAR scheme would be the most cost efficient augmentation option compared to other alternatives.

The introduction of an MAR scheme requires detailed knowledge of the ambient water quality in the various parts of the aquifer together with potential water-rock interactions. A considerable amount of water quality data is available on the source water for injection, i.e. the Von Bach Dam and the reclaimed water. Water quality information for the Windhoek Aquifer also exists but no detailed analytical data are available for the various host rocks. This implies that the expected water-rock interaction and the hydrochemical evolution have to be inferred from the observed data from the saturated zone. Other important sources of information would be groundwater quality trends with time, or from other events, such as unintended recharge and the impact of pollution sources. One potentially major aspect that cannot be fully assessed beforehand is the water-rock interactions that will take place when the unsaturated zone becomes saturated during injection. From the pilot scale testing some data are available that will aid in assessing this aspect.

A preliminary investigation into the quality of the water to be used for injection concluded that without pre-treatment none of the sources would be of sufficiently high quality to be used as such for injection into the Windhoek Aquifer (Tredoux, 2003). In the case of the treated water of the Von Bach Dam, the presence of

dissolved organic carbon (DOC) at levels often exceeding 5 mg/L is considered unacceptable for injection. With regard to the reclaimed water, the salinity is such that the water would be suitable for injection only in certain parts of the aquifer. Issues regarding injection water quality have been addressed by Van der Merwe *et al.* (2005). The report also considered improvement of the source water quality in detail.

This paper presents the natural water quality in the aquifer and discusses impacts of full scale injection of surface water (polished with granulated activated carbon and chlorinated) on ambient groundwater composition in the Windhoek Aquifer. Hydrochemical results obtained during the pilot scale injection tests are interpreted against the background of the natural groundwater quality and give an indication of the expected impact of water-rock interaction and water quality deterioration. This, in turn, affects the water quality requirements for the injected water and the injection practices.

## GEOLOGY

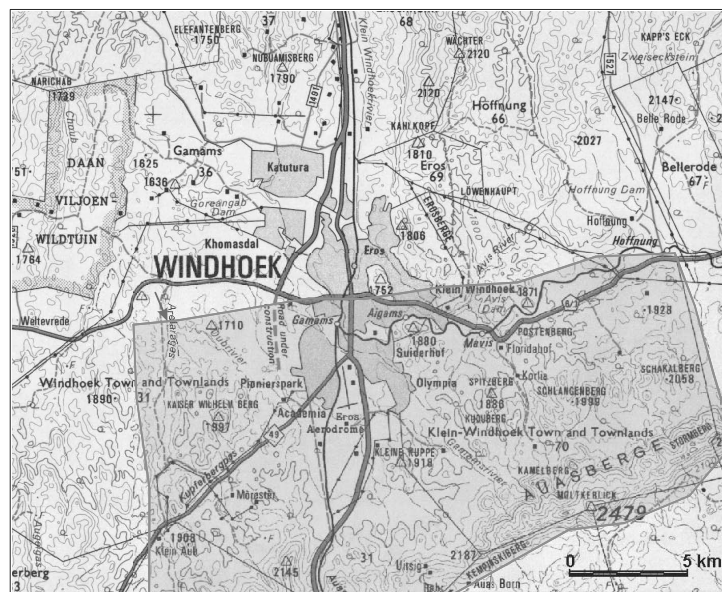
The geological setting and fracture systems of this mainly metamorphic environment are highly complex. Formations within the area have been folded in the process of orogenesis, and they have been subjected to a number of episodes of faulting including thrusting and rifting. It is important to note that quartzite and schist horizons with transverse faults and fractures that are prevalent throughout the Windhoek Aquifer dominate the hydrogeology of the area. The quartzite, being brittle, is highly fractured because of folding and faulting and has developed secondary porosity and permeability. The schist on the other hand is ductile and do not have well developed secondary permeability (Murray and Tredoux, 2002; Carr Barbour and Associates, 1999). Both the schist and the quartzite are thought to have no significant primary porosity (Colvin, 1994).

The geology of the Windhoek Aquifer can be divided into three main units of decreasing permeability: quartzite, micaceous quartzite, and schist. The Auas Formation quartzite in the south is relatively pure but the mica content in the quartzite horizons increases northwards and quartzite of the Kleine Kuppe Member (Kuseb Formation) in the north is highly micaceous. The degree and intensity of fracturing decreases as the mica content of the quartzite increases.

## Hydrogeology

The main natural recharge area of the Windhoek Aquifer is in the Auas Mountains which reach a maximum elevation of nearly 2500 m (Figure 1). The Auas Formation quartzite in the south is relatively pure but the mica content of the quartzite bands increases northwards while the permeability decreases, and as the aquifer dips below the city, the northerly flow of groundwater in the aquifer is impeded by mica schist and eventually blocked by the Matchless Amphibolite Belt crossing

the area from east to west. Large faults, mostly trending in a north-north-westerly direction, facilitate groundwater flow, partly also in the mica schist, and deep circulating groundwater which used to issue in the centre of the city and in Klein Windhoek as hot springs. Groundwater quality follows a gradation from high quality low salinity water in the recharge area in the Auas Mountains down to the city where higher salinities occur.



**FIGURE 1. Locality map showing the main features of the Windhoek Aquifer area. These include the Auas Mountains, the Kaiser Wilhelm Berg, and the Aretaregas River. The shaded area shows the approximate extent of the Windhoek Aquifer. (from 1:250 000 topographic sheet 2216, Surveyor General, Windhoek)**

The succession of metasediments hosting the Windhoek Aquifer dips from the Auas Mountains (in the southeast, Figure 1) northwards at an angle between 15 and 30°. The overall water level gradient indicates that water flows from the Auas Mountain range in the south to the north and the northwest. This is in agreement with the topographic gradient and with natural recharge mainly taking place in the highly fractured Auas Formation quartzite, with limited recharge in the micaceous quartzite, and practically no recharge in the areas underlain by schist. This was borne out by groundwater flow modelling (Zhang *et al.*, 2002). The Windhoek Aquifer is bounded by impermeable formations on all sides. The aquifer is characterised by shallow and deep flow with the latter presumably following the dip of the metasediments reaching a depth of approximately 4 km before being forced by the impermeable barrier in the north to emerge as hot springs (~80 °C) at the surface.



Radiometric age determinations (Vogel and Van Urk, 1975) using  $^{14}\text{C}$  indicated ages up to 23 000 years which confirm that recharge occurs in, and possibly also just south of, the Auas Mountains. Elevated water levels occur in the north around the hot springs in the centre of the city, in the central area in shallow boreholes in the schist (e.g. near borehole 9/5), and in the west around topographically higher areas. This latter area also includes the Kupferberg waste site. Groundwater leakage from the aquifer is only possible along the Aretaregas River valley in the north-western part of the aquifer (see Figure 1).

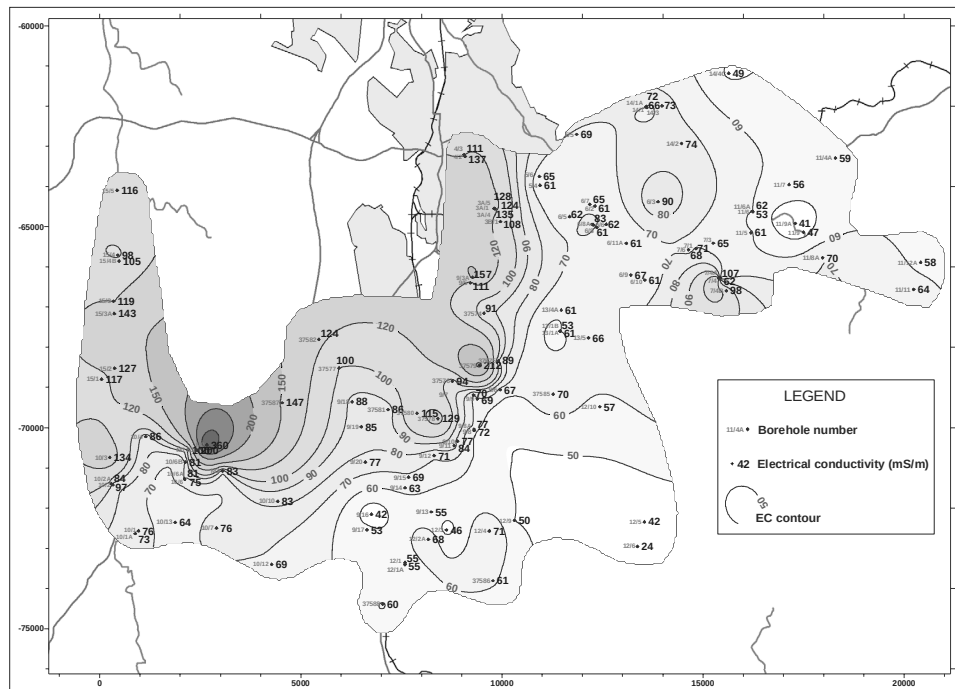
The country rock hosting the Windhoek Aquifer has no primary porosity but secondary porosity developed for several reasons (Carr Barbour and Associates, 1999). These include sympathetic fracturing of the country rock flanking the tension faults. The brittle nature of the quartzite leads to better secondary porosity development than in the schistose terrain. Therefore, a larger storage capacity is expected per unit volume of the fractured quartzite compared to an equal volume of fractured schist. Another potential source of secondary porosity (Carr Barbour and Associates, 1999) is of hydrochemical importance. Weathering, due to the breakdown of disseminated iron sulphide (pyrite), commonly occurring throughout substantial thicknesses of the quartzite, may have led to increased porosity.

## HYDROCHEMISTRY

The recharge water is being injected directly into the hard-rock fractured aquifer at a depth below the water table without any opportunity of seeping through the unsaturated zone. Hence the injected water generally needs to conform at least to drinking water quality requirements. As the ambient groundwater quality at each point in the aquifer will determine more specifically the required injection water quality at that point for ensuring that the abstracted water will conform to the specifications the groundwater composition in the Windhoek Aquifer had to be a studied in detail.

The hydrochemical evolution of groundwater in the aquifer takes place along the flow direction. In the Auas Formation quartzite in the south, the water represents recently recharged water containing mainly calcium-magnesium-bicarbonate. Sufficient calcium carbonate is present in the near-surface environment to allow for the initial development of groundwater of such hydrochemical composition. Along the flow path small quantities of sodium chloride dissolves, slightly increasing the sodium and chloride content of the groundwater. The calcium (and magnesium) concentration increases gradually along the flow path and at least part is exchanged for sodium from the aquifer matrix as the sodium concentration increases faster than that of chloride. Furthermore, the increase in bicarbonate in solution (measured as total alkalinity) confirms the further dissolution of calcium carbonate from the aquifer matrix followed by ion exchange of calcium for sodium.

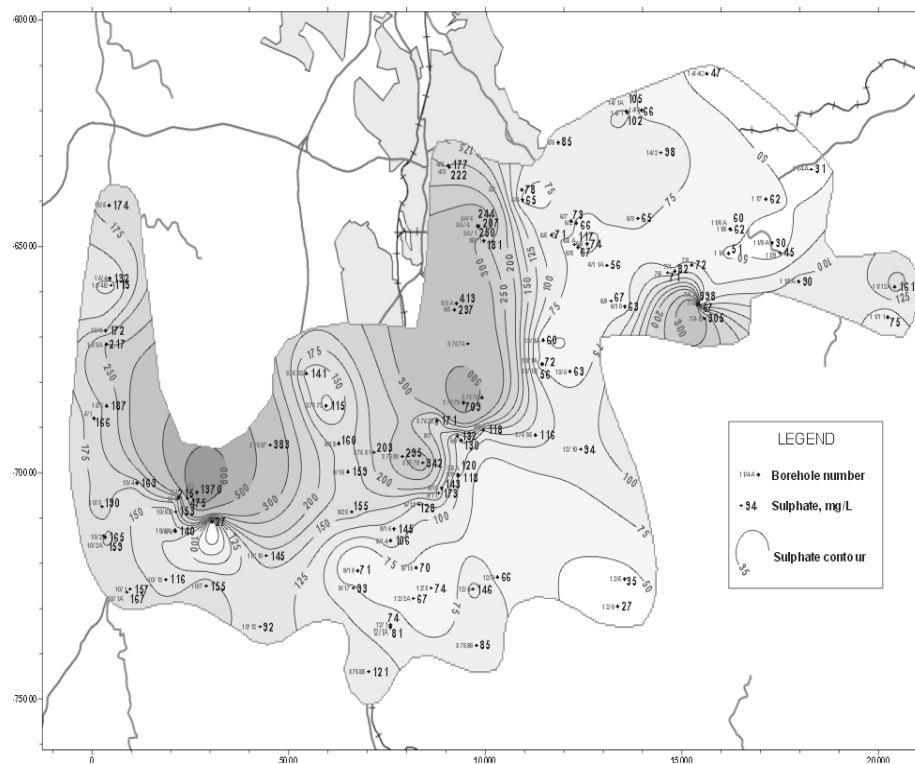
The electrical conductivity (EC) in Figure 2 shows the salinity distribution in the aquifer, confirming the very low salinity of the groundwater in the Auas Formation quartzite in the extreme southern and eastern parts of the aquifer. As part of the hydrochemical evolution in the aquifer, the salinity increases gradually in a northerly, north-westerly and westerly direction and reaches the highest levels in the mica schist underlying the southern part of the residential area, and in the west along the Aretaregas River. The deep circulating water emanating as hot water in boreholes in the city centre is slightly more saline. However, the highest salinities in the area occur at the Kupferberg waste site (to the south of the Kaiser Wilhelm Berg, Figure 1) where the EC reached 360 mS/m. These boreholes access groundwater in the micaceous schist which may form part of a perched aquifer and at this stage, it is unknown how much of this water will eventually reach the main aquifer, and at what rate.



**FIGURE 2. Geographic distribution of salinity as indicated by electrical conductivity (EC in mS/m) in the Windhoek Aquifer.**

The chloride levels in the aquifer are very low. The lowest concentrations, below 10 mg/L, occur in the south and the east. In the main aquifer the chloride concentration varies mainly between 8 and 14 mg/L also over most of the long flow paths of 5 to 10 km or more. The low chloride concentration in the aquifer also sets a

limit for chloride in the injected water. The increase of chloride along the flow path is very small. Only in the far west and in the north the concentrations approach 50 mg/L. Deep circulating water emanating in the Group 4 boreholes as hot water have a higher chloride concentration, reflecting the long residence times at depth. The hot water with higher chloride seems to be affecting the groundwater in the Group 3 boreholes to the south of Group 4. Deviations from these basic patterns will indicate either the existence of other flow systems or the presence of pollution sources. The highest chloride concentrations occur at two of the down-gradient monitoring boreholes at Kupferberg which confirms the site as being a point source of pollution.



**FIGURE 3. Sulphate concentration (mg/L) in groundwater in the Windhoek Aquifer.**

Figure 3 shows the distribution of sulphate in the aquifer and it is evident that sulphate occurs in high concentrations practically throughout the aquifer. The reason is that pyrite is disseminated throughout the host rocks of the Windhoek Aquifer. Dissolved oxygen entering the aquifer with the rainwater during natural recharge oxidizes the sulphide, generating sulphate in the aquifer until the dissolved oxygen is exhausted. As a result, the groundwater is largely devoid of dissolved oxygen and high in sulphate. Under natural conditions, the groundwater easily

attains sulphate concentrations of 200 to 300 mg/L. In areas where the soil has been disturbed, as in the residential area between Group 9 and Group 3, the sulphate

concentration increases as more of the pyrite is exposed to oxidation. Also at the Kupferberg waste site the natural soil and rock structure is disturbed and high concentrations of sulphate are being generated. This is reflected in the groundwater down-gradient of the site where the sulphate exceeds 1000 mg/L.

When the sulphide is oxidized, iron also enters into solution in the form of ferrous ions ( $\text{Fe}^{+2}$ ). In the unsaturated zone, sufficient oxygen may be available to oxidize the iron to the ferric ( $\text{Fe}^{+3}$ ) form, which is insoluble at neutral or alkaline pH. However, in the saturated zone ferrous iron will remain in solution once the dissolved oxygen is consumed. Analytical data showed that iron is distributed widely throughout the aquifer. High iron concentrations occur at Kupferberg waste site and a maximum of 22 mg/L was recorded in the slightly acidic water. Both ferrous and ferric iron is soluble in such groundwater.

Nitrate is often a useful indicator of pollution but its concentration in the Windhoek Aquifer is generally very low. The only polluted areas that can be identified with confidence are at the Avis Dam (borehole 6/3), in the Group 9 area between production boreholes 9/5 and 9/6, and at Kupferberg. The nitrate levels at Kupferberg are nevertheless very low. Stable isotope determinations confirmed that surface water is affecting borehole 6/3. This is also the borehole with the highest nitrate concentration of 8.3 mg/L.

With regard to the organic parameters, very limited reliable information was available for the groundwater. As a result, it was only feasible to determine the distribution of dissolved organic carbon (DOC) in the aquifer. Pristine, i.e. unpolluted, fractured aquifers are generally free of organic compounds. Only in certain acidic soil environments, mostly in moderate climatic zones, natural organic matter in the form of humic and fulvic acids may occur. In the extreme climatic conditions of arid regions organic compounds are generally absent. Hence, very low concentrations of DOC would be expected in the Windhoek Aquifer. In the southern and eastern parts of the aquifer, the DOC levels are very low and the main trend is a slight increase in a north-westerly direction. Some of these could be related to local pollution, e.g. from the garden refuse disposal in the vicinity of the Group 13 boreholes. An increase in the Group 6 area may also be related to pollution in the residential area. As can be expected, the DOC levels at Kupferberg are somewhat higher. The main conclusion is that the Windhoek Aquifer displays very low levels of organic compounds over most of the area.

The degree of mineralization of the various faults and the potential effect on groundwater quality needs further investigation and clarification. Extensive mineralization occurring along the Pahl Fault including massive disseminated iron

sulphide and various minerals were found during excavations along the fault for building construction (own observations, 1974). Most of these minerals may not be water-soluble at low temperatures and would not create a problem for groundwater exploitation. It is advisable to determine the range of minerals and constituents, e.g. heavy metals that may potentially enter the groundwater. For example, high arsenic levels could have serious health risk implications. Data available for arsenic in the Windhoek Aquifer indicates that the concentrations are generally very low and close to the detection limit of 0.005 mg/L. The highest concentration of 0.013 mg/L occurs in the deep circulating hot water of Group 4. In the same vicinity, the cooler waters of Group 3 also have measurable levels just above the detection limit. It is considered plausible that the mineralization of the Pahl Fault zone contributed to the presence of these arsenic concentrations. All of these are near or below the World Health Organization guideline value as well as the maximum allowable level of 10 µg/L. It is considered highly unlikely that the arsenic guideline levels will be exceeded because of the managed aquifer recharge operations. Nevertheless, during full operation, arsenic should be analysed, particularly in monitoring boreholes close to injection boreholes that are affected by increases in sulphate. At the time of sampling, the parameters pH, Eh, and temperature should be measured at the wellhead in order to determine the speciation of the arsenic. Speciation is important as the removal technologies need to be adapted to ensure efficient removal of all the arsenic compounds from the groundwater.

## GROUNDWATER POLLUTION

Groundwater pollution is clearly evident in the water quality deterioration observed at Kupferberg. Chloride concentrations down gradient of the waste site increased since 1997. In contrast, the chloride concentration in Group 10 production boreholes, surrounding the site, remained constant over the 45 year period of observation. Leachate from the waste site is considered to be the main source of chloride. Similar to chloride, sulphate concentrations have increased to very high levels at the down-gradient observation borehole M2. The increased sulphate is largely ascribed to the disturbance of the soil and enhanced weathering causing the oxidation of sulphide and the leaching of sulphate. The schist in this area is rich in sulphide (Mwiya, 2004). The extent of the salinity threat to the aquifer needs quantification as the linkage between the water in the schist and the main aquifer is still unclear. Presently, the relatively thick unsaturated zone may impede the exchange of water from the schist with the main, quartzite aquifer. However, the longer-term impact of the large differences in water quality between the shallower boreholes in the schist and the deeper aquifer will need to be elucidated as any significant rise in the water level in the main aquifer may enhance the contact with poorer quality water. Other incidences of pollution include the occurrence of nitrate at borehole 6/3 at the Avis Dam.

Groundwater pollution is confirmed by the presence of bacteria. The results for the indicator organisms varied over a wide range. However, procuring groundwater samples for reliable microbiological analysis presents many problems. When sampling is done at production boreholes where the pump has been running for many hours and if the sampling points are correctly sterilized, sampling should be representative of the groundwater. In several instances, the turbidity readings were available and some of these were very high. Bacteria often adhere to solid particles and in the case of high turbidity the results may not be a true reflection of the groundwater quality. Inspection of the microbiological results showed that practically none of the boreholes had consistent bacterial counts. Boreholes 6/3, 6/9, 7/6 and a number of the Group 9 boreholes showed counts of faecal coliforms, *E.coli*, faecal streptococci or *Clostridium* spp. As a result it was recommended that monitoring should continue at least once per annum with special attempts to ensure uncontaminated samples.

## OBSERVATIONS FROM PILOT INJECTION TESTS

The injection of surface water of high quality has the potential to bring about changes in the hydrochemistry in the aquifer. This is mainly due to the injected water, saturated with oxygen at atmospheric pressure, chemically interacting with the ambient groundwater and the host rock. The pilot injection tests provided the opportunity to observe impacts on the hydrochemistry in the vicinity of the injection boreholes. Four pilot scale injection tests were carried out and the period of injection and the volume of water injected during each test are listed in Table 1. Although the tests were of relatively short duration, with the longest one at borehole 9/8A lasting just more than six months, the volume of water injected was sufficient to bring about a significant rise in the water table and to cause measurable changes in the hydrochemistry at the observation points.

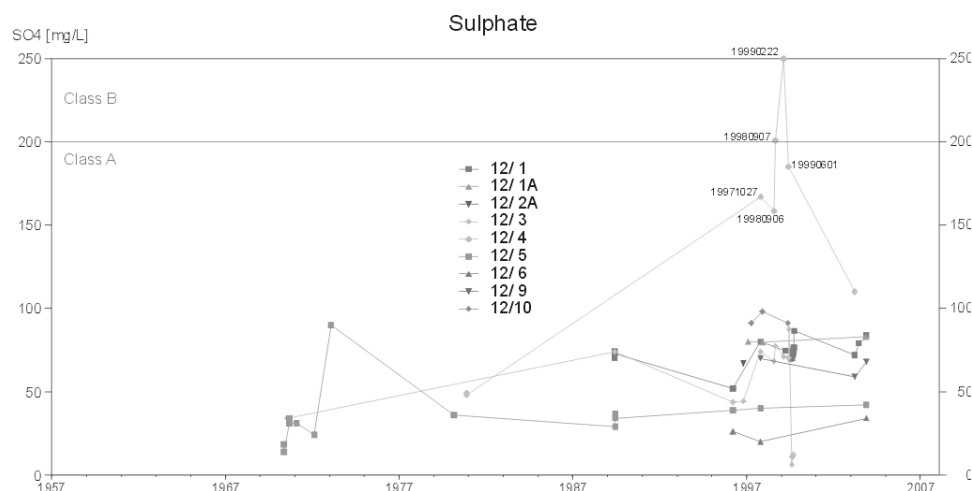
**TABLE 1. Windhoek aquifer pilot scale injection tests.**

| Injection borehole | Injection period         | Volume injected (m3) |
|--------------------|--------------------------|----------------------|
| 9/5                | 28-01-1997 to 18-02-1997 | 5 000                |
| 9/8A               | 12-08-1998 to 23-02-1999 | 289 000              |
| 9/16               | 12-04-1999 to 27-05-1999 | 24 000               |
| 12/3               | 10-08-1999 to 14-09-1999 | 99 000               |

During the pilot scale injection of surface water at borehole 9/8A, the salinity, and particularly the sulphate concentration increased at observation borehole 9/11 when the water level rose (Murray and Tredoux, 2002). Similar phenomena were also observed elsewhere during water level recovery. This implies that, in general, any water level variation in the aquifer may increase sulphate in the aquifer. A receding water level will cause ingress of oxygen into the (moist part of the) unsaturated zone

and oxidation of sulphide and other reduced material will take place. Once the water level rises again the sulphate formed during the oxidation process will be dissolved and dispersed into the groundwater. Similarly, oxygenated rainwater recharging the aquifer will cause a rise in the water level but will lead to oxidation of sulphide both in moist parts of the unsaturated zone, and in the saturated zone, until the dissolved oxygen is exhausted.

The increase in sulphate and salinity in monitoring boreholes surrounding the injection boreholes was the main effect noticed during the pilot scale injection tests. The slightly higher EC was also due to an increase in the concentrations of some of the other main cations. Alternative wetting and drying associated with the introduction of oxygen into the aquifer enhances this process. The data evaluation showed that the increase in sulphate during the injection test at borehole 9/8A was also evident at borehole 12/4 where a maximum of 250 mg/L of sulphate was measured on 22 February 1999, i.e. one day before the test ended compared to the background value of 50 mg/L (Figure 4). Insufficient data are available to determine whether there was any change in chemistry at borehole 12/4 during the injection test in borehole 12/3. However, the sulphate concentration had already increased prior to the injection tests when it reached 160 mg/L in October 1997. Comparing this with the water level information, it became evident that the sulphate increase followed after a period of water level recovery starting from the end of December 1996. The recovery period was preceded by continuous drawdown for at least four years and therefore in this case the sulphate increase was directly related to the borehole abstraction regime.



**FIGURE 4. Sulphate concentration trends in groundwater of the Group 12 wellfield during pilot injection testing at borehole 9/8A.**

Both sulphate and iron are mobilized when pyrite (i.e. FeS) is oxidized. In the Auas quartzite, graphitic schist occurs that will promote reducing conditions keeping  $\text{Fe}^{2+}$  ions in solution. Addition of organic chemical compounds to the aquifer during injection will have a similar effect in creating a reducing environment that enhances the solubility of iron. Some information is available on the occurrence and mobilization of iron during the pilot scale injection tests. The reported values generally refer only to total iron. The highest concentration measured as a direct result of the injection run at borehole 9/8A was 3.15 mg/L and in many instances total iron exceeded 1 mg/L. Boreholes located down-gradient of injection borehole 9/8A, showed elevated iron concentrations reaching a maximum of 2.63 mg/L. The total iron concentration was also recorded during the high rate injection test at borehole 12/3. Again, very high levels were recorded.

It is evident that iron is widely distributed and mobilized during injection of water into the aquifer. The key question is to what extent the presence of iron could contribute to borehole clogging over the longer term. This will only become evident through detailed monitoring of the iron concentrations, the pH, and the oxidation-reduction potential (Eh), which will allow the determination of conditions in the aquifer. Once such information is available, stability diagrams will provide a means for estimating the actual quantities of mobile iron in the aquifer.

## DISCUSSION

A range of other chemical constituents were studied in the Windhoek Aquifer, including arsenic and boron. Although assays were not done on rock samples it can be concluded from the water analyses that these elements should not create any chemical hazards due to water-rock interaction during long term injection. The only other element that entailed a health risk was fluoride which occurred in the hot water at concentrations well above potable guidelines. As injection was not intended to affect the deep flowing system the presence of fluoride was not considered of significance for the MAR operation.

Potential hydrochemical interactions and changes in water quality during and after injection are related to a number of factors. These include chemical differences between the injected water and the ambient groundwater, e.g. oxygen content and chemical composition, which will lead to re-establishment of chemical equilibria. Other important interactions will take place between the injected water and the aquifer matrix, both in the saturated and unsaturated zones. The oxidation-reduction potential of the injected water and the groundwater as well as the carbon dioxide partial pressure will determine the extent of such reactions. From the available data, it would seem that changes in the oxidation-reduction potential in the subsurface might be the main factor affecting the hydrochemical environment of the Windhoek Aquifer.



## QUALITY GUIDELINES FOR RECHARGE WATER

With the investigation into injection water quality it was accepted that the water quality from both treatment plants (Von Bach Dam Water Treatment Plant and the New Goreangab Water Reclamation Plant) could be improved subject to certain process adjustments. The injection water quality criteria were set based on the following guiding principles (Van der Merwe *et al.*, 2005):

The recharge scheme should meet a number of demands, which are listed below:

1. No negative environmental impact of significance.
2. Sustainable use of water from the Windhoek Aquifer for drinking water purposes preferably without treatment or at most with limited treatment such as stabilisation and disinfection:
  - a) The recharge water should meet modern drinking water standards (e.g. Rand Water and RSA)
  - b) No additional health risk for the residents of Windhoek as compared to present sources (2004)
3. No technical problems should arise due to injection water quality, such as clogging, corrosion and demand for extensive treatment of water before distribution.
4. Accept a deterioration of certain quality parameters of the water within the aquifer provided that the water quality after abstraction complies with acceptable water quality guidelines (RSA and Rand Water)

The interim water quality guidelines for injection water are based on the Rand Water and South African potable water quality guidelines with special requirements for DOC, AOC, EC, Chloride, Sulphate and Nitrate and Nitrite. The guidelines will be revisited after 2 years after intensive data collection and evaluation after further pilot injection studies.

Determining the longer term water quality impact will only be possible through mass transport modelling and this has to be considered as a next step for predicting the sustainability of the operation. The mass transport model will be based on the existing hydraulic simulation model. The outcome will strengthen the scientific base for refining the water quality requirements.

## CONCLUSIONS

The Auas Formation quartzite constitutes the most important part of the Windhoek Aquifer. This is where natural recharge is taking place, the best quality groundwater

is found, and high permeability combines with high storativity. Only low salinity water free of organic compounds should be injected into this part of the aquifer.

Low salinity water occurs over most of the aquifer. Groundwater in the Auas Mountains generally has very low chloride concentrations ranging from 4 to 10 mg/L and increasing to a maximum of approximately 60 mg/L in the deep circulating hot water issuing in Group 4 and the associated area of Group 3. In contrast, the chloride concentration exceeds 300 mg/L in the polluted part of the aquifer at Kupferberg waste site. It is also high in the shallow boreholes located in the schist. From the observations it would seem that these polluted parts may have limited hydraulic connection with the main aquifer.

Significant changes in salinity (EC) are mainly associated with the pilot scale injection tests, but these are also linked to the abstraction regime, e.g. in Group 12. These variations in EC are ascribed to fluctuations in the water level in the aquifer, the concomitant enhanced oxidation of sulphide, and subsequent dissolution of sulphate. High intensity natural recharge events may also cause salinity fluctuations in intake areas. Such a phenomenon was observed at Kupferberg following the 1999/2000 rainy season but due to a lack of detailed and accurate data this could not be studied for the aquifer as a whole.

The detailed water quality assessment showed that the presence of sulphate throughout the aquifer is an important characteristic of the hydrochemistry. Sulphate reaches very high concentrations of several hundred mg/L in certain areas. Its source is the iron sulphide (pyrite) which occurs widespread throughout the aquifer. Disturbing the soil and rock enhances weathering and oxidation of the sulphide. In disturbed areas, such as at Kupferberg, sulphate reaches several thousand mg/L.

Iron is ubiquitous in the groundwater in the Windhoek Aquifer. Similarly, manganese is present in the aquifer. The iron and manganese concentrations in the groundwater vary widely and are not accurately known. The solubility of iron depends on a number of factors and iron speciation is needed for a full understanding of the conditions in the aquifer.

Considering artificial groundwater recharge, the presence of pyrite in the aquifer is of some concern and injection should be controlled to minimize the oxidation of pyrite. Deep injection of the water, i.e. directly into the saturated zone, would be necessary to prevent cascading of the water through the unsaturated zone. Where possible, the change in water table should be minimised.

Recharge water quality has to be managed to maintain the high quality of the water stored in the aquifer. This has to be done according to the criteria set for this purpose.

An important outcome of artificial groundwater recharge may be the improvement of water quality in certain higher salinity areas.

## ACKNOWLEDGEMENTS

Thanks are due to the City of Windhoek for funding the research and providing water quality and other data: Tribute is also paid to the men and women that meticulously gathered the production volumes, water levels, and data on other parameters over many decades without ever knowing what this data would mean one day. The Geological Survey, Ministry of Mines and Energy, is thanked for the provision of geological map sheet 2216 of the Windhoek area.

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## INNOVATIVE WATER QUALITY STUDIES ON THE SANTA ANA RIVER, CALIFORNIA

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### ABSTRACT

In an average year, the Orange County Water District (OCWD) diverts 246,700 million m<sup>3</sup> per year of Santa Ana River (SAR) water for recharge into the Orange County Groundwater Basin. OCWD's recharge facilities include 26 facilities that cover over 6.1 km<sup>2</sup> (1,510 acres). SAR base flow, which is comprised primarily of tertiary-treated effluent, contains approximately 4 to 6 mg/L of dissolved organic carbon (DOC). SAR storm flow contains up to 25 mg/L of DOC. OCWD has conducted extensive water quality studies to verify the safety of recharging the basin with SAR flows. These studies include chemical characterization and fish biomonitoring evaluations.

Research performed by the United States Geological Survey (USGS) in Colorado applied advanced analytical techniques to characterize the majority of bulk organic matter present in recharge waters. Complex chemical and physical separation methods were used to comprehensively characterize the dissolved and colloidal organic matter following fractionation. The techniques were used to evaluate the amount, nature and reactivity of dissolved organic matter in SAR water. Following a fractionation/isolation process, dissolved organic matter was characterized by infrared, carbon-13 nuclear magnetic resonance spectrometry (<sup>13</sup>C-NMR), and liquid chromatography/mass spectrometry. These new analytical tools allowed the USGS approach to successfully characterize the majority of the carbon present.

Results determined that the majority of dissolved organic matter in SAR water was of natural origin, and most were biodegradable colloidal materials, which were removed during recharge. The trace amounts of anthropogenic organic matter identified in groundwater were mostly derivatives of linear alkyl sulfonates (detergents). Changes in the relative abundance of the different DOC fractions were measured on a synoptic groundwater recharge pulse tracked and sampled over time (~2 years) in various sites along groundwater flow paths originating at a single large recharge facility. The relative abundance of the hydrophobic acid fraction decreased systematically along the flow path relative to the other fractions. Relative abundances of the hydrophilic and transphilic fractions were most stable along the flow path. The colloidal fraction was largely removed in the initial period of recharge and remained low during groundwater flow.

SAR water and Colorado River water have very different chemistries. These differences are also evident in organic composition, with lower DOC concentrations, lower specific UV absorbance (SUVA) values and lower disinfection by-products formation potential in Colorado River water compared to SAR water. The fractions of the dissolved organic matter (DOM) from SAR water contain greater amounts of each fraction, and these fractions are more aromatic than DOM in Colorado River water. DOC fractionation analyses were conducted on two sets of water samples collected along the flowpath following recharge at a large recharge basin. The first set of samples was taken when water in the basin was comprised

entirely of Colorado River water, and the second set of samples when the basin contained only SAR water. As the waters moved downgradient, the amount of material in each fraction, with the exception of the hydrophobic neutral fraction, decreased significantly with distance, with a corresponding decrease in SUVA in the recharged SAR water. A comparison of the DOC fractions in downgradient wells indicates that the DOC in the waters originating from the Colorado River and SAR were similar. Further analyses of DOC concentration, SUVA and DOC fractionation data for all the downgradient wells sampled indicate the DOC in the wells is similar, regardless of the DOC concentration or composition of source waters recharged.

OCWD also evaluated testing water quality with fish. Three rounds of testing were conducted in 2004 and 2005. In the first two rounds, Japanese Medaka fish were analyzed for tissue pathology, vitellogenin induction, reproduction, and gross morphology. In the third round, fish were analyzed for vitellogenin induction, reproduction, limited tissue pathology, and gross morphology. In the first two rounds, no statistically significant differences in gross morphological endpoints, gender ratios, tissue pathology, or reproduction were observed between the test water (shallow groundwater adjacent to the SAR) and the control water. In the third round, no statistically significant differences were observed in reproduction, tissue pathology (limited to evaluation of gonads and ovaries), or vitellogenin induction between the test water and the control water.

These studies provide substantial new evidence that the SAR is a suitable source of recharge water for the groundwater basin.

## INTRODUCTION

OCWD is located in coastal southern California. OCWD was established by the State of California in 1933 to manage the Orange County Groundwater Basin. The groundwater basin is the primary water supply for 2.3 million people. To manage the groundwater basin, OCWD engages in a wide range of activities, including water quality protection and enhancement, groundwater recharge, water recycling, and natural resources management.

The groundwater basin is located in the lower portion of the Santa Ana River Watershed. Upstream of OCWD, publicly owned treatment plants discharge approximately 180 million m<sup>3</sup> per year of tertiary-treated water to the Santa Ana River. The majority of this water flows down to the OCWD's recharge facilities, where it is recharged into the groundwater basin. Prior to reaching OCWD's recharge facilities, part of the water is treated at the OCWD's Prado Wetland.

In an average year, OCWD diverts 246,700 million m<sup>3</sup> per year of SAR water for recharge into the Orange County Groundwater Basin. The District operates 26 recharge facilities that cover over 6.1 km<sup>2</sup> (1,510 acres). SAR base flow, which is comprised primarily of tertiary-treated effluent, contains approximately 4 to 6 mg/L of dissolved organic carbon (DOC). SAR storm flow contains up to 25 mg/L of DOC.

OCWD has conducted extensive water quality studies to verify the safety of recharging the basin with SAR flows. These studies include chemical characterization and fish biomonitoring evaluations.

## METHODS

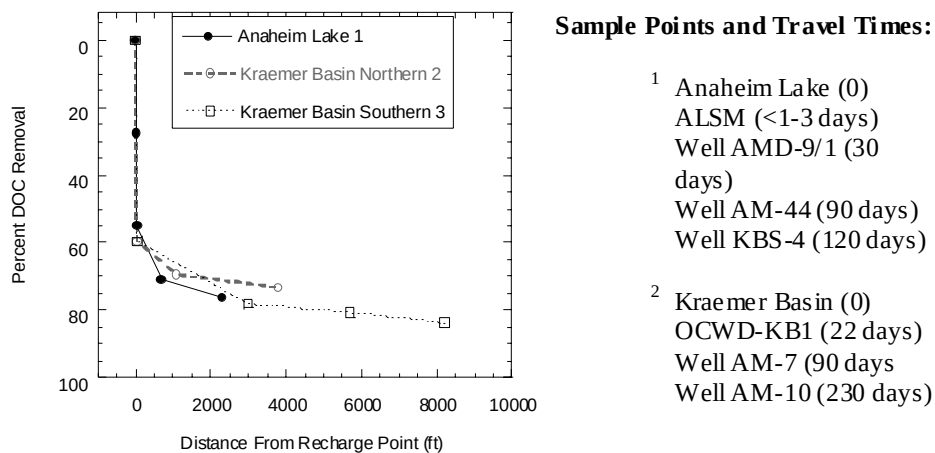
The United States Geological Survey (USGS) in Colorado applied advanced analytical techniques to characterize the majority of bulk organic matter present in recharge waters. Complex chemical and physical separation methods were used to comprehensively characterize the dissolved and colloidal organic matter following fractionation. The techniques were used to evaluate the amount, nature and reactivity of dissolved organic matter in the SAR system. Following a laborious fractionation/isolation process, dissolved organic matter was characterized by infrared, carbon-13 nuclear magnetic resonance spectrometry ( $^{13}\text{C}$ -NMR), and liquid chromatography/mass spectrometry. Details of these analyses are described in Leenheer (2004, 2005).

Methods used in the fish biomonitoring project are described in the project report for the project (Schlenk, et al, 2006).

## RESULTS AND DISCUSSION

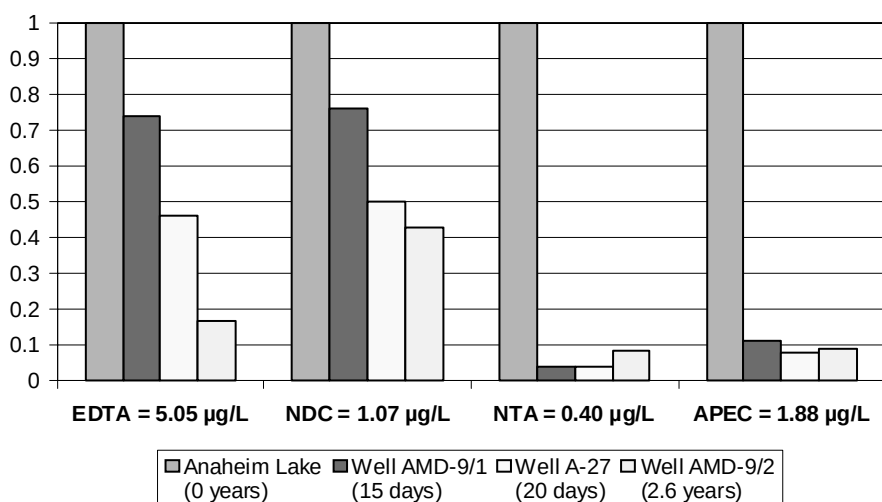
In the SAR at Imperial Highway, DOC concentrations of baseflow generally ranged between 4 to 6 mg/L. During stormflow conditions, DOC was generally higher, and during some storms ranged from 15 to 25 mg/L. During recharge and groundwater flow away from the recharge basins, DOC was reduced as shown in Figure 1. Within a short travel distance corresponding to one month or less of groundwater flow, DOC was reduced 50 percent or more. The finding of 50 percent removal was consistent throughout the study, during both saturated and unsaturated conditions in the aquifer underlying the recharge basins. Further DOC reduction occurred with additional groundwater flow away from the recharge basins.

Chemical compounds known as “wastewater indicator compounds” that were useful as tracers of wastewater derived organic compounds were evaluated, including: ethylenediaminetetraacetic acid (EDTA), alkylphenol ethoxycarboxylates (APEC) and nitrilotriacetic acid (NTA). Monitoring wells downgradient of Anaheim Lake provided data on changes in these specific organic compounds. Figure 2 shows relative concentrations in Anaheim Lake and three wells downgradient from Anaheim Lake, with concentrations normalized to Anaheim Lake. Concentrations in Figure 2 are normalized to the concentration in Anaheim Lake. EDTA and naphthalene dicarboxylic acid (NDC) concentrations decreased systematically away from Anaheim Lake. NTA concentrations, on the other hand, decreased to virtually zero before the first groundwater sampling point and was rarely detected at any of the subsurface locations. APEC concentrations were also reduced significantly during recharge and less than one month of subsurface flow (Reinhard, et al, 2000).



Source: OCWD Laboratory Data

**FIGURE 1. DOC Removal Following Recharge.**



Source: Reinhard, et al., 2000

**FIGURE 2. Relative Concentrations of Wastewater Indicator Compounds in Anaheim Lake and Wells.**

## Disinfection By-Products

Approximately one-third of the wastewater discharged to the SAR during the study underwent ultraviolet (UV) light disinfection, while the remaining two-thirds was disinfected with chlorine. Extensive studies of disinfection by-products (DBPs) in both recharge water and groundwater were conducted by the Metropolitan Water District of Southern California (MWD) Water Quality Laboratory. Samples were collected from the SAR, the Prado wetlands influent and effluent, the Forebay recharge basins, and groundwater from production and monitoring wells. Major wastewater treatment plant discharges were also sampled to evaluate differences in DBPs in traditional chlorinated/dechlorinated effluent compared to effluent disinfected using UV light. Samples were analyzed for in-situ concentrations of a variety of DBPs, including: trihalomethanes (THMs), haloacetic acids (HAAs), formaldehyde, acetaldehyde, chlorate and bromate. The study also evaluated DBP precursors through formation potential testing and through simulated distribution system (SDS) tests.

A wide range of in-situ DBPs was detected in wastewater, but most were removed prior to recharge. The average total THM concentration in the wastewater discharges to the SAR was 26 µg/L, but the THMs were removed significantly in river transit (Figure 3). The HAAs were reduced during river transit but not to the same degree as the THMs due to their lower volatility compared to THMs. During groundwater recharge and subsurface flow, the HAAs were reduced further. Low concentrations of some DBPs persisted in the SAR below Prado Dam, including formaldehyde, acetaldehyde, and chlorate. Overall, the concentrations of DBPs were low to non-detect in recharged groundwater. Chlorate was detected in the range from 15 to 20 µg/L in some monitoring wells that have older groundwater, but the concentration was below the California Department of Health Services Action Level of 800 µg/L.

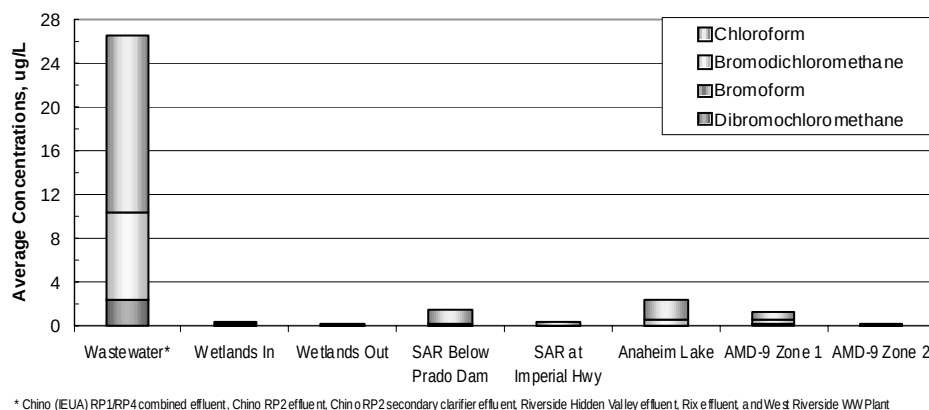
Additional DBP studies focused on changes in DBP formation potential through the Prado Wetlands and during recharge and along groundwater flowpaths. The formation potential of THMs and HAAs changed during groundwater recharge, suggesting transformation of molecular organic carbon during recharge. In general, formation potentials decreased along a particular flowpath due to the decrease in DOC with increasing subsurface residence time. The formation potential of chlorinated forms of DBPs decreased with increasing residence time, while brominated forms increased on a molar basis due higher bromide concentrations in older groundwater. The SDS testing determined that all production wells disinfected with free chlorine were able to meet primary drinking water standards for THMs and HAAs.

## Characterization of Organic Matter

The characterization by the USGS determined that the majority of dissolved organic matter in recharge water was of natural origin, and most were biodegradable colloidal



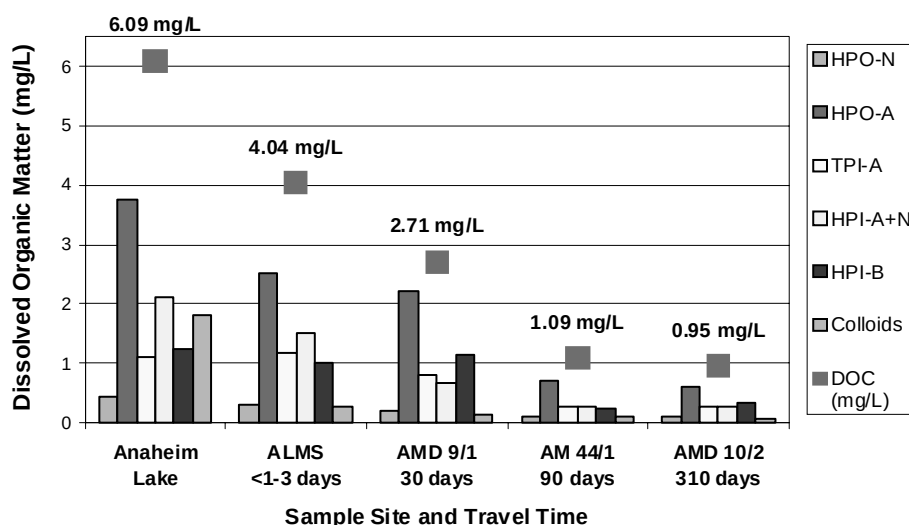
materials removed during recharge. The trace amounts of anthropogenic organic matter identified in groundwater were mostly derivatives of linear alkyl sulfonates (detergents).



**FIGURE 3. Relative Abundance of Trihalomethanes.**

Source: MWD Laboratory Data

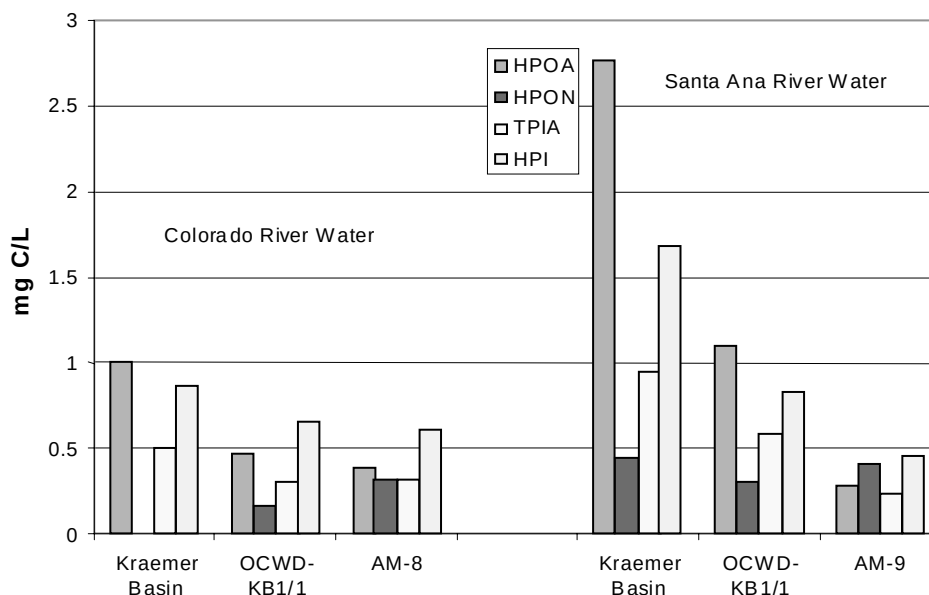
Changes in the relative abundance of the different DOC fractions were measured on a synoptic groundwater recharge pulse tracked and sampled over time (~2 years) in various sites along groundwater flow paths originating from Anaheim Lake. The relative abundance of the hydrophobic acid fraction decreased systematically along the flow path relative to the other fractions (Figure 4). Relative abundances of the hydrophilic and transphilic fractions were most stable along the flow path. The colloidal fraction was largely removed in the initial period of recharge and remained low during groundwater flow.



**FIGURE 4. DOC Fractionation in Anaheim Lake and Downgradient Wells.**

Source: Leenheer, 2004

SAR water and Colorado River water have very different chemistries. These differences are also evident in organic composition, with lower DOC concentrations, lower specific UV absorbance (SUVA) values and lower disinfection by-products formation potential (DBPFPs) in Colorado River water compared to the SAR. The fractions of the dissolved organic matter (DOM) from the SAR contain greater amounts of each fraction, and these fractions are more aromatic than the Colorado River. DOC fractionation analyses were conducted on two sets of water samples collected along the flowpath following recharge at Kraemer Basin. The first set of samples was taken when water in Kraemer Basin was entirely Colorado River water, and the second set of samples when only SAR was recharged. As the waters moved downgradient, the amount of material in each fraction, with the exception of the hydrophobic neutral fraction, decreased significantly with distance, with a corresponding decrease in SUVA in the recharged SAR water. A comparison of the DOC fractions in wells downgradient from Kraemer Basin indicates that the DOC in the waters originating from the Colorado River and SAR were similar (Figure 5). Further analyses of DOC concentration, SUVA and DOC fractionation data for all the downgradient wells sampled (AM-7, AM-10, AM-8, AM-9 and SCWC-PLJ2/1) indicate the DOC in the wells is similar, regardless of the DOC concentration or composition of source waters recharged.



**FIGURE 5. Changes in DOC of Colorado River Water and SAR Water.**

Source: Leenheer, 2004

Prior to the SARWQH Study, only about 10 to 20 percent of the organic compounds in recharge water, and in wastewater in general, had been identified and it was the remaining “unidentified” organics that concerned DHS. The USGS approach helped answer questions regarding the character of previously unknown organic matter in recharge water. These new analytical techniques enabled researchers to classify approximately 90 percent of the DOC entering the recharge basins during baseflow conditions.

### Fish Biomonitoring Project

Fish biomonitoring was evaluated in a feasibility study to determine if it could provide a supplement to chemical monitoring. If feasible, biomonitoring could provide additional assurance that unidentified or uncharacterized chemicals with potential toxicity are not present at biologically effective concentrations in the water. In general, three exposure groups were used: a negative control, positive control, and the test water. The test water was from a shallow well adjacent to the SAR.

Three rounds of analyses with fish were conducted in 2004 and 2005. In the first two rounds, Japanese Medaka fish were analyzed for tissue pathology, vitellogenin induction, reproduction, and gross morphology. In the third round, fish were

analyzed for vitellogenin induction, reproduction, limited tissue pathology, and gross morphology. In the first two rounds, no statistically significant differences in gross morphological endpoints, gender ratios, tissue pathology, or reproduction were observed between the test water (shallow groundwater adjacent to the SAR) and the control water. Table 1 summarizes the results from analyses of the fish for liver tumors in the second round of testing. In the third round, no statistically significant differences were observed in reproduction, tissue pathology (limited to evaluation of gonads and ovaries), or vitellogenin induction between the test water and the control water.

**TABLE 2. Incidence of Liver Tumors in Orange Red Medaka (Round 2)**

|                              | Exposure Group   |                  |                            |
|------------------------------|------------------|------------------|----------------------------|
|                              | Negative Control | Positive Control | Test (Recharged SAR water) |
| Female                       | 0% (0/15)        | 19% (7/37)       | 0% (0/35)                  |
| Male                         | 0% (0/9)         | 8% (3/38)        | 0 (0/27)                   |
| Source: Schlenk, et al, 2006 |                  |                  |                            |

## CONCLUSIONS

These studies provide substantial new evidence that the SAR is a suitable source of recharge water for the groundwater basin. An independent evaluation of the chemical characterization data was conducted by the National Water Research Institute (NWRI). NWRI's Scientific Advisory Panel determined that no chemicals of wastewater origin were identified in groundwater at concentrations that are of public health concern (NWRI, 2004).

These studies also provide new information regarding the water quality improvements that occur during groundwater recharge and subsequent subsurface flow away from the recharge facilities. Recharge and subsequent groundwater flow provides a substantial treatment process that improves water quality. OCWD continues to evaluate the changes in water quality that occur during recharge due to the importance of assuring the sustainability of the treatment process. Additionally, OCWD continues to adjust the monitoring program for the SAR and groundwater as new analytical chemistry tools become available and the science of water quality continues to evolve.

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## ALTERNATIVE SAT OF SECONDARY EFFLUENTS USING A COMBINATION OF UF AND SHORT TERM SAT FOR INCREASING INFILTRATION RATES AT THE SHAFDAN SITE (ISRAEL)

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### ABSTRACT

At the Shafdan site, secondary treated wastewater from the Tel Aviv area is recharged into an aquifer via a soil aquifer treatment (SAT) system with a retention time of 6-12 months. In order to increase infiltration rates a system of dug wells is considered. A pilot dug well site infiltrating UF treated secondary effluents is studied in this paper. The development of the site and the detailed hydrogeological characterisation including the development of a radial unsaturated-saturated solute transport model based on two tracer-tests is discussed. Geochemical batch modelling suggests dominant potential redox induced reactions, both in the saturated and the unsaturated zone due to the elevated concentration of dissolved organic matter (DOM) in the infiltrating water. In a future modelling step the models will be coupled and geochemically calibrated. Ultimately this will allow assessing the viability of the dug well system in terms of infiltration capacity and recovered water quality taking into account a residence time of around between 40 days.

### KEYWORDS

SAT, Shafdan, redox processes, flow and transport modelling, geochemical modelling, dissolved organic matter (DOM)

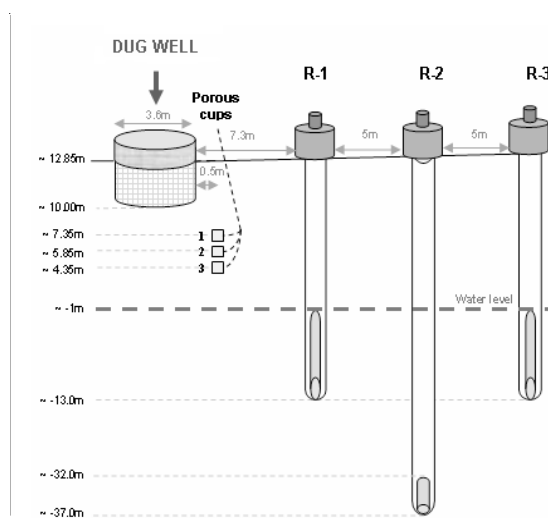
### INTRODUCTION

At the Shafdan site, secondary treated wastewater from the Tel Aviv area is recharged into an aquifer via a soil aquifer treatment (SAT) system with a retention time of 6-12 months. With an annual infiltration rate of 135 Mm<sup>3</sup> per year this SAT system is the biggest in the Middle East. The re-abstracted water is used for agricultural irrigation in the South of Israel. Water quality issues at the site are of great concern, especially redox related geochemical interactions since manganese

mobilisation has been observed recently on several occasions (Oren et al., 2007). The limits of the infiltration capacity using the existing infiltration basins are reached and in order to increase the recharged volume of water, a system of dug wells around the infiltration basins closer to the extraction wells, leading to a shorter residence time, is considered. The water infiltrated through the dug well would undergo ultra filtration as an extra treatment to guarantee the quality of recovered water and to compensate for the shorter residence time. Within the European RECLAIM WATER project, a pilot dug well site, including observation wells in the saturated zone and porous cups for water sampling in the unsaturated zone was set up in order to assess the performance and the resulting water quality in the system. A one year long infiltration test to monitor the water quality evolution has started. This paper describes the setup of the site, the hydrogeological modelling performed to characterise the flow regime in detail and the development of simple kinetic geochemical batch modelling to identify dominant redox reactions that can be attributed to the presence of dissolved organic matter (DOM) in the infiltrating water. In a final step both numerical models will be coupled and calibrated geochemically allowing for the description of the coupled flow and geochemical processes in detail. The results of the 1 year experiment on the dug well pilot and its modelling will, through up-scaling, contribute to the assessment of the viability of increasing infiltration rates at the Shafdan site using a system of multiple dug wells.

## **INSTALLATION OF THE DUG WELL SITE AND EXECUTION OF THE TRACER TESTS**

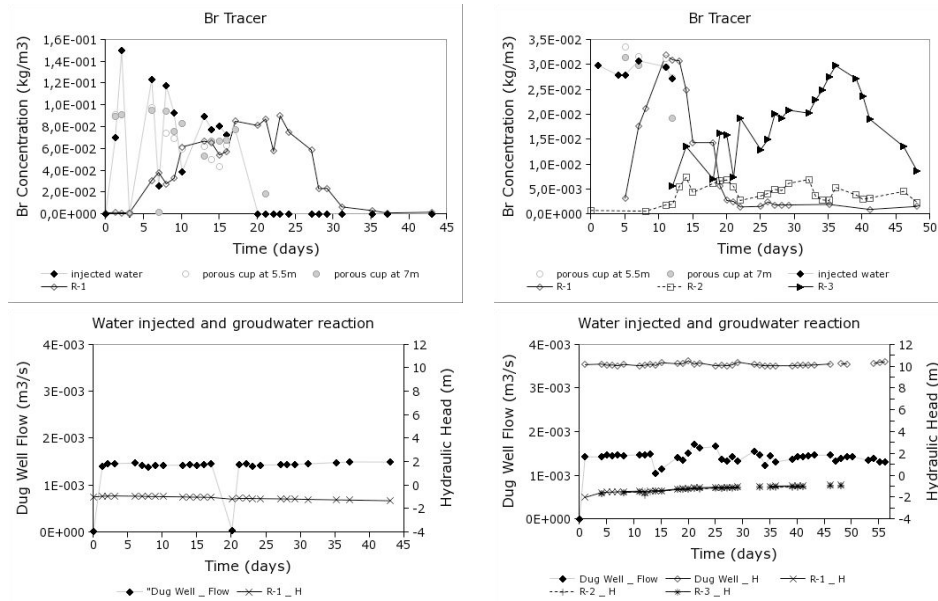
The installation of the dug well site consisted of a surface and a subsurface part. On the surface secondary treated effluents go through a 2 micron pre-filter and UV treatment to minimize fouling on the ultra filtration (UF) membrane. Then the effluent is filtered using the UF membrane and for a period of one year pumped into a dug well for infiltration through the unsaturated zone. The subsurface part consists of a dug well with a diameter of 3.5 m and a depth of 4 m. The dug well itself is constructed of a stainless steel framework of which the sides are covered with a textile to maximize the infiltration capacity. At 0.5 m from the dug well three porous cups are installed at various depths covering the thickness of the unsaturated zone (Figure 1). These porous cups allow water samples of the unsaturated zone to be taken (max 300 ml per sample). Downstream the regional groundwater flow three observation wells were installed in the saturated zone at distances between 7.3 to 17.3 m from the dug well. Two (R-1 and R-3) are screened at the same depth (14- 21m) at the top of the aquifer whereas the third (R-2) crosscuts a sandy clay layer and has a screen much deeper (deep of 46.3 51.3m).



**FIGURE 1. Installation of the dug well site for the 1-year infiltration test with secondary treated UF-filtered effluent aiming at a subsurface residence time between 40 and 60 days.**

Two tracer tests using KBr were performed to characterise the horizontal and vertical aquifer permeability and the residence time of the infiltrating water. A first one was conducted in September 2006 with one observation well (Figure 1: R-1) and a second one in February 2007 with three observation wells (Figure 1: R-1, R-2 and R-3). The tracer was mixed with tertiary treated effluent and infiltrated through the dug well. For the first tracer test the concentration averaged around 80 mg/L during the first 17 days. At the 18<sup>th</sup> day, the injection of the Br stopped, however infiltration of the water continued until day 43. The infiltrated flow through the dug well remained constant around approximately 5m<sup>3</sup>/h. Based on the results of the first tracer test and pre-dimensioning modelling R-2 and R-3 were installed to obtain an optimal residence time. For the second tracer test, starting in January 2007, the tracer concentration averaged around 28 mg/L during the first 11 days. At the 12<sup>th</sup> day, the injection of the Br stopped but infiltration continued until day 56 with an average infiltration rate of approximately 5.24 m<sup>3</sup>/h. Figure 2 represents the measured Br concentrations during the first (upper left) and the second (upper right) tracer test, while the lower graphs show the respective infiltrated flow and measured elevation of the water table. One can observe from the concentrations measured in the porous cups that the breakthrough in the unsaturated zone is almost instantaneously whereas it appears several days later in the observation wells R-1, R-2 and R-3 (saturated zone).





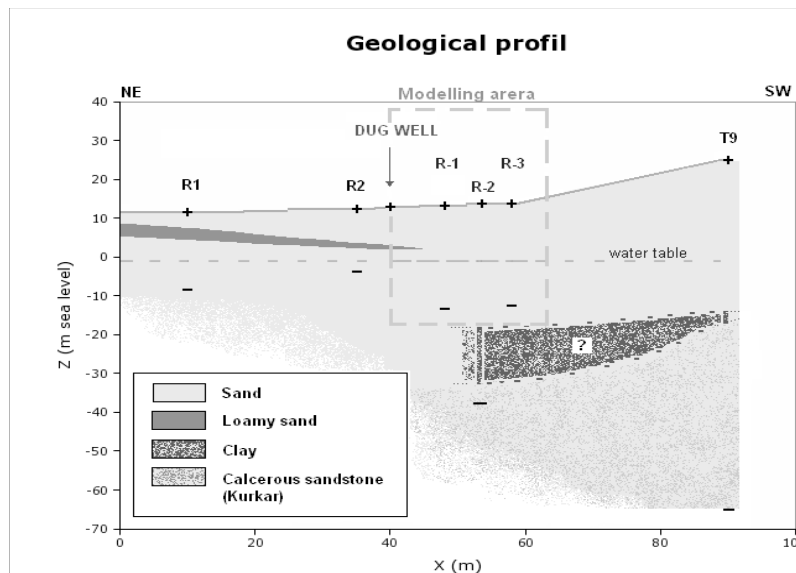
**FIGURE 2. Upper: Tracer concentrations in the dug well, the porous cups and the observation wells for the first (left) and the second tracer test (right). Lower: Measured infiltration rates and hydraulic heads during for the first (left) and the second tracer test (right).**

During the first tracer test the first appearance of Br in R-1 is 3-6 days after the infiltration started. Thus, the maximum velocity of the Br flow is  $3.5\text{-}2.10^{-5}\text{m/s}$ . Moreover, the top of the breakthrough is detected in R-1 after 16-18 days. The global velocity is thus  $6.5\text{-}6.10^{-6}\text{m/s}$ . For the second tracer test the first appearance of Br in R-1 occurs after 4-5 days after the injection started. Thus, the maximum velocity of the Br flow is  $2.5.10^{-5}\text{m/s}$ . Moreover, the top of the breakthrough is detected in R-1 after 11-12 days. The global velocity is thus  $9.10^{-6}\text{m/s}$ . In R-3, the first appearance is detected after 7-9 days with a maximum velocity around  $2,5\text{-}3.10^{-5}\text{m/s}$  and the top of the breakthrough at 36-37 days, the global velocity being  $6.10^{-6}\text{m/s}$ . Finally, the Br concentration seems to reach R-2 whereas it is located at great depth. Even if the vertical permeability was very high, and the intermediate clay layer was not continuous, this observation seems surprising.

## HYDROGEOLOGY AND CHARACTERISATION OF THE FLOW REGIME

A detailed geological description of the site was established and the tracer tests were interpreted using a radial flow model including both the saturated and the unsaturated zone.

A few auger holes (R1, R2), one observation well (T-9) and R-1, R-2, R-3, show subsurface characteristics which correspond to Quaternary alternating units as described in the cross section in Figure 3: sand, loamy sand, clay and calcareous sandstones (Kurkar). The studied site corresponds to the coastal sandstone aquifer of Israel (fresh water). The general flow direction of this aquifer is from east to west towards the Mediterranean Sea. However, locally there is a change in the flow direction as a result of the local pumping regimes. Based on piezometric maps, one could derive that on the site the gradient is very small and predominantly south west as a consequence of this pumping regime. Time series with groundwater level data exist for several piezometers. The aquifer has an unsaturated zone which has a large thickness (15m). The hydraulic conductivity of this coastal aquifer is reported to be between  $5.10^{-2}$  to  $5.10^{-3}$  m/s.



**FIGURE 3. Hydrogeological cross section through the wider area of the dug well pilot site.**

For the numerical modelling the MARTHE-code (Thi ry, 1993, 1994) was applied. MARTHE allows modelling density dependent flow and transport in saturated or

unsaturated zone using finite volumes discretisation in three dimensions. Radial symmetry was applied and proved a reasonable assumption since the regional groundwater gradient was very small as described above.

## Model Formulation

The first aim of this model is to represent the flow and the transport process of the aquifer at the pilot site and so to deduce the main hydrodynamic characteristics of the aquifer. This model is based on the two tracer tests (September 2006 and February 2007) and has to reproduce and predict the behaviour of conservative tracer (Br) from the dug well towards the three observation wells. The phenomena are modelled as a combination of two mechanisms: advection and dispersion in a porous unsaturated-saturated medium using Richards equation (1931) and Van Genuchten's retention model (1980). The flow and transport equations were solved using a radial axis-symmetric two dimensional grid, capable of representing the extension of the infiltrating tracer bubble. The transport scheme used is the Total Variation Diminishing scheme.

*Problem editing, domain and grid selection* - The simulation is performed in transient conditions during 45 days (second tracer test) with an initial time step between  $7.10^{-3}$  with 1 day. The porous medium is modelled as two homogeneous sand layers separated by a clay layer with at total depth at -47m sea level. The surface of the soil is defined at 12.85 m sea level. The aquifer is modelled by a domain with a  $500\text{m} \times 60\text{m}$  area cut into 3392 cells. The mesh of the domain is also conditioned by the observation points (position of observation wells R-1, R-2 and R-3 and porous cups) allowing the comparison between the measured data and the modelled data. The main input parameters for the flow are the measured infiltration rates in the dug well and the piezometric levels in the observation wells. Other parameters (hydraulic conductivity, porosity, retention parameters) are selected to represent a typical sandy aquifer with a clay layer initially (Table 1).

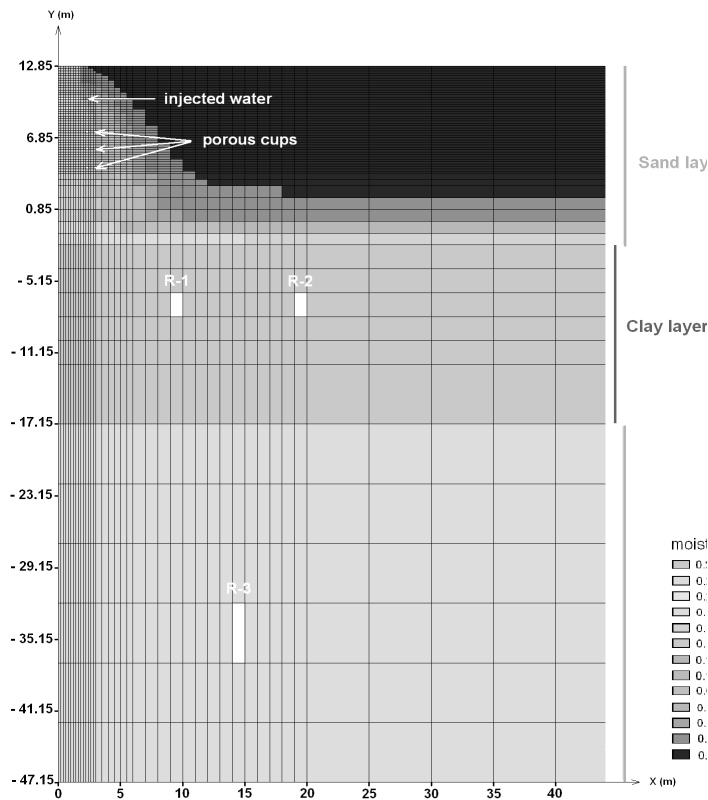
**Table 3: Data and parameters used for the hydrological modelling at the Shafdan dug well pilot.**

| Modelling parameters         |             | Retention Curves                                                                        |
|------------------------------|-------------|-----------------------------------------------------------------------------------------|
| Porosity (%)                 |             | Van Genuchten's model (1980)                                                            |
| - sand (unsaturated zone)    | 18          | $Se = 1 + (h_{rt})^{1/bt} \Big]^{n-1}$                                                  |
| - sand (saturated zone)      | 25          |                                                                                         |
| Hydraulic conductivity (m/s) |             | $\frac{K}{Ks} = \sqrt{Se} \left[ 1 - \left( 1 - (Se)^{1/n-bt} \right)^{1-bt} \right]^2$ |
| - sand                       | $9.10^{-5}$ | with:                                                                                   |
| - clay                       | $5.10^{-6}$ | $Se = \frac{\theta - \theta_r}{\theta_s - \theta_r}$                                    |
| Retention curve parameters   |             | $h_{rt} = \frac{h}{ht}$                                                                 |
| - ht (m)                     | 0.60        | $bt = \frac{1}{n} = 1 - m$                                                              |
| - bt                         | 0.40        | Mualem condition (1976)                                                                 |
| - $\theta_r$                 | 0.01        |                                                                                         |
| - $\theta_s$                 | 1.00        |                                                                                         |

*Initial and boundary conditions* - Initially the hydraulic head is fixed at -2m for the whole domain. Only an area which extends from the dug well to water table has an initial moisture content,  $\theta_i$ , equal to 10%. At the surface, the infiltrating water through the dug well is fixed at  $5.24\text{m}^3/\text{d}$  during the simulation. The outer boundary of the modelled domain ( $X=250\text{m}$ ) is defined as a constant hydraulic head at -2m. This represents the measured thickness of the unsaturated zone measured before the second tracer test in the observation wells, R-1, R-2 and R-3. In terms of transport, the initial Br concentration in the porous area is zero. The Br tracer is added to the injected water between the 1<sup>st</sup> and 11<sup>th</sup> day with a concentration of  $0.028\text{kg}/\text{m}^3$  (second tracer test) After the 11<sup>th</sup> day, the addition of Br in the water injected is stopped.

## Numerical Results

*Flow modelling* - Figure 4 shows the modelling grid, the observation points and the moisture content after 45 days of infiltration. While the dug well and observation well are fully saturated the porous cups remain unsaturated during the infiltration with a water saturation degree around 0.80 (near saturated state).

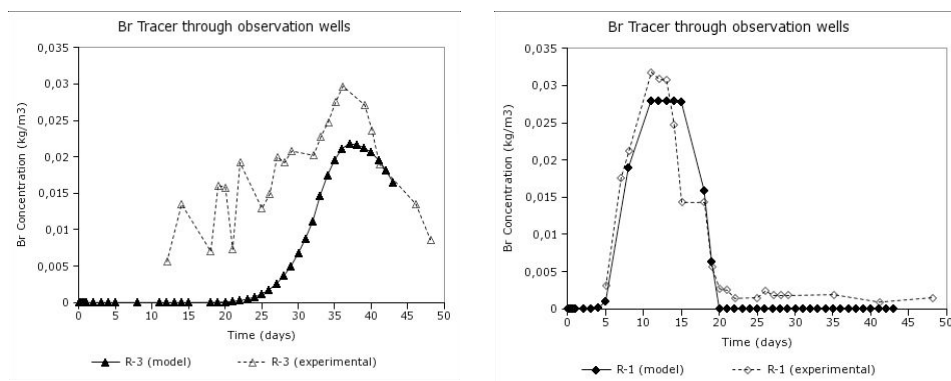


**FIGURE 4. Cross section (2D) of the MARTHE solute transport model grid showing the observation points and the moisture content at the dug well pilot after 45 days of infiltration.**

When comparing the simulated hydraulic heads to the measured hydraulic heads in observation wells R-1, R-2 and R-3 (not shown), and taking into account seasonal patterns from multi-annual piezometers, it becomes clear that the regional groundwater level increased during the tracer test. Most likely strong winter rainfalls in December and January 2007, lead to an increase in hydraulic head in the observation boreholes that is more pronounced than the one caused by the tracer test. However, since this is a regional change in heads, this does not have an impact of the flow regime induced by the infiltration in the dug well.

*Breakthrough (Br) results* - The simulated concentration is compared to the measured concentration in the observation wells R-1 and R-3 (Figure 5). While the fit for R-1 is very good, for R-3, the peak concentration is matched, but the maximum observed concentrations before the breakthrough are higher than the modelled concentrations. This suggests that the modelled properties are correct, but something happened with the background Br concentration in R3. The hypothesis is

that the winter rain infiltration remobilised trapped pore water of the first tracer test in the unsaturated zone towards the saturated zone and lead to a Br increase in R-3 before the Br of the second tracer-test arrived. Large measured concentration variations before the maximum breakthrough seem to suggest that this occurred in different large rainfall events. Modelling also confirmed that after the first tracer test Br tracer remains trapped in the unsaturated zone long after the tracer has passed in the saturated zone.



**FIGURE 5. Comparison between the simulated and measured tracer concentration (Br) during the second tracer-test in observation wells R-1 and R-3 using the MARTHE model.**

Optimal fit for the two tracer-tests was obtained when applying a uniform hydraulic conductivity of  $9.10^{-5}$  m/s for the top sand layer and assuming a porosity of 18% for the unsaturated zone and 25% for the saturated zone. A continuous clay layer with a low permeability (as observed for R-3) further optimised the fit. No anisotropy and dispersivity were introduced. The water saturation in the unsaturated zone below the dug well reaches a maximum of 80%. This calibrated flow and transport model will serve as a basis for the coupled flow and geochemical modelling.

## POTENTIAL REDOX PROCESSES INDUCED BY THE UF TREATED SECONDARY EFFLUENT FOR THE PILOT

In order to assess the potential redox impact of infiltrating UF secondary treated waste water into the pilot as well as the pilots DOM reduction capacity, geochemical modelling can be applied. Here simple batch modelling based on the composition of the infiltrating water and the simplified mineralogical composition (based on an averaged DRX analysis) of the sediment at the site is shown. This model will be coupled to the flow model described above and calibrated once results from the one

year infiltration test experiment become available. No calibration occurred so far and therefore timescales are arbitrary in this stage.

*Model development and data* - The PHREEQC2 code and the phreeqc.dat database (Parkhurst and Appelo, 1999) were used for the modelling. PHREEQC is designed to perform a wide variety of low-temperature aqueous geochemical calculations in contact with minerals both using equilibrium and kinetic geochemical interactions. Two different simulations were performed. The first simulation represents the reactions in the unsaturated zone. For this simulation a constant partial pressure of  $O_2$  corresponding to the atmospheric partial pressure was defined since  $O_2$  is assumed to be in excess in the unsaturated zone. Degradation of DOM occurs under the influence of aerobic bacteria. In this simulation oxidising conditions prevail. In the second simulation, the only oxygen present is the oxygen dissolved in the infiltrating water. This will be consumed very quickly and gradually a more reducing regime will develop. The second simulation represents the reactivity in the saturated zone and denitrifying bacteria trigger the DOM degradation.

**TABLE 4. Data and parameters used for the batch modelling of the redox interactions due to the degradation of dissolved organic matter (DOM ) at the Shafdan dug well pilot.**

| Composition of secondary treated UF water<br>(data from Mekorot 13-03-07) |         | Modelling parameters                   |                        |
|---------------------------------------------------------------------------|---------|----------------------------------------|------------------------|
| pH                                                                        | 7.57    | <i>Monod parameters</i>                |                        |
| pe ( $NO_3^-/NO_2^-$ )                                                    | 6.54    | $K_s$ (mol/L)                          | $7.64 \cdot 10^{-6xx}$ |
| $Ca^{2+}$ (ppm)                                                           | 81      | $K_b$ (mol/L)                          | $6.25 \cdot 10^{-6*}$  |
| $Mg^{2+}$ (ppm)                                                           | 30      | $Q_m$ (mol DOM/g cells/hr)             | $1.410^{-3*}$          |
| $Na^+$ (ppm)                                                              | 186     | $Y$ (g cells/mol DOM)                  | 200**                  |
| $K^+$ (ppm)                                                               | 24      | $B$ ( $hr^{-1}$ )                      | 0.00208*               |
| $Fe^{2+}$ (ppm)                                                           | 0.00033 | <i>Sediment composition (in %)</i>     |                        |
| $Mn^{2+}$ (ppm)                                                           | 0.00023 | Calcite ( $CaCO_3$ )                   | 10                     |
| $Cl^-$ (ppm)                                                              | 279     | Pyrolusite ( $MnO_2$ )                 | Traces                 |
| $HCO_3^-$ (ppm)                                                           | 348.63  | Ironhydroxide ( $FeOH_3$ )             | Traces                 |
| $SO_4^{2-}$ (ppm)                                                         | 93      | Quartz (not included)                  | 80                     |
| $NO_3^-$ (ppm)                                                            | 1       | Plagioclase, microcline (not included) | Traces                 |
| $NO_2^-$ (ppm)                                                            | 1.64    |                                        |                        |
| $PO_4^{3-}$ (ppm)                                                         | 2.9     |                                        |                        |
| $O_2$ (ppm)                                                               | 0.31    | $O_2$ partial pressure (atm)           | 0.2                    |
| DOM (ppm)                                                                 | 9.3     | Temperature ( $^{\circ}C$ )            | 25                     |

\* taken from Parkhurst and Appelo (1999) \*\*adapted for this study

In order to perform the modelling, the DOM species was added to the database and a generic chemical composition of  $\text{CH}_2\text{O}$  was assumed. The following multiplicative Monod rate expression, assuming that the DOM degradation is determined by microbial activity, is used to describe the rate of the DOM degradation:

$$R_{\text{DOM}} = -q_m X_m \left( \frac{c_{\text{DOM}}}{K_s + c_{\text{DOM}}} \right) \left( \frac{c_{\text{O}_2}}{K_a + c_{\text{O}_2}} \right) \left( \frac{c_{\text{NO}_3^-}}{K_a + c_{\text{NO}_3^-}} \right) \quad (\text{EQ 1})$$

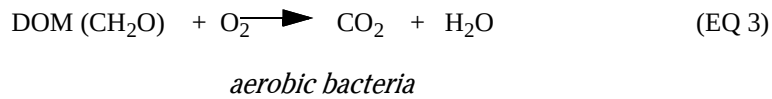
Where  $R_{\text{DOM}}$  is the rate of DOM degradation (mol/L/hr),  $q_m$  is the maximum specific rate of substrate utilization (mol/g cells/hr),  $X_m$  is the biomass (g cells/L),  $K_s$  is the half-saturation constant for the substrate DOM (mol/L),  $K_a$  is the half-saturation constant for the electron acceptor  $\text{O}_2$  (mol/L factor included in the unsaturated case only) and  $\text{NO}_3^-$  (mol/L factor included in the saturated case only), and  $c_i$  indicate concentration (mol/L). The rate of biomass production is dependent on the rate of substrate utilization and a first-order decay rate for the biomass:

$$R_{\text{Cells}} = -Y R_{\text{DOM}} - b X_m \quad (\text{EQ 2})$$

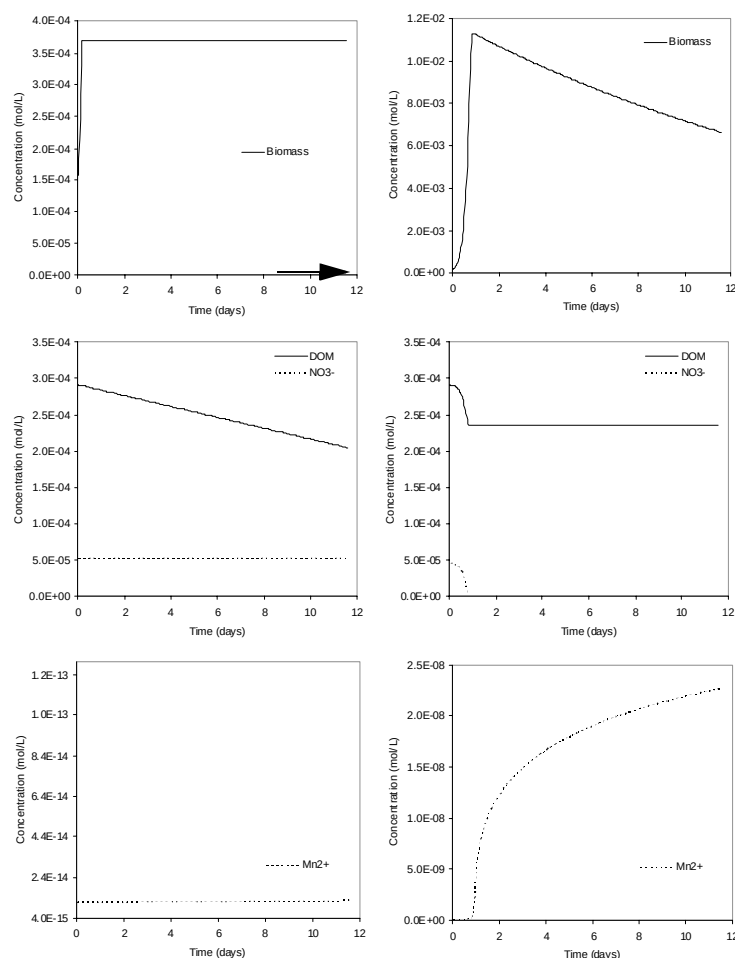
where  $R_{\text{Cells}}$  is the rate of cell growth (g cells/L/hr),  $Y$  is the microbial yield coefficient (g cells/mol DOM), and  $b$  is the first-order biomass decay coefficient ( $\text{hr}^{-1}$ ). The parameter values for these equations are listed in Table 2.

*Batch modelling results* - One can observe from the modelled results that the reactivity induced by the DOM in the infiltrating water is entirely different between the “unsaturated zone” case (Figure 6 - on the left) and the “saturated zone case” (Figure 6 - on the right).

In the “unsaturated zone” case the biomass increases quickly and reaches a plateau once growth and decay are neutralising each other. A significant amount of the DOM is consumed by the biomass having access to a constant supply of  $\text{O}_2$  (defined by its partial pressure). After 12 days still quite a bit of DOM is left but ultimately the decreasing concentration of the DOM will limit the DOM degradation rate (rate limitation by the donor concentration). Due to the presence of the  $\text{O}_2$ , the redox state of the infiltrating water does not change, the  $\text{NO}_3^-$  concentration remains constant and no metal mobilisation on the sediment takes place. The dominant reaction in the unsaturated zone can be summarised as:







**FIGURE 6. Batch modelling of the degradation of DOM using Monod kinetics using secondary UF treated wastewater in the saturated zone (right) and the unsaturated zone (left) taking into account a simplified mineralogy for the Shafdan sediment.**

In the “saturated zone” case the biomass grows fast as well, but does not reach a plateau. Its further growth is limited by the decreasing nitrate concentration. Very quickly the growth rate of the denitrifying bacteria is smaller than the decay rate and the biomass reduces again. During the growth phase some ( $0.5 \cdot 10^{-4}$  M) of DOM is consumed, but after that DOM reduction occurs at a near zero rate. In this case the DOM rate limitation is due to the receptor concentration ( $\text{NO}_3^-$ ). The dominant reaction in the saturated zone can be summarised as:



*denitrifying bacteria*

$\text{NO}_3^-$  disappears quickly since it reacts with the DOC, and this generates increasingly reducing conditions. Under these reducing conditions  $\text{MnO}_2$ , available in tiny amounts in the sediment, is no longer stable and starts to dissolve leading to a significant increase in Mn in the solution (in this example up to 0.00137 mg/L after 12 days). Reducing conditions are however not severe enough to mobilise  $\text{Fe}^{2+}$  also present in the sediment as  $\text{Fe(OH)}_3$  (not shown). The amount of  $\text{Mn}^{2+}$  released in the saturated zone is thus directly related to the amount of DOM consumed. However, because DOM is present abundantly this consumption seems unimportant. Because of the low drinking water limits for  $\text{Mn}^{2+}$  (Guide level 0.02 mg/l) a similar increase in  $\text{Mn}^{2+}$  can become of capital importance.

## CONCLUSIONS AND OUTLOOK

A dug well pilot site for testing the infiltration of UF treated secondary effluent in a SAT system was installed successfully with sampling points in both the saturated and the unsaturated zone. Tracer-tests and pre-dimensioning modelling allowed the site being developed to such extent that a radial flow pattern can be established and the recovered water quality can be monitored after a target residence time between 40 and 60 days.

These tracer tests using KBr were successfully used to calibrate a solute transport model that comprised both the saturated and the unsaturated zone. Based on a infiltration rate of  $5\text{m}^3/\text{h}$  through the dug well the flow and the tracer breakthrough are correctly represented if the model includes a continuous clay layer with a low hydraulic conductivity at medium depth. The water infiltration is quick through unsaturated zone which is almost saturated (saturation degree of 0.80). An optimal fit for the two tracer-tests was obtained when applying a uniform hydraulic conductivity of  $9.10^{-5}\text{m/s}$  for the top sand layer and assuming a porosity of 18% for the unsaturated zone and 25% for the saturated zone.

The potential importance of redox processes and subsequent water quality changes, due to the fact that the UF treated secondary effluent contains a slightly elevated concentration of dissolved organic matter (DOM), is explored using geochemical batch modelling based on bacterial driven degradation. Two cases representing the saturated zone on one hand and the unsaturated zone on the other hand are described. Results indicate that while in the unsaturated zone a large potential for degradation of organic matter exists due to the availability of  $\text{O}_2$  in the saturated zone, degradation becomes much less important and can potentially induce reducing

conditions thereby mobilising manganese that is present in tiny amount in the sediment as  $\text{MnO}_2$ . Calibration of the model is necessary to determine the time-frame and the total impact of these redox reactions.

The results of both models are to be fed into a coupled flow and geochemical model (TOUGHREACT) that will be calibrated during the ongoing one year infiltration test with ultra-filtrated secondary treated wastewater. This will allow for the reconstruction of the dominant redox reactions in the pilot system and the prediction of the behaviour of contaminants that might still be present in the infiltrated water. Laboratory experiments, characterising a selection of these contaminants will provide the necessary parameters for constraining these models. This will lead to a general appreciation of the recovered water quality and the assessment of the sensitivity of the system with respect to infiltration rates and infiltrated water quality.

## ACKNOWLEDGEMENTS

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## MASTER PLANNING RECHARGE FACILITIES TO PROVIDE FLEXIBILITY FOR THE FUTURE

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### ABSTRACT

When designing recharge facilities, the primary technical uncertainty is determining the long-term recharge potential of a site. A thorough hydrogeologic investigation can help manage this uncertainty. However, the actual performance of a site can not be known until operations begin and significant capital investments have been made. Utility managers should be concerned that interim decisions made during design do not preclude future options to expand the recharge and/or recovery capabilities of a potentially evolving facility. Tucson Water has learned a number of lessons over the years about what to do (and not do) when implementing recharge and recovery projects. Tucson Water planners and engineers seek to maximize flexibility by developing a conceptual master plan for the site. Considerations include: 1) Ensuring that the facility is designed to accommodate potential expansion; 2) Investing up front to provide sufficient capacity in critical raw and recovered water delivery infrastructure to minimize future expenses; and 3) Constructing the minimal initial recharge basin acreage to reduce near-term capital costs while making provisions for additional basin acreage if it is needed based on demonstrated facility performance. These flexibility factors help manage overall facility costs by allowing the Utility to develop the site over time reducing both the uncertainty and the associated risk.

### KEYWORDS

Resource management, planning, recharge, recovery, design

### INTRODUCTION

The City of Tucson is located in a semi-arid region of the Sonoran Desert in southeastern Arizona. Local surface water supplies are almost non-existent; most rivers that contain perennial flow are effluent-dominated streams located downstream from municipal wastewater treatment plants. Until the early 1990s, the Tucson community relied almost exclusively on local groundwater resources to meet water demand. Due to rapid growth in population and associated water demand

following World War II, the groundwater system transitioned from an approximate state of equilibrium to one of accelerating depletion. Despite the successful implementation of water conservation programs and the “desert landscape” ethic of Tucson residents, groundwater withdrawals for municipal use continued to increase through the year 2000. Rapidly declining water levels in the metropolitan and surrounding areas have resulted in land subsidence, increased pumping costs, and the gradual loss of native riparian habitat.

Tucson Water’s need to develop renewable water supplies to reduce reliance on mined groundwater and to meet future demand is a critical goal of *Water Plan: 2000-2050* (Tucson Water, 2004). The City of Tucson has two available renewable water resources which will be increasingly utilized in order to satisfy projected demand: imported Colorado River water delivered through the Central Arizona Project and locally-generated municipal wastewater. Recharge and recovery will continue to be a critical water resource utilization strategy for these renewable source waters.

Tucson Water has operated large-scale recharge facilities for over 20 years and about 50% of potable water supply and up to 75% of reclaimed water supply is currently managed through surface-spreading recharge facilities and wet-water recovery. The use of recharge technologies has provided a number of benefits to the Utility including water quality improvements (*via* soil aquifer treatment), long-term storage, seasonal storage and recovery, supply reliability, enhanced drought tolerance, and public acceptance of the use of renewable water resources (Thomure and Marra, 2005). One of the critical lessons learned over time is the need to consider uncertainty and future flexibility when conceptualizing and designing large-scale recharge and recovery facilities.

In the recharge field, the primary design uncertainty is the ultimate recharge potential of a potential site. A thorough hydrogeologic investigation is key to managing this uncertainty; however, even the best investigative techniques can only provide an approximate indication of the long-term recharge potential of a prospective site. The only way to truly determine what is practically possible is to put water on the ground. This can require significant up front capital investment and may involve significant financial risk if the design lacks flexibility. It is critical that the decisions made during design do not preclude achieving the full potential of the facility should there be interest in expanding recharge and/or recovery capacities in the future.

To manage the uncertainty and the associated risk, Tucson Water explicitly considers how flexibility can be built into each project’s conceptual master plan. Considerations include: 1) Ensuring that the facility is designed to accommodate recovery components and future expansion; 2) Investing up front to provide sufficient capacity in the raw and recovered water delivery infrastructure so that potential long-term goals can be met without having to expensively redo previous work; and 3) Constructing the minimal initial recharge basin acreage to reduce the

near-term capital cost while making provisions for additional basin acreage if and when it is needed based on actual facility performance. These flexibility factors help manage both the uncertainty and the initial and long-term costs of such facilities by allowing the Utility to develop projects over time in a low-risk logical progression. Lessons learned from past decisions that did not fully consider future flexibility have helped shape how Tucson Water currently plans its future large-scale facilities.

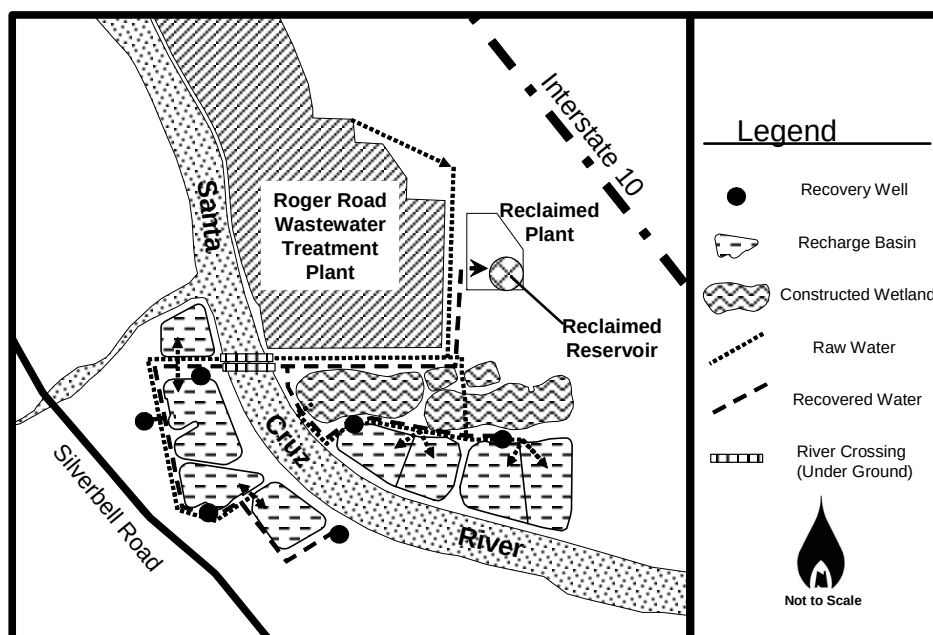
## **KEY EXISTING TUCSON WATER RECHARGE FACILITIES**

Tucson Water has practical experience in designing, constructing, and operating large-scale recharge facilities. To illustrate some of the more important lessons learned over time, this paper draws examples from two main projects: the Sweetwater Recharge Facilities (effluent recharge and recovery for non-potable uses) and the Central Avra Valley Storage and Recovery Project (a component of the Clearwater Program designed to maximize utilization of Colorado River water for potable supply).

### **Sweetwater Recharge Facilities**

In the early 1980s, Tucson Water constructed one of the first reclaimed water systems in the country. The system began operation with 10 miles of pipeline and only one customer – a destination resort golf course. Since then, the system has grown to include 150 miles of transmission pipelines and serves multiple golf courses, parks, schools, industrial sites, and homes. Tucson Water's reclaimed water system remains an industry leader and meets approximately eight percent of Tucson Water's total water demand. This reuse of wastewater effluent reduces groundwater pumping and conserves higher quality water sources for potable supply.

The Sweetwater Recharge Facilities are one of the core reclaimed water production facilities for Tucson's reclaimed water system and are located on the northwest side of the City of Tucson along the Santa Cruz River (Figure 1). These facilities are composed of eight surface-spreading recharge basins totaling about 11.3 hectares (28 acres), a constructed wetland treatment system, and a recovery well field. This project is currently permitted to recharge and recover up to 8 million m<sup>3</sup> per year (6,500 acre-feet per year) of reclaimed water with a planned potential expansion to 16 million m<sup>3</sup> per year (13,000 acre-feet per year).



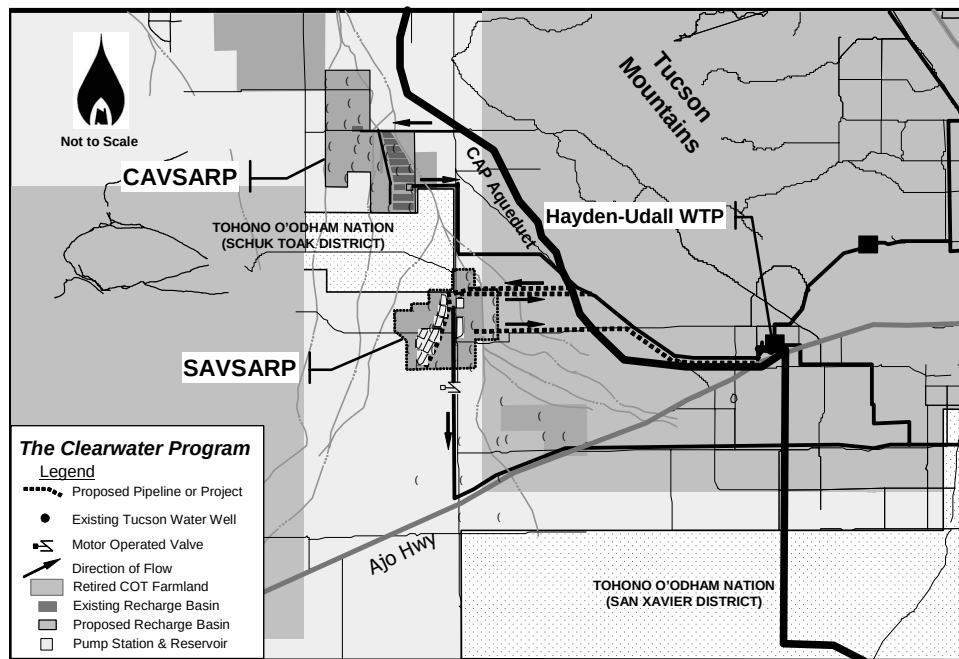
**FIGURE 1. Sweetwater Recharge Facilities.**

### Central Avra Valley Storage and Recovery Project and the Clearwater Program

The initial plan for using Colorado River water in Tucson in the early 1990s was similar to programs being implemented by other water providers. A large conventional surface water treatment facility, the Hayden-Udall Water Treatment Plant (WTP), was constructed adjacent to the Central Arizona Project canal. From 1992 to 1994, direct deliveries of Colorado River water through the Hayden-Udall WTP satisfied the potable demands of approximately 50 percent of the Utility's customers formerly served by Tucson Water's well fields which tapped the regional aquifers. However, a series of operational problems besieged Tucson's Colorado River water delivery system. Customer complaints of rust-colored water, taste and odor concerns, and damage to household plumbing and appliances were widely profiled. These issues ultimately resulted in the passage of a local ordinance in 1995 which prevented Tucson Water from continuing direct delivery of Colorado River water unless certain conditions were met (Johnson et al, 2003).

In order to utilize Colorado River water while complying with the legal constraints, Tucson Water constructed a large-scale underground storage and recovery facility in central Avra Valley. The Central Avra Valley Storage and Recovery Project (CAVSARP) is used to annually recharge up to 100 million m<sup>3</sup> (80,000 acre-feet) and

recover up to 86 million m<sup>3</sup> (70,000 acre-feet) of Colorado River water. The facility consists of 11 recharge basins totaling 130 hectares (318 acres), 27 high-capacity recovery wells, a reservoir and booster station, and approximately 40 km (25 miles) of pipeline. Through recharge and recovery, Colorado River water mixes with local groundwater to produce a blended water supply. CAVSARP and the Hayden-Udall WTP currently form the core of the Clearwater Program (Figure 2).



**FIGURE 2. The Clearwater Program.**

## LESSONS LEARNED IN DESIGNING RECHARGE AND RECOVERY FACILITIES

The most critical design lessons learned over time can be described in three topical categories: 1) Facility Build-out; 2) Pipeline Infrastructure; and 3) Recharge Capacity.

### Facility Build-out

Recharge facilities are large capital investments that should be designed to achieve the highest potential benefit to justify their cost. A flexible site master plan that considers not just near-term goals but also a wide range of future uncertainties and sizing potentials can help maximize the facility's return on the Utility's investment.



Such design considerations might include how the facility may evolve over time due to changes in how it may be operated, potential changes in surrounding land uses which may encroach on the site, and potential pressure for facility expansion(s) to meet projected future supply needs. In order to address such potential uncertainties, a site master plan should be developed to conceptualize all facility infrastructure components from an initial phase through potential build-out. Planners and design engineers need to look beyond initial near-term project targets to potential longer-term needs and put rudiments of the necessary infrastructure in place to accommodate those potential needs.

### **Issues for Consideration**

Tucson Water develops recharge projects that have integral wet-water recovery components. In contrast, many recharge projects in the southwestern United States are primarily long-term banking facilities where recovery is not expected to occur for years or even decades after constructing the recharge facilities. Even if recovery is not an initial goal, providing such considerations up front can ensure smooth implementation. Near-term design decisions are often made for the convenience of raw water delivery and recharge infrastructure needs. When such decisions are made without regard to the needs of eventual recovery systems, it can be more difficult to develop recovery infrastructure at a future date. By including recovery components at the master planning level, designers can anticipate a situation where pumping facilities become a natural extension of the recharge project rather than an engineering afterthought to be designed later in time. Recovery considerations include on-site spatial needs (reservoirs, booster stations, treatment/disinfection facilities, etc), pipeline corridors, power infrastructure, and future system hydraulics.

In many parts of the world, especially in fast-growing areas such as the urban centers in the southwestern United States, development pressure is a constant companion. Recharge projects are often developed to bring additional water supplies into use or provide enhanced resource reliability for existing supplies in fast-growing regions. Urban areas tend to spatially spread over time and often will encroach on what were previously rural, out-of-the-way locations. Planners need to consider potential impacts to and from current and future neighbors of a recharge facility. Questions to be addressed include whether recovery operations might be constrained by neighboring wells, how noise concerns will be factored into facility lay-out decisions, and whether currently available land should be purchased and preserved for future infrastructure needs. By considering the potential impacts of increased development near proposed recharge sites, resource managers can develop projects that are neighborhood friendly while preserving the opportunity to meet the future water supply needs of the community.

Finally, the site master plan should reflect the ultimate potential capacity for the recharge site. While considerable attention must be paid to achieving the initial project goals, it is valuable to plan the near-term components within the context of the planner's vision of complete build-out. It is a rare situation in the Southwest when utility managers can state with certainty that they have a complete understanding of the future water supply needs of their community. Many situations can arise over time that could affect future capacity needs. Significant community growth can create a demand for expanded infrastructure. The acquisition of additional renewable water supplies requires infrastructure to put it into use. Community preferences can change over time. Neighboring water providers may be willing to invest in the expansion of existing recharge facilities to bring their supplies into use through wheeling arrangements.

### **Experiences from CAVSARP**

Two examples of how a build-out master plan would have been beneficial can be drawn from Tucson Water's experiences at CAVSARP. When the CAVSARP site was initially constructed, it was based on a fixed view of project capacity. The ultimate potential of the site was not considered throughout the design process; rather, the project team became narrowly focused on achieving the initial project goals. This factor drove decision-making as the project design progressed. As a result, millions of dollars of infrastructure was constructed that adequately satisfied near-term recharge and recovery goals, but made it very difficult and costly to expand certain critical design elements. As Tucson Water brings more of its Colorado River water allocation into use, the CAVSARP site is limited in how it might be developed. Utility planners and engineers have crafted some creative ways to marginally expand the facility at reasonable costs; however, the full potential of the site would only be realized by virtually duplicating the initial capital investment. By considering future uncertainty through the development of a build-out master plan and investing adequately in the development of the initial phase for potential expansion, the full capacity of CAVSARP would be less costly to achieve.

Another issue with the development of CAVSARP was that the three, 8-hectare (20-acre) recharge basins that were constructed as part of the pilot phase were not designed to be directly adapted into the permanent facility. The pilot basins as well as their delivery pipelines, controls, and inlet structures were completely re-worked as the permanent facility was constructed. In retrospect, these additional costs could have been avoided if the pilot basins had been designed from the start to be incorporated into the permanent facility. Project components should be designed with consideration of how they can remain viable even as the facility expands over time.

## **Pipeline Infrastructure**

One of the most common phrases currently used in Tucson Water planning discussions is: “We’ve never built a pipeline that was too big.” It is often followed by the corollary: “Almost every pipe we’ve ever built is too small – eventually.” These statements tend to hold true in fast-growing communities. Practical experience suggests that consideration of these statements during the planning phase of a recharge project can help preserve, rather than limit, future flexibility. While not all pipelines and related infrastructure need to be over-sized to accommodate future expansions, key linkages that have significant construction, permitting, or coordination challenges could be designed to accommodate potential build-out. The marginal costs of over-sizing such components during the initial phase of project development should be weighed against the future costs and difficulty of expanding the facility if such investments are not made initially.

## **Experience from Sweetwater**

At the Sweetwater Recharge Facilities, Tucson Water is fast approaching certain infrastructure constraints that could have been alleviated when the facility was first constructed. The constraints relate to facility pipelines that cross beneath the Santa Cruz River. Both a raw water delivery pipeline and a separate recovered water pipeline were installed beneath the Santa Cruz River at relatively large expense. As shown on Figure 1, these pipelines are the critical connections between the west bank recharge basins and the recharge water source (Roger Road Wastewater Treatment Plant) as well as the west bank recovery wells and the reclaimed water delivery system (originating at the Reclaimed Water Treatment Plant). All current and future recharge and recovery that could potentially occur on the west side of the Santa Cruz River is entirely dependent and constrained by the size of these pipelines. Replacing these lines with larger diameter mains or installing additional parallel pipelines under the river channel are both prohibitively expensive options. Therefore, their initial diameters effectively capped the build-out potential of the western half of the project. Installing larger diameter pipelines at the outset would have provided significant expansion potential with a minor additional capital investment.

## **Experience from CAVSARP**

A similar lesson can be drawn from CAVSARP. As noted previously, this recharge project was implemented with a narrow focus on initial recharge and recovery capacity targets with little thought given to the potential for significant expansion. As such, decisions that were made during the design process tended to erode project flexibility in favor of near-term capital cost savings. For instance, the sizing of the Central Arizona Project raw water turnout was originally conceived for a large diversion volume. However, as the design progressed, the turnout was downsized to

be only equivalent to the initial project delivery target to save near-term capital costs. The diameter of the raw water pipeline was similarly downsized. No provisions were made for potential expansion of the raw water delivery infrastructure. These actions have effectively limited the ultimate recharge capacity of the project unless additional capital investments on the order of the original expenditures are made. Aside from the significant future cost impacts, expansion of the turnout structure would bear a significant permitting and coordination burden with one or more agencies. Tucson Water could have made the most of its initial investment in the turnout by sizing it for potential build-out at minimal increased costs while avoiding having to repeat the lengthy permitting and coordination process to tie-into the Central Arizona Project infrastructure.

### **Recharge Capacity**

Basic engineering principles can be readily applied to most aspects of a recharge project – the geometry of a reservoir determines its capacity, pump curves almost always adequately reflect real-world performance, and designers can accurately project pipeline carrying capacities. The actual performance of the primary recharge systems (basins or wells) is much more difficult to predict, however. A thorough hydrogeologic investigation is key to managing this uncertainty. However, even the best investigative techniques can only provide an approximate indication of the recharge potential for a site. The only way to truly determine what is practically possible is to put water into the ground.

Projecting surface-spreading infiltration rates, well injections rates, or hydraulic loading rates become a key design sizing factor when developing recharge sites; the range of the projected magnitudes can be uncertain. One common reaction is to apply conservative assumptions and design sufficient recharge surface area in the initial project to be absolutely certain they can meet the stated project goals. This can result in recharge projects where the constructed recharge capacity far exceeds the initial design targets and the ability of the raw water delivery infrastructure to optimize delivery rates. CAVSARP is an example where the attained surface infiltration rates are almost three times the assumed design standard. While this is in ways fortuitous, it does create a mismatch with the installed raw water delivery infrastructure elements.

An alternative approach is to build the initial recharge capacity based on realistic (even slightly optimistic) recharge assumptions within the context of a facility master plan that reserves sufficient acreage to facilitate the construction of additional recharge basins if needed. This action creates the possibility to save money in the near-term while achieving the project goal in most instances. In the rare instance where actual recharge rates are on the low-end of expectation, the added cost of building additional recharge capacity is only driven by the inflation and other cost escalations that occur over a couple of years. Stated differently, in a situation

where limited funding is available, it is preferable to invest more initial money into the project components that are very difficult to correct (turnout, pipelines) than the recharge components themselves.

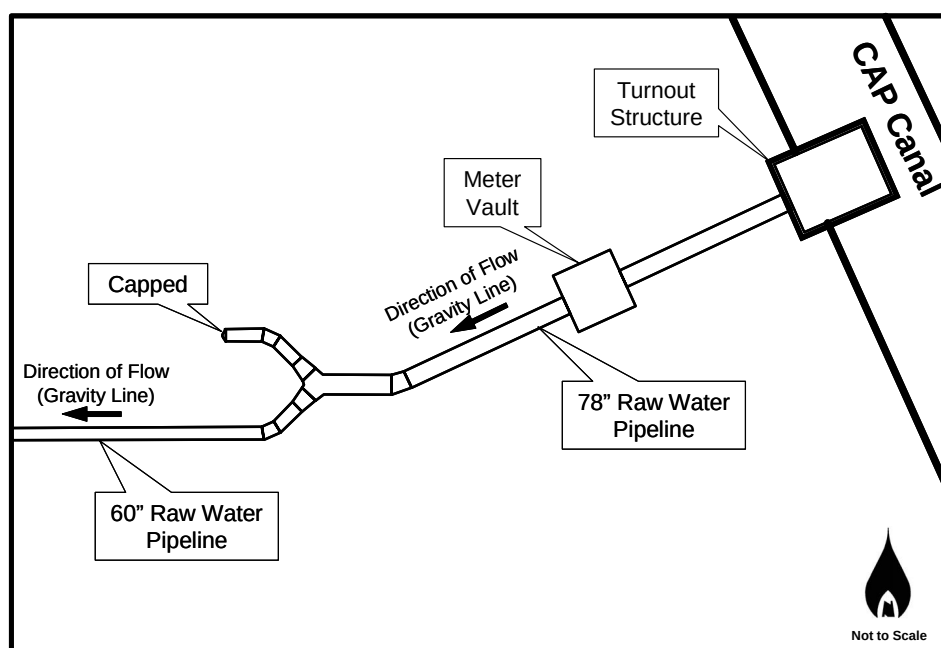
## **PUTTING THE LESSONS TO WORK**

Tucson Water is applying its lessons learned to the development of its newest recharge facility, the Southern Avra Valley Storage and Recovery Project (SAVSARP). This facility is an additional component of the Clearwater Program (Figure 2) and will go into operation in 2008. The facility's initial target capacities are to annually recharge and recover up to 74 million m<sup>3</sup> (60,000 acre-feet) of Colorado River water. However, the master planning of the facility was based on a potential build-out capacity of 173 million m<sup>3</sup> per year (140,000 acre-feet per year) of recharge with up to 123 million m<sup>3</sup> per year (100,000 acre-feet per year) of recovery.

To ensure that potential build-out targets were kept prominently under consideration throughout the initial project development, Tucson Water and its primary project consultants (Malcolm Pirnie, Inc. and CH2M Hill) included a comprehensive master planning effort in the scope of services for the SAVSARP raw water and recharge basin design contract. This effort included a series of design workshops which focused on developing a conceptual layout for the potential build-out facility (including all components of recharge and recovery) as a framework within which the initial project components must fit. While all facilities included in the build-out master plan might not be built, this exercise helped to ensure that the near-term development of the facility did not preclude future expansion.

Tucson Water analyzed its options for sizing the SAVSARP raw water delivery infrastructure based on cost and future flexibility considerations. In order to preserve long-term build-out options while minimizing initial project costs, a manifold raw water pipeline structure was developed as shown on Figure 3. The cost of constructing the turnout structure and meter vault for the potential build-out flow rate was only marginally higher than the cost to serve only the initial flow rate. In addition, if these structures were sized for the initial flow rate in the near-term, achieving the build-out flow rate in the future would come with higher risk and significant additional costs. Therefore, it was decided to build the turnout and meter vault for the build-out flow rate of 173 million m<sup>3</sup> per year (140,000 acre-feet per year). However, the cost of also sizing the entire raw water delivery pipeline to the build-out volume was significantly higher than that required for the initial flow rate. The manifold structure is designed to accept the potential full build-out flow from the turnout to feed an initial raw water pipeline sized for the initial flow with a blind flange that would connect to a future pipeline to carry the additional flow necessary to achieve build-out, if necessary. The easements acquired for the SAVSARP raw

water system were designed to include sufficient room to accommodate both the initial and potential future raw water pipelines.



**FIGURE 3. SAVSARP Raw Water Components.**

The number and total area of surface spreading recharge basins that will be built for SAVSARP also were partially driven by past experience. The hydrogeologic investigations conducted at the site have yielded somewhat mixed results. While surface infiltration rates throughout much of the site are quite high, there exists a fairly significant potential fine-grained layer at a relatively shallow depth of 15 to 20 m (about 50 to 70 feet) below land surface. The impacts that this fine-grained layer will have on recharge performance will not be known until the facility goes into operation. A conservative view of the initial facility design would yield an expectation that the nine planned recharge basins totaling 89 hectares (220 acres) might only achieve a recharge rate of 56 million m<sup>3</sup> per year (45,000 acre-feet per year). However, if one utilizes more optimistic (but nonetheless realistic) assumptions, the initial recharge volume target of 74 million m<sup>3</sup> per year (60,000 acre-feet per year) may be achievable. To save initial costs without sacrificing the ability to meet the project goals, staff decided to build nine basins initially based on the more optimistic assumptions, operate the facility for a period of time, and determine if additional basins are required. Space has been reserved for additional basins under the site master plan and all other required delivery infrastructure will

be in place to accommodate potential future recharge expansion. To further expedite the addition of more basins, the application filed with the Arizona Department of Water Resources to permit the SAVSARP facility included such contingency basins should they be needed.

Finally, to ensure that the recovery capacity of SAVSARP does not become constrained (and/or to possibly accommodate expanded recovery from the neighboring CAVSARP facility), the SAVSARP recovered water transmission main is being designed for 123 million m<sup>3</sup> per year (100,000 acre-feet per year). This lengthy pipeline is difficult to route due to significant easement issues and utility conflicts; in addition, it must be sized to accommodate future options. When constructed, the SAVSARP recovery pipeline will not only serve the initial needs of SAVSARP but will also allow for facility expansion and/or increased system reliability by providing an alternate route for conveying water recovered at CAVSARP to the Tucson community.

## SUMMARY

Tucson Water has over 20 years of experience in developing and operating recharge facilities. Over the years, the Utility has learned a number of valuable lessons that it is currently applying when designing projects. A comprehensive master plan for a large-scale recharge project can help prevent near-term actions from constraining the achievement of potential future goals and/or adding significant costs to future needs. By devoting time in the planning phase to evaluate the potential build-out future of the site, near-term decision-making can occur with a higher degree of confidence while reserving the potential for expanding the facility to its ultimate potential. In addition, by considering the potential long-term effects of near-term actions, future flexibility can be preserved at minimal costs.

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## INJECTION WELL HYDRAULIC PERFORMANCE AND CLOGGING DETECTION

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### INTRODUCTION

Over the past 20 years, the application of groundwater storage technology using direct well injection has expanded greatly. With many agencies across the world striving to put these systems together and make them work, a flood of information on the technology has been developed for the potable water industry. In 2002, the United States of America National Academy of Science held a brainstorming session with the nations leading experts from industry to better understand the application and spread of this technology. It was quickly recognized that the success in the application of the technology was very sensitive to key components in order to maintain a Sustainable Underground Storage (SUS) program. Failure in any of these simple hydraulic concepts resulted in ultimate system failure and loss of confidence in the technology.

In 2005, the American Water Works Association Research Foundation (AWWARF) contracted with ASR Systems LLC (ASRS) to develop a national guideline for the Design, Installation, Operation, and Maintenance of Sustainable Underground Storage Facilities. Under the Project 3034, the scope focuses on documenting the successful SUS practices from across the world, for a variety of geologic settings, in order to provide guidance to other agencies pursuing the SUS technology. This information on direct well, vadose, and infiltration basin recharge will hopefully increase the success of each technology application and acceptance to the communities, as well as, help entice additional water agencies to pursue the technology through better understanding.

The field research for the project yielded a pile of great information that would otherwise have been kept local to the facility originating the concept. It also reviled struggling system operators in numerous other non-potable water disciplines utilizing groundwater injection concepts. Theses applications included salt-water intrusion barrier systems, contaminated groundwater remediation systems, treated sewage effluent disposal systems, and flood water control management systems. Many of these agencies are not connected to the local water purveyors and do not communicate on the same technical concourse. In short, these application groups have missed out on the technical advances of the potable water agencies.



This report will focus on some of the key technical flaws observed in the direct well injection technology. Many system designers consider non-potable injection wells as simple drain holes requiring less rigorous construction than wells pumping water. The sub-par construction is then accompanied by operation methods that entrain air, resulting in well failures within one year of operation startup. In many cases, no baseline injection performance data or operation data is available to determine the well performance factors related to normal hydraulic performance versus wellbore clogging. In one way or another, the key technical flaws are all relate to some basic understanding of injection well hydraulics.

## WELL CONSTRUCTION HYDRAULICS

Many of the non-potable water industry injection wells were constructed undersized, partially penetrated the aquifer, and contained perforations above the water table. The required (design) injection performance for the wells was found to be as great as 3-times the recovery rate of the largest pumping equipment that could be placed in the well. This type of well design resulted in well damage during operation that could not be reversed. The following sections will discuss the key lessons learned in the sizing and screen design of injections wells.

*Well Size:* The diameter of the well casing should be based upon the largest size well development pump necessary plus any specialty equipment to be installed with the injection column. The development pump should yield 125% of the nominal safe well production yield to stabilize the filter pack allowing sand free production at the lower design discharge rate. The pump should be sized to discharge nearly twice the injection rate. This provides energy to remove well plugging particles during back-flushing at twice the force that placed them into the well. These pumps may require shrouds for submersible motor cooling, increasing the overall size of the pumping unit. Therefore, in sizing the well casing, the required development pump dimensions and operational constraints should be carefully considered.

In nearly all injection well applications, a recovery or back-flush pump is required to be placed in the well to maintain performance sustainability from year to year. In many applications, a submersible pump is used, requiring a borehole flow control valve. These specialty valves allow injection into the well casing from the pump column, below the water table, from a point above the pump bowls. These valves require shrouds to prevent formation jetting or screen damage. The diameter of this valve, plus the pump power cable and any access tubes that need to pass by this valve body, may define the limits for the well casing size.

Many proponents of injection through wells are looking at a basic design philosophy of limiting the injection water flow in the casing to a 1 foot/second travel velocity. This is to reduce the ability of the water to carry air and particulate matter down the water column and into the aquifer. In addition, the approach sets up the condition

of surplus capacity that can be temporarily tapped during back-flush events of other wells, well maintenance, and emergency capture operations during storm or flood water events.

*Well Design:* The design depth of the well and location of the screens can also have substantial impacts on the well's injection performance. Many direct injection wells are completed to lesser depths than typical groundwater production wells, reducing the overall injection yield of the well. These partial penetration wells are common in the groundwater remediation industry. At these locations, the recovery wells are shallow and typically penetrate only the top of the aquifer, allowing focused removal of contaminants near the water table surface. The injection wells are designed to the same depth as the recovery wells. This limits the injection yield of the well and creates a tight operation window. Increasing the injection well depth similar to typical water production wells in the area can result in greater injection capacity and operation flexibility.

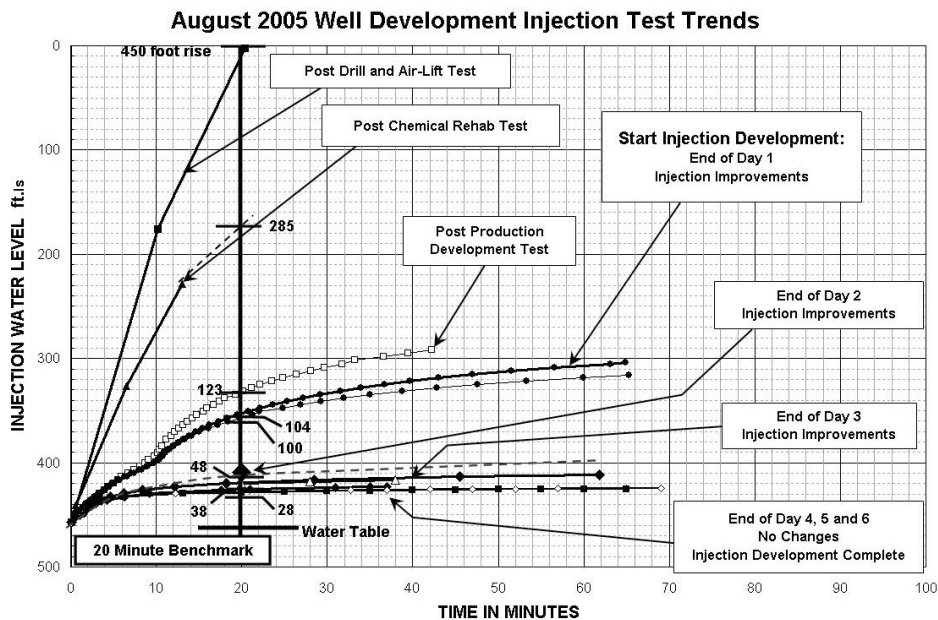
Many believe that screening the injection well above the water table will get more injection capacity from the well without increasing well depth. Others claim this to be their future saturation zone in their groundwater banking program. These are misguided thoughts that can be troublesome during future operation. All injection wells will require back-flushing to maintain sustainable performance. Screens above the water table are not cleared during back-flush events. Well screen near the water table will be exposed during back-flushing or recovery pumping events, creating cascading water and possibly entraining air into the discharge water. Eventually the exposed perforations will become damaged by this action, reducing the injection capacity of the well. Additionally, many receiving aquifers for groundwater injection are confined. The water table in the wellbore represents an expression of the confining pressure, with the actual formation aquifer capped by a low-permeability barrier some distance below the piezometric surface. Screening the well to this water table and above could result in perching injection water in overlying formations. This perched water (aerobic in nature) can potentially leach minerals and metals from the incorrect aquifer, draining these constituents back down into the target aquifer as the injection mound dissipates.

## WELL DEVELOPMENT HYDRAULICS

Developing an injection well is not like developing a groundwater production well. Nearly all the potable groundwater injection wells in the USA are developed by production well standards with injection performance being tracked as a function of lost production performance. In the other industries, it has been found that the development of injection wells is less rigorous than methods used for a monitoring well. Both of these approaches resulted in wells that clogged easily during injection service. During a typical production well development program, the formation material is co-mingled into the gravel pack to form a filter matrix. Surge pumping

develops the fine particles out of the passage ways and grades the sand and gravel towards the wellbore in the most optimum arrangement to prevent further sand movement. During injection, the extended reverse energy and vibration eventually breaks this material out of the production packing arrangement and flips it the opposite direction, clogging the well. The packing this time is not so optimum, so upon reversal of the energy to recovery pumping mode, the grains flip back towards the wellbore in some sort of disorganized packing that no longer stops the migration of sand to the discharge water.

If the well is to be used for injection, then the formation and gravel pack must be developed for injection using injection forces. The well development program should be designed to bulletproof the gravel and develop the well for “no particle movement” under production or injection forces. This requires simulating the full injection forces as would be assumed under operation to clog the well, and then use the surge development pumping equipment to work out the damage. Repeat clogging the well through simulated injection operation, and unclogging with the temporary development pumping equipment. This is repeated until the well cannot be clogged during normal restarts from a relaxed state, or from slam injection starts immediately following a pumping round. The injection specific capacity is tracked at a key control time point for validation of injection development improvements. Figure 1 provides an example of this well development approach.



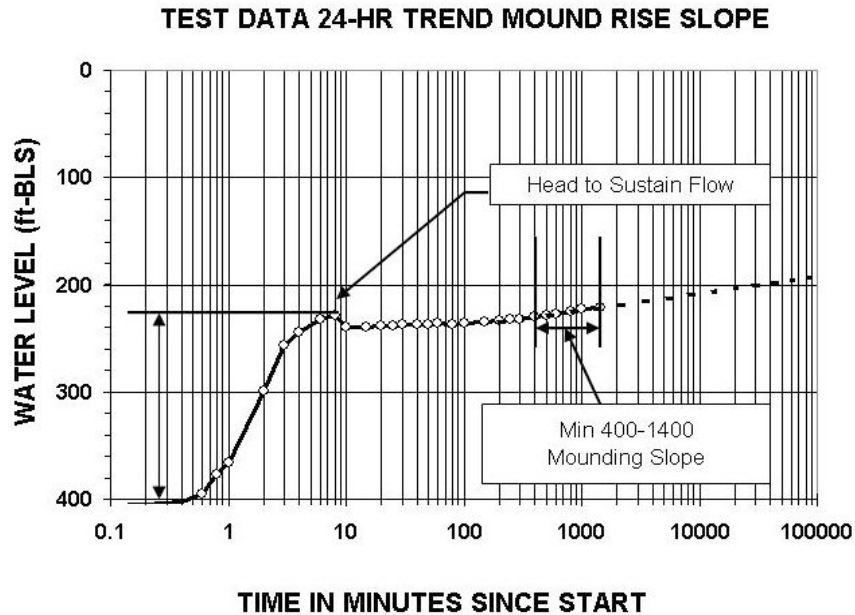
**FIGURE 1. Injection Performance Improvements from Development Methods.**

## OPERATION PERFORMANCE HYDRAULICS

Many entities calculate the injection design rate based upon a short term pumping test. This commonly results in injection design rates in excess of the actual sustainable rate. Operators need to have a confident design backed with baseline injection performance data. The following sections will attempt to define the pertinent operation and maintenance hydraulic parameters that will need to be defined to properly operate the injection well.

*Baseline Performance:* The baseline injection performance for the well is established during the injection development. The 60-minute water level from the “best” back-flush of the development program becomes the injection baseline control point for the life of the well. Under the controlled back-flush testing, this point is proven to be under minimal wellbore clogging influence. The performance is measured as Injection Specific Capacity (ISC) and Effective Hydraulic Area (EHA). The ISC is a ratio of the rate versus the water level rise. This value is valid at the control static water level and within 10% of the test flow value. The EHA represents the hydraulic communication of the wellbore to the aquifer, defined as an orifice equivalent, and is used to determine the proportional water level response to a flow rate. The parameter is independent of aquifer saturation changes and is a valid cornerstone to calculate the injection hydraulic response at virtually any rate. This allows development of an aquifer water level rise versus injection rate performance curve from a single test point.

*Pinnacle Mound Growth:* The final hydraulic parameter to capture prior to designing the injection equipment is the wellbore “pinnacle” mound growth slope. This is the tight cone-like mound that develops immediately around the injection well. This mound can assume 10-times to 100-times the water level rise as observed at the local monitoring well 300 feet away. This mound most likely dissipates to the wellfield “pancake” mound within 15 to 30 feet away from the wellbore (my current research is on the geometry of this pinnacle mound and the dispersion hydraulics). The beginning slope of this mound can be captured in a 24 to 72-hour continuous injection test. The data from minute 400 to test end best represents this mound growth slope. This hydraulic data is then used to determine the practical operation ranges of rigid flow control devices versus variable flow control valves. Figure 2 shows the hydraulic components of the injection water level rises versus time trend.

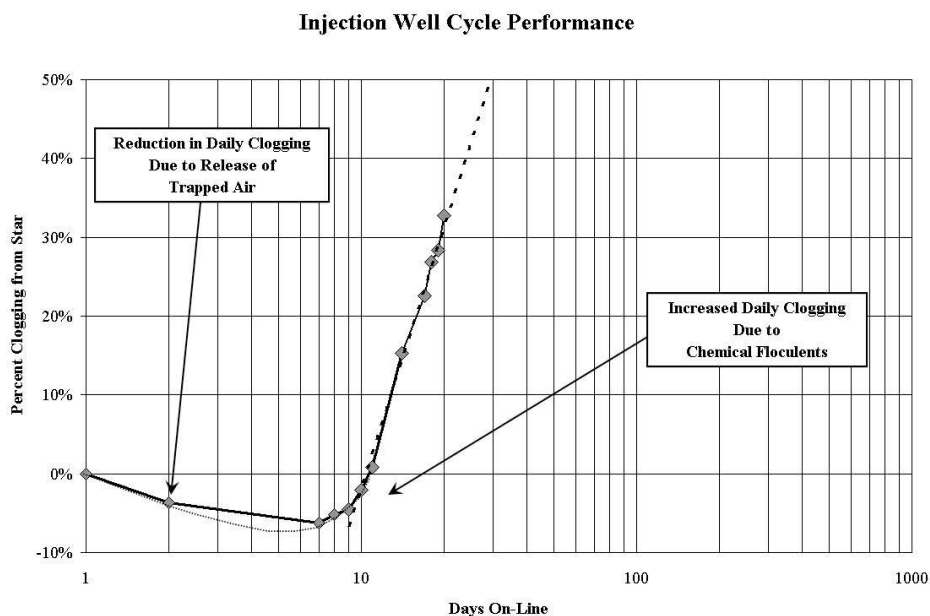


**FIGURE 2. Initial Hydraulic Response and Mound Slope Development.**

*Demonstration Testing:* Upon equipping the well with the permanent equipment, including a back-flush pump and receiving structure, the well can be placed into a “Demonstration Test” to evaluate other external influences to the injection performance. This could include well interference, wellfield mound development, natural recharge, and clogging. During this period the true operation behavior of the well can be evaluated. Impacts by water quality, delivery methods, distribution system components, and operation procedures can be flushed up and evaluated. This is where many new system operators become frustrated and abandon the technology. After approximately 6-months of cycle and continuous duty testing, the final design of the water treatment, delivery, and injection well equipment can be assembled. This will include determining back-flush limits and durations to maintain consistent injection performance.

During the test period, it is very important to obtain the correct data to analyze the injection performance. This includes a pre-start static water level, followed by injection parameters at 1-hour, 1-day, daily for first week, weekly for first month, every 2-weeks through month 3, then every month after that. If the injection operation is interrupted, pressure changed, or flow changed, a full suite of data should be taken just prior to the adjustment, followed by the same data collection

sequence as used for startup consisting of 1-hour data, 1-day data, and so forth. Full suite injection operation data includes injection water level or resulting well casing pressure, injection column pressure at the wellhead, and flow rate.



**FIGURE 3. Daily Clog Analysis from Startup.**

*Operation and Maintenance:* The Operation and Maintenance program should be able to track the injection performance with enough precision to determine the degree of performance loss induced by system startup, day by day operations, and by emergency events such as leaking air-vacuum valves, pipeline turbidity surges, or pipeline drains down the well during a power loss shutdown. How much does the column air clog the well at startup and does this air dissipate in time? How much clogging was reduced by the back-flushing event? Figure 3 displays a daily clog analysis showing an injection well that clogged about 10 percent by column air during startup. The air was relived from the pore passages within 5 to 7 days. Though by this point, the 0.5% per day clog induced by chemical flocculants is taking the lead on well performance reduction. As can be seen in the performance trend, the well should be back-flushed about every 9 days.

The EHA worksheet is used for the clogging analysis. The routine captures the wellbore hydraulics, mounding and external influences and then calculates the expected performance. Deviations from this hydraulic performance are reverse calculated to determine the altered pore area communication to the wellbore, allowing determination of the change from a pervious measurement or cycle

(clogging). The tool was introduced to the scientific community in 1995 with the latest versions of day-to-day clog analysis published in 2003. The mathematical tool is available free to the public at our informational web site [www.asrform.ws](http://www.asrform.ws).

## CONCLUSION

With the benefit of the scientific research in the potable water industry, we have the tools and knowledge to construct, develop, test, operate, and maintain direct injection wells based upon injection performance parameters not production performance. The key is getting the critical information into the hands of system designers and operators working with this technology in multiple disciplines. Injection wells are not like production wells; therefore the rules of construction and operation are different. The well should be sized for the largest redevelopment pump and flow control equipment. The well depth should fully penetrate the aquifer, maintaining well screen in the sections of the well that are saturated under all phases of operation. The injection wells should be developed with injection forces through full scale injection surges followed by pumping surges. The well's gravel pack and formation interface needs to be locked up to prevent sand migration under production or injection forces towards or away from the wellbore.

The well's operation must be approached in stages. This is to include short term back-flush events to tag the best hydraulic response of the aquifer, followed by tests to define the mounding impacts, external influences, and then separate the clogging aspects caused by the well, equipment, and operational practices. Upon completion of the well installation and basic hydraulic response tests, a Demonstration Test is initiated using the permanent well equipment, proposed operational methods, and injection source water. This test allows operation or performance issues to be exposed prior to finalizing the equipment installation and placing the facility into extended operation.

The proper field data must be collected within a specific schedule to provide the information for performance analysis. Using the Effective Hydraulic Area program, the wellbore clogging induced by startup, daily operation, and catastrophic events can be determined. This information is used to pinpoint and solve clogging issues, as well as define the necessary back-flush schedule to maintain a sustainable performance.

# APPLICATION OF THE HAZARD ANALYSIS AND CRITICAL CONTROL POINT (HACCP) RISK MANAGEMENT FRAMEWORK TO MANAGED AQUIFER RECHARGE

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## ABSTRACT

The limitations of conventional monitoring programs in facilitating the management of water quality risks are being increasingly recognised by both regulators and the greater community. This study describes the novel application of a Hazard Analysis and Critical Control Point (HACCP) framework for quality assurance of stormwater utilised for an Aquifer Storage Transfer and Recovery (ASTR) project. While HACCP has a long history in the food industry, it has only recently gained acceptance for potable water supplies; this application of HACCP is the first in the risk management of water quality in a managed aquifer recharge project.

A preliminary qualitative HACCP plan was initially developed to assist in the management of the ASTR project taking account of both human health and environmental risks. The intent in preparing this HACCP plan for ASTR was similar in concept to that for drinking water provision, where water quality and hazards are considered from source to end-user in a risk-based approach. Next, the preliminary qualitative HACCP plan and input from a steering committee of project stakeholders guided the design of a risk-based water quality monitoring program and the collection of data for direct input into a quantitative risk assessment. Water quality data was collected on the efficacy of stormwater harvesting and associated treatment processes (settling basins and a cleansing reedbed) and the ability of these processes to reduce the concentrations of contaminants (chemical, physical and microbiological) as a reliable pre-treatment step for managed aquifer recharge.

## KEYWORDS

Risk management, HACCP, QRA, stormwater quality and treatment

## INTRODUCTION

Although Hazard Analysis and Critical Control Point (HACCP) plans were first introduced over 40 years ago (CAC 1999), their development for use in drinking water



quality management has only been recently promoted in the Australian Drinking Water Guidelines (NHMRC 2004) and internationally (WHO 2004). This has resulted in an increased usage of the methodology with the management of water quality moving to incorporate more pro-active HACCP frameworks for drinking water (NHMRC 2004) and now recycled water (NRMMC/EPHC 2005) quality management systems. Further to this, Dillon (2005) suggested that HACCP would be suitable to manage water quality in managed aquifer recharge applications. This stems from a key limiting factor in the application of aquifer recharge type applications, namely for managing the spacio-temporal variability and uncertainty surrounding perceived water quality risks both to human health and the environment.

While previously HACCP approaches were applied to the Aquifer Storage Transfer Recovery (ASTR) project (Rinckf-Pfeifer *et al.* 2005) located at Salisbury in South Australia in a largely qualitative manner (Swierc *et al.* 2005), this study builds on the previous study by moving towards a quantitative risk assessment. The ASTR project is a major demonstration project which involves injecting stormwater harvested via an engineered wetland into a brackish aquifer and aims to recover the water at a potable quality. Unlike traditional aquifer storage and recovery (ASR), this scheme involves using separate injection and recovery wells to extend residence time and enhance passive treatment within the aquifer. Periodic analysis and review of the HACCP plan allows for further fine tuning of the risk-based water quality monitoring program and associated research to support the implementation of the evolving quantitative HACCP plan for the ASTR project.

## METHODS

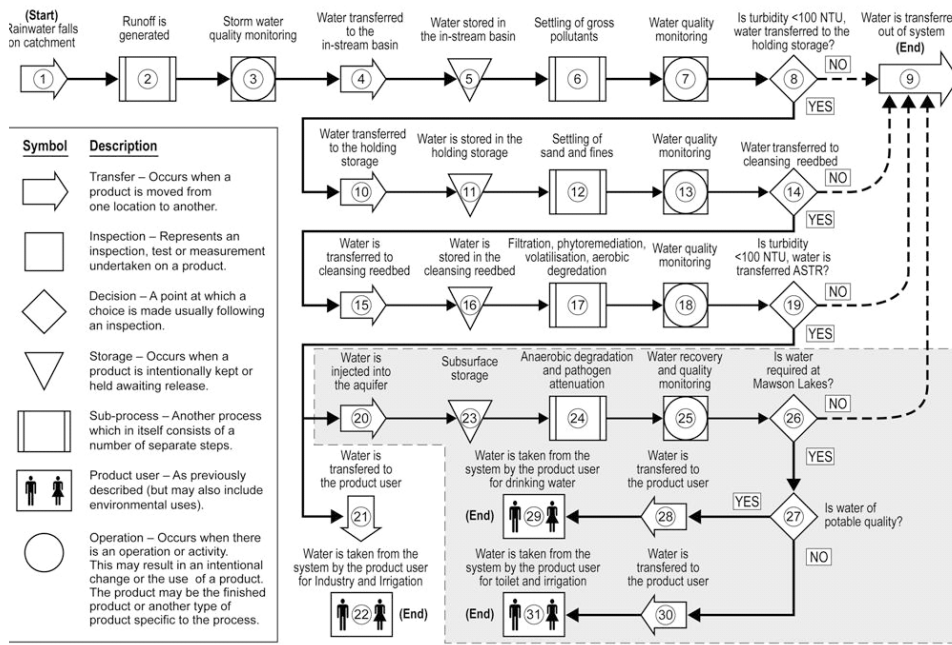
Swierc *et al.* (2005) performed a qualitative risk assessment for the ASTR project, including suggestions for appropriate Critical Control Points (CCPs) to manage the system. Review of CCP real-time monitoring data by Page *et al.* (2006) demonstrated that the ASTR system as a whole could be effectively managed, but hazards such as micropollutants (e.g. pesticides, herbicides, endocrine disruptors) which were not currently monitored in real-time required further research to validate that they were either absent or being effectively removed by the stormwater harvesting pre-treatment system. A risk-based water quality monitoring program was instituted based upon on the preliminary qualitative plan to further refine the risk management to ensure a high quality product.

## RESULTS AND DISCUSSION

With each refinement of the HACCP plan there is an increasing move from qualitative to quantitative risk assessment as more information is incorporated into the plan. Of primary importance is the continual review and improvement of the system by re-evaluating the CCPs and the supporting monitoring programs.

## Review of Critical Control Points, Quality Control Points and Supporting Programs

In managing the ASTR project, the system is first represented conceptually then simplified and represented as a process diagram (much like a water treatment plant) complete with the CCPs (decision points) used to manage water quality (Figure 1).



**FIGURE 1. Process diagram for the ASTR system the greyed area represents subsurface component.**

In 2006, the main objective of the ASTR project was the continued establishment of the site; this included the development of injection and recovery wells and the first stage of conditioning (flushing) of the brackish aquifer. During this initial period the focus of the water quality monitoring program was on understanding and quantifying changes in water quality through the stormwater harvesting system from the inflow at the in-stream basin to the outflow of the cleansing reedbed. This aimed at determining the hazard removal efficiencies of the stormwater harvesting system.

## Risk-based Water Quality Monitoring and System Assessment

A key limitation identified by Swierc *et al.* (2005) in the preliminary risk assessment was the absence of any hydrologically-linked water quality data (i.e. event-based monitoring). For example, during the rising limb of the runoff hydrograph there is a higher concentration of hazards due to the washout of deposited materials by

rainfall known as a “first flush” effect. The characterisation of this phenomenon was a principal objective in the risk-based sampling described here. The first flush has been noted as an important and distinctive phenomenon and produces higher hazard concentrations early in the runoff event and a concentration peak preceding the peak flow. During winter-spring 2006, five storm events were monitored; this data was coupled to a water and chloride balance for the system to estimate removal efficiencies for the stormwater harvesting system (Table 1).

The runoff quality from the Parafield residential catchment is typical of many urban environments. The inflow stormwater quality varied considerably during the event-based sampling period but the outflow quality was much less varied, demonstrating both the dampening of hazard spikes (Page *et al.* 2006) as well as good water quality generally meeting the Australian water quality guidelines. Exceptions to this include colour which is related to high dissolved iron and organic carbon loads as well as the faecal indicator bacteria which are likely to result from extraneous input sources of animal faecal matter (wildlife). Preliminary modelling by Pavelic *et al.* (2004) has estimated that the subsurface residence time of the water will be in the order of eight months. This provides sufficient residence time to be confident that no injected faecal indicator bacteria or enteric pathogens are likely to survive (Dillon and Toze 2005). This is further discussed below with the quantitative risk assessments below.

The preliminary HACCP plan also identified land uses and associated micropollutants which may be found within the stormwater catchment. As part of the quantitative risk assessment and in addition to the routine parameters described in Table 1, micropollutants were also investigated during the event-based sampling. A series of suites of common micropollutants (organochlorine, organophosphate, phenoxy acids, triazines) based on catchment land uses were analysed but no detections resulted. This was thought to be due to the “spikey” nature of concentrations of micropollutants in stormwater. It is well known that standard sampling/analytical techniques often have limits of detections that are orders of magnitude above the low concentrations that may pose a risk to the environments in which they occur.

**TABLE 1. Water and chloride budget, mean inflow and outflow stormwater quality parameters and removal efficiencies from the Parafield stormwater harvesting system for the period 22/08/06 ñ 15/09/06.**

| Initial Volume = 21 ML           | Water (ML)                   |                              | Chloride (kg)      |                                         |                                |
|----------------------------------|------------------------------|------------------------------|--------------------|-----------------------------------------|--------------------------------|
| Inflow                           | 61                           |                              |                    | 1,474                                   |                                |
| Outflow                          | 46                           |                              |                    | 1,584                                   |                                |
| Final Volume = 38ML              |                              |                              |                    |                                         |                                |
|                                  | Inflow untreated stormwater* | Outflow treated stormwater * | Removal Efficiency | Ecosystem Protection Guideline Value*** | Drinking Water Guideline Value |
| Major Ions (mg/L)^               |                              |                              |                    |                                         |                                |
| Alkalinity                       | 67.5                         | 86.6                         | 0.11               |                                         |                                |
| Bicarbonate                      | 125                          | 105                          | 0.43               |                                         |                                |
| Bromide                          | 0.38                         | 0.08                         | 0.85               |                                         |                                |
| Sulphate                         | 23.0                         | 9.9                          | 0.71               |                                         | 500                            |
| Chloride                         | 24.0                         | 34.5                         | 0**                |                                         |                                |
| Fluoride                         | 0.35                         | 0.21                         | 0.60               |                                         | 1.5                            |
| Calcium                          | 37.9                         | 26.9                         | 0.52               |                                         |                                |
| Magnesium                        | 17.1                         | 6.3                          | 0.75               |                                         |                                |
| Potassium                        | 6.8                          | 5.4                          | 0.46               |                                         |                                |
| Sodium                           | 22.3                         | 21.0                         | 0.36               |                                         | 180                            |
| Microbiological (number/100mL)   |                              |                              |                    |                                         |                                |
| Coliforms                        | $3.5 \times 10^5$            | $2.3 \times 10^3$            | 0.99               |                                         |                                |
| <i>E. coli</i>                   | $2.4 \times 10^2$            | $2.3 \times 10^1$            | 0.94               |                                         | 0                              |
| Enterococci                      | $7.0 \times 10^2$            | $6 \times 10^0$              | 0.99               |                                         | 0                              |
| Faecal coliforms                 | $2.4 \times 10^2$            | $2.6 \times 10^1$            | 0.93               |                                         | 0                              |
| Faecal Streptococci              | $7.0 \times 10^2$            | $6 \times 10^0$              | 0.99               |                                         | 0                              |
| Physical characteristics (mg/L)^ |                              |                              |                    |                                         |                                |
| True Colour (HU)                 | 50.9                         | 51.6                         | 0.32               |                                         | 15                             |
| Suspended Solids                 | 3.5                          | 1.3                          | 0.75               |                                         |                                |
| Turbidity (NTU)                  | 4.5                          | 2.9                          | 0.56               | 100                                     | 5                              |
| Metals (mg/L)^                   |                              |                              |                    |                                         |                                |
| Aluminium (t)                    | 0.18                         | 0.17                         | 0.35               |                                         |                                |
| Iron (d)                         | 0.09                         | 0.33                         | -1.55              |                                         |                                |
| Iron (t)                         | 0.3                          | 0.55                         | -0.24              |                                         | 0.3                            |
| Lead (t)                         | 0.003                        | <0.0005                      | 0.87               |                                         | 0.01                           |
| Manganese (d)                    | 0.01                         | 0.02                         | -0.40              |                                         |                                |
| Manganese (t)                    | 0.043                        | 0.042                        | 0.33               | 1.9                                     | 0.1                            |
| Nickel (t)                       | 0.00123                      | <0.0005                      | 0.86               | 0.011                                   | 0.02                           |
| Zinc (t)                         | 0.10                         | 0.03                         | 0.82               | 0.008                                   | 3                              |
| Nutrients (mg/L)^                |                              |                              |                    |                                         |                                |
| Total Nitrogen                   | 0.91                         | 0.45                         | 0.67               | 1                                       |                                |
| Ammonia                          | 0.07                         | 0.02                         | 0.84               | 0.1                                     | 0.5                            |
| Nitrate + Nitrite                | 0.0061                       | <0.0005                      | 0.97               | 0.1                                     | 50                             |

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|                       |       |       |      |      |
|-----------------------|-------|-------|------|------|
| Kjeldahl Nitrogen (t) | 0.90  | 0.48  | 0.64 |      |
| BOD <sub>5</sub> (t)  | 3.7   | 1.2   | 0.78 |      |
| BOD <sub>5</sub> (d)  | 2.1   | 1.1   | 0.64 |      |
| COD <sub>5</sub> (t)  | 82.7  | 42.4  | 0.65 |      |
| COD <sub>5</sub> (d)  | 59.8  | 22.9  | 0.74 |      |
| Organic Carbon (t)    | 0.90  | 0.48  | 0.64 |      |
| Organic Carbon (d)    | 15.7  | 7.9   | 0.66 |      |
| Phosphorous (t)       | 0.023 | 0.016 | 0.55 | 0.1  |
| Phosphorous (d)       | 0.13  | 0.04  | 0.78 | 0.04 |
| Sulphur (t)           | 7.5   | 3.3   | 0.70 |      |

\* volume weighted concentration (n = 5 storm events).

\*\* was set to zero and all other reactive contaminants scaled to this in the calculation of removal efficiencies..

\*\*\* for 95% protection

^ unless otherwise indicated

t= total, d = dissolved.

To address this and in view of the limitations of conventional sampling and analytical techniques, time integrated passive sampling techniques (Komarova *et al.* 2006) were deployed for the monitoring of organic micropollutants throughout the ASTR stormwater harvesting system. Passive sampling techniques provide a measure of the concentration of analytes that are detected in the samplers (Stephens *et al.* 2005). Average water concentrations in the environment during the time of deployment are derived from the concentrations sequestered in the sampler using calibrations conducted in the laboratory (Table 2).

**TABLE 2. Estimated concentrations (ng/L) of selected micropollutants in repeat measurements for the stormwater harvesting system using passive samplers.**

|                   | Field<br>Blank | In-stream<br>basin inlet |       | Holding storage<br>inlet |      | Cleansing<br>reedbed inlet |       | Cleansing<br>reedbed outlet |       |
|-------------------|----------------|--------------------------|-------|--------------------------|------|----------------------------|-------|-----------------------------|-------|
| Sampler<br>number |                | 1                        | 2     | 1                        | 2    | 1                          | 2     | 1                           | 2     |
| diuron            | < 50           | 168                      | 159   | 97                       | 84   | 221                        | 208   | 416                         | 345   |
| trifluralin       | < 0.5          | < 0.5                    | < 0.5 | < 0.5                    | 1    | < 0.5                      | < 0.5 | < 0.5                       | < 0.5 |
| simazine          | < 30           | < 30                     | < 30  | < 30                     | < 30 | < 30                       | < 30  | 104                         | 104   |
| atrazine          | < 8            | < 8                      | < 8   | < 8                      | < 8  | < 8                        | < 8   | 31                          | 32    |
| diazinon          | < 6            | < 6                      | < 6   | 13                       | 15   | 23                         | 22    | < 6                         | < 6   |
| terbutryn         | < 3            | < 3                      | < 3   | < 3                      | < 3  | < 3                        | < 3   | 7                           | 6     |
| metolachlor       | < 8            | < 8                      | < 8   | < 8                      | < 8  | < 8                        | < 8   | 12                          | 11    |
| chlorpyrifos      | < 0.5          | 3                        | 4     | 3                        | 3    | 3                          | 3     | < 0.5                       | < 0.5 |

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|                           |        |       |       |        |        |     |     |       |       |
|---------------------------|--------|-------|-------|--------|--------|-----|-----|-------|-------|
| chlordane trans           | < 1    | < 1   | < 1   | < 1    | < 1    | < 1 | < 1 | < 1   | < 1   |
| chlordane cis             | < 1    | < 1   | < 1   | < 1    | < 1    | < 1 | < 1 | < 1   | < 1   |
| nonachlor trans           | < 1    | < 1   | < 1   | < 1    | < 1    | < 1 | < 1 | < 1   | < 1   |
| oxadiazon                 | < 0.5  | 2     | 2     | 2      | 2      | 3   | 3   | 1     | 1     |
| propiconazol              | < 2.5  | < 2.5 | < 2.5 | 7      | 8      | 13  | 14  | < 2.5 | < 2.5 |
| isomers                   |        |       |       |        |        |     |     |       |       |
| endosulfan                | < 2    | < 2   | < 2   | 5      | 4      | 5   | 4   | < 2   | < 2   |
| sulphate                  |        |       |       |        |        |     |     |       |       |
| piperonyl                 | < 0.5  | 2     | 3     | 2      | 3      | 3   | 3   | < 0.5 | < 0.5 |
| butoxide                  |        |       |       |        |        |     |     |       |       |
| oxyfluorfen               | < 0.5  | 1.8   | 1.7   | 2      | 2      | 3   | 3   | < 0.5 | < 0.5 |
| Polyaromatic hydrocarbons |        |       |       |        |        |     |     |       |       |
| phenanthrene              | < 0.08 | 1     | 1.1   | < 0.08 | < 0.08 | 0.3 | 0.3 | 0.1   | 0.1   |
| fluoranthene              | < 0.08 | 1.2   | 1.4   | 0.4    | 0.5    | 0.6 | 0.6 | < 10  | < 10  |
| pyrene                    | < 0.08 | 1.8   | 1.8   | 1.4    | 1.9    | 1.4 | 1.6 | 0.3   | 0.3   |

Diuron was found at the highest levels at all sites followed by simazine and atrazine. Discussions with local councils, maintenance contractors and the rail authority did not identify any current users. From Table 2 it appears that concentrations of micropollutants appear to increase with passage through the system as a result of non-isokinetic uptake by the passive samplers at the locations deployed. The sampler at node 3 (in-stream basin inlet) is exposed to much greater variations in hazard concentrations, spikes of hazards are the norm as stormwater is quickly pumped from the in-stream basin to the holding storage, while the sampler at node 18 (cleansing reedbed outlet) is exposed to more uniform flow over sustained periods.

### Quantitative risk assessments

In progressing the risk assessment, a revision of the qualitative work performed by Swierc *et al.* (2005) was undertaken. Principal amongst the chemical hazards identified by Swierc *et al.* (2005) were benzene from road runoff as a surrogate for hydrocarbons and the herbicides simazine and chlorpyrifos. These were identified based on the previous *ad hoc* detections in the stormwater quality monitoring program, their known toxicity and the availability of a suitable guideline to identify an acceptable maximum concentration. In keeping with this initial assessment, though these micropollutants were not detected in the results of the conventional monitoring, their presence was again indicated (but not sufficiently quantified) through the use of passive samplers (Table 2). As such these three hazards were chosen for a quantitative risk assessment (QRA). Analogously, the potential microbial hazards identified were represented by the commonly used “index” pathogens, rotavirus, *Cryptosporidium parvum* and *Campylobacter* representing virus, protozoa and bacterial hazards, respectively consistent with current guidelines (NHMRC 2004; WHO 2004). These hazards will be the focus of the microbial QRA and are described as “index” hazards. The behaviour of these higher risk hazards form an upper bound to the risks associated from the multitude of

contaminants presently in stormwater that may impact on human or environmental health. If these hazards can be managed and brought under control in the HACCP context, then other hazards not explicitly managed will also be brought under control by the multiple barriers present within the ASTR system. For micropollutants the risk assessment used the current guideline values as endpoints, though simplistic, they are easy to assess by regulatory bodies, and hence establishment of critical limits for CCPs is feasible. However, for the pathogens a more sophisticated QRA model was adopted where, the yearly disease burden in terms of Disability Adjusted Life Years (DALYs) was calculated and compared to current WHO acceptable thresholds (WHO 2004). The QRA model was further developed to facilitate Monte Carlo simulation (using @RISK Industrial v4.5, Palisade Corporation, USA), which entails generating hypothetical scenarios in terms of the values attributed to the identified factors in the exposure and dose-response assessments. The simulation represents the inherent variability in the process of stormwater harvesting, wetland treatment and subsurface storage and treatment characteristics based on the modelling by Pavelic *et al.* (2004) and influence on expectant risk as well as the uncertainty in the mathematical model of the process. The results of the simulations for the risk associated with drinking recovered water from the ASTR system are shown in Table 3.

**TABLE 3. QRA results for micropollutant and microbial risk assessments, simulated concentrations relative to hazard guidelines or DALYs**

| Parameter (µg/L)                     | Benzene               | Simazine             | Chlorpyrifos          |
|--------------------------------------|-----------------------|----------------------|-----------------------|
| Drinking water guideline (ADWG 2004) | 1.0                   | 0.5                  | 0.05                  |
| Ecosystem guideline (ANZECC 2000)*   | 950                   | 3.2                  | 0.01                  |
| Mean                                 | $5.4 \times 10^{-3}$  | 0.4                  | $9.8 \times 10^{-5}$  |
| Minimum                              | 0                     | 0.3                  | $1.4 \times 10^{-11}$ |
| Maximum                              | 0.5                   | 0.7                  | $9.4 \times 10^{-3}$  |
| 5%                                   | $1.2 \times 10^{-15}$ | 0.3                  | $2.0 \times 10^{-8}$  |
| 50%                                  | $2.1 \times 10^{-6}$  | 0.4                  | $5.6 \times 10^{-6}$  |
| 95%                                  | $2.4 \times 10^{-2}$  | 0.6                  | $4.5 \times 10^{-4}$  |
| Standard Deviation                   | $4.4 \times 10^{-3}$  | $7.6 \times 10^{-1}$ | $3.9 \times 10^{-4}$  |
| Mode                                 | $4.3 \times 10^{-21}$ | 0.4                  | $8.3 \times 10^{-10}$ |
| Parameter                            | <i>C. parvum</i>      | <i>Campylobacter</i> | Rotavirus             |
| Guideline value (DALYs) (WHO 2004)   | $1.0 \times 10^{-6}$  |                      |                       |
| Mean                                 | $4.0 \times 10^{-9}$  | 0                    | $1.8 \times 10^{-9}$  |
| Minimum                              | 0                     | 0                    | 0                     |

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|                    |                      |   |                      |
|--------------------|----------------------|---|----------------------|
| Maximum            | $1.8 \times 10^{-5}$ | 0 | $9.0 \times 10^{-6}$ |
| 5%                 | 0                    | 0 | 0                    |
| 50%                | 0                    | 0 | 0                    |
| 95%                | 0                    | 0 | 0                    |
| Standard Deviation | $2.3 \times 10^{-7}$ | 0 | $1.0 \times 10^{-7}$ |
| Mode               | 0                    | 0 | 0                    |

\* for 95% protection

For the three micropollutants (Table 3), all mean concentrations are less than the guideline values. Indeed for each micropollutant, even the maximum estimated concentrations were below the guideline values. Given the inputs used for the QRA model, it seems unlikely they pose a significant risk to water quality in the ASTR system. Also given that the resultant distribution of risk is non-normal, the use of the arithmetic mean to describe central tendency may result in an over estimate of risk, due to the proportionally larger contribution of high values. For this reason it may be more prudent and practical to use the 50<sup>th</sup> percentile (median) to describe expectant risk much as for the removal efficiencies calculated in Table 1. For each of the micropollutants, this results in an even lower estimate of risk. Finally it is worth noting that the estimated concentrations of each of the micropollutants at the point of supply to the end user are below the current instrument detection limits and as such verification and validation of the HACCP plan may be problematic. Alternative approaches such as the use of solid phase membrane devices (Table 2) may be critical in validating the HACCP plan. Where differing guideline values for protection of human health and aquatic ecosystems exist, the lower limit was used providing a tighter constraint to the risk assessment.

The results for the microbial risk assessment are also given in Table 3 and similarly to the micropollutant risk assessments, the use of the mean to characterise microbial risk is problematic. When considering the results it is important to note that there is no one probability for illness resulting from the consumption of water, but rather there is a range of risk. The conditional probability of DALYs given the particular process scenario described by an individual iteration of the QRA model is usually infinitesimal. As shown in Table 3, for the risk for most simulations (as exemplified by the mode) is actually 0, well less than  $1.0 \times 10^{-6}$  DALYs and the median and 95<sup>th</sup> percentile are also several orders of magnitude below this value. As such, most water derived from the ASTR system is predicted to present a negligible risk. The exception to this is the maximum risk posed by *Cryptosporidium* and rotavirus which are above the DALYs guideline value. As previously noted, the median risks posed by each of the index microbial hazards were all estimated to be zero when compared to the DALYs guideline value (Table 3). This largely due to the large residence time of the injected water in the aquifer prior to recovery, with mean residence times in the order of 8 months. Given the likely decay rates for each of the index pathogens, each of them will achieve  $\sim 10$  to  $100$  log removal during the subsurface storage treatment component (Dillon and Toze 2005). For such a large



predicted decay rate in the subsurface treatment barrier, even stormwater of sufficiently poorer quality than reported in Table 1, could be effectively treated. Possibly then the greatest risk could come from risk associated with short circuiting in the aquifer, leading to a large reduction in the effective residence time. The travel time of injected water within the aquifer is currently being validated during the ASTR trial.

In moving to a quantitative HACCP plan, QRA has been used to numerically describe the distribution of risk attributable to several index hazard micropollutants and pathogens. However, the predicted risk should be interpreted carefully. Any single water sample may pose no risk or a very high risk considering the efficacy of stormwater harvesting and subsurface storage treatment processes. However it is the mean value of risk that is experienced by an average person, drinking water processed in the described manner. The diversity of individual's immune competence and consumption patterns was averaged into this expected value of risk. No confidence limits or uncertainty bounds can be put on the expected value of risk because the distribution represents both uncertainty and variability. Treatment of variability and uncertainty separately would be necessary to estimate uncertainty bounds on the risk estimate, requiring more sophisticated simulation techniques.

The QRA model was developed to provide an initial means to analyse the relationship between risk and factors which might be used to mitigate risk, specifically within the HACCP framework. A risk manager implementing a HACCP plan is likely to be more interested in the sensitivity analysis and the requirements of risk mitigation strategies than the risk distributions themselves. Sensitivity analysis of the QRA model input parameters identified several factors which contribute significant uncertainty to the total uncertainty of the risk. Hence, possible interventions can be deduced from controllable variables which have an important contribution to risk. For example, for micropollutants where degradation in the aerobic environment of the aquifer is minimal and source water protection management strategy would yield the most efficient reduction in risk. Using this approach for comparison of possible management options de-emphasizes the importance of the actual risk estimate, and emphasizes the relative risk estimates under possible intervention options. This allows consideration and allocation of resources to potential risk reduction strategies that may be immediately feasible, while at the same time identifying priorities for focused and longer term research to better understand, and intervene at steps (which may or may not be classified as CCPs) in the process. In this study, several parameters were modelled using data from a wide variety of processes, such as the log reduction in pathogen counts resulting from literature values. A decision-maker could estimate the value of changing the current treatment barriers to a more effective or less costly combination by altering the model parameters.

Obviously, the results of this QRA are only accurate to the extent that the model accurately represents the processes. Model validation is an important consideration in QRA and equally in HACCP. Other estimates of risk may not be specifically comparable to the predictions based upon the particular scenario of the QRA. However the scope of HACCP is not necessarily to accurately estimate the absolute risk but instead to manage risk. QRA facilitates both implementation and verification of HACCP by providing a conceptual model of the risk assessment.

## CONCLUSIONS

The evolution of risk assessment from qualitative to quantitative is supported within the HACCP framework. This is used to manage risk for the ASTR project by the use of QRA in verifying the functioning of barriers in the system aiming to produce potable water quality from urban stormwater. The water produced by ASTR can be regarded as having a high degree of safety if all of the CCPs of the system are validated and treatment processes meet performance targets. Development of a QRA model for the ASTR project to support HACCP principals and a revised risk-based monitoring program were presented. Further development and process understanding of the system is required particularly if CCP with surrogate parameters for the hazards such as micropollutants are to be used to operate the system.

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# A STOCHASTIC EVALUATION OF THE SUBSIDENCE POTENTIAL OF THE HAWTHORN GROUP IN SOUTH FLORIDA AS A RESULT OF THE CERP ASR SYSTEM

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## ABSTRACT

The proposed Comprehensive Everglades Restoration Project (CERP) represents a massive re-plumbing of southern Florida. The CERP also includes a very large Aquifer, Storage and Recovery (ASR) well system that is unprecedented in both size and scope. The CERP ASR plan includes 330 ASR wells to provide flexible water storage and water supply for Lake Okeechobee and other urbanized areas as well as important natural areas. The CERP ASR plan is well advanced and includes the development of a regional groundwater model and many other technical studies. One of the key completed studies evaluated possible impacts from large pressure changes within the Floridan Aquifer System (FAS) and the overlying Hawthorn Group confining unit.

Both analytical modeling and desktop spreadsheet evaluations were completed to provide a preliminary review of the pertinent issues. In general, the preliminary evaluation concluded that effective stress changes within the FAS and Hawthorn Group were likely but that potential impacts would probably be minor and could be controlled further through operational constraints. This paper builds upon the original work completed for the CERP to evaluate subsidence potential within the Hawthorn Group. For this paper, further subsidence evaluations have been completed using a stochastic analysis approach. Principal variables were assigned reasonable ranges based upon lab testing results and literature values so that a full probabilistic evaluation could be completed.

This paper will endeavor to discuss the original preliminary subsidence findings along with the results from the new stochastic simulations. The stochastic simulation approach will be outlined and final results presented. Then, ASR operational considerations will be discussed in order to bracket expected subsidence. Lastly, the paper will discuss and comment upon the overall findings and issues that have been identified.

## KEYWORDS

ASR, subsidence, stochastic model, deformation

## INTRODUCTION

The Central and Southern Florida Project Comprehensive Review Study (USACE & SFWMD, 1999) – developed jointly by the South Florida Water Management District (SFWMD) and the U.S. Army Corps of Engineers (USACE) – provides a sound basis for Everglades restoration. Now known as the Comprehensive Everglades Restoration

Plan (CERP), this plan contains 68 components, including structural and operational changes to the Central and Southern Florida (C&SF) Project. The overarching purpose of CERP is to restore the Everglades by improving the quantity, quality, timing and delivery of water for the natural ecosystems of south Florida. A key component in the overall restoration strategy is the provision of more dynamic storage of freshwater. Storage is envisioned to combine surface reservoirs with underground aquifer storage via aquifer storage and recovery (ASR) wells.

The use of ASR is increasing in the United States and abroad and has been utilized in a wide variety of geologic environments Brown (2005). The CERP relies heavily upon ASR technology to provide additional system storage. The proposed use of ASR for this project is unprecedented with 330 large-capacity wells proposed to store over 3.8 million cubic meters of water per day. Due to project uncertainties, the SFWMD and the USACE have undertaken an important regional study to further evaluate the overall feasibility of the project (USACE & SFWMD, 2004). One issue that has been evaluated is the potential for large-scale deformation of the Hawthorn Group clays due to pumping-induced subsidence. A preliminary study was prepared and discussed by Brown et al. (2004). Based upon the preliminary results of that study, further numerical investigations were undertaken for this paper. Specifically, a more sophisticated analytical analysis was completed using a stochastic modeling approach. Key model variables were assigned to the model using standard probabilistic distributions and then the model was run using a classic Monte-Carlo simulation in the Crystal Ball® software package. The model was run for 20,000 iterations and the resulting probability cumulative density functions were plotted and analyzed. This paper discusses the problem, the modeling effort, and the results.

## LITERATURE REVIEW

Land subsidence (downward movement) can be attributed to several causes: tectonic movement; chemical dissolution; consolidation of sedimentary materials; vibrations and water table decline due to groundwater withdrawals. Groundwater withdrawals, however, are one of the most obvious causes for land subsidence (Brown et al. 2004). Land subsidence can be severe where pumping exceeds the aquifer safe yield, and water level or the piezometric surface has declined. Numerous cases of subsidence attributed to groundwater level lowering have been documented. In Baton Rouge, Louisiana, one foot of subsidence has been observed in an industrial district where the piezometric surfaces have dropped about 135 feet due to pumping of groundwater since 1890 (Davis and Rollo, 1969). Subsidence of clay confining units caused radial cracks to form around ASR wells at the Washoe County ASR project (Bureau of Reclamation, 1996a). Subsidence in New Orleans has been discussed by several authors including (Poland and Davis, 1969; Sever, 2006). Groundwater mining in China has led to significant subsidence in the Yangtze Delta (Zhang et al. 2007).

The use of Monte-Carlo simulations to evaluate consolidation problems stochastically rather than deterministically is becoming an accepted practice. The use of stochastic models dates to the late 1970s when Freeze (1977) evaluated the use of such a model to simulate the one-dimensional consolidation of heterogeneous formations. Freeze shows that the standard deviations associated with the input soil properties in heterogeneous soils can lead to large uncertainties in predicted hydraulic head values and consolidation rates. Freeze also notes that uncertainties are increased by increasing soil heterogeneity and decreased by the presence of spatial trends. Massman (1986) outlines the potential advantages of using a stochastic consolidation model rather than a deterministic model. Van Vliet et al. (1997) discuss the use of probabilistic input soil parameters for consolidation of mine tailings. They assigned probability distributions for the parameters describing the constitutive large strain consolidation relationships from the results of a previously conducted laboratory testing program. Li and Choa (1999) used a stochastic model to simulate the consolidation of clay. They assigned probability distributions to model input soil parameters in order to produce a consolidation cumulative probability density function. Frias et al. (2004) evaluated heterogeneous storage reservoirs in a similar fashion but developed more sophisticated log-normally distributed stationary random hydraulic conductivity fields using a coefficient of variation to govern the magnitude of the field heterogeneity.

## PROBLEM DEFINITION

ASR well fields are typically designed for the short-term storage of excess water for use during peak demands. In addition, the well fields are designed to recover the maximum amount of water possible at the required demand rate. Many ASR well field storage zones are located in a brackish portion of an aquifer and as a result individual ASR wells tend to be closely spaced to optimize the capture zones and maximize recovery of injected water of superior quality (Brown et al. 2004). Mixing of waters occurs due to a number of mechanisms including groundwater advection, dispersion, and diffusion. Excessive mixing of the injected water with the ambient groundwater can degrade the recovered water quality over time and this can be detrimental to the overall recovery efficiency. This approach, of closely spaced ASR wells, seems reasonable as long as care is taken to evaluate aquifer pressure increases or decreases. ASR wells located in close proximity can lead to significant hydraulic interference with large head increases or decreases as a result. Large well fields such as those proposed by the CERP (USACE and SFWMD, 1999), could lead to extreme head increases or decreases across large areas. It is important to manage these pressure changes so that significant regional changes are kept to a minimum. Increased or decreased aquifer pressures can also have unintended geotechnical consequences that should be evaluated carefully. Excess pressures generated during ASR recharge cycles can potentially lead to hydraulic fracturing of the aquifer

medium. If clay confining units are affected by the ASR recovery, deformation or subsidence may result.

All of the subsidence cases mentioned previously involve thick clay deposits, which have high compressibility, thus are more vulnerable to subsidence (Brown et al. 2004). The magnitude of the subsidence was directly associated with groundwater level declines that were caused by long-term groundwater extraction. Usually, the greatest magnitude of subsidence occurs where water-level declines are greatest, and where confining units are thicker and most compressible. Generally, clay confining unit effective stress fields change as a result of aquifer dewatering. The overall effective stress within a clay confining unit is a function of the total overburden vertical stress minus the formation pore pressure (Terzaghi and Peck, 1948). The effective stress equation is shown as equation (1).

$$\sigma_e = \sigma - u \quad (\text{EQ 1})$$

Where  $\sigma_e$  is the effective vertical stress;  $\sigma$  is the total vertical stress; and  $u$  is the pore pressure. If the overall confining unit pore pressure is reduced due to aquifer pumping, the effective vertical stress will increase in a commensurate fashion. Consolidation of clays is a long-term, non-linear process and has been thoroughly discussed in the literature and tested in practice. Lambe and Whitman (1969) provide a thorough theoretical discussion of long-term consolidation of clay formations. The consolidation equation is shown as equation (2).

$$\delta = C_c (H / (1 + e_0)) \log (P/P_0) \quad (\text{EQ 2})$$

Where  $\delta$  is the total settlement or deformation;  $C_c$  is the compression index;  $H$  is the thickness of the consolidating clay unit;  $e_0$  is the initial unit void ratio;  $P$  is the total effective pressure acting on mid-height of the consolidating clay unit; and  $P_0$  is the initial effective overburden pressure of the overlying sediments. Equation (2) provides a means to estimate the total magnitude of the deformation. Equation (3) is an analytical model developed to estimate the time to reach a particular settlement increment.

$$t = (T_v / c_v) H^2 \quad (\text{EQ 3})$$

Where  $t$  is the time to reach a particular settlement increment (e.g., 10%);  $T_v$  is the non-dimensional settlement time factor;  $c_v$  is the coefficient of consolidation corresponding to  $P$  in equation (2); and  $H$  is the thickness of the consolidating unit. It should be noted that if the clay unit can be drained on top and bottom as the Hawthorn Group can, half the thickness of the unit should be substituted for  $H$ . Equations (1) to (3) were utilized in the stochastic model discussed below.

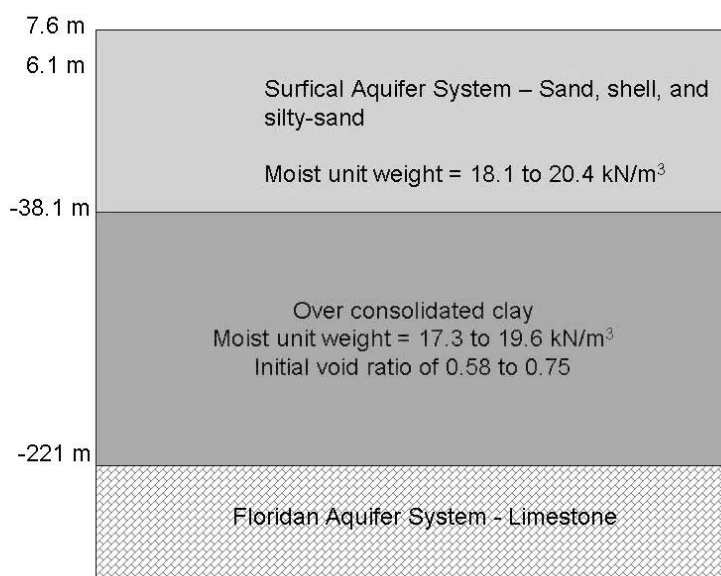
The stress history of the deposit also plays an important role in its vulnerability to subsidence (Brown et al. 2004). Generally, soil consolidation is naturally caused by the weight of overlying materials, thus, the stress history depends upon the age and the depth of the deposit, as well as geological processes that the deposit has experienced. The older age and deeper buried the sediment, the greater consolidation it may have experienced. In contrast, a young and shallow deposit may experience less pre-consolidation. Natural phenomena have acted on many soil deposits around the world. Glaciers present within the northern hemisphere subjected local soil deposits to massive loads, erosion, and re-deposition. In Florida, or other marine environments, past sea level stands several hundred feet higher than present day have undoubtedly stressed the existing soil deposits (Miller, 1997). The opposite can also occur. Dissolution of carbonate materials can lead to gradual unloading of a limestone deposit. Therefore, soil deposits have generally experienced a range of historic stresses and loads. Soils subjected to large historic loads (glaciers, sea level rise, etc.) are less likely to subside as compared to virgin deposits of river silts and clays created in recent time. Lambe and Whitman (1969) provide a detailed discussion of consolidation theory as it relates to stress history. Depending upon the soil deposits past stress history, a compressible clay unit could be described as “normally consolidated” or “over consolidated”. Over consolidated clay units have been exposed to very large loads some time in the past that are greater than current loads, whereas normally consolidated units are recent materials that have only experienced a relatively short stress history.

According to Lambe and Whitman (1969), the overconsolidation ratio (or OCR) is a useful metric in determining the susceptibility of a particular deposit to additional subsidence. The OCR can be determined from one-dimensional consolidation testing of confining unit sediments. The OCR is simply a ratio of the pre-consolidation pressure or stress to the current effective stress. Generally, normally consolidated clays are much more vulnerable to consolidation and consolidate at a faster rate. Normally consolidated clays have OCRs less than 1.0. Over consolidated clays tend to consolidate rather slowly until new stresses exceed those the material may have experienced in the past. Over consolidated deposits have an OCR greater than 1.0. Recent testing as part of the Everglades ASR project indicates that the Hawthorn Group confining unit is mostly over consolidated with an OCR of 1.2 to 2.0 (USACE, 2005). For this paper, a theoretical future ASR well field was located in central Martin County, Florida east of Lake Okeechobee. In this location, the Surficial Aquifer System (SAS) overlies the Hawthorn Group which in turn overlies the carbonate Floridan Aquifer System (FAS). A conceptual geologic section of the study area is shown on figure 1. At the idealized project site, the SAS is 45.7 meters thick and is composed of sand, shell, and silty-sand with a few limestone layers. The consolidating clay layer is the Hawthorn Group which is approximately 183 meters



thick. Underlying the Hawthorn Group is the carbonate FAS. Subsequent paragraphs describe the character of the Hawthorn Group in more detail.

### Conceptual Geologic Section

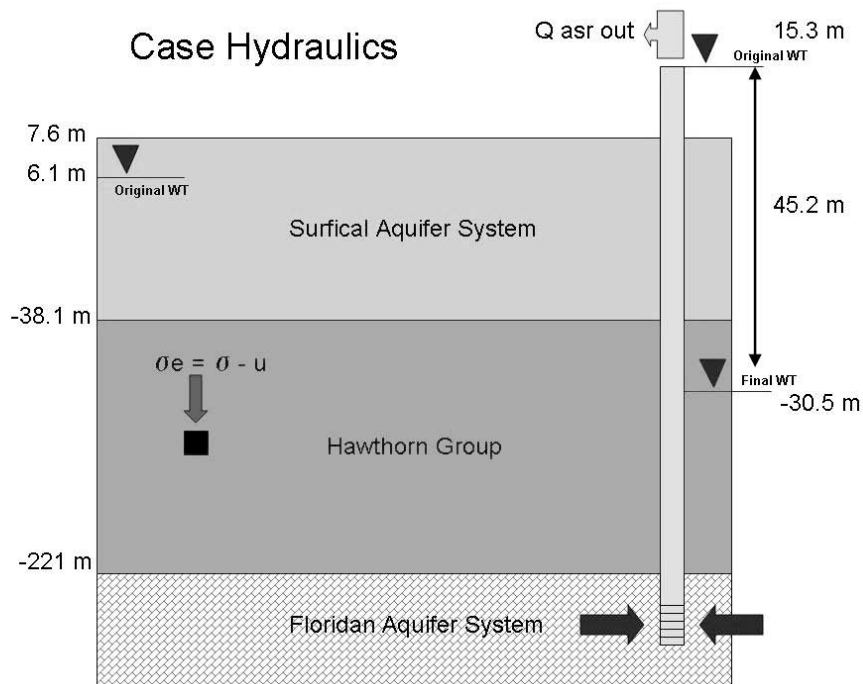


**FIGURE 1. Conceptual Geologic Section of Theoretical ASR Well Field in Martin County, Florida.**

The Hawthorn Group is a complex geologic unit composed of interbedded and intermixed carbonate and siliciclastic sediments containing varying percentages of phosphate grains (Scott, 1988). The upper portion of the Group is typically less well indurated and contains primarily interbedded sands and clays. The lower portion of the Group is more indurated and contains more abundant carbonate layers. Recent core borings completed into the Hawthorn Group in Martin County confirm this. Two deep exploration holes in Martin County reveal that the character of the unit is quite variable. In one hole near Lake Okeechobee in western Martin County, the Hawthorn Group was composed of 57% clay or silt, while at a hole in central Martin County, the Group only contained 15% clay or silt.

For this paper, it was assumed that the artesian piezometric head in the FAS was lowered during ASR recovery by approximately 45 meters such that the pore pressures in the Hawthorn Group clays began to dissipate causing deformation.

Figure 2 depicts the case study hydraulics including the SAS water table elevation and the initial and final piezometric head in the FAS.



**FIGURE 2. Case Study Initial and Final Hydraulics.**

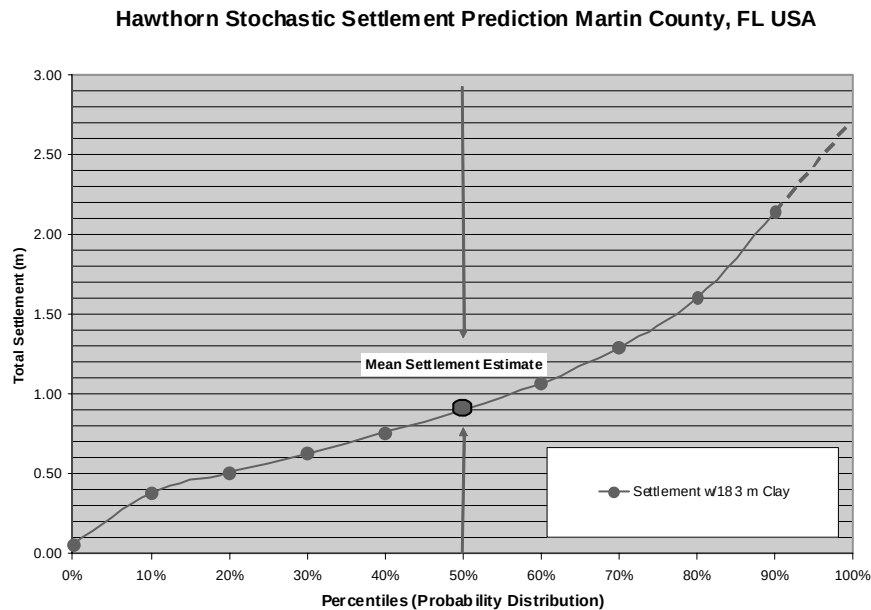
## RESULTS

Based upon the problem definition and using classical soil mechanics and consolidation theory as discussed previously, a stochastic model using Crystal Ball® was developed to examine the potential subsidence at the project site. Two models were developed. First, the magnitude range of total subsidence was estimated using classical clay consolidation theory. Second, the rate of consolidation was modeled to determine what percentage of the total magnitude can be realized during an ASR project life of 50 years. For the total magnitude of the consolidation, the following variables were assigned probabilistically based upon observed ranges in laboratory test data:

- $C_c$  – range from 0.04 to 0.40 log-normally distributed variable with a mean value of 0.08;
- SAS  $\gamma_m$  – range from 17.5 to 20.5 kN/m<sup>3</sup> normally distributed variable with a mean value of 19 (component of  $P_0$  and  $P$ );

- Hawthorn Clay  $\gamma_m$  – range from 15.5 to 18.5 kN/m<sup>3</sup> normally distributed variable with a mean value of 17 (component of  $P_0$  and  $P$ );
- $e_0$  – range from 0.575 to 0.75 normally distributed variable with a mean value of 0.66.

The model was then run forward for 20,000 iterations and cumulative probability functions were prepared. The mean settlement or deformation (50 percentile) predicted by the model is 0.91 meters at the ASR well head. Figure 3 depicts the percentiles generated from the model. The 90% percentile or the 90% confidence interval indicates that the model is 90% sure that that the total deformation will not exceed 2.14 meters. The overall probability of having 2.14 meters of deformation is only 10% though ( $1 - 0.90$ ).

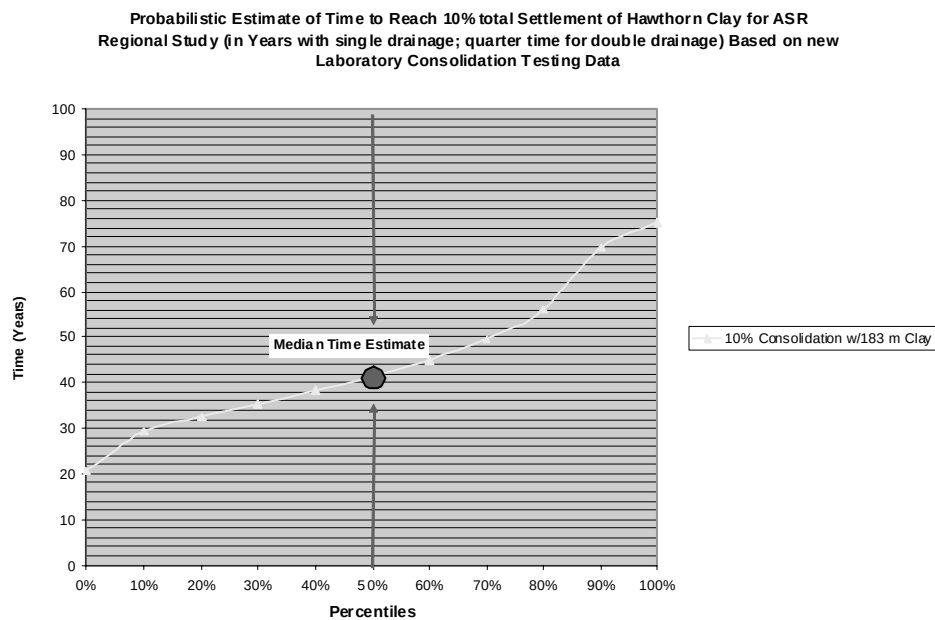


**FIGURE 3. Probability percentiles for model predicted total settlement or deformation.**

After determining the total magnitude of the settlement, the time rate of settlement was analyzed stochastically using equation (3) presented previously. Only one variable was input as a probability distribution. The variable assigned distribution based upon observed ranges from laboratory test data (cv):

- $c_v$  – range from  $1.08 \times 10^{-5}$  to  $5.38 \times 10^{-4}$   $\text{cm}^2/\text{day}$  normally distributed variable with a mean value of  $2.7 \times 10^{-4}$ .

The model was then run forward for 20,000 iterations and cumulative probability functions were prepared. The mean settlement time to reach 10% consolidation is predicted by the model to be 41 years. The earliest possible time to reach 10% consolidation based upon the 10% probability is 29 years. Similarly, the latest time to reach 10% consolidation is 70 years. Figure 4 depicts the percentiles generated from the model.



**FIGURE 4. Probability percentiles for model predicted time to reach 10% consolidation.**

## DISCUSSION

The stochastic modeling performed for the theoretical ASR well project in Martin County, Florida predicts that some subsidence of the Hawthorn Group clays will occur during a 50 year project life. Over the 50 year project life, the model predicts that between 0.04 m and 0.21 m of subsidence will occur based upon the estimates for settlement magnitude and rate. The model assumes that the ASR wells will be recovering continuously for the life of the project when in fact; the amount of recovery time could at most be 50% of the total time. Therefore, the actual range of

subsidence will be one half or less than the model predicts depending upon actual ASR recovery durations. In addition, the model assumes that the Hawthorn Group is made up of 100% clay or silt that is subject to long-term consolidation. In fact, much of the Hawthorn Group is composed of sands and carbonates such that subsidence would likely be reduced even further. Therefore, it can be concluded that ASR well fields sited in Martin County should not induce a large magnitude of subsidence. Assuming that ASR recovery only occurs 50% of the time during a 50-year project life and that the character of the Hawthorn Group is at most 50% compressible materials, the estimated subsidence should range from 0.01 m to 0.05 m (~2 inches).

## CONCLUSIONS

This study continued work started by the USACE and SFWMD to evaluate the potential subsidence resulting from the proposed CERP ASR project. This study evaluated a theoretical ASR well field located in central Martin County, Florida and determined that the range of estimated subsidence would be 0.01 m to 0.05 m (~2 inches). This subsidence envelope should be tolerable considering that the actual subsidence should not impact any nearby structures since the CERP ASR wells will be sited in more rural locations in south Florida. In addition, due to well field hydraulics, the ASR drawdown during recovery will rapidly approach less than 10 meters several 100 meters away from the well field such that the area affected may not be large as compared to other published subsidence cases.

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## AN INVESTIGATION OF AQUIFER RECHARGE SITES IDENTIFICATION USING SPATIAL TECHNOLOGIES

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### ABSTRACT

Proper site selection for a recharge structure is no less important than the other aspects of the entire recharge program. This is again more realistic where the country rock is hard and fractured. Upper catchment area of Kangsabati river basin, Purulia, eastern India bounded by latitude  $23^{\circ}05'$  to  $23^{\circ}30'N$  and longitude  $86^{\circ}$  to  $86^{\circ}20' E$ , the present study area belongs to peninsular India and comprises of hard crystalline gneissic rock. The terrain has low porosity (less than 5 percent) and very low permeability ( $10^{-1}$  to  $10^{-5}$  Darcys). This area is agriculturally drought prone with an average annual rainfall of 1200 mm. The regional slope is moderate and this prevailing situation promotes high surface run-off and low infiltration. On the basis of above context, the entire area has been studied for appraisal of terrain characteristics pertaining to identification of artificial recharge sites and creation of a digital data base. Composite map(s) (Scale 1:50,000) involving the individual thematic layer like drainage density, hydrogeomorphology and lineament density reveals the suitable sites for locating artificial recharge structures. Areas having low values of drainage density ( $1.55\text{km}/\text{km}^2$  to  $2.97\text{ km}/\text{km}^2$ ) and characterized by alluvial and/or weathered residuum and palaeochannel overlying fractured basement crystallines appear to be prospective sites of groundwater recharge.

### KEYWORDS

Composite map(s), Kangsabati River, recharge sites

### INTRODUCTION

Aquifer recharge occurs in nature by precipitation, seepage from canals, reservoirs and return flow from irrigation. In order to augment the depleting groundwater resources, it is essential that the surplus monsoon run-off that flows non dedicated, is conserved and recharged to increase groundwater resources. In the wake of high demand of groundwater, certain areas are over exploited, causing decline of groundwater levels and increased pumping lift, thereby necessitating deepening of structures. Excessive groundwater depletion affects major regions of North Africa, the Middle East, South and Central Asia, North China, North America and Australia, and localized areas throughout the World (Konikow and Kendy, 2005). In such areas artificial recharge schemes may be taken up to augment the depleted groundwater reserve, and to arrest or reverse the declining trend of groundwater level. Artificial recharge has a big role to play in water management in India (Das, 2006). Watershed

management, using contour bonding, gully plugs, check dams, percolation tanks and subsurface dyke is found to be very effective for augmenting groundwater recharge in hard-rock areas. On the other hand, recharge wells and basin-spreading techniques can be utilized in alluvial areas (Kittu, 2003). Johad, popularly known as tank in most part of India, is a well known traditional system of water storage for lean periods in several parts of peninsular India. Undoubtedly, Johad (figure 1) construction plays a major role in recharging groundwater (UN-IAWG-WES, 1998). It also initiates large-scale economic and ecological regeneration. In the above connections, appropriate site selection for aquifer recharge is an essential exercise.



**FIGURE 1. A typical Johad in Peninsular India. Water storage is seen in the central portion of the photograph. Photo taken during dry season (February, 2007) at Alwar, Rajasthan.**

About 79 percent area of India is drought prone (Sahai, 1988). According to Indian National Water policy 2002, water resources development and management will have to be planned for hydrological unit such as drainage basin as a whole or for a sub-basin, multisectorally, taking into account surface and groundwater for sustainable use incorporating quantity and quality aspects as well as environmental conditions. In India, Central Groundwater Board (CGWB), Govt. of India, has classified groundwater areas into white, grey and dark blocks, respectively, corresponding to areas, where groundwater can be harnessed, where it can be harnessed with cautions, and where it cannot be harnessed any more. Artificial recharge is called for grey and dark blocks (Thatte, 2003). Present study area falls in the grey block. The area is located in the upper catchment of Kangsabati (Kasain) river basin, Purulia, situated in the West Bengal state of eastern India and is bounded by latitude  $23^{\circ}05'$  to  $23^{\circ}30'N$  and longitude  $86^{\circ}$  to  $86^{\circ}20'E$ . It belongs to peninsular India and comprises of



mainly hard crystalline gneissic rocks. The terrain has low porosity (less than 5 percent) and very low permeability ( $10^{-1}$  to  $10^{-5}$  Darcys) (Sahu, 2001). Here the rural population is suffering from poverty and occurrence of drought adds misery to their lives. However, the average annual rainfall of this region is 1200 mm, and about 80 percent of which takes place during July-August. High surface run-off due to uneven relief of the terrain, crystalline nature of the country rock and temporally non uniform distribution of rainfall are responsible for inadequate groundwater recharge. In semi-arid to sub humid climate, granite usually exhibits a pediplain-inselberg topography. Therefore, a geomorphological approach has been found successful for proper appraisal of the hydrological condition of the area. One of the major advancements made in geomorphology is the quantitative analysis of drainage basin. Again, one of the purposes of fluvial morphometry is to derive information in quantitative form about the geometry of fluvial system that can be correlated with hydrologic information (Strahler, 1964). In general a well developed drainage pattern implies relative low infiltration and relative impervious materials, while absence of surface drainage is indicative of high infiltration and general perviousness (Rao, 1984). It has also been observed that the terrain transmissibility is inversely proportionate to the square of drainage density (Omar, 1990). Das and Kader (2003) observed that combined effect of drainage density ( $0.15\text{km/km}^2$ - $14.76\text{km/km}^2$ ), stream frequency ( $0.95\text{km}^2$ - $12.11\text{km}^2$ ), bifurcation ratio (2-10) and granitic lithologies favor high surface run-off and low infiltration. The presence of lineaments has immense importance in hard-rock hydrogeology as they can identify rock fractures that localize groundwater (Das, 1990). Hydrogeologists usually infer subsurface hydrological condition through surface indicators, such as aerial geological features and linear structures.

### **Role of Spatial Technology**

Remote sensing provides multi spectral, multi temporal, multi-sensor and multivariate data of the earth's surface. Geographical Information System (GIS) has emerged as a decision support system with capabilities of efficient data storage and analysis of the spatial data from diverse sources. The practice of integrating remote sensing and GIS is relatively new. Combination of the two techniques has been proved to be an useful tool in groundwater studies (Gustafsson, 1993; Krishnamurthy et al. 1996; Shankarnarayan et al. 1996).

### **Aims of the Study**

In the backdrop of foregoing observations, an investigation on the identification of the sites suitable for aquifer recharge is carried out with the following objectives.

- 1) To make an appraisal of terrain characteristics (geomorphology), geological structures (lineaments), drainage and land use/land Cover pertaining to groundwater recharge.
- 2) To test the capability of integrated remote sensing and GIS techniques in identification of the sites for aquifer recharge.
- 3) To prepare a digital data base revealing the locations for aquifer recharge.

## METHODOLOGY

The methodology adopted in the present study involves several steps mentioned below and is explained schematically in figure2.

1. Analysis of Survey of India(SOI) topographical sheet(no.73I/3, scale 1:50,000) and geocoded Indian remote sensing satellite imagery(IRS-IB, LISS II, FCC data, scale 1:50,000 and 1:2,50,000) dt.25<sup>th</sup> February 2001. IRS-LISS-II(Linear Imaging Self Scanning Sensor) operates in the visible-near infrared (NIR) spectral range of the electromagnetic spectrum and has a spatial resolution. of 36.25m. IRS imagery is interpreted visually by analyzing the elements like tone, texture, shape, size and pattern and the geomorphic expressions like drainage, hills, ridges and plains. False color composite(FCC) from bands 4,3,2 coded in red, green and blue color scheme which highlights the vegetation cover, land use and geomorphology(Saraf et al, 2001). Satellite data of February month provide the scenario of perennial and ephemeral streams and other water bodies.

2. Consultation of existing data.

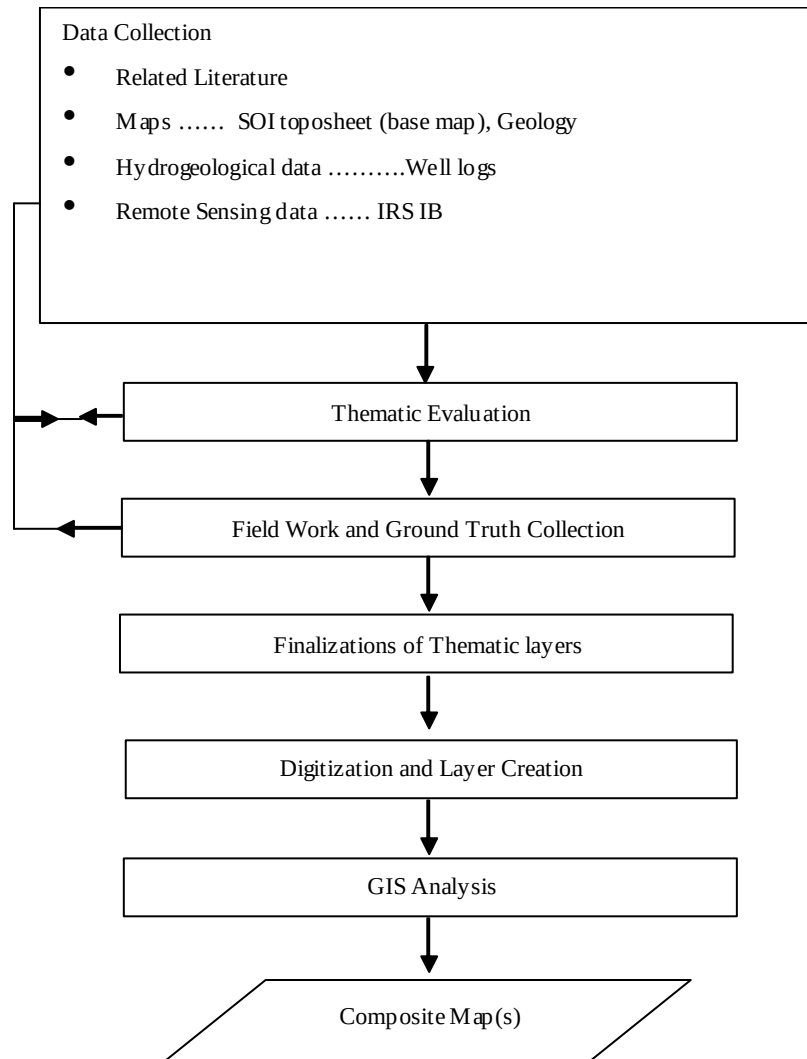
3. Preparation of drainage basin map using SOI topographical sheet (used as a base map) and satellite imagery. Consultation of satellite imagery in this regard helps to update the drainage information in particular. Subsequent morphometrical analysis of 3<sup>rd</sup> order drainage (Strahler, 1952) sub-basins has been done. Third order sub-basins are chosen as they comprehend considerable area for morphometrical measurements.

4. Preparation of pre-field thematic working maps on geology, hydrogeomorphology, structure emphasizing the lineaments in particular is performed. Lineaments are marked based on the alignments of streams and valleys, vegetation, and structural features. Lineament density is derived from grid analysis.

5. Verification of pre field thematic working maps is done with selected field checks. Assessment of water table conditions is performed through well inventory method.

6. Preparation of final thematic maps.

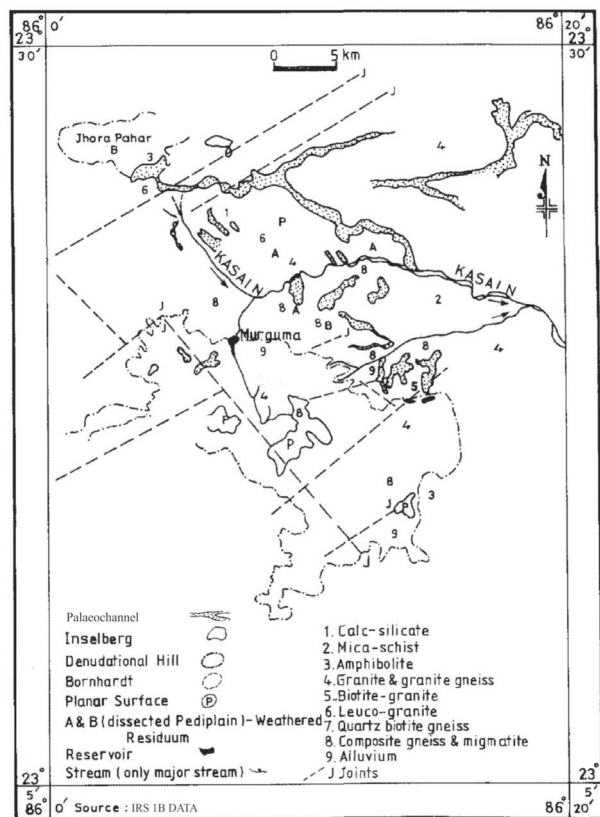
7. Digitization of all the thematic layers and subsequent overlay/cross analysis of the pertinent layers in GIS environment using TNT mips software package ( 7.2 version) for delineating the suitable sites for aquifer recharge in the produced composite map(s) (digital data base) is accomplished.



**FIGURE 2. Schematic of the data flow and GIS analysis followed in the present study.**

## RESULTS AND DISCUSSION

In the study area, among the two physiographic divisions, high land is composed of denudational and structural hills. Physiographically, the area is a southeasterly extension of Chhotanagpur plateau which is believed to be a rejuvenated landmass. Various erosional landforms like planar surface, bornhardt (at Jhora Pahar), inselberg, and undulatory valleys at different altitudes (240m to 520m) are the evidences of works of tectonics and erosional processes for a vast interval of time on the Precambrian basement rocks. The area has been subjected to several cycles of erosion aided by neotectonic upliftment (Singh, 1969). The major rock types of the area are different varieties of granite, granite gneiss, composite gneiss and migmatite with some occurrences of calc-silicate, mica-schist and amphibolite. The rocks are traversed by several sets of joints among which NNW-SSE and ENE-WSW master joints are prominent (figure 3) which show a dip amount of  $60^{\circ}$  -  $70^{\circ}$ . Low lying region

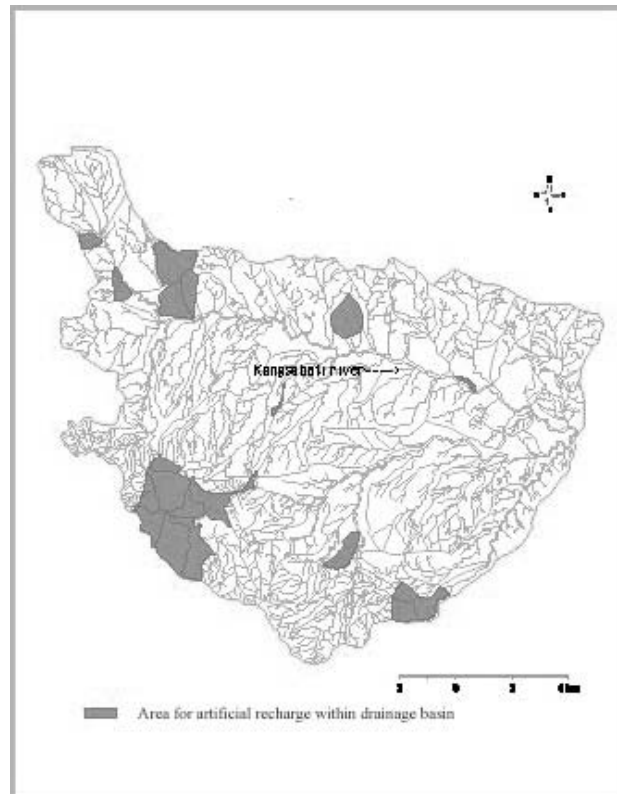


**FIGURE 3. A generalized geomorphological and geological map of the area.**

of the study area is undulatory and forms the pedepain region. The thickness of the weathered residuum as revealed from field evidence varies from 5m to 15m. Buried pediment occurs both in pediplain and planar surfaces of higher elevation. Spatial

extension of alluvium is limited. Natural forest cover is predominant in the southern hilly region.

Observed drainage pattern is over all dendritic (figure 4). However, parallel and to some extent rectangular pattern of drainage development is also present when observed locally. Presence of rectangular and parallel drainage pattern corroborates structural control over drainage development. The higher order streams of the area show the evidence of structural control over their development with the presence of straight segments in their course. The number of the 3<sup>rd</sup> order streams is 63 and the Kangsabati river is of 6th order and flows south easterly. The tributaries are ephemeral and a few of them maintain a small amount of flow (figure 5) during dry season (during drought) probably due to their effluent nature. Close to the source of the streams both valley widening and down cutting are active, the former as a result of slope retreat phenomena and the later due to the high gradient. The valley often experience floods during monsoon rains. Field evidence shows that the Kangsabati is an under fit river. Identification of older fluvial cycle (palaeochannel) with the help of satellite data aided by field checks suggests that the Kangsabati river basin is probably a part of an older fluvial system.



**FIGURE 4. Digitized drainage map is draped on a composite map.**



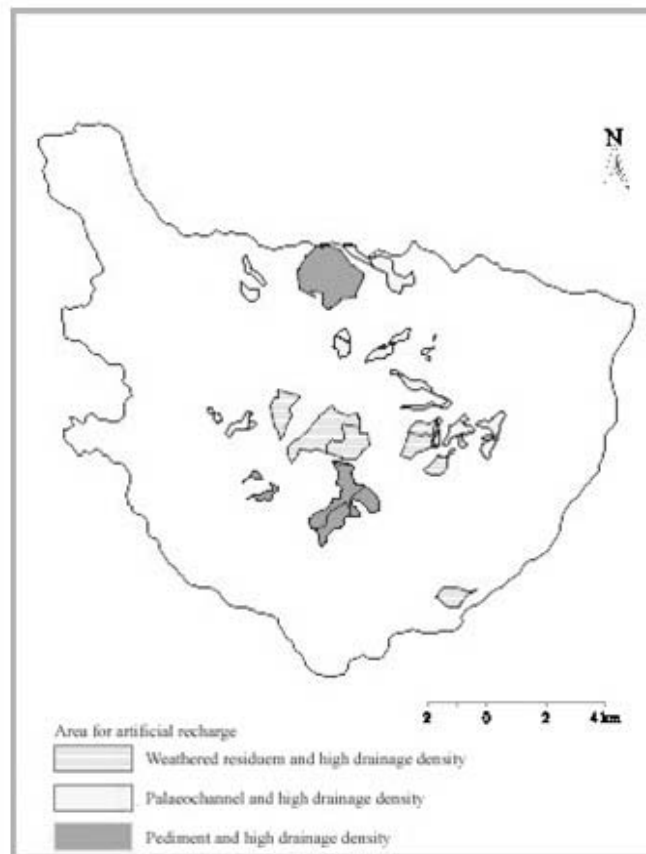
**FIGURE 5. An effluent tributary flows along a structural joint. A pre-monsoon photograph.**

### GIS Analysis

When a wide range of mono-disciplinary resource maps are available, the users must seek ways in which the available information can be combined to give an integrated overview, or reclassification or generation as needed (Burrough,1990).The criteria for any analysis are dependent on the objective and also on the data sets. In the present investigation, different thematic maps are generated on the basis of hydrogeomorphology and other surface geological expressions of a potential aquifer. Thematic mapping is done through visual interpretation of satellite data aided by field checks. Subsequently digitization, layer creation and overlay/cross analysis (using Boolean logic) involving pertinent thematic vector layers are carried out for proper accomplishment of the present work. Generated composite maps (figure 4 and figure 6) reveal the suitable sites for artificial recharge.

According to the surface-subsurface water interrelationship, it has been observed that mostly the inselbergs, basin divides and linear ridges act as run-off zones. The pediment-inselberg complex, denudational hills, shallow weathered pediments and other upland areas act as run-off cum recharge zones. The pediplain and upper reaches of alluvial plains generally act as recharge-cum-storage zones capable of storing and supplying large quantities of groundwater (Reddy and Rao2003). In the present investigation, intersection areas (figure 4) with low drainage density values( $1.55\text{km}/\text{km}^2$  to  $2.97\text{ km}/\text{km}^2$ ),deep seated interconnected open fractures (lineaments), buried pediments, weathered residuum and/or paleochannels appear

to be the best potential sites to act as rechargeable aquifers. Drainage map has been draped on this composite map (figure4) to show the spatial association of rechargeable areas with the relevant sub-basins. Sub-basins with high drainage densities are favorable for surface run-off and do not encourage much natural recharge. However, intersections (figure 6) of sub-basins with higher drainage density values ( $3.13\text{km/km}^2$  to  $7.41\text{km/km}^2$ ) with weathered residuum/palaeochannel/buried pediments may encourage aquifer recharge when those intersections are bounded by water impounding structures. Moreover, these created water reservoirs may act as surface water storage which in turn can be used in conjunction with groundwater in dry seasons for various uses. Nevertheless, arresting high surface run-off is not always desirable as it may adversely affect down stream aquatic ecology and natural process of aquifer recharge by influent streams. In fact, hydrologic alteration from dams, diversions, canals and other water developments invariably manipulates the natural flows of rivers to some extent (Moore, 2005).



**FIGURE 6. Areas for artificial recharge are shown within basinal boundary.**

During the present work, it is also observed in the field that areas with gentle inclination (less than  $40^\circ$ ) of structural joints show little fluctuation of water table whereas, the area with higher inclination (greater than  $50^\circ$ ) of structural joints show higher recession of water table with the approach of dry months. The wells which have tapped the depth less than the entire thickness of the weathered residuum, found to contain meager amount of water during dry season. Several small surface water storages including a large one at Murguma (figure 3) have been constructed in the area mostly by the local people applying their own traditional wisdom of water harvesting. The prevailing land use/land cover of the area is not very congenial to aquifer recharge. Many of the recharge ponds have been constructed in the foot hills of barren slopes (figure7), and this situation simply aggravates huge silt accumulation in the ponds during monsoon run-off. Combating this adverse geo-environmental condition needs a separate study.



**FIGURE 7. A recharge pond is situated in the vicinity of a barren hill slope. A pre-monsoon photograph.**

## CONCLUSIONS

Integrated geographical information system (GIS) and remote sensing is attracting increasing attention in natural resource studies including groundwater. The development of satellite remote sensing technology is obviously influencing hydrogeological research and applications. In this regard, improved characterization of the landforms owing to better and more efficient satellite observation of the geomorphic features and land cover/land use related to groundwater recharge is quite noteworthy. Movement and occurrence of groundwater are controlled by the lithology, structural pattern, geomorphology, soils and land use/land cover of an area. All these information are available from a single satellite imagery. Delineation of a technically feasible area (open deep-seated fractures, weathered residuum and



palaeo channel courses) in the composite map is one of the most desired task for groundwater development and siting artificial recharge/water harvesting structures. Since the crystalline rocks of the area have low permeability, rechargeable aquifer is located within channel fill of present and older fluvial systems, which are mostly engraved in the deep-seated interconnected fractures and in the weathered zone.

In recharge site selection, it is well known that electrical resistivity method is widely used in the hard-rock terrain. However, according to Integrated Water Resources Management (IWRM), research pertaining to find out low cost and time efficient technique is necessary so that even the smaller non-governmental organizations (NGOs) could use these techniques (Das,2006). According to Hoffman (2005), there is an effective communication gap between the research community driving the instrument development and operation and the hydrogeologic research community. It is a fact that satellite imageries of visible (0.38  $\mu\text{m}$ -0.72  $\mu\text{m}$ ) to near infrared (0.72  $\mu\text{m}$  -1.3  $\mu\text{m}$ ) region of electromagnetic spectrum are very much useful in extracting information on aerial aspects of drainage basin and various hydrogeomorphic features (Solomonson, 1983). These extracted features are quite useful in hard-rock hydrogeological studies as groundwater condition in hard-rock terrain vary widely and its understanding is necessary for locating suitable sites for aquifer recharge. The case study discussed in this paper represents only the hydrogeological application area of remote sensing and GIS techniques. Various remote sensing and GIS tools have provided the opportunity to analyze geo-environmental data in ways that have not been possible in the past. Application of this integrated technique in creating a digital database related to hydrogeomorphology/geology of rechargeable sites is time and cost effective. Research in geo-informatics based low cost instrumentation is necessary and NGO staffs should be trained in operating such instruments and in qualitative interpretation of the results.

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## FORMATION AND ATTENUATION OF DBPS IN GROUNDWATER AT ASR SITES

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### KEYWORDS

ASR; biodegradation; disinfection by-products; trihalomethanes

### EXTENDED ABSTRACT

Quantitative understanding of the fate of disinfection by-products (DBPs) in groundwater is of value due to concerns regarding human health effects. This paper summarizes the main outcomes of work on the fate of selected DBP compounds in groundwater as was previously reported by Dillon and Toze (2005), Pavelic *et al.* (2005) and Pavelic *et al.* (2006). Rates of THM attenuation were quantified and related to the geochemical characteristics of the subsurface environment at multiple ASR sites. In addition, the observed incidences of THM formation in groundwater were noted and predictions made on THM formation kinetics within the aquifer.

Eight ASR field sites from Australia and the USA were selected for this study: Bolivar, South Australia; Jandakot, Western Australia; Charleston, South Carolina; East Bay, California; Las Vegas, Nevada; Memphis, Tennessee; Oak Creek, Wisconsin and Talbert Gap, California. Collectively, they cover a broad range of subsurface environments (eg. alluvial sands and gravels through to fractured rock and limestone aquifers) and operating conditions (eg. injected volumes of  $10^3 \text{ m}^3/\text{yr}$  to  $>10^5 \text{ m}^3/\text{yr}$ ).

Trihalomethanes (THMs) and a conservative tracer (either chloride or fluoride ions, or electrical conductivity as a surrogate), were monitored in the injectant and the groundwater during the storage and/or recovery phase (limited haloacetic acids data have been analyzed but are not discussed here). A mass balance approach was used to determine the proportion of injected THMs that were recovered relative to conservative behaviour, as indicated by the conservative tracer. First-order attenuation rates were calculated from the rate of change in the mixing-corrected DBP concentration over time. Formation was examined by an assessment of

measured concentration changes and using an existing pipeline model by Clark and Sivaganesan (1998).

Mass balance calculations based on an input/output analysis showed evidence of THM removal within the aquifer at five of the sites. At two sites recovered concentrations of THMs were higher than that injected, even when mixing was taken into account, demonstrating that there had been significant in situ formation within the aquifer.

Three orders of magnitude difference in total TTHM attenuation rates occurred between the smallest half-lives at Charleston (< 1d) and the greatest half-lives at East Bay (120-550d) and Las Vegas (>120d). The ranking of total THM half-lives was as follows: Charleston < Jandakot ^ Bolivar ^ Oak Creek < Talbert Gap ^ Memphis < Las Vegas ^ East Bay. For individual compounds this ranking was not maintained due to differences in the proportion of chlorinated to brominated THM compounds between field sites. Generally, the order of removal for individual compounds was observed to be:  $\text{CHBr}_3 > \text{CHClBr}_2 > \text{CHCl}_2\text{Br} > \text{CHCl}_3$ .

It is well established that microbial processes are responsible for any substantial degradation that occurs (Bouwer and Wright 1988; Landmeyer *et al.* 2000). Account of the differences in half-lives focused on an examination of the geochemical environment within the zone occupied by the injected water mass during the period of monitoring in the groundwater that best represented the redox conditions in the aquifer when monitoring occurred. Degradation rates were found to be strongly dependent upon the redox state. Sites which were aerobic had half-lives in excess of 50 days, those that were anoxic (nitrate-reducing) had half-lives less than 70 days (with one exception), and those that were anaerobic (sulphate-reducing or methanogenic) had half-lives less than 20 days.

Evidence for the continued formation of DBPs in aquifers during ASR is strong given current knowledge of DBP processes, but there has been little evidence, apart from studies by Singer *et al.* (1993) and Fram *et al.* (2003) to conclusively demonstrate actual concentration increases and the kinetics involved.

Formation, defined as an increase in DBP concentration in groundwater relative to that in the injectant, was estimated by a variety of methods, including: i) comparing concentrations in the injectant and in the groundwater, ii) applying a formation model for distribution systems using known values of input water quality, and iii) back-projecting according to the rate of DBP degradation to predict the peak concentration relative to the injected concentration.

Continued formation of THMs in the aquifer during the storage phase was observed in a number of cases, and formation of chloroform was most commonly detected. Modelling suggests that all of the source waters have sufficient residual chlorine and NOM for substantial increases in THMs to occur. However, the low frequency of

sampling, combined with concomitant attenuation means that this is not always identified. Variance in the formation estimates between the various methods was large.

Formation takes the form of a growth curve that reaches a plateau once the residual oxidant (chlorine and/or bromine) in the source water is consumed. Attenuation (corrected for mixing with ambient groundwater) is characterized by exponential decay and persists until DBP concentrations are exhausted. The combined effect of these processes may give an initial peak where formation is dominant, followed by a gradual decay where attenuation becomes dominant. Failure to consider DBP formation with the ASR Risk Index (ASRRI) model was found to underpredict DBP concentrations in recovered water (Dillon and Toze 2005). Further work to more accurately reflect formation processes during ASR is the focus of current studies.

## ACKNOWLEDGEMENTS

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# OVER 100 YEARS OF DRINKING STORMWATER TREATED THROUGH MAR: ASSESSING THE RISKS OF STORMWATER RECHARGE ON THE QUALITY OF THE BLUE LAKE

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## ABSTRACT

Improved management of the urban water cycle, by putting all the available water resources to use instead of discarding them as waste is a high priority for semi-arid regions like those found in Australia. As water sources such as storm- and waste- water are generally lower quality than the current domestic supply, their reuse potential can be enhanced through the natural treatment provided by managed aquifer recharge (MAR). The city of Mount Gambier is an example of indirect potable reuse of stormwater, which has been in operation for over 100 years, allowing examination of the potential contaminant attenuation or treatment processes. Mount Gambier reuses urban stormwater by discharging it into an unconfined karstic limestone aquifer that recharges the Blue Lake, the city's domestic water supply. Understanding the risks associated with this stormwater to potable reuse scheme relies on estimates of aquifer residence time and attenuation processes within the system; including pre-treatment prior to aquifer discharge, passage through the aquifer and residence time within the Blue Lake.

## KEYWORDS

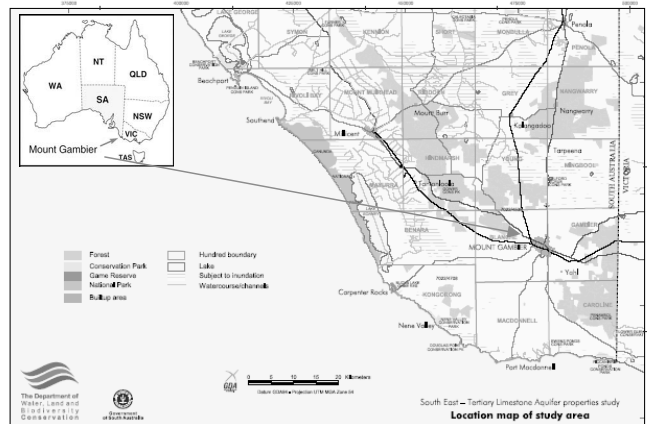
Stormwater reuse, risk assessment, karstic aquifer

## INTRODUCTION

Water shortages in urban areas necessitate the use of all the available resources, including storm- and waste- waters, often discharged as waste. However these resources are generally of lower quality than the existing domestic supply and the risks associated with their reuse must be considered. The reuse potential of resources such as storm- and waste- waters can be enhanced through the natural treatment provided by managed aquifer recharge (MAR). This paper presents a case study where a regional city in Australia has been using stormwater recharge for indirect reuse as drinking water, for over 100 years.

Mount Gambier is located in the south east corner of South Australia (Figure 1) on the flank of a volcanic complex formed around 28,000 years ago (Leaney *et al.*, 1995). Since settlement in the late 1800s, the city of Mount Gambier has grown to occupy an area of around 27 km<sup>2</sup>, with a population of approximately 23,000. The city's domestic

water supply is drawn from the Blue Lake, a volcanic crater lake that is recharged from the unconfined karstic Gambier Limestone aquifer beneath the city. Urban stormwater is also discharged directly into the unconfined aquifer via over 500 purpose-built drainage bores and karst features such as sinkholes. Thus, Mount Gambier reuses urban stormwater by discharging it into an unconfined aquifer that recharges the drinking water supply.



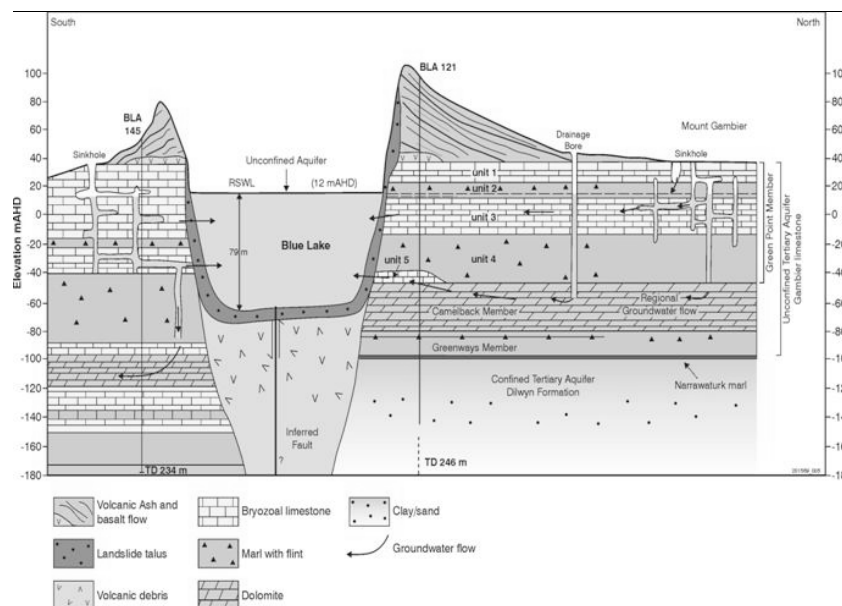
**FIGURE 1. Location of Mount Gambier (adapted from Mustafa and Lawson, 2002).**

The Gambier Limestone consists of alternating beds of bryozoal limestone and marl overlying a lower dolomitic Camelback member and again marl (Figure 2) (Hill and Lawson, in press). Near the Blue Lake, the thickness of the Gambier Limestone is approximately 100 m (Waterhouse, 1977). The top of the Camelback member is encountered at approximately -30 to -50 m Australian Height Datum (AHD) (Hill and Lawson in press) and this sub-unit is approximately 30 to 40 m thick. A confining layer of glauconitic and fossiliferous marls and clays separates the Gambier Limestone from the lower Tertiary Dilwyn Formation which consists of a series of unconsolidated sandstones and gravels with carbonaceous clay interbeds (Waterhouse, 1977). The major structural features are related to rifting and post-depositional tectonics, which have formed a northwest-southeast structural trend through the region. Such faulting is thought to represent a zone of regional structural weakness through which the volcanic activity occurred (Lawson *et al.*, 1993) and the major karstic features in the vicinity of Mount Gambier, are predominantly aligned in the northwest-southeast direction.

The unconfined aquifer within the Gambier Limestone has characteristics of a dual porosity medium with secondarily-developed karstic flow supplementing primary intergranular porosity. Pumping-test derived transmissivities vary over two orders of magnitude from 200 m<sup>2</sup>/day to greater than 10,000 m<sup>2</sup>/day, with porosity estimates

between 30% and 50% (Love *et al.*, 1993). Thus there is considerable potential for variation in groundwater flow rates to Blue Lake via pathways through the porous matrix of the aquifer and preferential flow through karst features. The regional groundwater flow direction is toward the southwest. However in the lake vicinity, the hydraulic gradient is low ( $10^{-4}$ ) and local flow systems and the karstic domain can reverse the flow direction (Richardson, 1990; Schmidt *et al.*, 1998). The dolomitic Camelback Member is believed to be the dominant source for groundwater flow into the lake. However, there is a lack of hydraulic and water quality data for the subunits of the Gambier Limestone unconfined aquifer that could be used to ascertain flow directions and rates.

Contaminant transport modelling in karstic aquifers is a significant challenge due to the large variation in possible flow rates and residence times within the aquifer, which in turn impacts on the potential for time-dependent attenuation. This work uses a risk assessment approach (NHMRC and NRMCC, 2004) to examine the impacts of stormwater recharge on the quality of Blue Lake. The risk assessment takes into account the uncertainty regarding travel time estimates when determining the potential contaminant attenuation or treatment processes within the system.

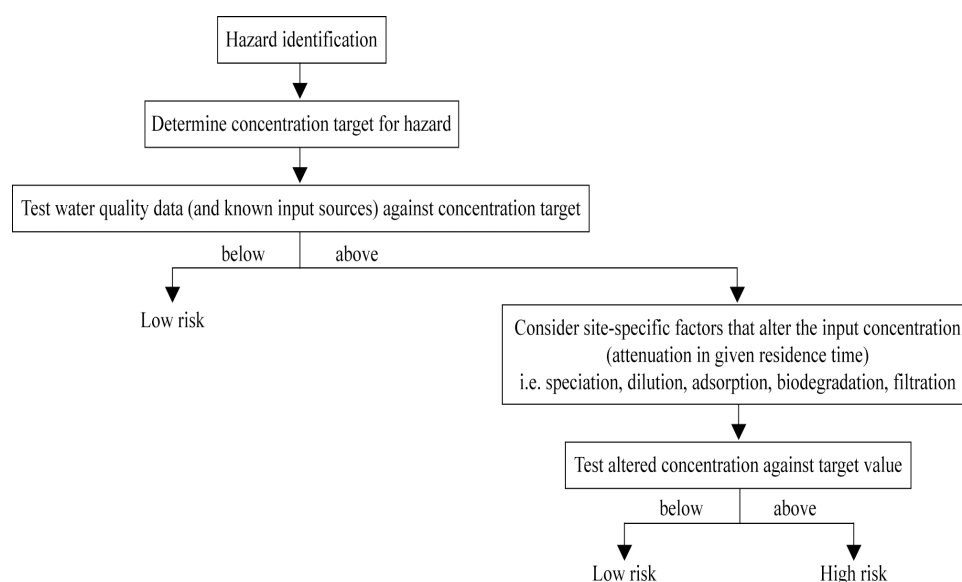


**FIGURE 2. Recent stratigraphic assessment in the vicinity of the Blue Lake (Hill and Lawson, in press).**



## METHODS

The risk assessment method used to assess the impact of stormwater recharge on the water quality of Blue Lake was consistent with the Australian guidelines for managing drinking water (NHMRC and NRMMC, 2004) and also for water recycling (NRMMC and EPHC, 2006) and is outlined in Figure 3. The first step was to identify hazards within the system and to set target values for those hazards. Target values were established with respect to potable water quality guidelines (NHMRC and NRMMC, 2004) and aquatic ecosystem water quality criteria (ANZECC and ARMCANZ 2000; SA EPA 2003). Land use mapping and examination of existing water quality data were used to identify stormwater contamination hazards. The concentration of potential contamination hazards, before and after site specific attenuation processes, was then tested against the target value to determine whether the risk level was high or low. The method allows for the risk assessment to progress from a simple qualitative assessment through to a fully quantitative assessment (@Risk<sup>®</sup> software was used in this case study) as the supporting input data develops.



**FIGURE 3. Framework to assess the impact of stormwater recharge on the water quality of the Blue Lake. Target values can be based on potable and environmental water quality guidelines.**

Site specific processes that may alter the input concentrations can include pre-treatment prior to stormwater discharge or natural attenuation within the aquifer or Blue Lake. The risk assessment was applied conservatively, taking a worst case scenario, where the minimum travel time in the aquifer (relevant for fracture flow)

was used to calculate attenuation. Sorption has not been included as the capacity of the sediments may be limited and the process may also be reversed. The processes included in this study were dilution and degradation in the aquifer, and degradation and volatilisation in the receiving water body (Blue Lake). Experimental programs are underway to improve the estimates of groundwater travel times, using SF<sub>6</sub> and pharmaceutical tracers, and to quantify the potential for removal of contaminants in Blue Lake in conjunction with an annual calcite precipitation cycle.

## RESULTS AND DISCUSSION

The urban activities deemed as posing the greatest risk to the water quality of Blue Lake through stormwater recharge were: high density traffic; dry cleaning; bulk fuel storage; service stations; timber processing sites using copper chrome arsenic (CCA) or creosote; and works depots storing bitumen. The associated hazards are perchloroethene (PCE) from dry cleaners, the fuel based hydrocarbons benzene, toluene, ethylbenzene and xylene (BTEX), polycyclic aromatic hydrocarbons (PAHs) and copper chrome arsenic (CCA) wood preservative. From the PAH group, phenanthrene, fluoranthene and pyrene were chosen for examination as they have been quantified in Mount Gambier's stormwater and are most commonly reported in urban stormwater (Makepeace *et al.*, 1995).

There are over 300 stormwater drainage catchments in Mount Gambier and each was given a score based on the number and type of high risk activities they contained (Figure 4). As a result, sixteen high priority stormwater catchments were identified (risk score =7) for detailed assessment of the measures taken to prevent contamination, including community awareness programs, chemical storage and handling procedures and stormwater pre-treatment prior to discharge.

The initial risk assessment presented here deals only with the potential organic hazards. In-lake attenuation, thought to be significant for the inorganic hazards (CCA), is currently under investigation and will be included in a revised risk assessment. Pathogens are not considered a high risk as the loadings in stormwater discharge are low and pathogen attenuation in the aquifer is efficient.

The behaviour of the potential organic contaminants was assessed by simulating attenuation or treatment through dilution and degradation in the aquifer and degradation and volatilisation in Blue Lake. The initial concentration for stormwater was based on the solubility limit of each species due to the lack of stormwater quality data for these species (Table 1). In all cases, the stormwater quality used in the risk assessment was far in excess of either measured concentrations in Mount Gambier's stormwater or literature values for stormwater. The groundwater concentration used for dilution was assumed to be zero, based on existing water quality data for the ambient groundwater. Degradation calculations used an aquifer residence time of 1.5-20 years, based on tracer test results to date and a Blue Lake

residence time of 6-10 years (Herczeg *et al.*, 2003). The ratio of the median simulated concentration to the target value was used as a risk indicator. A ratio above 1 is high risk and below 1 is low risk.



**FIGURE 4. Stormwater catchments prioritised by potential contamination from urban land use activities contained in each. Sixteen catchments scored =7 and were at the highest risk of contamination.**

Based on the maximum solubility, benzene was the highest risk hazard due to the potential for high concentrations in solution (Figure 5). The degradation of benzene in the aquifer is so rapid, with a half-life of 10-16 days, that predicted concentrations in the groundwater approached zero, which in turn resulted in lower concentrations in Blue Lake after a half-life of 5-16 days was applied. Benzene thus became among the lowest risk organic contaminants considered. Similar behaviour was exhibited for the other BTEX compounds.

Despite starting with median concentrations orders of magnitude greater than those recorded for Mount Gambier, calculated medians for the three PAHs (phenanthrene, fluoranthene and pyrene) after treatment in the aquifer alone or both the aquifer and the Blue Lake were below the aquatic ecosystem target of 3 “g/L. The simulations for pyrene, with the highest resistance to degradation (Table 1), predicted a

concentration in the groundwater close to the target value and a concentration in Blue Lake close to the current analytical capabilities for trace concentrations (0.1 “g/L).

**TABLE 1. Input data for assessment of risk posed by organic hazards in stormwater.**

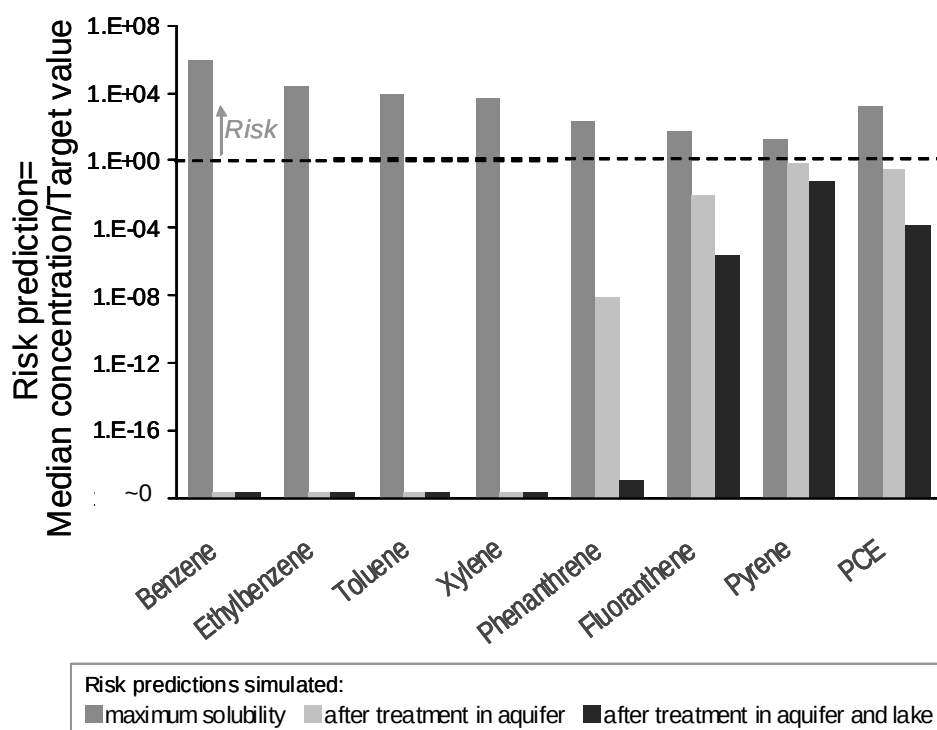
|                 | Target value       | Mount Gambier stormwater | Solubility limit (1) | Half-life (days) (1) |          |
|-----------------|--------------------|--------------------------|----------------------|----------------------|----------|
|                 | (µg/L)             | (µg/L)                   | (µg/L)               | Aquifer              | Lake *   |
| Benzene         | 1 <sup>d</sup>     | <1                       | 1.8 x10 <sup>6</sup> | 10-16                | 5-16     |
| Ethylbenzene    | 3 <sup>d,a</sup>   | <2                       | 1.5x10 <sup>5</sup>  | 6-10                 | 3-10     |
| Toluene         | 25 <sup>d, a</sup> | <2                       | 5.2 x10 <sup>5</sup> | 7-28                 | 4-22     |
| Xylene          | 20 <sup>d, a</sup> | <4                       | 1.8 x10 <sup>5</sup> | 14-100               | 7-28     |
| Perchloroethene | 50 <sup>d</sup>    | not monitored            | 1.5 x10 <sup>5</sup> | 360                  | 180-360  |
| Phenanthrene    | 3 <sup>e</sup>     | 0.0026-0.014             | 1.2 x10 <sup>3</sup> | 30-200               | 16-200   |
| Fluoranthene    | 3 <sup>e</sup>     | 0.0022-0.0077            | 2.7 x10 <sup>2</sup> | 280-400              | 140-400  |
| Pyrene          | 3 <sup>e</sup>     | 0.0018-0.0057            | 1.3 x10 <sup>2</sup> | 420-1900             | 210-1900 |

<sup>d</sup> based on the drinking water quality guideline (NHMRC and NRMCC, 2004)

<sup>e</sup> based on the aquatic ecosystem target value (SA EPA, 2003)

<sup>a</sup> aesthetic; \* surface water half-life is based on degradation and volatilisation; (1) Adams *et al.*, 2007

Perchloroethene (PCE) has high water solubility, but the degradation half-life estimates are longer than for the BTEX compounds, at 180-360 days. Starting from the solubility limit for PCE indicates that degradation and dilution in the aquifer reduces the PCE concentration to around one-tenth of the drinking water guideline. Additional treatment within Blue Lake leads to further reduction. Clearly the reduction in risk is less for PCE than for the BTEX compounds.



**FIGURE 5. Change in risk as stormwater based organic compounds (initial concentration is the solubility limit) are attenuated in the aquifer and the Blue Lake.**

## CONCLUSIONS

Indirect stormwater reuse has been operational in Mount Gambier for over 100 years. A risk assessment was used to evaluate some of the high risk activates in the city that could impact on the quality of drinking water source. The organic chemical hazards identified were perchloroethene (PCE), benzene, toluene, ethylbenzene and xylenes (BTEX) and polycyclic aromatic hydrocarbons (PAH). Despite the uncertainty in contaminant transport within the karstic aquifer system (1.5-20 years), the preliminary risk assessment suggests there is sufficient treatment capacity within the aquifer and the lake to reduce the risk from these organic chemical hazards. Nonetheless, the natural treatment should be coupled with management at the source and prior to discharge, in order to minimise the potential for contaminants to migrate through the system. It must be noted that the residence times estimates are crucial to degradation calculations and in this case study, residence times in the

karstic aquifer are quite long. Such treatment may not be achieved if residence times were of the order of days.

## ACKNOWLEDGEMENTS

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## RECENT RECHARGE POSSIBILITIES: DETERMINATION OF THE PLEISTOCENE AQUIFER SYSTEM OF WADI EL-ASSIUTI BASIN, EGYPT, USING HYDROGEOCHEMICAL AND ENVIRONMENTAL ISOTOPIC CRITERIA

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### ABSTRACT

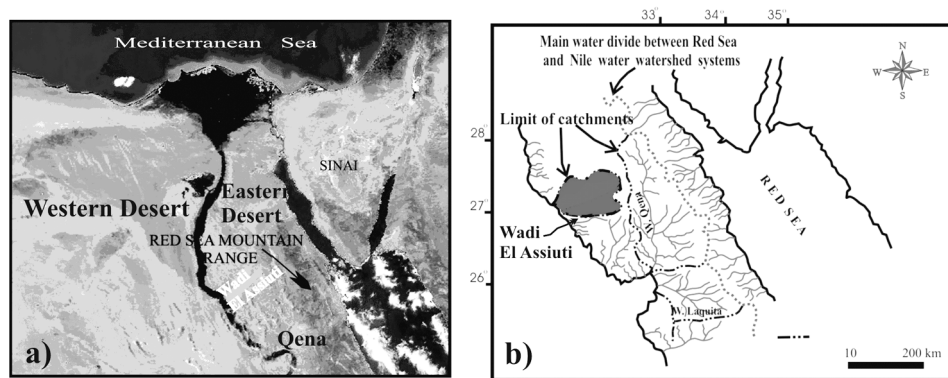
Wadi El-Assiuti is one of the largest dry valleys in Middle Egypt. It comprises prolific groundwater potentialities and suitable lands for agricultural expansion. Twenty one groundwater samples are recently collected from the drilled water wells tapping the Pleistocene groundwater aquifer. The groundwater salinity values vary generally from 580 mg/l to 2445 mg/l indicating fresh to brackish water. The variation in water genesis could reflect the diversity of the recharge sources for the Early Pleistocene aquifer. The isotopic results of the collected surface and groundwater samples are widely varied from -8.9 to 2.81 for  $O^{18}$  and from -61.07 to 21.11 for Deuterium. Almost 86 % of these samples fall in the depleted isotopic range, whereas the remaining percentage (14 %) is gradually enriched. The low slope and intercept values obtained from this correlation addresses the old origin of the main bulk of groundwater, which is represented by 86 % of samples. These samples carry the depleted isotopic signature of the paleowater characterizing the Eastern Desert of Egypt. The  $^{14}C$  age determination for two samples indicated that the Pleistocene water of Wadi El-Assiuti is mostly paleowater with a significant portion of relatively recent water recharging the aquifer from the rainfalls.

### INTRODUCTION

Wadi El-Assiuti represents the eastern expansionable developmental corridor for Assiut Governorate. The main trunk channel of it reaches about 110 km in length with an area of about 2700 km<sup>2</sup>. It is located between latitudes 27 00' N and 27 30' N and longitudes 31 00' E and 31 30' E. (Figs 1 a & b). The catchment basin of this wadi is situated entirely in the elevated Maaza Eocene limestone Plateaux that forms a distinctive landscape feature in the desert east of the Nile Valley.

Wadi El-Assiuti includes potential soils for agricultural development if irrigation water is available. Thus, development plans should be accompanied by high activities in searching for new water resources, especially the groundwater. Recently, the determination of recent recharge possibilities of the Pleistocene aquifer system of Wadi El-Assiuti became an urgent issue.





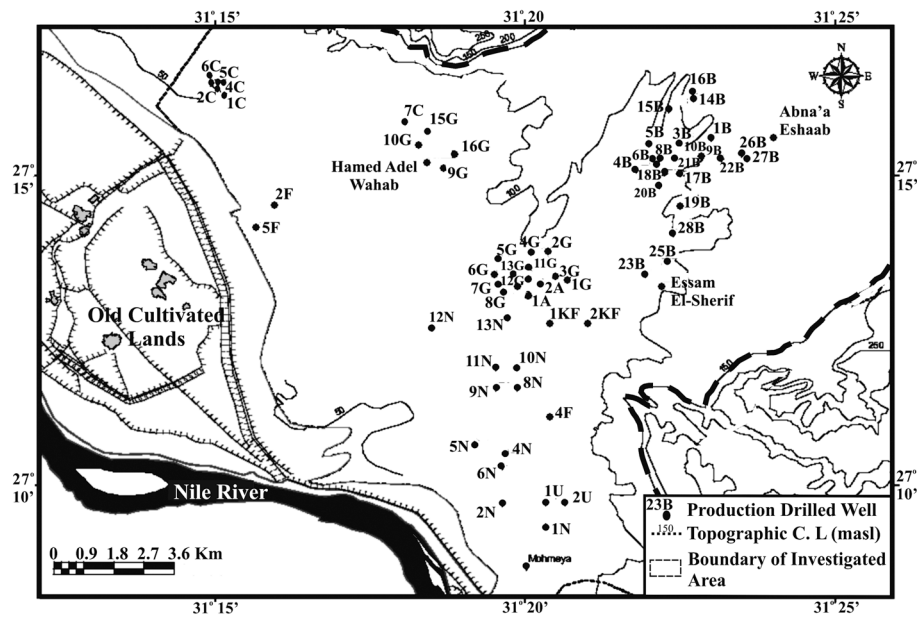
**FIGURE 1. Location map of the area of study: a) Satellite image showing the main catchment areas for Wadi El Assiuti watershed system. b) Wadi El Assiuti watershed and the main River Nile-Red Sea water divide.**

The area of study represents a part of the arid belt of North Africa, which is characterized by hot, dry and rainless climate in summer and being mild with rare rainfall in winter. Naturally, the air temperature increases during the summer periods and decreases during the winter periods.

### Methodologies and Approaches

In the present study, the authors investigated the recent recharge possibilities of the Pleistocene aquifer system. Twenty one water samples are collected during the year 2004 from the Pleistocene groundwater and Nile surface water. Subsequently, these samples were analyzed for major constituents, environmental stable isotopes and  $^{14}\text{C}$ . The interpretations of these data, besides the available geological, hydrogeochemical and hydrogeophysical information are integrated to assess the recharge mechanism and possibilities. The drilled production water wells locations are determined using field truth Geographic Positioning System (GPS) and a location map is constructed (Fig. 2). The drilled water wells are partially penetrating the aquifer successions of the Early Pleistocene times. The number of drilled wells is about 85 water wells, especially with the involvement of the private sector into the rehabilitation activities of Wadi El-Assiuti.

A total number of 30 vertical electrical soundings (VESes) are carried out in this area. The Schlumberger four symmetrical electrodes configuration is used. From the gained results of both quantitative and qualitative interpretations, a hydrogeophysical cross-section A-A' is introduced.



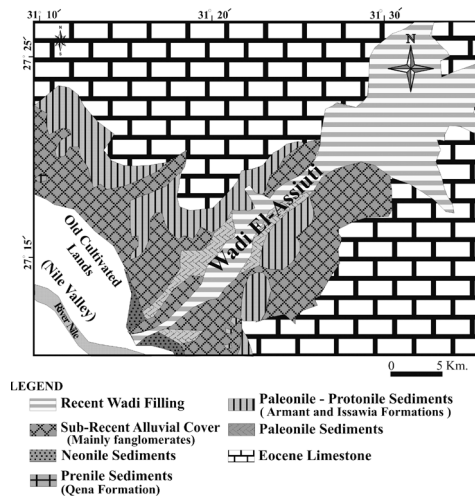
**FIGURE 2. Location map of the drilled water wells of Wadi El-Assiuti.**

## GEOLOGIC SETTING

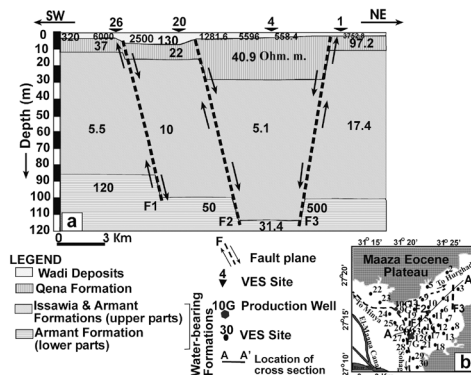
According to Said (1981), the geological sequence in Wadi El-Assiuti is starting (from top to bottom) by the Neogene of the Holocene, followed by the Preneogene of the Middle-Late Pleistocene, which is followed by the Proto/Preneogene and the Proterozoic of the Early Pleistocene, followed by the Late Paleogene of the Late Pliocene and the Early Pliocene, and ends at last by the Early – Middle Eocene (Fig. 3).

According to the drilled production wells in the area (Fathy and Nagaty 2005), from top to bottom, the sequence starts by loose gravels, sands and carbonate materials with a thickness that could reach 18 m, followed by Qena Formation with a maximum thickness of 27 m, followed by the Issawia Formation (Early Pleistocene) and finally the Armant Formation (Early Pleistocene). The Armant Formation is made up of alternating beds of locally derived gravels and fine-grained clastic rocks. The fine-grained clastic beds are calcareous, sandy, shaly, or phosphatic depending on the nature of the nearby source rocks. Issawia Formation overlies the Armant Formation and is made up of bedded travertines with minor conglomerate lenses, composed of limestone pebbles of local derivation and the overlying beds of red breccia with highly angular limestone pebbles cemented in a matrix of red muds. The Eocene sediments represent the oldest outcropping rocks and form a plateau land that occupies almost the entire basin. (Fig. 3). The structural setting of the area of study and its surroundings was studied by different authors as a part of the Nile Valley.

Youssef *et al.*, (1977) believed that the NW-SE faults that affect the Eocene rocks around Assiut are the result of a Post-Eocene uparching of rocks that caused the transmission of older NW-SE fractures vertically from the basement to affect the sedimentary cover.



**FIGURE 3. Geologic map of the area of study (after Said, 1981).**



**FIGURE 4. Illustrative subsurface aquifer situation by a) hydrogeophysical cross-section A-A' across Wadi El-Assiuti b) key map of cross-section.**

The constructed hydrogeophysical cross-section A-A' reveals the existence of two main trends for deep faults, the NW-SE and NE-SW (Fig. 4). These deep-seated faults are efficient conduits through which the upward leakage of Pre-Upper Cretaceous (Cenomanian) Nubia Sandstone groundwater takes place, thus providing an effective recharge source for the Early Pleistocene aquifer system of Wadi El-Assiuti. The upward leakage occurs where the low permeable cover of the Nubian strata is missing or its effects are reduced due to special lithological or structural conditions. However, the upward leakage of groundwater from the underlying Pre-Upper Cretaceous (Nubia Sandstone) aquifer to recharge the overlying Tertiary and Quaternary Formations is clearly indicated in the area occupied by Wells 6G, 7G, 12G, 13G, 9G and 10G by hydrochemical evidences given by the stable isotopes ( $O^{18}$  & D), as will be discussed later. These wells are lying in the vicinity of a fault plane (F1) (Fig. 4).

## Hydrogeological Setting

The hydrogeologic setting of Wadi El-Assiuti has been worked out by various workers, among them; Abu El-Ella (1999) and Mousa et al (1993), Yan et al (2004), Fathy et. al. (2005) and NARSS (2005).

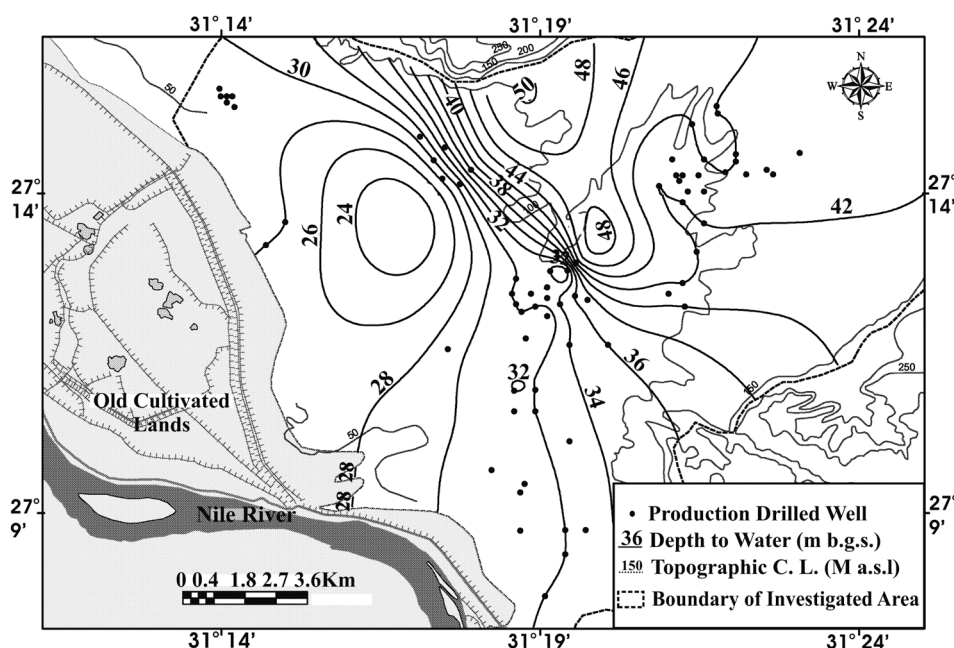
The hydrogeological setting of the area of study is as follows, from top to bottom:

5. The subsoil aquifer: It has no significance as an aquifer due to its composition, and it is of very small thickness.
6. The Qena Formation aquifer: It represents the main groundwater aquifer in the Nile Valley and its vicinities that reach the entrance of Wadi El-Assiuti. Inside Wadi El-Assiuti, it loses its significance as an aquifer due to its small thickness and its altitude. It is of the Middle – Late Pleistocene age.
7. The Issawia Formation (Early Pleistocene aquifer): It is the aquifer that is tapped by the shallow drilled wells in the area of study.
8. The Armant Formation (Early Pleistocene aquifer): It underlies the Issawia aquifer, and most of the drilled deep water wells ( $\pm 260$  m) are tapping its upper parts.

Wadi El-Assiuti watershed is one of the main Eastern Desert wadis that depouchs towards the River Nile. Wadi El-Assiuti hydrologic system comprises two flow components: 1) the surface water system of the Wadi El-Assiuti watershed and 2) the Wadi El-Assiuti groundwater flow system. The watershed collects rainfalls over the area of  $5589 \text{ km}^2$  and is channeled toward River Nile conveying precipitation over the Red Sea Hills and the adjacent mountains. The Quaternary alluvial aquifer floors the main channels within Wadi El-Assiuti watershed, whereas the surrounding outcrops are mainly Tertiary limestones (Fig. 3). Gheith and Sultan (2002) estimated that the infiltration to the limestone is limited and transmission loss to the alluvial aquifer along the stream channels is significant at an average annual rate of  $6 \cdot 10^6 \text{ m}^3$ . The flow to the outlet of the watershed is also minimal, only about 7 % of the total rainfall (Yan et. al, 2004). At the adjacent areas of the Nile Valley (near the old cultivated lands), the Quaternary sediments are mostly influenced by Nile recharge, and to a less extent by the surface runoff water from the Eastern Desert wadis. Traveling farthest inside the eastern tributaries of Wadi El-Assiuti, the influence of the River Nile diminishes, and the main recharging sources are from the upward leakage from older formations such as Nubia Sandstone or from the Red Sea mountainous ridges, outcropping to the far northeast of the area of study, and the wadis.

The depth to water is topographically controlled. The groundwater is available at depths ranging between 27.5 m (Well 5F) near the Nile Valley and 44.2 m (Well Abna'a Eshaab) below ground surface. It increases generally from west to east- northeast directions (Fig. 5). However, the depth to water contours configure approximately the topographic features of the hills and valleys, where the water being low where the surface of the ground is high.

It is noticed that the pumping of Well 9G started at 1992 with a depth of water of 31 m, and now its water depth became 34.1 m. Accordingly, there should be another recharge source that feeds these wells. This source could be the in situ local rainfalls on the wadis, such as Wadi Dahasa, Wadi Imu, etc, in addition to any other possible upward leakage through faults from older water-bearing formations.



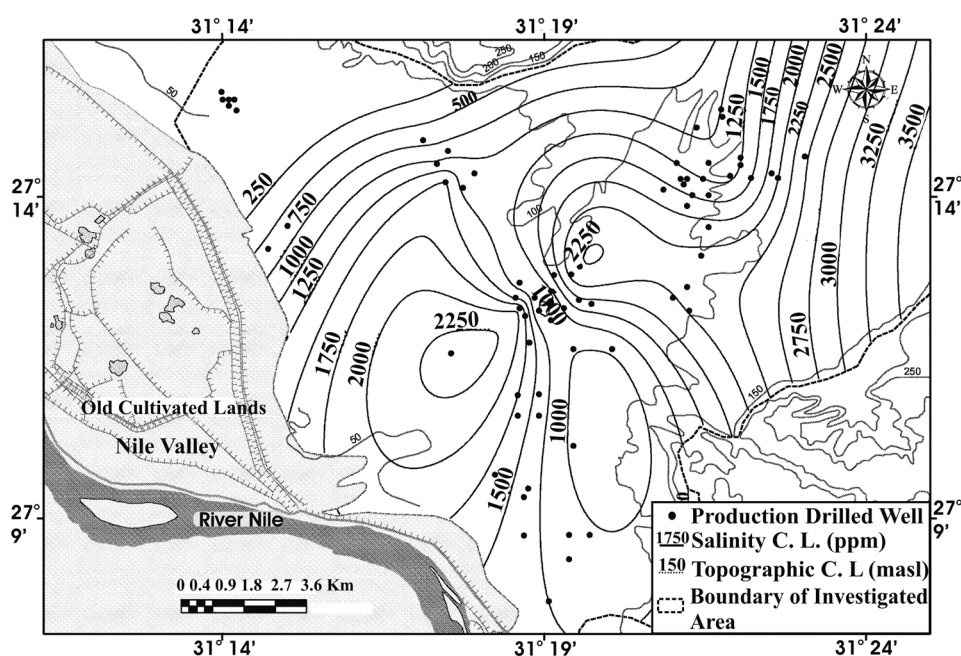
**FIGURE 5. Depth to water contour map of the Early Pleistocene aquifer system.**

### **General hydrogeochemical characteristics and recharge sources of the Pleistocene groundwater aquifer**

The routine chemical analysis was carried out for the major cations and anions. However, for the objective of this study, in order to determine the groundwater recharge processes, environmental stable isotopes determinations are also carried out. Twenty one groundwater samples are collected from the drilled production water wells tapping the Early Pleistocene aquifer of Wadi El-Assiuti area (Tables 1 and 2) during the year 2004. The chemical analyses for the environmental isotopes ( $^{18}\text{O}$  and  $\text{D}$ ) and  $^{14}\text{C}$  were performed in the laboratories of the Nuclear Energy Authority, which were funded by NARSS (NARSS, 2005) (Tables 3 and 4).

All the collected groundwater samples lie in the alkaline medium ( $\text{pH} > 7$ ), in the range from 8.3 to 8.7. The gradation of TDS in the investigated area can be considered to be graded from fresh to brackish water. However, most of the water samples of

Wadi El-Assiuti are brackish. The groundwater salinity values vary generally from 580 mg/l to 2445 mg/l indicating fresh to brackish water. The relative decrease of salinity was directed toward the River Nile (southwestern direction). It could point to some recharge from the Nile water in the Nile Valley districts. However, the area located near to the old cultivated land of the River Nile (< 500 mg/l) reveals clearly the influence of the Nile water. The Pleistocene aquifer salinity is high at the far northeastern part of the area of study (2445 mg/l). The over consumption will lead to the advance of high salinity front found to the northeast of the area of study (Fig. 6). This is attributed to the fact that, there are two different environments confronting each others at the entrance of Wadi El-Assiuti (Fathy et. al. 2005).



due to the prevalence of  $\text{Na}^+$  and  $\text{Cl}^-$  ions, except the Well no. 5 F (W. Delta) has  $\text{HCO}_3^-$   $\text{Na}$  as typical fresh water character. However, the groundwater of Wadi El-Assiuti and its entrance in the Nile Valley is considered as fresh water to slightly brackish.

### Hypothetical salts combination

Three salt assemblages are detected according to ions concentrations as (Table 1):

- $\text{Na Cl}$ ,  $\text{Mg Cl}_2$ ,  $\text{Mg SO}_4$ ,  $\text{Ca SO}_4$ ,  $\text{Ca (HCO}_3)_2$  in Wells 10G, Hamed Abdel Wahab, 9G, 16G, 6G, 7G, 2G, 13G, 3G, 1G, 1KF, Essam El-Sherif, 19B.
- $\text{Na Cl}$ ,  $\text{Na}_2 \text{SO}_4$ ,  $\text{Mg SO}_4$ ,  $\text{Ca SO}_4$ ,  $\text{Ca (HCO}_3)_2$  in samples 2F (Diab Salem), 4F, 8G, 12G (Assiut 2000), Abna'a El-Shaab.
- $\text{Na Cl}$ ,  $\text{Na}_2 \text{SO}_4$ ,  $\text{Na HCO}_3$ ,  $\text{Mg SO}_4$ ,  $\text{Ca SO}_4$ ,  $\text{Ca (HCO}_3)_2$  in samples 5F, 9B (Gamal Abdel Motaal).

**TABLE 1. Hydrochemical aspects of the collected water samples.**

| S. No. | Well Name                            | Depth (m)        |                   | PH   | EC $\mu\text{S/cm}$ | TDS Mg/l | Na                   | $\text{NH}_4$ | K     | Mg    | Ca    | Cl    | $\text{SO}_4$ | $\text{HCO}_3$ | $\text{NO}_3$ |
|--------|--------------------------------------|------------------|-------------------|------|---------------------|----------|----------------------|---------------|-------|-------|-------|-------|---------------|----------------|---------------|
|        |                                      | St. level (mast) | Well depth (mbgl) |      |                     |          | Concentration in epm |               |       |       |       |       |               |                |               |
| 1      | 10 G                                 | 38               | 120               | 8.63 | 1.65                | 1056     | 11.97                | ----          | 0.11  | 1.19  | 3.39  | 12.98 | 1.87          | 3.00           | ----          |
| 2      | Hamed Abdel W.                       | 28               | 130               | 8.50 | 2.48                | 1487     | 14.11                | ----          | 0.16  | 6.18  | 8.80  | 20.29 | 5.00          | 2.11           | ----          |
| 3      | 4 F                                  | 37               | 120               | 8.60 | 1.49                | 954      | 10.78                | 0.27          | 0.17  | 2.98  | 3.32  | 9.64  | 2.71          | 3.89           | ----          |
| 4      | 5 F                                  | 18               | 100               | 8.60 | 0.907               | 580      | 5.85                 | 0.11          | 0.20  | 1.94  | 2.34  | 2.94  | 2.05          | 4.11           | 0.99          |
| 5      | 9 G                                  | 30               | 158               | 8.72 | 1.87                | 1199     | 10.96                | 0.30          | 0.14  | 2.64  | 6.71  | 13.34 | 3.41          | 2.78           | ----          |
| 6      | 16 G                                 | 31               | 133               | 8.30 | 1.71                | 1092     | 11.32                | ----          | 0.15  | 2.86  | 5.87  | 13.08 | 3.53          | 2.66           | ----          |
| 7      | 6 G                                  | 38               | 160               | 8.60 | 2.32                | 1485     | 15.96                | ----          | 0.15  | 4.41  | 6.14  | 18.66 | 4.40          | 2.44           | ----          |
| 8      | 7 G                                  | 34               | 158               | 8.47 | 2.40                | 1882     | 11.32                | 0.23          | 0.14  | 5.83  | 8.92  | 13.74 | 8.06          | 2.78           | 0.65          |
| 9      | 8 G                                  | 37               | 210               | 8.70 | 1.57                | 1007     | 11.50                | 0.42          | 0.09  | 2.03  | 4.28  | 11.36 | 2.24          | 3.33           | 0.11          |
| 10     | 2G                                   | 37               | 158               | 8.56 | 3.32                | 2061     | 19.45                | 0.22          | 0.30  | 6.90  | 11.76 | 25.66 | 7.86          | 2.00           | 0.70          |
| 11     | 13G                                  | 30               | 220               | 8.43 | 1.905               | 1219     | 12.30                | 0.26          | 0.19  | 2.46  | 5.67  | 14.71 | 6.46          | 2.33           | ----          |
| 12     | 12G                                  | 33               | 220               | 8.60 | 1.64                | 1052     | 11.65                | ----          | 0.17  | 3.61  | 3.72  | 11.46 | 4.17          | 3.00           | ----          |
| 13     | 2KF                                  | 36               | 180               | 8.80 | 1.64                | 1052     | 10.67                | 0.68          | 0.80  | 2.60  | 3.50  | 11.16 | 2.97          | 4.22           | ----          |
| 14     | 3G                                   | 40               | 160               | 8.45 | 2.86                | 1702     | 16.87                | ----          | 0.36  | 6.55  | 9.57  | 2.10  | 7.07          | 2.89           | ----          |
| 15     | 1G                                   | 42               | 150               | 8.30 | 2.71                | 1651     | 17.82                | 0.16          | 0.08  | 4.39  | 5.88  | 19.68 | 5.80          | 2.78           | ----          |
| 16     | 1KF                                  | 34               | 220               | 8.65 | 1.603               | 1026     | 11.33                | 0.38          | 0.12  | 3.33  | 3.59  | 11.16 | 3.45          | 3.33           | ----          |
| 17     | Essam Sh.                            | 40               | 221               | 8.50 | 3.15                | 2016     | 24.69                | ----          | 0.16  | 3.91  | 6.90  | 25.34 | 6.41          | 3.33           | 0.64          |
| 18     | 19B                                  | 42               | 160               | 8.60 | 2.87                | 1837     | 22.50                | ----          | 0.13  | 2.92  | 4.82  | 24.34 | 4.21          | 3.00           | ----          |
| 19     | 9B                                   | 45               | 200               | 8.70 | 1.87                | 1200     | 17.65                | ----          | 0.11  | 0.75  | 1.83  | 13.79 | 2.43          | 3.33           | 0.18          |
| 20     | Abna'a Shaab                         | 60               | 190               | 8.30 | 3.82                | 2445     | 35.80                | ----          | 0.18  | 2.49  | 3.12  | 32.05 | 5.38          | 3.55           | ----          |
| 21     | River Nile                           | —                | ----              | 8.00 | 0.10                | 194      | 1.018                | ----          | 0.138 | 0.543 | 0.699 | 0.228 | 0.214         | 1.934          | ----          |
| 22     | Nubia S.S. (Eastern D.) <sup>*</sup> | Flowing          | 420               | 8.00 | 3.25                | 2080     | 31.42                | ----          | ----  | 2.00  | 1.12  | 15.55 | 8.33          | 1066           | ----          |

\* Data obtained from Nubia Sandstone of the Eastern Desert (Wadi Qena) (Elewa et al 2001)

The first assemblage characterizes water of marine salts due to the presence of  $\text{Mg Cl}_2$  salt. The enrichment of  $\text{SO}_4$  minerals that are found in the form of  $\text{Na}_2 \text{SO}_4$ ,  $\text{Mg SO}_4$

in the second assemblage reflects the dissolution of terrestrial salts, which are represented by gypsum or anhydrite dominating the aquifer matrices. The second and third assemblages indicate the meteoric origin of water. It could be noticed that these groundwater wells of different water origins are located very close to each others (about 400 m distant from each others). The depth of penetration is about the same for these wells, but the well design is not the same. It was reported earlier by Fathy et al. 2005, that the wells tapping both the Issawia and Armant Formations are characterized by marine water genesis, whereas the wells that tap the Armant Formation only are characterized by meteoric water genesis. In addition to this, different recharge sources are resulted and expected from mixing of different recharge sources along the deep seated faults that, characterizes this area.

**TABLE 2. Hydrochemical parameters of the Early Pleistocene aquifer system.**

| Well No.           | SAR   | Na %  | rNa/rCl | rCa/rMg | (rCl-rNa)/rCl | rCa/rNa | rSO <sub>4</sub> /rCl |
|--------------------|-------|-------|---------|---------|---------------|---------|-----------------------|
| 10 G               | 7.91  | 71.84 | 0.93    | 2.84    | 0.08          | 0.28    | 0.14                  |
| Hamed Abdel Wahab  | 5.16  | 48.25 | 0.70    | 1.43    | 0.30          | 0.62    | 0.25                  |
| 4 F                | 6.07  | 61.53 | 1.14    | 1.11    | -0.12         | 0.31    | 0.28                  |
| 5 F                | 4.00  | 56.05 | 2.06    | 1.20    | -0.99         | 0.40    | 0.70                  |
| 9 G                | 5.07  | 53.07 | 0.83    | 2.54    | 0.18          | 0.61    | 0.26                  |
| 16 G               | 5.41  | 55.17 | 0.85    | 2.06    | 0.13          | 0.52    | 0.27                  |
| 6 G                | 6.95  | 59.87 | 0.86    | 1.39    | 0.15          | 0.38    | 0.24                  |
| 7 G                | 4.17  | 43.19 | 0.83    | 1.53    | 0.18          | 0.79    | 0.59                  |
| 8 G                | 6.47  | 63.42 | 1.02    | 2.10    | -0.01         | 0.37    | 0.20                  |
| 2 G                | 6.37  | 50.09 | 0.77    | 1.70    | 0.24          | 0.60    | 0.31                  |
| 13 G               | 5.76  | 56.36 | 0.85    | 1.64    | 0.16          | 0.46    | 0.44                  |
| 12 G               | 6.09  | 60.02 | 1.03    | 1.03    | -0.02         | 0.32    | 0.36                  |
| 2 KF               | 6.14  | 60.97 | 1.03    | 1.35    | 0.04          | 0.33    | 0.27                  |
| 3 G                | 5.94  | 49.57 | 0.82    | 1.46    | 0.20          | 0.57    | 0.34                  |
| 1 G                | 7.86  | 63.24 | 0.91    | 1.34    | 0.09          | 0.33    | 0.29                  |
| 1 KF               | 6.09  | 61.15 | 1.03    | 1.08    | 0.02          | 0.32    | 0.31                  |
| Essam Sh.          | 10.62 | 68.51 | 0.98    | 1.76    | 0.03          | 0.28    | 0.25                  |
| 19 B (Abdel Hafez) | 11.44 | 74.08 | 0.93    | 1.65    | 0.08          | 0.21    | 0.17                  |
| 9 B (G)            | 15.55 | 86.81 | 1.29    | 2.43    | -0.28         | 0.10    | 0.18                  |
| Abna'a Shaab       | 21.37 | 86.07 | 1.12    | 1.25    | -0.12         | 0.09    | 0.17                  |



### **Ion ratios (hydrochemical parameters)**

Five ion ratios (hydrochemical parameters) were calculated to detect the mineralization sources in the Pleistocene aquifer (Table 2). The high concentration of sodium and chloride ions among cations and anions in most samples indicates the abundance of marine leached salts, as presented in low ratios of  $rNa/rCl$ ,  $rCa/rNa$  and  $rSO_4/rCl$ . However, the increase of  $rCa/rMg$  ratio indicates a fingerprint of calcareous facies, represented by calcite, dolomite and or limestone rocks prevailing in the surrounding limestone plateaux and the deep-seated carbonate rocks. These facies are leached by the deeply percolated Nubia Sandstone groundwater via the deep seated geological structures. The evidence of Nubia Sandstone as one of the sources of old groundwater recharging to the Pleistocene aquifer is obtained from the results of stable isotopes ( $^{18}O$  & D) and  $^{14}C$  age determination, as will be discussed later. Furthermore, the positive values obtained from the calculation of ion base exchange  $\{(rCl-rNa)/rCl\}$  in some samples refer to the marine origin of this water as previously indicated.

### **Discussions about the isotopic data of Wadi El-Assiuti groundwater**

The most important elements in this study are the isotopes of hydrogen and oxygen (D &  $^{18}O$ ), the two constituents of water molecule which occur naturally. As previously mentioned, these isotopes are used as natural tracers to investigate the origin of water or the processes that have affected water since it was formed. The isotopic measurements are carried out for 21 groundwater samples. The results obtained are shown in Table 3.

The isotopic results of the collected surface and groundwater samples in the studied area are widely varied from -8.9 to 2.81‰ for  $O^{18}$  and from -61.07 to 21.11 ‰ for Deuterium.

Almost 86% of these samples fall in the depleted isotopic range with a mean value equal -7.45‰, whereas the remaining percentage (14%) is gradually enriched in isotopes (Figs 7 and 8). Plotting the data points with respect to the Global Meteoric Water Line (GMWL:  $\delta D = 8 \delta^{18}O + 10$ ) in the conventional relationship of  $\delta O^{18}$  and  $\delta D$  is indicted in Fig. 9. The distribution of these points fall below the global line with a regression line following the equation:

$$\delta D = 7.08 \delta^{18}O - 1 \quad (EQ 1)$$

Low slope and intercept values obtained from this equation indicate the old origin of the main bulk of water that is represented by 86% of these samples, which have the depleted isotopic signature of the paleowater in Eastern Desert of Egypt.

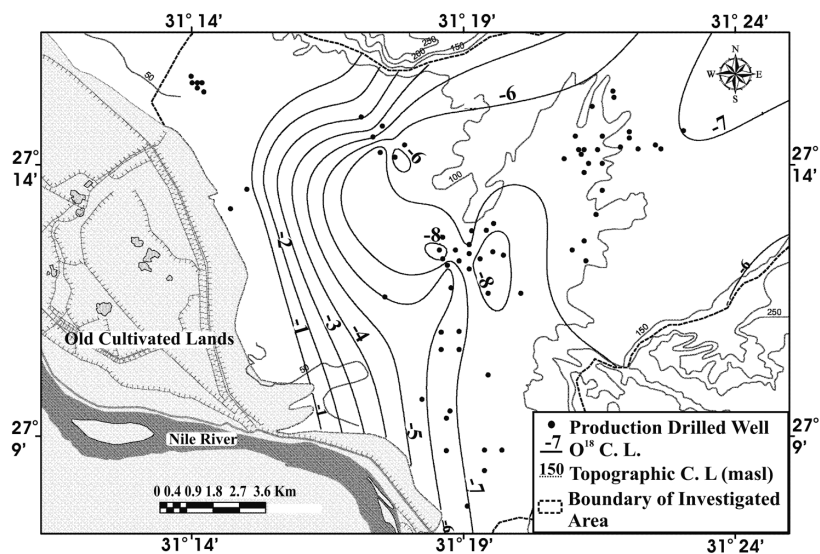
**TABLE 3. The isotopic results of the collected water samples.**

| Well name            | $\delta O^{18}$ | $\delta D$ | d*    |
|----------------------|-----------------|------------|-------|
|                      | ‰               | ‰          | ‰     |
| 10 G                 | -4.88           | -34.89     | 4.15  |
| Hamed Abdel Wahab    | -7.73           | -57.25     | 4.59  |
| 4 F                  | -7.69           | -55.73     | 5.79  |
| 5 F (Delta)          | 2.81            | 21.11      | -1.37 |
| 9 G                  | -5.73           | -42.45     | 3.39  |
| 16 G                 | -6.38           | -46.48     | 4.56  |
| 6 G                  | -8.16           | -59.14     | 6.14  |
| 7 G                  | -8.02           | -57.79     | 6.37  |
| 8 G                  | -6.35           | -43.49     | 7.31  |
| 2 G                  | -7.65           | -56.39     | 4.81  |
| 13 G                 | -6.99           | -49.59     | 6.33  |
| 12 G                 | -7.55           | -53.32     | 7.08  |
| 2 KF                 | -7.63           | -53.22     | 7.82  |
| 3 G                  | -8.94           | -63.29     | 8.23  |
| 1 G                  | -8.31           | -61.07     | 5.41  |
| 1 KF                 | -8.25           | -60.11     | 5.89  |
| Essam Sh.            | -6.49           | -49.16     | 2.76  |
| 19 B (Abdel Hafez)   | -6.76           | -53.66     | 0.42  |
| 9 B (G Abdel Motaal) | -6.72           | -44.5      | 9.26  |
| Abnaa Shaab          | -7.01           | -47.87     | 8.21  |
| Nubia Sandstone**    | -10.66          | -78.10     | 12.2  |
| River Nile *         | -0.5            | + 6.3      | --    |

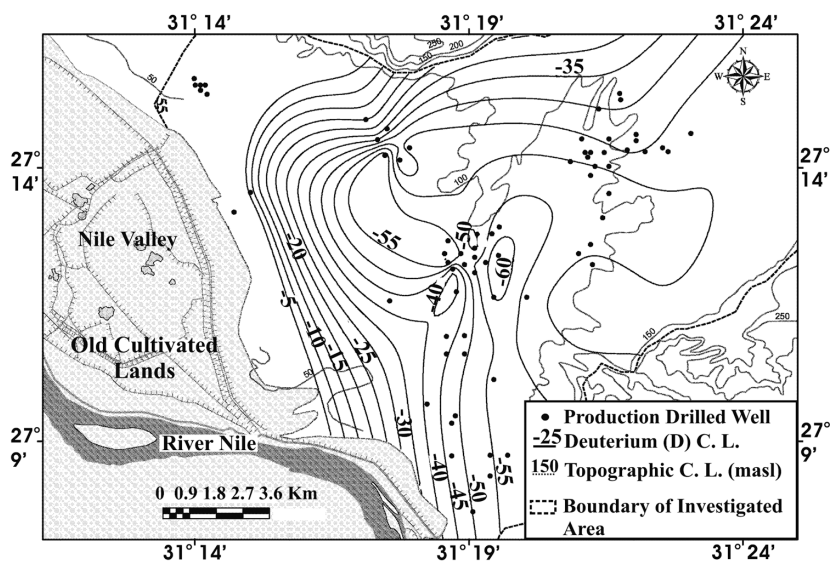
\* & \*\* referenced data taken from GMBH, 1979.

The replenishment of this water is attained from the underlying Nubia Sandstone aquifer through the deep seated faults, which provide active recharge zones or conduits through which the Nubian water upwardly leaked to recharge the overlying aquifers (upper and lower Eocene fissured limestone aquifer or the Pleistocene aquifer). However, the groundwater in the Nubia Sandstone aquifer system is confined by the upper shale members of the Nubian Formation. The drilled boreholes in the Nile Valley (i. e. Wadi Qena wells (Elewa et al. 2001) demonstrate that free-flowing artesian conditions develop in the Nile Valley itself, and in the lower sections of the tributary wadis of its eastern flank. It is most likely that the shallow aquifers, which are regularly encountered saturating the alluvial fill of these wadis are fed by discharge of the Nubian aquifer. The groundwater from deep artesian Nubian aquifer has the lightest isotopic composition (GMBH, 1979). This fact clearly proven by Piper (1944) trilinear diagram, as the two water groups nearly occur in the same water class (Sample 22 in Fig. 10) confirming the idea of direct hydraulic interconnection of the Pre-Upper Cretaceous aquifer (Nubia Sandstone) with the

Early Pleistocene aquifer system by upward leakage, as deduced also from the isotopic composition.



**FIGURE 7.  $O^{18}$  contour map of the Early Pleistocene aquifer of Wadi El-Assiuti.**



**FIGURE 8. Deuterium contour map of the Early Pleistocene Aquifer system of Wadi El-Assiuti.**

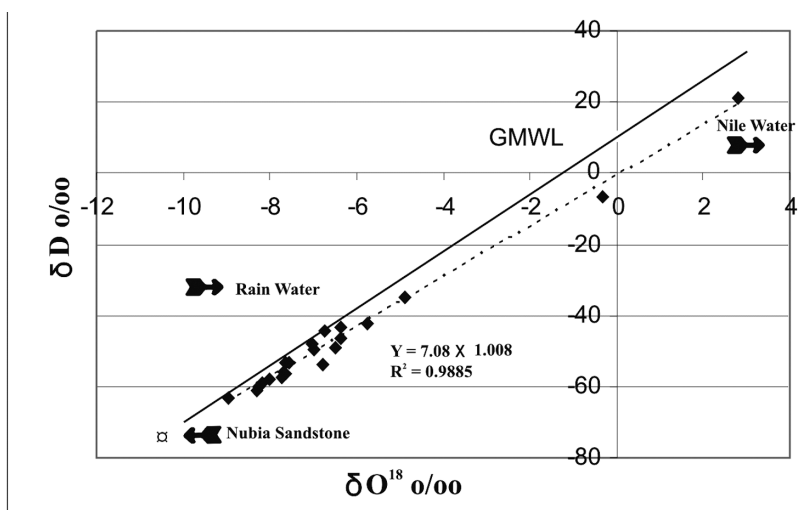


FIGURE 9. Relationship of Delta O<sup>-18</sup> versus Delta D (in per mil).

### Legend

S. N. Well Name

- 1 10G
- 2 Hamed Abdel W.
- 3 4F
- 4 5F
- 5 9G
- 6 16G
- 7 6G
- 8 7G
- 9 8G
- 10 2G
- 11 13G
- 12 12G
- 13 2KF
- 14 3G
- 15 1G
- 16 1KF
- 17 Essam sh.
- 18 19B
- 19 9B
- 20 Abna'a Sha'ab
- 21 Nile Water
- 22 Nubia S. S. Aquifer (Eastern Desert)

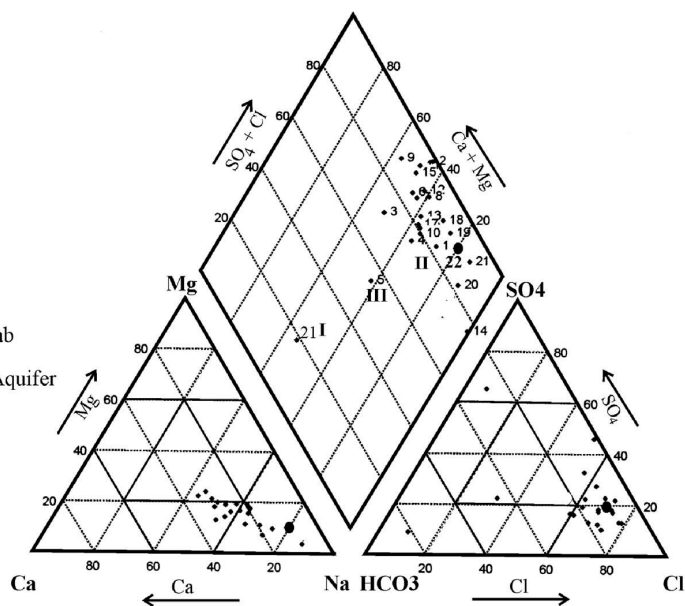


FIGURE 10. Groundwater hydrochemical evolution trend by Piper's trilinear diagram.

On the other hand, the obvious shift of the enriched samples towards the Nile point confirms the existing of two different aquifers, confronting each others at the entrance of Wadi El-Assiuti. The eastern one is the main Early Pleistocene aquifer of the Wadi proper. This aquifer could be recharged by the Nubian aquifer through a direct contact (through the deep-seated faults) between them. The western one is the main aquifer of the Nile Valley, which is recharged by the River Nile. This part of the aquifer is characterized by exceptionally high hydrogen and oxygen values which are found in samples of 2F, 5F, and 10G. Compared with the isotopic composition of the Nile water it becomes obvious, that the groundwater in this area must have been recharged from the River Nile. The trend of water evolution is clearly given by Piper (1944) trilinear diagram (Fig. 10).

#### **Evidence of groundwater old age by Carbon-14**

The idea of the old age of the Early Pleistocene aquifer, which is diluted at certain areas under certain circumstances by the recent ground and surface water, was confirmed by carrying out the  $^{14}\text{C}$  activity measurements of DIC precipitated from two selected groundwater samples at the northeastern boundary (Well 9B) and the center of the basin (Well 3G) (Table 4). These samples were precipitated in the field as barium carbonate. The precipitate was converted to  $\text{CO}_2$  gas and synthesized to benzene. Accordingly, radiocarbon activity was measured by Beta Liquid Scintillation Counter. The calculated value of  $^{14}\text{C}$  activity measurements in the sample of Well 9B reach 24.7 pmc (percent modern carbon) indicating water age up to 10,000 yBP. Whereas, at the center of the basin, low pmc was recorded in Well 3G (pmc = 3), where the age of this water reaches 27,000 yBP. Both results are consistent with the stable isotopic values of  $^{18}\text{O}$  and D. The relative enrichment in stable isotopes and short age detected in the northeastern mountainous front zone gives evidence about the significant amount of recent precipitation replenishing the aquifer. On the other hand, at the center of the basin, high isotopic depletion and long age confirm the upward leakage of paleorecharge water from the Lower Cenomanian aquifer (Nubia Sandstone aquifer), where the major replenishment occurs in this part could point to an area of a major fault zone (Fig. 4).

**TABLE 4.  $\text{C}^{14}$  data and results of the Pleistocene aquifer of Wadi El-Assiuti.**

| <b>Sample no.</b> | <b>PMC %<br/>(percent modern<br/>carbon)</b> | <b>Age (yr.b.p)<br/>Years before present</b> |
|-------------------|----------------------------------------------|----------------------------------------------|
| Well no.3G        | 3                                            | 25,000 27,000                                |
| Well no. 9B       | 24.7                                         | 8000 10,220                                  |

The estimated travel time of the groundwater along the flow path from the northeastern boundary to the center of the basin is very long, which is attributed to the low hydraulic conductivity of the aquifer (0.030 m/day) (NARSS 2005). The distance from the catchments mountainous northeastern areas and the center of the basin of the area of study is about 90 km. Accordingly, the average travel time needed is about 8219 years via the subsurface to reach the center of the aquifer basin. Thus, this water gives another positive recharge contribution side-by-side with the original stored groundwater from the pluvial periods and the water coming from the upward leakage via the deep-seated faults from the Nubia Sandstone aquifer system.

## CONCLUSION

The hydrogeological, hydrogeophysical and hydrochemical tools of investigation elaborated different recharge sources and environments in the Early Pleistocene aquifer system of Wadi El-Assiuti area. These recharge sources includes the River Nile water in the neighborhood area of the Nile Valley, the local rainfall of the northeastern and surrounding mountainous areas and the upward leakage of the deeply penetrated Nubia Sandstone aquifer system. The  $^{14}\text{C}$  results indicate that the Pleistocene water of Wadi El-Assiuti is mostly paleowater. This is obvious in the sample taken from Well 3G, whereas at the Well 9B, which is located in the northeastern part of the study area, reveals a significant portion of relatively recent water recharging the aquifer from the local rainfalls in addition to that of the Red Sea mountainous area. This confirms the slight enrichment in stable  $\text{O}^{18}$  and D in the second sample than the former one.

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