Fifth Biennial Symposium on Artificial Recharge of Groundwater

Symposium Proceedings

“Challenges of the 1990s”

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Salt River Project

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Fifth Biennial Symposium on Artificial Recharge of Groundwater

Symposium Proceedings

“Challenges of the 1990s”
# TABLE OF CONTENTS

## FOREWORD  
vi

## LIST OF CORPORATE SPONSORS  
vii

## Reuse — Reclaimed Water  
1

Groundwater Recharge with Wastewater:  
Pre and Post-treatment  
*Herman Bouwer, U.S. Water Conservation Laboratory, ARS-USDA*

Groundwater Recharge with Reclaimed Water:  
Resolving Regulatory Issues  
*Martin Rigby and Bill Mills, Orange County Water District*

Siting of Spreading Basins for Underground Storage  
of Treated Municipal Wastewater  
*Peter Mock, CH2M HILL, Inc.*

Fate of Chlorination By-Products During Soil Aquifer Treatment  
*Aimee Conroy, John Chahbandour, Gary L. Amy,  
L.G. Wilson, and Bruce Johnson, University of Arizona*

Virus and Bromide Transport Through Sandy Alluvium  
with Infiltrated Treated Sewage  
*D.K. Powelson, D. Cline, M.T. Yahya, L.G. Wilson,  
and C.P. Gerba, University of Arizona*

## Innovation and Regulation  
63

Regulatory Considerations for Aquifer Protection Permits  
*William Marceau, Arizona Department of Environmental Quality*

Interpreting Arizona’s Recharge Statutes from a Technical Perspective  
*Wayne Cooley and Greg Bushner,  
Arizona Department of Water Resources*

Drywells: Pre-Treatment Designs and Technology  
*Steve DeTommaso, McGuckin Drilling, Inc.*

Twenty-First Century Methods of Cleaning Deep Recharge Lakes  
*James A. Goodrich, and Allan Flowers,  
Orange County Water District*
Recent Technical Advances in Aquifer Storage Recovery (Abstract)  
R. David Pyne, CH2M HILL, Inc.

Renewable Urban Water Supplies, Nogales and the Microbasins of the Santa Cruz River: A Case of Natural Water Banking  
Leonard Halpenny and Philip Halpenny, Water Development Corp.

Aquifer Storage and Recovery

Testing of a Saline Aquifer for Aquifer Storage Recovery Potential  
Sean Skegan, Kevin Brall, Albert Muniz, and Peter Kwiatkowski, CH2M HILL, Inc.

Kerrville, Texas: A Case Study for Aquifer Storage Recovery (Abstract)  
John McLeod, Rich Petrus, Larry Amans, CH2M HILL, Inc. and B.W. Bruns, Upper Guadalupe River Authority

Recharge Permitting in the Wake of H.B. 2612: Redefining DEQ and DWR Roles in the Cooperative Process (Abstract)  
Greg Bushner and Jim DuBois, Arizona Department of Water Resources

Hydrogeochemistry and Chemical Compositional Changes of Groundwater from a Deep Well Recharge Operation Using River Water Subjected to Limited On-Site Treatment  
Mario Lluria, Timothy Gorey and Bruce Mack, Salt River Project

Research, Models, and Demonstration Projects – Basins

Results and Significance of an Unsaturated-Zone Tracer Test at an Artificial Recharge Basin, Tucson, Arizona (Abstract)  

Gravity Response to Storage Change in the Vicinity of Infiltration Basins (Abstract)  

Vadose Zone and Saturated Zone Flow Modeling, Tucson Recharge Feasibility Assessment Project, Pima County, Arizona  
Laura Strauss and Errol Montgomery, Errol L. Montgomery & Associates, Inc.

Investigation of the Mechanism of Percolation Reduction in a Deep Recharge Basin (Abstract)  
Donald Phipps, Grisel Gordon and Harry Ridgway, Orange County Water District
Kern Fan Element: Feasibility of Conjunctive Use
John Fielden and Terry Erlewine,
California Department of Water Resources

Gravel-Filled Trenches for Recharging
R.J. Lutton, Geotechnical Laboratory,
USAE Waterways, Experiment Station

Research, Models, and Demonstration Projects
Continued — Well Injection and Basins

The Rillito Creek Recharge Project: Goals and Status
J. Craig Tinney, Pima County Flood Control District

Groundwater Recharge Field Testing on O'Neill Unit, Nebraska
Roger Burnett, Larry Cast, and Michael Kube,
U. S. Bureau of Reclamation

Implementation of "The High Plains States' Groundwater
Demonstration Program Act of 1983"
Richard Lasson, U. S. Bureau of Reclamation

Nonunique Simulations of the Quality of Water Recovered
Following Injection of Freshwater into a Brackish Aquifer
Michael Merritt, U.S. Geological Survey

Coupling Diffuse and Conduit Flow Models to Simulate
Injection and Recovery of Water Through a Single Well
Vicente Quinones-Aponte and Miguel Medina, Jr., Duke University

Analysis of Injection Well Plugging During Long-Term
Pilot Recharge Tests, Tucson Basin, Arizona
Mark Cross, Laura Strauss, and Errol Montgomery,
Errol L. Montgomery and Associates, Inc.

BIOGRAPHICAL PROFILES
FOREWORD

Artificial recharge of groundwater is a technology whose time is now. Although there is still a lot to be learned about the where, when, and how of artificial recharge, projects around the country have logged enough operating history to establish this technology among the tools of choice for water management. This exalted position for artificial recharge is a significant change from little more than ten years ago, when some speakers at the first Symposium argued that artificial recharge of groundwater would not and could not work in Arizona.

Science and experience change minds, even in water management. The new challenges facing scientists, engineers, and others concerned with artificial groundwater recharge in the 1990s go beyond convincing a skeptical water management community. The water using public wants results. This means answering the public health and environmental quality concerns of the public and regulatory agencies, developing efficient strategies for and avoiding physical obstacles to aquifer storage and recovery, and advancing the technology beyond demonstration projects to full-scale implementation.

These are the kinds of challenges that bring out the best in the research and development community, as we hope you will witness from this collection of papers of the Fifth Biennial Symposium on Artificial Recharge of Groundwater, held May 29 to 31, in Tucson, Arizona.
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GROUNDWATER RECHARGE WITH WASTEWATER:
PRE-AND POST-TREATMENT

Herman Bouwer
U.S. Water Conservation Laboratory
Phoenix, AZ 85040

ABSTRACT

Reuse of municipal wastewater requires treatment so that the water meets the quality requirements for the intended use. Irrigation of vegetables consumed raw requires absence of pathogenic organisms. To achieve this, fecal coliform concentrations should be essentially zero and the water should be filtered. This requires intensive treatment. For developing countries, however, where adequate sewage treatment often is lacking, the World Health Organization allows up to 1000 fecal coliform bacteria per 100 ml, which can be obtained by lagooning. Potable reuse requires much more intensive treatment, including lime clarification, denitrification, carbon adsorption, reverse osmosis, and disinfection. Where hydrogeologic conditions permit groundwater recharge with surface infiltration facilities, considerable water quality improvement is obtained by the movement of the wastewater through the soil, vadose zone, and aquifer. The resulting soil-aquifer treatment reduces the total treatment costs for potable reuse by more than 60%. Soil-aquifer treatment systems also are simple and robust, offer storage of the water to absorb differences between supply and demand, and enhance the aesthetics of potable reuse of wastewater. Hence, they can play an important role in wastewater reuse.

INTRODUCTION

Reuse of municipal wastewater has two advantages: (1) it eliminates adverse health or environmental impacts of discharging the effluent into surface water, and (2) it provides for the beneficial use of the effluent as a water resource. The latter includes use for irrigation, cooling and other industrial applications, household use, and drinking. Reuse of wastewater is especially important in areas where water is scarce or very expensive. If municipal wastewater is reused, it must be treated so that it meets the quality requirements for the particular use. This paper discusses quality criteria for irrigation and potable reuse, and the treatment necessary to achieve those criteria, particularly the role of groundwater recharge in the treatment process.
Irrigation

The main concern in using municipal wastewater for irrigation is about pathogenic microorganisms, although agronomic quality standards for irrigation (nitrogen, salinity, trace elements, etc.) also must be met (Bouwer, 1985; Bouwer and Idelovitch, 1987). Primary or other partially treated effluent still contains large numbers of microorganisms, which may include pathogens. Such effluent can generally be used for irrigation of non-edible crops if irrigators are properly trained and public access to the irrigated areas is controlled. More stringent standards apply for irrigation of food crops that are cooked or otherwise processed to kill microorganisms. This is still a restricted use, however. The most stringent standards apply to unrestricted irrigation, which includes sprinkler irrigation of crops consumed raw or brought raw into the kitchen, and of parks and playgrounds and landscaped areas with unrestricted public access. There are now two standards for such unrestricted irrigation: one for developed, industrialized countries, and one for developing countries.

The standard for developed countries is based on zero risk and it requires essentially complete absence of fecal coliform bacteria, viruses, Entamoeba hystolytica, and eggs of parasitic worms (Bouwer, 1985; Bouwer and Idelovitch, 1987, and references therein). This standard follows the California regulations and the resulting effluent is commonly called Title 22 effluent, after the section in the California code dealing with wastewater reuse. Title 22 effluent typically is obtained with conventional primary and secondary treatment followed by coagulation, sedimentation, filtration, and chlorination or other disinfection. This kind of treatment requires considerable capital investment and operating expenses, plus adequate personnel, energy, and supplies to properly operate and maintain the treatment plant. These resources generally are not available in developing countries, with the undesirable result that sewage often is not treated at all and raw sewage is used for irrigation, including fresh vegetables. From a public health standpoint this is, of course, completely unacceptable. In view of this, the World Health Organization has adopted less stringent standards for irrigation with sewage effluent that can be achieved with simple, inexpensive treatment facilities. The standards are based on case-history studies of sewage irrigation systems and disease outbreaks, which indicated that parasitic worms are the main danger and that treatment of the effluent to obtain a fecal coliform level of less than 1000 per 100 ml produces an effluent that is essentially safe for unrestricted irrigation (Shuval, 1990). This level of treatment can be achieved by lagoons, allowing adequate detention times (about one month in warm climates) for complete die off of parasitic worms and significant die off of bacteria and viruses. The big advantage of lagoon systems is that they are inexpensive, robust, and simple to operate, and, hence, suitable for developing countries. For large flows, however, they require relatively large areas of land, which can be a problem.
Potable Reuse

There are many rivers in the world that receive sewage effluent and at the same time supply drinking water for adjoining cities. Often, the sewage receives only conventional treatment, and so does the drinking water. For drinking water, this treatment consists of coagulation, sedimentation, filtration and disinfection. When the river is badly polluted, more treatment steps, such as carbon adsorption, are added. For direct, pipe-to-pipe recycling of sewage effluent into drinking water, the normal drinking water quality standards are not adequate because these standards generally apply to situations of relatively clean and unpolluted source water. Sewage effluent as such, however, may contain hundreds and perhaps thousands of natural and synthetic chemicals. It is not feasible to develop health standards for each chemical and to do regular testing of the product water to see if the concentrations of all these chemicals are below maximum limits. Instead, a certain treatment train is specified which, based on chemical and biological analyses and biomonitoring of the product water from pilot studies, produces a safe drinking water. An example of such a pilot/demonstration project is the Denver, Colorado, Potable Water Reuse Demonstration Project (Lauer, 1990).

The Denver project takes conventional unchlorinated effluent (activated sludge, plus coagulation and sedimentation, and some denitrification) and treats it into drinking water with the following treatment train: lime clarification, recarbonation, granular media filtration, ultraviolet irradiation, activated carbon adsorption, reverse osmosis, air stripping, ozonation, and chloramination. These steps were selected to provide the necessary treatment, redundancy, and multiple barriers against the various contaminants. Major contaminant groups and the multiple barriers against them provided by the treatment train listed by Lauer (1990) are shown in Table 1. The total cost of this treatment, projected to a 0.4 million m³ per day (100 mgd) plant, is about $600 (August 1988 dollars) per 1000 m³ or $740 per acrefoot which is 1234 m³ (personal communication, W.C. Lauer, 1990). This figure is based on 100% flow through the reverse osmosis system and it includes the amortized capital costs and the annual operation and maintenance costs. The total cost of conventional primary and secondary treatment (activated sludge) is on the order of $80 per 1000 m³. These figures show that the total cost of recycling municipal wastewater into drinking water is not all that much and often may be less than the cost of developing new water resources. Public acceptance is an important factor and education and community programs must be provided to assure the public of the purity and safety of the recycled water. In the future, more and more cities will have to recycle their wastewater as water demands rise but new water resources are no longer available.
Table 1. Major Contaminant Groups and Barriers in the Treatment Train of the Denver Potable Water Reuse Demonstration Project (from Lauer, 1990).

<table>
<thead>
<tr>
<th>Contaminant groups</th>
<th>Barriers</th>
</tr>
</thead>
<tbody>
<tr>
<td>bacteria and viruses</td>
<td>High pH lime clarification reverse osmosis</td>
</tr>
<tr>
<td></td>
<td>ultraviolet irradiation chloramination</td>
</tr>
<tr>
<td>protozoa</td>
<td>high pH lime clarification reverse osmosis</td>
</tr>
<tr>
<td></td>
<td>filtration chloramination</td>
</tr>
<tr>
<td></td>
<td>ozonation</td>
</tr>
<tr>
<td>metals and inorganic</td>
<td>high pH lime clarification reverse osmosis</td>
</tr>
<tr>
<td>compounds</td>
<td>activated carbon adsorption</td>
</tr>
<tr>
<td>organic compounds</td>
<td>high pH lime clarification reverse osmosis</td>
</tr>
<tr>
<td></td>
<td>activated carbon adsorption air stripping</td>
</tr>
</tbody>
</table>

GROUNDWATER RECHARGE AND SOIL-AQUIFER TREATMENT

Infiltration Systems

Soil-aquifer treatment. When sewage effluent is applied to land and allowed to infiltrate the soil and move through the vadose zone to underlying groundwater, considerable improvement in the water quality is obtained by the natural filtration process. For suitable soils, this "soil filtration" removes essentially all suspended solids, biochemical oxygen demand (BOD) and microorganisms, most of the metals and phosphates, and considerable nitrogen. Some of these processes, like removal of BOD, nitrogen, and microorganisms, are renewable and can go on indefinitely. Other constituents, like metals and phosphate, can accumulate in the soil. In an experimental system near Phoenix, Arizona, soils were calcareous and phosphate precipitated as calcium phosphate in the vadose zone and aquifer (Bouwer et al., 1980). The rate of accumulation was so slow, however, that decades and maybe even centuries would be required before soil porosity, hydraulic conductivity, and infiltration rates would be noticeably reduced. Concentrations of metals in municipal wastewater are much lower than those of phosphates. Metals also accumulate in the soil, but much slower than phosphates and mostly in the upper soil layers. Thus, soil-aquifer treatment systems have a very long useful life.
Infiltration rates are maintained by alternating flooding and drying periods, which may range from several days to several weeks each depending on wastewater quality, climate, and soil. Suspended solids accumulate on the soil surface and these must be periodically removed, for some systems with every drying period, for others once a year. After the water has moved through the vadose zone, additional purification can take place when it moves through the aquifer, but usually aquifer materials are so coarse that only "polishing" and "aging" benefits are obtained (dying of microorganisms, adsorption and decay of synthetic organic compounds, taste and odor improvement, etc.). Thus, the combination of soil-aquifer treatment can play an important role in the treatment and reuse of municipal wastewater.

Soil-aquifer treatment (SAT) systems generally are designed and managed so that all the water that infiltrates as sewage effluent will be recovered with wells, drains, or via seepage into surface water. Typical SAT recharge and recovery systems are shown in Figure 1. SAT systems with infiltration basins require aquifers that are unconfined, vadose zones that are free from restricting layers, and surface soils that are coarse enough to give high infiltration rates but fine enough to give good filtration treatment. Thus, sandy loams and loamy or fine sands are preferred for the surface soils in SAT systems. Where surface soils are too fine or not available for infiltration systems, vadose zones have severely restricting layers, and/or aquifers are confined, the only other practical way to recharge groundwater is with injection wells, as discussed in the last section.

**Quality improvement and reuse.** An example of the improvement of the quality of secondary effluent in a groundwater recharge (SAT) system is shown in Table 2. These data are the results of a demonstration project in the Salt River bed west of Phoenix, Arizona (Bouwer and Rice, 1984), except for the metal concentrations and effluent virus level, which were obtained from an earlier experimental project in the Salt River bed (Bouwer et al, 1980). The groundwater table in the demonstration project was at a depth of about 17 m, and the vadose zone and aquifer consisted mainly of sand and gravel layers (some very coarse). The secondary effluent was sampled as it entered the infiltration basins. The well for pumping sewage water after SAT was located in the center of the 16 ha basin area and was perforated from 30 to 55 m. The layout of the system was similar to system C in Figure 1. The four basins of the project were operated on a schedule of two weeks flooding-two weeks drying to enhance denitrification in the soil and to allow recovery of infiltration rates between flooding periods. Infiltration rates during flooding were about 0.5 m/day but since the basins were dry half the time, hydraulic loading rates were about 100 m/yr. Thus, 1 ha of basin area could infiltrate about 1 million m³ of secondary effluent per year.

The quality parameters of the water after SAT in Table 2 show that the water meets the agronomic requirements for crop irrigation and the health standards for California Title 22 effluent (Bouwer,
Figure 1. Schematic of soil-aquifer treatment systems with natural drainage of renovated water into stream, lake, or low area (A), collection of renovated water by subsurface drain (B), infiltration areas in two parallel rows and lines of wells midway between (C), and infiltration areas in center surrounded by a circle of wells (D).
1985; Bouwer and Idelovitch, 1987; and references therein). Hence, it is suitable for unrestricted irrigation (including sprinkler irrigation of fruits and vegetables consumed raw) and for unrestricted recreation (including swimming and fishing). For potable reuse, post-treatment with activated carbon would be necessary because the TOC of 1.9 mg/l includes small concentrations (often on the µg/l level) of a wide spectrum of synthetic organic carbon compounds such as halogenated and nonhalogenated aliphatic and aromatic compounds (E.J. Bouwer et al., 1984). However, because the TOC is so low, the activated carbon adsorption process will be very efficient and carbon regeneration will be less frequent than for water with more organic carbon (Semmens and Field, 1980). Also

Table 2. Quality parameters from Phoenix, Arizona, SAT system for mildly chlorinated secondary effluent (activated sludge) as it enters the infiltration basins (left column) and after it has moved through the vadose zone and aquifer to a well in the center of the infiltration basin area (right column).

<table>
<thead>
<tr>
<th></th>
<th>Secondary effluent</th>
<th>Recovery well samples</th>
</tr>
</thead>
<tbody>
<tr>
<td>Total dissolved solids</td>
<td>750 mg/l</td>
<td>790 mg/l</td>
</tr>
<tr>
<td>Suspended solids</td>
<td>11</td>
<td>1</td>
</tr>
<tr>
<td>Ammonium nitrogen</td>
<td>16</td>
<td>0.1</td>
</tr>
<tr>
<td>Nitrate nitrogen</td>
<td>0.5</td>
<td>5.3</td>
</tr>
<tr>
<td>Organic nitrogen</td>
<td>1.5</td>
<td>0.1</td>
</tr>
<tr>
<td>Phosphate phosphorus</td>
<td>5.5</td>
<td>0.4</td>
</tr>
<tr>
<td>Fluoride</td>
<td>1.2</td>
<td>0.7</td>
</tr>
<tr>
<td>Boron</td>
<td>0.6</td>
<td>0.6</td>
</tr>
<tr>
<td>Biochemical oxygen demand</td>
<td>12</td>
<td>0</td>
</tr>
<tr>
<td>Total organic carbon</td>
<td>12</td>
<td>1.9</td>
</tr>
<tr>
<td>Zinc</td>
<td>0.19</td>
<td>0.03</td>
</tr>
<tr>
<td>Copper</td>
<td>0.12</td>
<td>0.016</td>
</tr>
<tr>
<td>Cadmium</td>
<td>0.008</td>
<td>0.007</td>
</tr>
<tr>
<td>Lead</td>
<td>0.082</td>
<td>0.066</td>
</tr>
<tr>
<td>Fecal coliforms per 100 ml</td>
<td>3500</td>
<td>0.3</td>
</tr>
<tr>
<td>Viruses, PFU/100 l</td>
<td>2118</td>
<td>0</td>
</tr>
</tbody>
</table>

the water after SAT would have to be disinfected and some of the water (for example, one-half) would have to go through reverse osmosis to reduce the salt content. The lead concentrations in Table 2 are unusually high. However, these values were determined about 20 years ago and present values are more in the range of 0.001 to 0.005 mg/l. Average metal concentrations in the secondary effluent in 1990 were zinc 0.036, copper 0.008, cadmium 0.0001, and lead 0.002 mg/l (personal communication, Pat Wokulich, City of Phoenix, Water and Wastewater Department, 1990). These lower values may be due to better control of industrial waste discharges, better wastewater treatment, and/or better analytical techniques.
The recent values were determined with atomic absorption spectrophotometry using a graphite furnace. Thus, passing half the flow through reverse osmosis indeed is adequate for potable reuse. The cost of the carbon adsorption, reverse osmosis for one-half of the flow, and disinfection for a 0.4 million m$^3$ per day plant is estimated to be about $230 per 1000 m^3$ (personal communication, W.C. Lauer, 1990). This is less than 40% of the cost of the complete in-plant treatment to produce potable water from secondary sewage effluent in the Denver project, which was $600 per 1000 m^3$. Thus, SAT in this case is worth about $370 per 1000 m^3$. The cost of SAT, however, is quite low and often consists mostly of that for pumping the renovated water from the aquifer. This may be about $2.5 to $25 per 1000 m^3$ depending on the depth of the groundwater (3 to 30 m, respectively). Thus, SAT gives about $370 worth of treatment for about $2.5 to $25 pumping costs. Other advantages of groundwater recharge and SAT systems are

1. They are robust and fail-safe.

2. They offer underground storage to absorb seasonal or other differences between supply and demand.

3. They break the pipe-to-pipe connection of the direct recycling of sewage effluent with in-plant treatment only. This enhances the aesthetic aspects and public acceptance of potable reuse of municipal wastewater, because the water is pumped from wells and has lost its identity as sewage water.

**Pretreatment.** Before municipal wastewater is used for SAT, it usually receives conventional primary and secondary treatment, at least in the United States where such treatment is required to meet discharge permit requirements. However, since SAT systems can remove a lot more BOD than is in secondary effluent, secondary treatment really is not necessary where the effluent is used for groundwater recharge and SAT. As a matter of fact, the higher organic carbon content of primary effluent actually may enhance nitrogen removal by denitrification in the SAT system (Lance et al., 1980). Also, it may enhance removal of synthetic organic compounds by stimulating greater biological activity in the soil and resulting increase in co-metabolism and secondary utilization (McCarty et al., 1984). Where primary effluent has been used for SAT systems, satisfactory results have generally been obtained (Carlson et al., 1982; Lance et al., 1980; Rice and Bouwer, 1984). Since the total cost of primary and secondary treatment in a large plant (0.4 million m$^3$ per day, for example) may be about $80 per 1000 m^3$, elimination of the secondary treatment step would save about $40 per 1000 m^3$, thus increasing the value of SAT by another $40 per 1000 m^3$. This estimate is based on the assumption that the cost of primary treatment is about the same as the cost of secondary treatment. Secondary treatment requires a lot more energy and could be more expensive than primary treatment, but the cost of sludge handling and disposal in the primary treatment can also be high, depending on local conditions. Since primary effluent generally has a higher suspended solids content than
secondary effluent, hydraulic loading rates of the infiltration basins may be lower and they may have to be cleaned more often. This increases the cost of SAT.

Other treatment methods prior to groundwater recharge and SAT could be lagooning, overland flow, wetlands, or similar "natural" method that does not require a lot of capital and skilled personnel. Infiltration problems, however, could arise if the water from these treatment processes contains a lot of suspended algae because these can form a filter cake or other clogging layer on the bottom of the infiltration basins for the SAT system. The infiltration basins then should be shallow to avoid compaction of the clogging layer and to promote rapid turnover of the water in the basins to minimize additional algae growth (Bouwer and Rice, 1989).

Well Injection

Where SAT with infiltration systems is not feasible because surface soils and/or vadose zones are unsuitable or aquifers are confined, groundwater recharge can be achieved with injection wells. Since aquifer materials often are quite coarse, the treatment benefits of flow of sewage water through an aquifer tend to be small. Also, to prevent clogging of the aquifer interface around the recharge well, the water should be treated to remove all suspended solids, BOD, and microorganisms, and to have a residual chlorine content to minimize bio-clogging of the well and aquifer. Thus, sewage water for well injection should be treated to essentially drinking water standards before it goes into the well. This makes groundwater recharge through wells much more expensive than recharge with infiltration basins. Therefore, where injection wells are used to recharge groundwater with sewage effluent, the pretreatment requirements are much greater and the SAT benefits much smaller than for recharge with infiltration basins. However, the recharge process with wells still offers the benefits of storage in the aquifer, enhanced aesthetics and public acceptance for potable reuse of the water, and the polishing treatment obtained in the aquifer. To maximize the latter, production wells should be a significant distance (1 km or more, for example) from injection wells to allow for sufficient distance and time of underground travel. An example of the sequence of advanced wastewater treatment-injection wells-pumped wells is the system used by the city of El Paso, Texas, to reuse its wastewater for municipal water supply.

CONCLUSION

Soil-aquifer treatment, or SAT, as obtained by using partially treated municipal wastewater for groundwater recharge and recovery, can play an important role in the treatment and storage of wastewater for reuse. Under favorable hydrogeologic conditions, SAT systems produce water that can be used as such for irrigation of vegetables and other crops consumed raw or brought raw into the
kitchen. For potable reuse, additional treatment such as carbon adsorption, reverse osmosis on part of the stream, and disinfection, is needed. The cost of this treatment, however, is less than 40% of the cost of complete in-plant treatment to convert secondary sewage effluent into drinking water. In addition, SAT systems are simple and robust, offer opportunities for water storage, and enhance the aesthetics of potable reuse of wastewater by breaking up the pipe-to-pipe connection of direct recycling.

REFERENCES


GROUNDWATER RECHARGE WITH RECLAIMED WATER:
RESOLVING REGULATORY ISSUES

Martin G. Rigby, Ph.D
William R. Mills Jr., P.E.
Orange County Water District
Fountain Valley, California

ABSTRACT

Groundwater recharge operations along the Santa Ana River provide a significant percentage of the groundwater utilized by 2 million residents in Orange County. During the past 20 years, the percentage of treated wastewater in the Santa Ana River has gradually increased to an estimated 90 percent of the flow from March through November. As the percentage of wastewater in the Santa Ana River has increased, nitrate levels increase along with other inorganic and organic constituents have also increased. Although monitoring data has shown that constituents of concern to health and regulatory officials are effectively removed by soil filtration, health officials have expressed concern that insufficient data exists regarding the toxicological properties of the River water used for spreading. Health agencies have recommended that the Orange County Water District document the changes in water quality throughout the Santa Ana river system, with particular emphasis on soil purification processes. The Orange County Water District has recently assembled a research team to determine both the pollutant removal efficiency of the Santa Ana River operations and answer related toxicological questions. The health effects study will take 3-4 years to complete, with the first 2 years focusing on data collection and the third and fourth years on data analysis and interpretation.

INTRODUCTION

The base flow of the Santa Ana River at Prado Dam currently averages 130,000 acre-feet/year, with projected flows of 280,000 by the year 2010. Storm flows are becoming greater as impervious areas such as roofs, streets, and lined channels proliferate in the growing urban areas above Prado. Even in dry years, the combination of base flows and occasional large runoff events exceed the recharge capacity of the District's facilities.

The District operates an extensive groundwater replenishment system covering about 1,500 acres (Figure 1). In order to recharge the additional supplies available from the Upper Basin, the District
has developed a master plan to enhance the infiltration capability of the existing recharge system and to construct new recharge facilities.

Construction of a pump station and pipeline to transfer water from Burris Pit to Santiago Creek was completed in 1989 (Figure 2). This system will increase the District's capacity to capture Santa Ana River storm flows for groundwater recharge by 20,000 to 30,000 acre-feet in wet and normal years. A pump-out facility is planned for this facilities as at the other recharge basins.

HEALTH AND REGULATORY ISSUES

The Santa Ana River has always functioned as the primary method of recharging the Orange County groundwater basin, providing a significant percentage of the water utilized in the central and northern regions of Orange County (Figure 3). During the past 20 years, the percentage of wastewater in the Santa Ana River has gradually increased to the point where from March through November the percent wastewater exceeds 90 percent (Figure 4). As the percentage of wastewater in the Santa Ana River has increased, the nitrate levels have increased, as have levels of other inorganic and organic constituents.
Figure 2. Santiago Creek recharge facilities.

Figure 3. Consumption and sources of groundwater within the Orange County Water District boundaries.
Figure 4. The percentage of reclaimed water in the Santa Ana River has increased over time.

Several years ago, a blue ribbon panel was convened to review groundwater recharge with reclaimed water. The conclusions of this panel are published in The Report of the Scientific Advisory Panel on Groundwater Recharge With Reclaimed Water, prepared for the State of California Department of Water Resources in November 1987. This report recommended that a panel of health experts be convened to formulate guidelines for spreading and injection of wastewater, replacing the current case-by-case approach. As water becomes an increasingly scarce resource, it was realized that a growing number of agencies would seek to augment groundwater supplies by spreading or injecting reclaimed water.

The spreading operations along the Santa Ana River are unique. The Orange County Water District does not hold discharge permits; it receives wastewater flows from discharges in both Riverside and San Bernardino counties (Figure 5). These discharges dominate Santa Ana River flows during the spring, summer, and winter months. For this reason, the State Department of Health Services and the Regional Water Quality Control Boards have recommended that the Orange County Water District document the changes in water quality throughout the Santa Ana River, with particular emphasis on the purification benefits derived from soil filtration.

With this goal in mind, the District has recently expanded its water quality monitoring program as described below.
Expanded Water Quality Monitoring Program

The Prado surface water quality monitoring program consists of monthly sampling of eight locations. Samples are analyzed for complete general minerals and volatile organics. Five sites are located above Prado Dam at key tributaries receiving secondary or tertiary treated sewage treatment plant (STP) effluent discharging into the Santa Ana River (Figure 6). This sampling program will determine:

- Sources of recharge water.
- Total amount of applied reclaimed water.
- Total amount of recharge water.
- Hydrogeologic characteristics of the underlying groundwater basin.
- Probability of dilution of reclaimed water with natural groundwater.
- Residence time of recharge water in recharge facilities.
The District's current and proposed reclamation projects may be potentially impacted by upcoming stringent Department of Health Services (DOHS) Reclamation Criteria. To ensure that adequate data are provided, the Prado/Upper Watershed monitoring program has been greatly expanded (Table 1).

The recharge facilities monitoring program is designed to 1) address health effects concerns of using reclaimed water for recharge, 2) assess degradation of pollutants with flow and residence time through the Santa Ana River, and 3) determine water quality from the point of recharge to point of extraction.

Within the basins, increased sampling programs have also been instituted. Water Quality staff monitor the lakes for electrical conductivity, dissolved oxygen, pH, temperature and light penetration using a Hydrolab instrument, irradimeter, and secchi disk. Parameters are measured at one meter increments from water surface to the bottom to provide a depth profile. Field observations include wind direction, speed, and a brief description of weather conditions. Core samples are coordinated with divers at the initial filling of the basins. In 1989, chlorophyll samples were added to the monitoring program.
**TABLE 1**

**SANTA ANA RIVER MONITORING PROGRAM:
WATER QUALITY CONSTITUENTS CURRENTLY ANALYZED**

<table>
<thead>
<tr>
<th>Constituent</th>
<th>Analyte</th>
</tr>
</thead>
<tbody>
<tr>
<td>EC @ 25°C (umhos/cm)</td>
<td>Org-N</td>
</tr>
<tr>
<td>TDS (mg/L)</td>
<td>TKN</td>
</tr>
<tr>
<td>pH/Temperature (°C)</td>
<td>OH⁻ as CaCO</td>
</tr>
<tr>
<td>Na⁺</td>
<td>CO₃⁻ as CaCO</td>
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<tr>
<td>K⁺</td>
<td>HCO₃⁻ as CaCO</td>
</tr>
<tr>
<td>Mg²⁺</td>
<td>Tot. Hardness as CaCO</td>
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<td>Ca²⁺</td>
<td>F⁻</td>
</tr>
<tr>
<td>Fe</td>
<td>Cl⁻</td>
</tr>
<tr>
<td>Mn</td>
<td>NO₂⁻⁻ as N</td>
</tr>
<tr>
<td>Ag</td>
<td>Br⁻</td>
</tr>
<tr>
<td>Al</td>
<td>NO₃⁻⁻ as N</td>
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<tr>
<td>As</td>
<td>PO₄³⁻⁻ as P</td>
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<td>SO₄⁻²</td>
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<td>B</td>
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<td>Cu</td>
<td>TOC</td>
</tr>
<tr>
<td>Hg</td>
<td>MBAS (Surfactants)</td>
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<tr>
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<td>Suspended Solids</td>
</tr>
<tr>
<td>Se</td>
<td>Settleable Solids (mL/L)</td>
</tr>
<tr>
<td>Zn</td>
<td>Turbidity (NTU)</td>
</tr>
<tr>
<td>NH₃⁻⁻ as N</td>
<td></td>
</tr>
</tbody>
</table>

The monitoring requirements for the percolation study will continue through 1990. Anaheim Lake and Kraemer Basin will be focal points for collecting data for the health effects study. These data, combined with other data from Prado, the Recharge Facilities program and wells near recharge facilities, should provide a thorough overview of water quality of origin to points of extraction.

The Non-Point Source Discharge Program is a new program to determine the water quality of major tributaries discharging into or adjacent to our recharge facilities (river and off-channel). Seven locations have been tentatively selected as major discharge points into the Santa Ana River. The flow from these facilities includes runoff from 7,500 acres. Four county flood control channels, representing surface runoff of about 10,500 acres, will also be monitored. Data from this program will be beneficial to develop the District's Forebay Hazardous Materials Release and Response Plan.

Finally, a new storm flow monitoring program has been established to assess surface water quality generated from storm flows. All sites identified in the Prado/Upper Basin program will be monitored to determine changes in constituents and concentration during storm events. In addition, all recharge facilities and all sites identified in the Non-Point Source Discharge Program will be

19
monitored to assess water quality flowing into our basins and percolating to groundwater.

All of the above described baseline data will be utilized by the newly formed Scientific Advisory Committee.

Establishment of Scientific Advisory Committee

The Orange County Water District has assembled a Scientific Advisory Committee consisting of 19 recognized experts to determine the efficiency of the Santa Ana River operations and to develop appropriate epidemiological and toxicological tests for both river and imported supplies. The full committee met on February 19, 1991 to develop the outline of an appropriate scope of work. The committee is currently in the process of finalizing this scope of work.

In addition to the water quality monitoring program described previously, toxicological tests will be conducted at sample sites, as shown in Figure 7. Due to the very high costs associated with quality toxicological research, it is necessary to select sample locations carefully. Epidemiological studies would also be conducted in communities that pump groundwater from the vicinity of the spreading basins.

Removal efficiency studies would be conducted for the entire Santa Ana River system (from the points of discharge in the river to the nearest potable wells). Particular attention will be paid to nitrates, organics, viruses, and all constituents of concern identified by health and regulatory agencies.

It is estimated that this study will take 3-4 years to complete and cost $5-$6 million. The first 2 years will focus on data collection, and the third and fourth years on data analysis as outlined below:

- **Spring 1991:** Scientific Advisory Committee to prepare full scope of work.
- **Summer 1991:** Obtain funding.
- **January 1992:** Begin study.
- **January 1994:** Complete data collection.
- **Summer 1995:** Complete final report.

**CONCLUSIONS**

Throughout the arid west, there is a surge of new interest in the use of reclaimed water. Health and regulatory agencies have recognized this trend and are finalizing draft regulations for both injection and spreading of reclaimed water. The Orange County Water District, in cooperation with state and local health and regulatory officials, plans to fully analyze and then optimize current spreading operations. Since the wastewater content of the Santa Ana River is high, this study would provide extremely valuable data that could be utilized to develop appropriate regulations for other recharge operations that utilize higher percentages of reclaimed water.
The Orange County Water District has developed an extensive water quality monitoring program and is developing a state-of-the-art recharge system. The Orange County Water District shares and supports the requests of health and regulatory officials that a comprehensive study be initiated on the Santa Ana River to document both percent removal of constituents of concern and toxicological and epidemiological effects for current and planned spreading operations. Many agencies will be able to utilize the data from this study to formulate sensible groundwater recharge programs using reclaimed water. The widespread interest in this topic is evidenced by widespread support received by all agencies affiliated with this study. The District is currently in the process of obtaining the funding necessary to expand this study to add toxicological and epidemiological studies to the already growing water quality data base.
REFERENCES


Crook, James, Asano, Takashi, Nellor, Margaret, August 1990. Groundwater Recharge with Reclaimed Water in California. Water Environment & Technology, Water Pollution Control Federation.


SITING OF SPREADING BASINS FOR UNDERGROUND STORAGE OF TREATED MUNICIPAL WASTEWATER

Peter Mock, CH2M HILL, Inc. Tempe, Arizona

ABSTRACT

The siting of spreading basins for underground storage of treated municipal wastewater requires consideration of both operational and institutional requirements. Operational requirements typically include high surface infiltration rates, correspondingly high vadose zone transmission rates, aquifer capacity to dissipate the resulting mounding in proportion to the vadose zone transmission, and aquifer characteristics favorable to tracking and recovery of the recharged wastewater. Some variations on these requirements can be tolerated if land and operations costs are small for the area considered. A mismatch between surface infiltration rates and vadose zone transmission rates should be avoided where it results in excessive or complex spreading in the vadose zone. Institutional requirements are those of the regulatory programs as well as addressing the compatibility of recharge with existing land and water uses. ADWR and ADEQ have structured permitting programs whose requirements are typically addressed by the data and interpretations developed for maximum operational efficiency described above. Likewise, the considerations of land and water use compatibility in basin siting typically also address the requirements of the permitting programs. Examples of using commonly available data for siting wastewater recharge basins are provided to illustrate these concepts.

INTRODUCTION

Purpose

This paper describes the approach taken to site surface spreading basins in order to recharge treated municipal wastewater. The reasons for recharging municipal wastewater and the basic concepts of siting spreading basins are described in numerous existing articles and textbooks. However, the practical application of the standard concepts to siting in Arizona deserves an update as the 1990s begin in light of the practically available data and the regulatory atmosphere in Arizona.

Experience indicates that the data needed and approaches taken to implement a cost-effective recharge program for treated municipal
wastewater over the long-term are largely coincidental with the statutory mandates of the present ADWR and ADEQ permitting programs. Recharge requires substantial investments and it will be conducted responsibly for its own sake. The challenge is to demonstrate this to the agencies within the framework of the statutes. When the feasibility-level preparations are adequate to design a cost-effective recharge project with a high likelihood of success, the permitting process should be a minor effort.

This report is organized into four sections: project operational requirements, institutional requirements, recent Arizona siting experience, and conclusions.

**Project Operational Requirements.** Project operational requirements consist of data and analyses needed to estimate the feasibility of implementing a successful recharge project. Surface infiltration, vadose zone transmission, aquifer transmission, and aquifer recovery capacity are the primary categories evaluated under project operational requirements.

**Institutional Requirements.** The permitting programs of ADWR, ADEQ, the local flood control district, and the Army Corps of Engineers need to be addressed as appropriate. Other institutional requirements include addressing compatibility with existing and projected land and water uses.

**Recent Arizona Recharge Siting Experience.** Recent siting studies in the west Phoenix area and in the Tucson area provide indications of the data typically available and a current approach for addressing operational and institutional requirements.

**Conclusions.** Conclusions to be drawn at this time focus on how operational and institutional requirements overlap and can be addressed concurrently in an efficient manner.

**PROJECT OPERATIONAL REQUIREMENTS**

**Surface Infiltration**

A successful recharge project will be sited in an area with high surface infiltration rates. This allows minimization of the area required for recharge basins and directly affects land and operational and maintenance (O&M) costs. Depending on the soils characteristics, very high (>6 feet/day) infiltration rates may lead to reduced nitrogen species removals. The impact of the trade-off between nitrogen removal and infiltration rate is specific to the site and to project goals. Depending on local land and O&M costs, lower surface infiltration rates can be tolerated by creating larger basins.
Vadose Zone Transmission. In order to take advantage of high surface infiltration rates, a vadose zone characterized by high vertical hydraulic conductivity is advantageous. An absence or low occurrence-frequency of silt-clay intervals is indicative of higher vertical hydraulic conductivity in the vadose zone. Some resistance to vertical flow can be tolerated if the areal extent of saturated ("perched") conditions does not extend beyond a practical recovery area or into undesirable areas (e.g., landfill materials) and if the depth of saturated conditions does not rise up into the floors of the basins. Often, isolated intervals may build up a stable 5 to 20 feet of positive head during infiltration which may not significantly affect recharge operation.

Aquifer Transmission. Once the water has passed from the land surface through to the areally extensive local water table, tracking of the recharged water is simplified if its movement in the water table aquifer is characterized by low, flat mounding and small regional transport velocities. Such conditions are provided by a relatively homogeneous, aerially extensive aquifer of high transmissivity and small regional hydraulic gradients.

Aquifer Recovery Capacity. Recovery of recharged wastewater for seasonal storage and recovery objectives is challenging in that the peak recovery capacities are very large (20,000 to 30,000 gpm) on some typical projects. An aquifer of very high transmissivity is typically required to provide the needed recovery well capacity. Longer-term (decades) recovery does not require such unusually high capacities.

Another factor has to do with vertical transmission of the recharged water. If large vertical gradients are locally present, precise control of the recharged water may be difficult and seasonal recovery of all of the recharged water may not be practical. However, for longer-term recovery such precise control is typically not required and interception of the recharged wastewater can take place some distance below and down gradient from the recharge basins if the wastewater is treated to ambient water quality levels for key parameters.

Interrelationships. The above operational requirements are typically accommodated in a hydrogeologic setting of essentially continuous coarse-grained sediment deposition since early Quaternary to Tertiary time. In the Phoenix and Tucson AMAs, some locations adjacent to large river beds have been located which provide large surface infiltration rates, adequate vadose zone transmission with minimal mounding, and simple movement of the recharged water near the water table in the aquifer.
INSTITUTIONAL REQUIREMENTS

Compatibility with Land Uses

Surface spreading projects for recharge of treated municipal wastewater (typically groups of 1 to 10 acre basins) are most compatible with industrial/commercial or agricultural land uses. Spreading projects are typically not as compatible with high-visibility areas because of the alternating wet-dry cycling required for the continued success of recharge. The wet-dry cycling causes the spreading basins to be inconsistent water features; half of the basins are dry or nearly so at any one time. The basin facilities also need to be protected by fencing from entry as any disturbance of the basins' floors will reduce infiltration.

If the project needs to be located within the 100-year floodplain, excavation of the basin floors rather than building up of berms is typically practiced to keep from raising the flood profile during infrequent flood events. When conducting recharge in floodplains, it is often important to monitor and conduct recharge so as to keep water from laterally entering deeply excavated gravel mining operations.

Compatibility with Water Uses. Recharge of treated municipal wastewater is compatible with existing water uses to the extent that the operations do not unfavorably impact 1) the pumping lift or the water quality of adjacent groundwater wells, or 2) the natural infiltration of surface water in streambeds during infrequent flood events.

Design of the recharge project can protect against local hydraulic impacts on land and well owners. An example of this would be to place large-capacity recovery wells to the side of the project away from adjacent existing groundwater wells. Design should also include monitoring and operation to avoid saturating large areas of Recent Alluvial materials below ephemeral stream channels.

For seasonal storage and recovery, design incorporating hydraulic control of the recharged and recovered water can protect against degrading the water extracted by adjacent well owners. For longer-term recharge, additional treatment of the wastewater to ambient water quality levels for key parameters is often pursued prior to recharge, making hydraulic control of flow paths less important.

ADWR Permitting Program. ADWR’s Underground Storage and Recovery permit process essentially addresses the water use compatibility requirements described above. Hydraulic and water quality impacts within an "area of hydrologic impact" are reviewed. A benefit of the ADWR permit program is that it provides the framework for water rights concerning the recharged water.
ADEQ Permitting Program. ADEQ's Aquifer Protection Permit (APP) program essentially addresses the water quality aspects of water use compatibility. The statutes require that a point of compliance be identified on a site-specific basis for meeting numerical standards. In most cases, control of recharged wastewater such that drinking water standards are not exceeded at adjacent wells complies with the statutes and protects those who need protection. These adjacent wells make logical points of compliance for the APP. Monitoring cost-effective operations of either hydraulic control for seasonal storage projects or of treatment for longer-term recharge projects would allow correction of any deficiencies long before impacts on adjacent wells would be indicated by violation of numerical standards at these compliance points.

ACE-FCD Permitting Programs. The U.S. Army Corps of Engineers (ACE) handles Section 404 Permits (Clean Water Act) for modifications of river channels. Since many of the hydrogeologic requirements for optimal operation of recharge occur adjacent to large river channels in the Tucson and Phoenix AMAs, ACE 404 Permits are often required. Design of recharge basins within floodplains to eliminate flood-profile increases as described above addresses the primary concerns of this permit program. The ACE 404 permit in the past has also included consideration of surface water quality changes by ADEQ. Contingency plans to keep treated wastewater out of infrequent ephemeral flood flows can address these concerns.

County flood control districts manage floodplains and have floodplain activity permitting programs. Typically these programs require that if recharge is conducted in the floodplain, the basins must be excavated rather than having built-up berms. This addresses requirements for no increase greater than 0.1 foot in the flood profile, and the mined gravel and sand can sometimes be sold.

Interrelationships. Obviously, a great deal of overlap is present in the permitting programs (especially between ADWR and ADEQ) as the various agencies ensure that the public is not harmed by the recharge activities. As can be seen from the previous discussions, long-term, successful, cost-effective wastewater recharge operations have operational and institutional requirements that coincide with the requirements of the permitting programs. The challenge is to coordinate the submittal and discussion of permit application materials with the concerned agencies so that they are assured of the integrity of the recharge design and implementation plan.
RECENT ARIZONA RECHARGE SITING EXPERIENCE

Typical Existing Data and Uses

Experience in the Tucson and Phoenix AMAs has led to a consistent program of using all of the available data to infer hydrogeologic conditions favorable or unfavorable for conceptual recharge projects with wastewater. The Tucson AMA has been more extensively studied and investigated than the Phoenix AMA, therefore, a more comprehensive database and interpreted framework for the hydrogeology is typically available for Tucson projects. However, useful preliminary evaluations of feasibility can typically be made in both AMAs with existing data and interpretations. As with any large investment, significant on-site testing should be conducted prior to financial commitment to a full-scale wastewater recharge project. The results of this on-site testing are typically also used to prepare the agency permit applications.

The following are descriptions of current typical existing data sources and approaches used to evaluate recharge feasibility at the conceptual level in the Phoenix and Tucson metropolitan areas.

Land Uses. Classes of land uses for recharge siting are typically general in nature: agricultural, industrial/commercial, residential, parks, floodplain, or unused. Land uses are typically evaluated by inspecting current aerial photography. Land uses are further evaluated by driving through the area. Past land uses can also be reviewed with aerial photography which is often available extending back in time to the 1940s.

Land uses of particular concern are landfills and uncontrolled hazardous waste disposal sites. Landfill Advisory Committees, and County and ADEQ staff are particularly helpful in locating such land uses in particular areas. Figure 1 shows a map of floodplain, river channel, and less-desirable land uses created for a recent siting study.

Soils. Soil Conservation Service (SCS) mapping is available for most areas of interest. The estimated infiltration rates provided by the SCS for each soil type should not be taken literally in the context of artificial recharge. Rather, the relatively younger and coarser soil series are a good start for concentrating the search for likely areas for siting surface spreading basins compared to older (and more cemented) and/or finer-grained series. SCS surveys typically investigate to only 5 feet, so the mapping does not necessarily represent deeper horizons. Figure 2 shows a portion of a soils map created for a recent siting study from SCS mapping.

Geotechnical exploration holes have sometimes been drilled for various projects in the study area and the resulting logs supplement the SCS work. Large contrasts in soil horizons are often noted in cable tool driller’s logs and these can be used, if
available in large numbers (dozens to hundreds), to infer distinctive soil horizon trends. Figure 3 presents a portion of a map created for a recent siting study which interprets the thickness of finer-grained surficial soils.

Vadose Zone. Driller's or geologists's logs are typically the only available information on the general nature of the vadose zone. If large numbers of these logs are available, distinctive trends can often be inferred. Sequences of silt/clay in the vadose zone are of particular concern. If such sequences are consistently described in lithologic information, that particular area may be screened from consideration.

The thickness of the vadose zone is commonly interpreted from recent depth-to-water measurements in available shallow wells. Thicknesses of at least 50 feet are usually required as a preliminary criterion for basin siting. This allows some room for uncertainty in estimating the depth to water and room for mounding in the preferred high-transmissivity aquifers. Greater accuracy in estimating depth-to-water is achieved by subtracting a grid of land surface elevations from water-level elevations as opposed to contouring depth-to-water measurements. Figure 4 presents a portion of a depth-to-water map from a recent siting study where a minimum depth to water of 20 feet was allowed for siting because of very high (500,000 gpd/ft and greater) transmissivity in the area.

Aquifer. Driller's or geologists's logs are typically the only information available on the general nature of aquifers in an area. If large numbers of these logs are available, large-scale distinctive trends can be inferred. Significant aquifers, aquitards, and hardrock boundaries can often be inferred which are refined in description with on-site stratigraphic testing. Figure 5 shows a portion of a map used to locate coarser-grained areas in the water-table aquifer for a recent siting study. In this case, percent-fines is used in a preliminary manner as a surrogate for transmissivity data.

Water-quality evaluations typically center on finding areas where existing water quality is similar to that of the treated wastewater. Fortunately, such areas have been found where hydraulic factors are also favorable for recharge in the Phoenix and Tucson AMAs. Groundwater near the water table with high total dissolved solids and high nitrates is particularly common in the Phoenix AMA. These are the primary parameters used in siting wastewater recharge projects. Other chemical or biological constituents distinctive to wastewater, e.g., pathogens or trace organics, are often substantially removed by surface spreading basins with alternating wet-dry cycling. Of particular concern are zones of existing groundwater contamination associated with landfills or uncontrolled hazardous waste disposal sites (see land uses). Design of the recharge facility should address minimizing undesirable additional movement of these zones. Various hydraulic simulations can assist in the design phase. For preliminary
Figure 4
Depth to Water (in Feet)
(December 1982)
siting studies, spreading basins should be placed at least one-half mile from such zones to minimize impacts from recharge operations.

Figure 6 shows a portion of a map of groundwater uses created for a recent siting study. Domestic wells are of particular concern in siting recharge projects.

CONCLUSIONS

Consistency Between Operational and Institutional Requirements

It is apparent that there is consistency between the operational requirements of a cost-effective long-term wastewater recharge project and institutional requirements, particularly agency permitting programs. The challenge lies in communicating that consistency to regulatory agencies in the framework of the statutes they address with permitting programs.

Data Availability

Data are commonly available for useful feasibility studies and preliminary siting of wastewater recharge projects in the Tucson and Phoenix AMAs. Prior to actual investment in design and construction of a recharge projects, on-site testing is required to reliably estimate operational parameters.

ACKNOWLEDGEMENTS

CH2M HILL would like to acknowledge the contributions to the practice of recharging treated municipal wastewater in Arizona shared by the staffs of Tucson Water, the City of Phoenix, and the Town of Gilbert. Special thanks also go to Dr. Herman Bouwer and Dr. Gray Wilson for innumerable impromptu lessons given with great patience.
FATE OF CHLORINATION BY-PRODUCTS DURING SOIL AQUIFER TREATMENT

Aimee Conroy
University of Arizona
Tucson, Arizona

John Chahbandour
University of Arizona
Tucson, Arizona

Gary L. Amy
University of Colorado, Boulder
Boulder, Colorado

L. Gray Wilson
University of Arizona
Tucson, Arizona

Bruce Johnson
City of Tucson - Tucson Water
Tucson, Arizona

ABSTRACT

A research study, sponsored by the Tucson Water and Salt River Project, was established to investigate the possible use of soil aquifer treatment as a treatment process and its feasibility in the Tucson and Phoenix areas. Two 12 foot by 12 foot test-basins located at the Sweetwater Underground Storage and Recovery Facility and the University of Arizona's Water Resources Research Center (WRRC) water farm were used to study the treatment obtained during effluent recharge. The study used both secondary (Sweetwater test-basin) and tertiary (WRRC test-basin) effluent. Lysimeters were installed in both test-basins, so the vadose zone could be sampled during recharging. A favorable amount of removal has been achieved for both test-basins in terms of dissolved organic carbon (DOC) and total organic halides (TOX). Microbiological activity in the soil appears to be enhancing treatment.

INTRODUCTION

Increased development and population growth have caused water scarce areas, such as metropolitan Phoenix and Tucson, Arizona, to
investigate alternative sources of water. Tucson is one of the largest metropolitan areas in the United States to rely solely on groundwater. The Central Arizona Project (CAP) water, as well as strict conservation measures will help lighten the burden on Tucson's groundwater supply, but this is not enough to help meet projected demands. The Salt River Valley, on the other hand, is dependent on groundwater and CAP water as well as reservoirs in northern Arizona, which are not always at full capacity. As a result, the City of Tucson and the Salt River Project (SRP) decided to investigate the use of groundwater recharge of treated wastewater as a means to augment their water supplies.

A research study, sponsored by SRP and Tucson Water, was established to investigate the possible use of effluent recharge as a treatment process and its feasibility in the Tucson and Phoenix areas. This project is primarily interested in direct surface recharging of treated wastewater using high-rate rapid infiltration through excavated basins otherwise known as soil-aquifer treatment (SAT). Rapid infiltration takes advantage of the soil's natural ability for biodegradation, sorption, and filtration of applied effluent (Bouwer, 1980). Once the renovated wastewater reaches the aquifer where it receives additional treatment due to dilution and other mechanisms, and it becomes stored.

Research Objectives

The major objective of this project was to study the mechanisms of soil aquifer treatment using a test-basins at two sites located in the Tucson area, the Sweetwater Underground Storage and Recovery Facility and the University of Arizona's Water Resources Research Center (WRRC) site. Characterizing the chemical properties of water recovered from such operations will elucidate treatment requirements to upgrade recovered water to potable standards.

The specific objectives of this research are to 1) examine two chemical parameters of applied and recovered secondary effluent: total organic halide (TOX), and dissolved organic carbon (DOC), 2) determine the treatment of the recharged water in the vadose zone, 3) determine if the recovered effluent from the site is suitable for use as drinking water with or without additional treatment, and 4) study the secondary test-basin's effluent over a long time period to determine if any change in treatment occurs.

SITE LOCATION AND DESCRIPTION

Sweetwater Site Test-Basin

The City of Tucson operates its reclamation and recharging facilities at the Sweetwater Underground Storage and Recovery Site. The Sweetwater site is located a quarter mile west of the Roger Road Treatment Facility, on the west bank of the Santa Cruz River. General groundwater flow is toward the northwest, roughly paralleling the Santa Cruz River, with a hydraulic gradient of
0.002 to 0.003 ft/ft (Weissenborn, 1988). The site currently consists of four excavated spreading basins. Groundwater beneath the four basins are monitored by a total of ten monitoring wells.

A 12 ft by 12 ft test-basin was constructed in the south end of Basin 1 along with a series of eight monitoring wells. The test-basin was constructed study the chemical, hydrological, and microbiological behavior of the site when secondary effluent is recharged through virgin alluvium. The test-basin is equipped with seven stainless steel suction samplers or lysimeters. These units were installed at depths of 1, 2.5, 5, 7, 9.5, 15, and 20 feet in order to sample the vadose zone during recharge. Four tensiometers were also installed at various depths. A 15 foot access tube was installed for neutron moisture logging. The tensiometers and neutron logging are used to track the recharged water front as it moves through the vadose zone. The Sweetwater test-basin's influent is received from the nearby Roger Road Wastewater Treatment Facility, which treats municipal wastewater using primary clarification followed by biotowers, secondary clarification and chlorination. The average water quality is shown in Table 1.

<p>| TABLE 1 |
| AVERAGE WATER QUALITY CHARACTERISTICS FOR THE SOURCE WATERS |</p>
<table>
<thead>
<tr>
<th>SOURCE</th>
<th>DOC (mg/l)</th>
<th>UV ABS @ 254nm</th>
<th>TOX (ug/l)</th>
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<tbody>
<tr>
<td>Reclaimed Water</td>
<td>11.4</td>
<td>0.170</td>
<td>181.3</td>
</tr>
<tr>
<td>Secondary Effluent</td>
<td>12.2</td>
<td>0.150</td>
<td>93.5</td>
</tr>
</tbody>
</table>

The soil profile beneath the Sweetwater secondary test-basin is approximately 55% to 65% large cobbles and gravel intermixed with coarse sand, silt and clay until approximately 16 feet. At 16 feet, there is an interface between alluvial and basin fill material. This interface has a high percentage of silt and clay (approximately 15%). Silt/clay lenses also seem to be present in the upper 3.5 feet and between 12.0 and 13.5 feet of the soil profile. These silt/clay lenses appear to promote the lateral movement of recharged water. Figure 1 shows the locations of the silt/clay lenses. The $f_{oc}$ is relatively higher at the surface (0.098) than elsewhere in the profile.

WRRC Test-Basin

The WRRC test-basin, at the University of Arizona's Water Resources Research Center water farm, is located on the east bank of the Santa Cruz River. The test-basin is 12 feet by 12 feet and was constructed on the existing land surface. It was equipped with ceramic lysimeters at depths of 0.5, 1, 2, 5, 8, 15, and 20 feet in order to sample the vadose zone during recharge. Tensiometers were
Figure 1. Variations in Silt and Clay as a Function of Depth for Site 1 (WRRC) and Site 2 (Sweetwater).
also installed at various depths throughout the soil profile. The WRRC test-basin was flooded with reclaimed wastewater piped from Tucson Water's Reclamation Facility and had been chlorinated before entering the system. The average water quality characteristics are shown in Table 1.

The soil profile beneath the WRRC test-basin consists entirely of alluvial stream and floodplain deposits. In general, the soils between the land surface and about 10.5 feet below the surface can be characterized as sands and sandy loams. Within the interval, between the land surface and 3.0 feet below the surface the soils are sandy loams and loamy sands. From 4.5 feet to 13.5 feet the soils are comprised of mostly sands and gravels and are classified as sands. At 13.5 feet, there is a well defined change to a sandy loam soil. Below 13.5 feet, the soil profile quickly changes back to sand with virtually no silt or clay present. The percentage of silt and clay as a function of depth is shown in Figure 1. By comparing the treatment of the two basins the effect of textural characteristics on treatment can be determined.

MATERIALS AND METHODS

Sampling Protocol

Samples were collected from the influent, surface and lysimeter units at the Sweetwater and WRRC test-basins during each of the flooding cycles. A vacuum pump was used to apply a constant vacuum pressure on the lysimeters through a tygon tubing manifold so the sample could be collected through the porous cup. To recover samples collected in the lysimeters, a positive pressure was applied to the pressure/vacuum line using a modified bicycle pump. As a positive pressure was applied, the sample was drawn through the sampling port and collected in a amber 250 ml sample bottle, headspace-free, if possible.

Analytical Protocol

Influent, basin, and lysimeter samples were analyzed according to the following analytical protocol:

Dissolved Organic Carbon (DOC) was analyzed using a Dohrmann DC-80 Total Organic Carbon Analyzer.

Total organic halide (TOX) concentration was measured using a Dohrmann DX-20A Total Organic Halide Analyzer. TOX was measured using a two step process which involved an adsorption step on to activated carbon followed by a quantification step.
RESULTS AND DISCUSSION

Sweetwater Site Test-Basin

Three flooding cycles of approximately 168 hours (excluding Cycle 3 which lasted 192 hours) were used for this research. Flooding cycles were separated by a 7-day drying period. The drainage cycle allowed rejuvenation of intake rates due to desiccation of the algal mat formed at the soil surface. Initial infiltration rates approached 50 ft/day for Cycle 1 and declined to a steady rate of approximately 2 ft/day. The surface of the test-basin was not raked or rehabilitated between the organic flooding cycles. A total of 9330 ft$^3$ for Cycle 1, 8712 ft$^3$ for Cycle 2, and 6810 ft$^3$ for Cycle 3 were infiltrated during the flooding periods. A constant head of approximately one foot was maintained during flooding.

Sweetwater Test-Basin Organic Carbon Removal. One of the main objectives of the three flooding cycles was to establish trends in the removal of organic material as the secondary effluent percolated through the vadose zone. To this end averages for specific time intervals during the cycles have been calculated. The averages for all three cycles appear in Table 2. This table shows overall trends in DOC within each recharge cycle and between cycles. The last average for each cycle shows the average DOC concentration at the end of the flooding period and during the drying cycle. Treatment continues during the drying cycle. Deeper depths appear to have achieved up to 23% additional DOC removal for Cycle 1 and up to 26% for Cycle 2. This additional removal continues during Cycle 3 but not to the same extent as the previous cycles. This could be due the flooding cycle being longer and that little drying had occurred before the last sample was collected.

As previously mentioned, one purpose of this project was to track the overall treatment level of the system over a long period of time. This gives a basis to predict overall treatment performance. Intra-cyclic increases in DOC removal were also observed as the profile matured despite variable influent DOC concentrations.

Treatment effectiveness for each cycle can be compared by examining the relative percent removal or C/C$_o$ relationship through the effective soil profile of 15 feet. For Cycle 1 after approximately 4 days of flooding, the overall percent removal is 41%. After approximately 16 days (9 days after flooding stopped), the percent removal over the whole cycle is approximately 46% on the average. For Cycle 2 after approximately 3 days of flooding the percent removal is 47%. After approximately 10 days, the percent removal based on the average is 70%. This continues for Cycle 3 where after 3 days, the percent removal is 65%. After 10 days when the last sample was collected, the percent removal over the whole cycle based on the average is 77%. It is quite evident that removal increases over time.
### TABLE 2

AVERAGED DOC CONCENTRATIONS FOR CYCLES 1, 2, AND 3
FOR SECONDARY TEST-BASIN LYSIMETERS

<table>
<thead>
<tr>
<th>TIME RANGE (HOURS)</th>
<th>BASIN</th>
<th>1.0 ft</th>
<th>2.5 ft</th>
<th>5.0 ft</th>
<th>7.0 ft</th>
<th>9.5 ft</th>
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<td>7.14</td>
<td>8.32</td>
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<td>7.04</td>
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<td>6.27</td>
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<td>7.54</td>
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<td>5.71</td>
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<td>5.59</td>
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<td>0.5-72.5</td>
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<td>6.15</td>
<td>6.14</td>
<td>5.89</td>
<td>6.10</td>
<td>4.30</td>
<td>6.15</td>
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</table>
Several removal mechanisms could be responsible for the continued increase in DOC removal from cycle to cycle. Two possible removal mechanisms are adsorption and biodegradation which are intimately linked. As the effluent percolates through the vadose zone, organic material is adsorbed onto the soil. As the drying cycle ensues, aerobic conditions are reestablished in the vadose zone promoting the growth of bacteria. The bacteria, in turn, utilize the organic material adsorbed on to the soil. Therefore, the soil's ability to adsorb organic material is constantly renewed. As the wetting and drying cycles continue larger and well assimilated colonies of bacteria are developed. Therefore, as cycling continues the bacteria continue to improve the soil's ability to adsorb organic material, thus explaining the improvement in treatment from cycle to cycle. Also the bacterial population is constantly being augmented from the bacteria present in the effluent. The bacterial population is dynamic in both character and size. Additional treatment also occurs as the profile becomes clogged, because mechanical removal is increased.

Sweetwater Test-Basin Total Organic Halide Removal. While the overall goal of SAT systems is the removal of organic matter from treated effluent, it is important to quantify the amount of organic halides (TOX) removed. This is due to the toxic and carcinogenic nature of some forms of organic halides.

Averages for specific time intervals during the three flooding cycles have been calculated and appear in Table 3. For Cycles 1 and 2, the last average shows the average TOX concentration at the end of the flooding period and during the drying cycle. As with the DOC data, when the drying cycle TOX concentrations are compared to the previous flooding period, it was shown that TOX removal continued as the basin was allowed to drain. During the first two drying cycles, there was additional removal throughout the entire profile of up to 48% for Cycle 1 and 40% for Cycle 2. Unlike the DOC data, the additional removal during the drying cycle was pronounced throughout the entire soil profile and not just at deeper depths.

The TOX database for Cycle 3 was incomplete because equipment failure during the analysis of this data. Therefore, it is difficult to identify any trends in the Cycle 3 TOX data.

TOX removal of each cycle is compared by examining the percentage of removal \((C/C_0)\) through 15 feet of soil. For Cycle 1, there was 31% removal after approximately 4 days and 75% removal over the whole cycle. For Cycle 2, there was 44% removal after 3 days and 68% removal over the whole cycle. Cycle 3 is not complete, but there was approximately 26% removal for the first 3 days. It is not actually understood what is occurring in Cycle 3 in terms of TOX removal. Still, it appears that there is an increase in removal in the first two cycles.

Discounting the incompleteness of the Cycle 3 data, it does not appear that TOX removal increases from cycle to cycle as it does.
# TABLE 3

**AVERAGED TOX CONCENTRATIONS FOR CYCLES 1, 2, AND 3**

**FOR SECONDARY TEST-BASIN LYSIMETERS**

<table>
<thead>
<tr>
<th>TIME RANGE (HOURS)</th>
<th>BASIN</th>
<th>1.0 ft</th>
<th>2.5 ft</th>
<th>5.0 ft</th>
<th>7.0 ft</th>
<th>9.5 ft</th>
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<tr>
<td>0-99.0</td>
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<td>87.0</td>
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<td>83.0</td>
<td>78.8</td>
<td>86.5</td>
</tr>
<tr>
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<td>83.0</td>
<td>78.6</td>
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<td>69.7</td>
<td>72.0</td>
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<td>80.1</td>
<td>85.0</td>
<td>83.8</td>
<td>81.1</td>
<td>62.4</td>
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</tbody>
</table>

* DUE TO EQUIPMENT FAILURE ALL SAMPLES WERE NOT ANALYZED.
** AVERAGE BASIN VALUE FOR PREVIOUS TWO CYCLES.
for DOC. This could be due to the inability of the indigenous bacteria to degrade halogenated organic. Therefore, the soil's ability to adsorb TOX is never fully restored.

**WRRC Test-Basin**

During the Summer of 1989, research was performed at the University of Arizona's Water Resources Research Center (WRRC) using tertiary effluent. The test-basin was flooded with tertiary effluent for 96 hours at a constant head of 0.6 feet. Approximately, 1340 ft³ of reclaimed water was applied to the profile at the WRRC site. Flooding was discontinued after it was determined that the wetting front had reached 20 feet and that representative samples of water moving past this point had been collected. A longer flooding period was not used to avoid conflicting with the permit requirements of Arizona's 1986 Recharge and Recovery Act.

**WRRC Test-Basin Dissolved Organic Carbon Removal.** There does appear to be removal of DOC in the WRRC basin, up to approximately 50% at 20 feet over the whole cycle. The concentration of DOC was initially high due to mobilization of in situ humic substances. The DOC found in the upper depths appeared to be distributed throughout the profile as flooding continued.

**WRRC Test-Basin Total Organic Halide Removal.** For the WRRC basin there was an 77 % reduction in TOX over the whole profile. The difference in removal for each basin can best be explained by the differing capabilities of each soil profile in terms of adsorption, filtration, and biodegradation.

The overall trend indicates there is a decrease in TOX concentration with depth. Yet, an increase in TOX concentration at a given depth with time does appear to be occurring as well. These increases could be a reflection of desorption processes and/or a deterioration of the mechanisms responsible for the disappearance of TOX at those depths. An example of this is the 8 foot depth which shows an increase in TOX concentration from approximately 60 µg/l at 42 hours to concentrations of about 80 - 85 µg/l at later times. It is likely that the adsorption capacity of the soil above the 8 foot level was becoming exhausted. Therefore, adsorption of organic halide compounds occurred at lower depths. As time progressed, the adsorption capacity of the soils at 8 feet was decreased. This is supported by an increase in TOX concentrations at the 15 foot depth at later times.

**CONCLUSIONS**

From the limited data available for the WRRC plot, it appears that the overall removal of chlorinated organics is better than that of the Sweetwater basin. Still, the Sweetwater basin has shown continued treatment as it ages without any apparent sign of breakthrough after 3 months of recharging cycles.
The amount of water applied to the basin is an important aspect of each system. For example, the infiltration rates at the Sweetwater test-basin usually start out at up to 50 ft/day and decrease to approximately 2.0 - 3.0 ft/day by the end of the 7 day flooding period. The infiltration rates at the WRRC test-basin were approximately 3 ft/day and decreased over the 96 hour flooding period to about 1.5 ft/day. If more importance is placed on the amount of water recharged into the system, the performance at the Sweetwater test-basin is significantly more appealing. There is often a trade-off between treatment, economy, and actual throughput. When this trade-off is considered, the Sweetwater site obtains significant treatment of the source water which is available and relatively inexpensive at a throughput which is very desirable.

Some type of post-treatment will be necessary if the renovated water from either site is to be used as a potable water source. The minimum amount of treatment necessary will be chlorination or some other disinfectant. Whenever disinfection is required, evaluation the amount of Trihalomethane precursors present in the water becomes necessary. As a result, an additional treatment step such as nanofiltration or activated carbon could be used to remove some of the precursors prior to disinfection. While any type of post-treatment will increase the final cost of the water, it will be more cost effective than importing water to augment existing water supplies.

REFERENCES


VIRUS AND BROMIDE TRANSPORT THROUGH SANDY ALLUVIUM
WITH INFILTERATED TREATED SEWAGE

D.K. Powelson, D.J. Cline, M.T. Yahya, L.G. Wilson, and C.P. Gerba
University of Arizona, Tucson, AZ 85721

ABSTRACT

Soil-aquifer treatment of sewage may aid in the removal of pathogenic microorganisms. Three experiments were conducted where two non-pathogenic bacterial viruses, MS-2 and PRD-1, and a chemical tracer, potassium bromide (KBr), were added to secondary-treated sewage before infiltration into a 12 ft by 12 ft basin of coarse sand alluvium at Tucson Water's Sweetwater Underground Storage and Recovery Facility. Samples of the percolating water below the basin were taken through porous stainless steel suction-samplers. Impeding layers, particularly at the interface between channel- and basin-fill units at 16 feet, resulted in a perched water table monitored by 15 to 20 ft deep wells near the basin. Tracers were observed in wells up to 148 feet from the infiltration basin. Bromide data indicated a high degree of lateral and vertical preferential flow. Initial breakthrough velocities for bromide and viruses were similar. Less removal of MS-2 was observed at all depths under ponded conditions compared to PRD-1, but MS-2 was removed at a higher rate during drying of the basins. For the high infiltration rate test (20 to 55 ft/day), numbers of MS-2 were reduced by 90% and PRD-1 by 99% by passage through 15 ft of alluvium. During low infiltration rate tests (0.6 to 6 ft/day) greater removal was observed. With slower flow through 15 ft, MS-2 was reduced by 99.7% and PRD-1 by 99.99%.

INTRODUCTION

Land application of domestic sewage effluent has been considered as an effective method of disposal and treatment (Bouwer at al., 1985). Rapid infiltration of sewage into sandy soils can result in improvements in its physical, chemical, and microbiological quality; reductions in turbidity, nitrates, biochemical oxygen demand (BOD), bacteria, and viruses may be achieved after passage of sewage effluent through a few meters of soil (Bouwer at al, 1985). The infiltrated wastewater may be used to recharge groundwater and subsequently be pumped to the surface for reuse.

Arizona is a state with limited water resources and reuse of domestic wastewater is critical to meet future water needs. The
City of Tucson has constructed the Sweetwater Underground Storage and Recovery Facility for the underground storage of sewage effluent during non-peak-demand periods. The wastewater is then pumped to the surface during the dry summer seasons when water demand in the community is at its peak. The recovered wastewater is currently used for irrigation of golf courses and other grassed areas. The possibility also exists of recovery and treatment of recharged effluent for potable purposes.

The objective of this study was to evaluate the fate of viruses in the upper 20 feet of the vadose zone during the land application of sewage. Viruses tend to be transported farther than bacteria or parasites and may be considered the worst case from a public health standpoint (Yates and Yates, 1988). Information gained from this research might be used to better design and manage soil-aquifer treatment sites for optimal removal of chemical and microbial contaminants.

MATERIAL AND METHODS

Experimental Site

The Sweetwater Underground Storage and Recovery Facility is located on the western edge of the city of Tucson, Arizona adjacent to the usually dry bed of the Santa Cruz River. The principle features at the facility are four large infiltration basins, covering approximately 14 acres, and three extraction wells. The basins are underlain by Santa Cruz River alluvium 15 to 20 feet thick. The alluvium is composed of coarse sand and gravel with some discontinuous clay layers. At the 15 to 20 foot depth is a clay rich layer at the alluvium - basin-fill interface. The basin-fill deposits are unsaturated to a depth of approximately 100 feet. Two mini-basins were constructed in one of the large infiltration basins at the Sweetwater Site (Fig. 1). Only the mini-basin designated for secondary treated sewage effluent was used in this study. The effluent was chlorinated after passing through trickle biofilters at the Roger Road Treatment Plant. The sewage effluent was obtained via a buried pipeline which supplied a nearby golf course.

The 12 ft by 12 ft mini-basins were constructed from four 12 ft by 4 ft steel walls buried two feet into the floor of Basin 1. A two inch brass discharge pipe, connected to the delivery pipe, discharged the sewage effluent into the mini-basins. The level in the pond was controlled with a float valve. An in-line flow meter measured the volume of water delivered to the mini-basins.

Eight monitoring wells were installed in Basin 1 at the locations shown in Fig. 1. (Only results for wells 3, 5, and 6 will be discussed since the farther wells were not sampled and well 4 was often dry.) The wells were drilled into the clay layer at the alluvium-basin fill interface (15 to 20 feet), and cased with 2-inch polyvinyl chloride (PVC) pipe. Each casing was slotted from
Fig. 1. Experimental Site.

about five feet to total depth. These wells were used to delineate the extent of lateral spreading in a perched water mound which develops at the alluvium-basin fill interface, and for collecting samples for chemical and microbiological analyses. Located within each mini-basin are seven stainless-steel suction samplers for collection of samples from the vadose zone. Stainless steel samplers were used since preliminary studies showed that use of this material resulted in no apparent adsorption or die-off of the tracer viruses (MS-2 and PRD-1). Depths of the individual samplers in the secondary-effluent minibasin were 1, 2.5, 5, 7, 9.5, 15, and 20 ft. (For simplicity, only sampler results from the 5 and 15 ft depths will be compared in this paper.) Samples were collected with the aid of a suction pump to maintain a constant suction (200 to 250 millibar) in the samplers.

A total of 9 floodings of the mini-basin with secondary effluent were conducted, mainly of 7 day duration, interspersed with 7 day drying periods. Three tracer test were included among the 9 floodings. The effluent temperature ranged from 28°C to 29°C during the August tracer test, from 27°C to 30°C during the September tracer test, and was 22°C during the December tracer test. The viruses were injected into delivery line of the secondary sewage effluent prior to entering the mini-basin with the aid of a chemical feed pump. Since the sewage was chlorinated, sodium thiosulfate (Na₂S₂O₃·5H₂O) was injected along with KBr tracer up stream of the virus injection point to dechlorinate the effluent. The resulting sodium thiosulfate concentration in the effluent was
5 mg/L. The feed pumps were adjusted to the infiltration rate to keep the pond concentration of the tracers approximately constant.

Virus and bromide tracers were fed for 2 days in the August Test (infiltration rate of 55 to 20 ft/day); and for 3 days in the September Test (infiltration rate of 3 to 2 ft/day). In the December Test (infiltration rate of 6 to 0.6 ft/day) virus tracer was injected for 10 days and bromide tracer for the first 3 days. Samples for bromide and virus assays were collected from the sewage effluent ponded in the mini-basin, the wells, and the stainless steel suction samplers. Samples were collected in 20 ml polypropylene sterile containers, and 2 mL subsamples for virus were kept on ice until assayed.

**Virus Propagation and Assay**

To study the possible fate of human enteric viruses, bacteriophage MS-2 and PRD-1 were selected because of their low adsorption to soils and their long survival time [Yates et al., 1985 (PRD-1); Powelson et al., 1990 (MS-2)]. MS-2 is similar in shape and size (28 nm) to poliovirus and PRD-1 is similar in shape and size (62 nm) to rotavirus (Fraenkel-Conrat et al., 1988). MS-2 was grown on agar overlay of host bacterium *Escherichia coli* (ATCC 15597) and PRD-1 on bacterium *Salmonella typhimurium* LT2. Viruses were assayed by the plaque forming technique (Adams, 1959) on trypticase soy agar (Difco, Detroit, MI), and enumerated as plaque forming units (PFU). Samples for virus assay were collected, stored on ice, and assayed within 48 hours.

**Bromide analysis**

Bromide is widely assumed to be conservative when used as a tracer in ground water where the concentration ranges from tens to thousands of milligrams per liter. A conservative chemical tracer was used in order to determine flow paths and velocities as a function of the infiltration rate. Bromide concentration were determined using a bromide ion-specific electrode. The potential developed across the membrane of the electrode is proportional to the bromide concentration (Orion, 1982).

**RESULTS AND DISCUSSION**

**August Test**

Total duration of this test was 3 days. The infiltration rate started at 55 ft/day and declined to 20 ft/day after 30 hours flow (Fig. 2). Results of the bromide concentrations are shown in Fig. 3. The bromide concentration from the 5.0 foot depth reached that of the pond in approximately 2 hours. The concentrations from the 15.0 foot depth and well 6 reached that of the pond by 7 hours. Bromide concentrations in wells 3 and 5 attained concentrations of
Fig. 2. Infiltration rates during the August Test.

Fig. 3. Bromide results during the August Test.
approximately 0.6 and 0.5 of the pond concentrations, respectively. Well 3 is approximately 148 feet from the mini-basin and reached its maximum concentration after 30 hours of flooding. Well 5 is approximately 80 feet from the mini-basin and reached its maximum concentration after 70 hours of flooding. These results suggest that flow occurs preferentially in the lateral direction at the alluvium-basin fill interface during flooding of the mini-basin.

Both MS-2 and PRD-1 were detected in all of the stainless steel suction samplers in less than 3 hours except the 20 ft sampler which was below the alluvium-basin fill interface and did not produce enough water to sample. Wells were not sampled. Results for depths 5 and 15 feet are shown in Figures 4 and 5. Virus concentrations rapidly rose to near the levels of the pond as the infiltrating water reached the samplers. PRD-1 persisted longer after application of effluent to the pond was stopped (Fig. 5).

September Test

Total duration of flooding was seven days. Effluent was applied without any tracers for 4 days to allow the infiltration rate to decline to 3 ft/day (Fig. 6). Tracers were then added continuously for 3 days. At the end of the experiment the infiltration rate was 2 ft/day.

As a result of the lower infiltration rate, bromide concentration at the 5.0 foot depth did not reach that of the pond until 20 hours into the test and the 15.0 foot depth and well 6 concentrations reached that of the pond after 35 hours (Fig. 7). Bromide concentration in well 3 attained approximately 0.5 that of the pond concentration after 120 hours. Bromide concentration in well 5 only attained a value of 0.1 that of the pond concentration.

Results of the virus concentration in depths 5 and 15 ft, and wells 3, 5, and 6 are shown in Fig. 8 and 9. Regardless of the infiltration rate, greater removal of PRD-1 occurred than of MS-2, but PRD-1 appear to survive longer when viruses were no longer infiltrating. Between 90 and 95 percent of the MS-2 was removed by the first 5 feet of soil and 99 percent or greater by 15 feet of soil. In contrast 99 to 99.9% of the PRD-1 was removed by 5 ft of soil and usually more than 99.99% by 15 ft of soil. Greater removal of PRD-1 may be due to its larger size or greater hydrophobicity than MS-2 [PRD-1 contains some lipid in its protein coat (Fraenkel-Conrat et al., 1988, pp.41,249.)].

Virus detection in the wells was fairly rapid after infiltration had begun. PRD-1 concentrations were detected in well 3, located 148 feet from the basin, within 5 hours of the beginning of tracer application (Fig. 9). First detection of bromide in this well also occurred in 5 hours (Fig. 7). Higher concentrations of PRD-1 were observed in the wells than MS-2, which is in contrast to results from the samplers beneath the mini-basin. Again this may reflect the greater survival of PRD-1.
Fig. 4. MS-2 virus results for the August Test.

Fig. 5. PRD-1 virus results for the August Test.
Fig. 6. Infiltration rates for the September Test.

Fig. 7. Bromide results for the September Test.
Fig. 8. MS-2 virus results for the September Test.

Fig. 9. PRD-1 virus results for the September Test.
December Test

Virus tracers were applied for 10 days and the bromide for the first 1.6 days of this period. Infiltration rates started at 6 ft/day and declined to 0.6 ft/day after 10 days (Fig. 10). These rates were similar to the September Test during the application of tracers.

Bromide concentrations at the 5.0 and 15.0 foot depths reached that of the pond after 36 hours of infiltration (Fig. 11). This arrival of the tracer at the same time for the two different depths may be due to preferential flow in the vertical direction. The bromide concentration in well 6 reached 0.90 that of the pond concentration after 70 hours of flooding. Wells 3 and 5 behaved similarly and bromide concentrations reached 0.20 that of the pond after 20 hours of flooding.

The virus results for 5 and 15 ft and wells 3, 5, and 6 are shown in Fig. 12 and 13. MS-2 and PRD-1 were removed with similar rate. Highest concentrations occurred early in the experiment (5 to 20 hours) during the highest infiltration rates. After this peak concentrations tended to decline to the end of the experiment at 200 hours. This may indicate that a large number of virus adsorption sites exist and all were not filled during application of virus, or that non-detected virus were inactivated. The results should also be related to infiltration rates and subsurface flow paths. As the surface clogs, more flow occurs through finer sequences of pores and with greater contact time.

Concentrations of virus in the wells were low except for PRD-1 in well 6 (Fig. 13), the well nearest the mini-basin. PRD-1 levels in this well were higher than at the 5 ft depth. In contrast, bromide concentrations at 5 ft reached the pond concentration in 23 hours, while in well 6 bromide concentration reached only 0.90 that of the pond after 70 hours of flooding (Fig. 11). This peculiar result may have been due to interaction of PRD-1 characteristics with preferential flow patterns.

CONCLUSIONS

High infiltration rates (20 to 55 ft/day) through 15 ft of coarse sand and gravel resulted in removal of 90 % of the MS-2 and 99% of the PRD-1. The corresponding removals at low infiltration rates (0.6 to 6 ft/day) were 99.7% for MS-2 and 99.99% for PRD-1. The data indicated up to 148 ft of horizontal transport of the viruses. This observation is supported by the bromide tracer data. However, horizontal transport is strongly direction dependent, as evidenced by the variability in bromide concentrations between wells 3, 5 and 6.
Fig. 10. Infiltration rates for the December Test.

Fig. 11. Bromide results for the December Test.
Fig. 12. MS-2 virus results for the December Test.

Fig. 13. PRD-1 virus results for the December Test.
These results suggest that different viruses will experience different degrees of removal by the same soil type. How much virus reaches the groundwater will depended on the survival rate, electrostatic characterics, hydrophobicity, size, and other characteristics of the virus. To better understand the transport of pathogenic viruses in soil, more than one virus should be tested. It appears that virus removal of at least 99% or more after passage through 15 feet of soil are possible. Preferential flow patterns in alluvial material, both horizontally and vertically, make more precise predictions difficult. Greater removals of enteroviruses and other human pathogenic viruses can probably be expected since they generally adsorb to a greater degree to soils than the bacteriophage used in this study (Goyal and Gerba, 1979). It was also demonstrated that transport of a fraction of the virus appears to be fairly rapid both vertically and horizontally. Further studies are underway to determine the effects of infiltration rate and turbidity on virus removal during soil-aquifer treatment.

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REFERENCES


REGULATORY CONSIDERATIONS FOR AQUIFER PROTECTION PERMITS

William Marceau
Arizona Department of Environmental Quality
Phoenix, AZ

ABSTRACT
Arizona's Aquifer Protection Permit (APP) rules were certified on September 26, 1989. The permitting process under these rules is outlined in R18-9-107 of the Arizona Administrative Code; the actual application requirements are itemized in R18-9-108. The rules for permitting recharge and underground storage and recovery (USR) projects are the same as for other Aquifer Protection Permits, except for the BADCt requirement, from which recharge/USR projects are exempt pursuant to R18-9-109.A. The application process entails demonstrating that the facility will not violate Aquifer Water Quality Standards at the points of compliance. The applicant must submit a detailed hydrogeologic study of the area where recharge is to occur, to demonstrate site feasibility. If site characteristics are to be used for pollutant reduction, this must also be demonstrated. The applicant has the option of proposing aquifer quality limits, alert levels, monitoring programs, and points of compliance. In addition, the applicant must demonstrate that he is financially and technically capable of constructing, operating and closing the facility, and that the facility complies with all applicable municipal or county zoning ordinances. The application must be submitted a minimum of 180 days prior to the expected date of beginning operations. R18-9-107 specifies time periods for each phase of the application. The primary factor causing unexpected delays is incomplete or incorrect applications.

Pursuant to A.R.S 49-250, ADEQ now issues APP's only for those recharge projects using effluent as source water. These permits will be processed jointly with recharge permits issued by ADWR. Permits for recharge of other source waters are issued by ADWR, with ADEQ retaining authority to recommend permit conditions pertaining to water quality concerns.

INTRODUCTION
Prior to October, 1990, all recharge and underground storage and recovery (USR) projects in Arizona were required to receive an Aquifer Protection Permit (APP) from ADEQ, pursuant to Arizona
Revised Statutes Title 49, Chapter 2, Article 3, and Arizona Administrative Code Title 18 Chapter 9, Article 1. In addition, all such projects were required to apply for a recharge or USR permit from ADWR. The permitting process for the ADEQ and ADWR permits was done cooperatively. A.R.S. 49-250 exempts CAP water and natural watercourse waters from the APP program. Recharge and USR projects using non-effluent waters continue to require recharge permits from ADWR, with ADEQ having authority under Title 45 to review the application for water quality concerns and to make water quality recommendations to ADWR for such permits. ADEQ continues to issue APP's for projects recharging effluent. The remainder of this paper discusses the permitting process for these projects. To date, effluent projects under consideration have all involved municipal effluent; there have thus far been no applications for industrial effluent recharge projects.

The cooperative permitting process by which ADEQ and ADWR review applications for non-effluent recharge and USR projects is discussed in a separate paper and presentation by ADEQ hydrologist Jim BuBois.

**ADMINISTRATIVE CONCERNS**

For recharge projects using effluent as source water, the permitting process remains as outlined in R18-9-107 of the Arizona Administrative Code. The actual application requirements are as itemized in R18-9-108. The process typically begins with a pre-application meeting, in which the applicant meets with an ADEQ hydrologist and permit writer and with representatives from ADWR. The ADEQ representatives outline the permitting process and basic hydrogeological requirements (R18-9-108.C.) for the applicant. In addition, an applicant is encouraged to submit to the Department for review and comment a proposal for meeting any of the information requirements of the permit process; the ADEQ will comment on this proposal within 30 days of its receipt. This proposal is usually presented as a scope of work. When the actual application is formally submitted, ADEQ has 30 days to respond in writing to indicate the application's completeness. If the application is incomplete, the applicant is informed in writing of the missing items. The ADEQ has 20 days to review each follow-up submittal of additional information.

Following written notification of an application's completeness, ADEQ is allotted 90 days to review the application for technical sufficiency; this time does not include the time required by the applicant to respond to any ADEQ requests for additional technical information. During this phase of permit development, an ADEQ hydrologist evaluates the hydrogeologic study included with the
application. Details of the hydrogeologic review are discussed in a later section, Hydrogeologic Study, of this paper. Also examined during the sufficiency review period are questions of financial and technical sufficiency, discussed in the section under that heading, and zoning considerations. Pursuant to R13-9-108.B.10., an applicant must submit evidence that a facility to be permitted complies with all applicable municipal or county zoning ordinances; the evidence accepted by the ADEQ is a signed statement from the appropriate zoning authority verifying this compliance with the ordinances in question.

When ADEQ has accepted all aspects of an application and is satisfied that the project will not cause or contribute to a violation of an aquifer water quality standard, the Water Permits Unit issues a letter stating ADEQ's preliminary decision to issue the permit. The permit is then drafted and public noticed within 30 days of the issuance of this letter signifying the preliminary decision to permit. The public notice is published in a newspaper distributed within the geographic area of the facility. The public has the right to submit written comments on the draft permit or the project during this period. The ADEQ Water Permits Unit will respond in writing to all significant comments. Also, all draft permits are reviewed by other units within the ADEQ and by other concerned agencies (EPA, health departments, local planning and zoning boards, etc.); this internal/external review usually occurs just prior to the public notice, although it is not required by law. If the Director determines that there is no need for a public hearing, the permit will be finalized and sent to the Director for signature within 30 days of the close of the public comment period. Prior to permit signature, the facility owner, operator, and landowner must sign a statement certifying that they have read the permit and agree to abide by its conditions.

PUBLIC HEARING

Sometimes the public interest in a project is such that a public hearing is requested. If the Department determines that significant public interest in a hearing exists, a hearing will be scheduled to begin on or before 75 days from the close of the comment period established by the public notice. During a hearing, a hearing officer presides over the proceedings, and all comments are recorded on tape as well as by a court reporter. The public also has the right to submit written comments by mail, so long as the hearing record remains open. (The record is closed within seven days of the end of the hearing.) ADEQ responds in writing to all comments. If the Department decides to issue a permit despite objections raised during a public hearing, the objecting parties may still appeal to the Water Quality Appeals Board.
A hearing may extend the permit process by several months. Concerns raised during a public hearing may result in requests by the Department for additional information, in order to resolve problems of which the Department or the applicant had previously been unaware. Following the hearing comment period, the Director has authority to delay the final decision on whether to issue or deny a permit by as much as 90 days from the close of the hearing record.

HYDROGEOLOGIC STUDY

The applicant for an APP for a recharge or USR project is required to submit a hydrogeologic study of the project site. A single study may be submitted for ADEQ and ADWR. ADWR's application packet contains guidance for such a study. This study should define the discharge impact area for the operational life of the facility and demonstrate that the facility will not cause or contribute to violation of an Aquifer Water Quality Standard at the applicable point of compliance. The study must demonstrate that the soils at the site chosen will accept the water to be recharged without causing flooding or resurfacing, and that the effluent is of suitable quality for recharge. The applicant must also demonstrate that the soils in the area will not contribute to groundwater pollution; for example, it must be clear that the soils are free of pesticides or other chemicals which could be flushed into the groundwater when exposed to large quantities of percolating water. In addition, it must be clear that contaminant plumes existing within the area impacted hydrologically by the recharged water will not be mobilized in such a way as to impact neighboring drinking water supplies. (Potential applicants should be aware that there is no BADCT requirement for recharge or USR projects, pursuant to R18-9-109.A.)

During this review for technical sufficiency, the applicant and ADEQ work out details of permit conditions, including groundwater and discharge monitoring programs. The applicant has the option of proposing monitoring parameters, alert levels, discharge limits, aquifer quality limits, and points of compliance.

In a typical recharge facility, groundwater monitoring is extensive. Groundwater monitoring parameters are chosen based on contaminants identified as potentially present in the source water, in the area soils, or in known contaminant plumes in the area. Aquifer quality limits are set as permit conditions. These limits are usually set at levels specified in Arizona's Aquifer Water Quality Standards, but may be set higher if groundwater has already been degraded to levels higher than those standards. Alert levels are warning levels set lower than the Aquifer Quality Limits, in order to provide an early warning system that contamination may in fact be occurring. Exceedance of an alert level at a facility
requires the permittee to implement contingency measures specified in the permit. Source water monitoring is also required in recharge/USR projects, to verify that proper treatment of the source water is continuing.

One common concern or interest is the use of site specific characteristics to attenuate pollutants in the source water. The ADEQ will consider data which demonstrates performance of site characteristics to accomplish this goal. The most obvious example is the removal of microbial contaminants during percolation. Also, the use of rapid infiltration basins with alternate wetting and drying cycles as a means of denitrification is acceptable. Pilot testing is often advisable to carefully determine operational parameters which will ensure pollutant removal.

FINANCIAL AND TECHNICAL CAPABILITY

All permit applicants must submit proof that they are technically capable of carrying out the terms of the permit. A demonstration of relevant experience or training, or the possession of pertinent licenses or certifications, will satisfy this requirement.

Applicants other than governmental entities must also demonstrate that they are financially capable of carrying out the terms of closure and post-closure of a permit. During the application review process, an estimate of anticipated closure and post-closure costs is agreed upon by ADEQ and the applicant. Pursuant to A.A.C. R18-9-108.B.8.c., the applicant must demonstrate capability to meet these costs by submittal of one of the following: (1) a copy of the applicant's 10K form filed pursuant to section 13 or 15(d) of the Federal Securities and Exchange Act of 1934; (2) a report describing the applicant's legal and financial status; (3) evidence of a bond, insurance or trust fund assuring that the applicant will be able to meet the costs agreed upon. If the applicant chooses to submit a financial report, this report will be reviewed by the ADEQ accounting section. ADEQ policy is to accept only copies of audited financial statements as constituting such a report.

STREAMLINING THE PERMIT PROCESS

Applicants frequently ask if the permit process can be expedited or otherwise streamlined. From an administrative point of view, the most significant and commonly encountered obstacle in permit development is the submittal of an incomplete application. Potential applicants should realize that the best way to expedite the permit process is to submit all documents and information correctly, according to the directions and requirements in the
permit application. Information should be clearly labeled or titled, and should not require that ADEQ personnel research documents of peripheral importance to the project. Also, if an applicant wishes to use in his application a document which has been submitted to another state agency or to another unit within ADEQ, a duplicate or a photocopy of this document must be submitted to the Water Permits Unit as part of the APP application.

Also, the proposal and pre-application processes are intended to clarify requirements and streamline the permit process. Communication with the permit writer regarding questions or concerns is always encouraged. This is especially important during early stages of permit development.
INTERPRETING ARIZONA’S RECHARGE STATUTES FROM A TECHNICAL PERSPECTIVE

Wayne R. Cooley and Greg L. Bushner
Arizona Department of Water Resources
Phoenix, Arizona 85007

ABSTRACT

The State of Arizona enacted legislation in 1986 that provided the necessary framework for implementing artificial groundwater recharge and underground storage of water. Several of the terms and definitions used in the statutes require further interpretations from a regulatory and technical perspective. For many of these terms it was necessary to make a technical determination from which a policy decision could be based. Technical interpretations that will be discussed in this paper are: (1) the area of hydrologic impact, (2) the recoverable amount of water that has reached the aquifer, as defined in the statutes, and (3) that the project will not cause unreasonable harm to land or other water users within the area of hydrologic impact of the project. The primary purpose of the paper is to present the technical basis from which the Department is interpreting the Recharge and Underground Storage and Recovery statutes. The secondary purpose of the paper is to discuss the impacts of such interpretations.

INTRODUCTION

In 1986 the Arizona State legislature passed the Recharge and Underground Storage and Recovery Act. This legislation established the permitting process for two types of artificial groundwater recharge projects that include (1) recharge and (2) underground storage and recovery (US&R). In this paper these projects are collectively referred to as recharge (although both projects allow for the artificial addition of water to an aquifer, only US&R projects allow for later recovery of the stored water by the project permit holder). The recharge programs were established to assist the state in using its maximum entitlement of Colorado River water, to help in reducing groundwater overdraft and achieve management goals, to store water underground for seasonal peak demand use and during years of drought, and to augment the water supply for future growth and development.
The Arizona Department of Water Resources (ADWR) administers the permitting program established by the legislature for artificial groundwater recharge in order to carry out the above mentioned goals. While the concepts presented here represent the current policy of the Department regarding the recharge program, final policy determinations have not yet been made. In administering the recharge program, the ADWR has tried to be consistent in the methodology and criteria it uses to evaluate applications. This is not an easy task due to the fact that all recharge projects and sites are different. The site-specific nature of each recharge project makes it difficult to administer the statutes using a single methodology. The methodologies used in evaluating recharge projects include physical monitoring, analytical, and numerical tools. Geologic, hydrologic, operational, qualitative, and quantitative parameters greatly affect the analysis and results of the method used. Therefore, after several years of evaluating and interpreting the statutes, the Department has concluded that no single methodology can be used in the characterization and regulation of all sites.

This paper presents three technical issues that are inherent in the administration of the recharge program. They are: (1) the area of hydrologic impact, (2) the recoverable amount of water that has reached the aquifer, as defined in the US&R statutes, and (3) that the project will not cause unreasonable harm to land or other water users within the area of hydrologic impact of the project (referred to as unreasonable harm). Each of these issues is discussed in detail below.

AREA OF HYDROLOGIC IMPACT

The Area of Hydrologic Impact (AOHI) is defined in ARS Sections 45-651.2 and 45-802.2 to mean "...as projected on the land surface, the areal extent of the migration of water stored [recharged] pursuant to an underground storage and recovery [recharge] project...."

The concept of the AOHI is used many times throughout the recharge statutes. The AOHI determines the area in which the Director will examine a proposed project for "unreasonable harm" to others (ARS Section 45-804.B.4), it determines the area in which monitoring of the project is required (ARS Section 45-804.G), it determines the area in which recovery wells may be located (ARS Section 45-807.A), and it determines the percent debit that storage accounts are charged when water is recovered (ARS Section 45-809.D.1.d.). Although this concept is widely used and very important to the recharge statutes, its definition is imprecise. The actual area of hydrologic impact will vary throughout space and time as water is added to or withdrawn from the project, thereby making it very difficult to identify as required by statute.
The AOHI can be identified by either direct measurement (monitoring) or by a calculation. The Department’s consideration of the two methods is summarized in the following paragraphs.

Monitoring the actual physical migration of water pursuant to an underground storage and recovery project is a monumental task and would require extensive field data (i.e. monitor wells). Stresses (such as well pumpage) other than the recharge project also effect the migration of water and need to be isolated. The Department has determined that such an undertaking could easily cause many proposed recharge projects to be uneconomic and in effect be unsupportive of the Department’s goals.

Calculating the AOHI offers the possibility of approximating the areal extent of the migration of stored water without the extreme economic burden of physical identification of the stored water. The areal extent of the AOHI is currently defined by a one foot rise in equipotential of the potentiometric surface due to recharge. The primary reason such a definition was made is because the driving mechanism for the migration of stored water is advective transport and migration of stored water outside the one foot rise in equipotential would probably be negligent. Table 2 (to be discussed in the Unreasonable Harm section) provides a comparison of the results from several numerical simulations to a simple particle tracking model. This table illustrates the fact that although the outer fringe of the AOHI or the one foot rise in equipotential can extend for several miles in twenty years, the actual water molecules or particles travel only several thousand feet in the same time frame. Furthermore, if for any reason the one foot rise definition needed to be physically substantiated, measurements of less than one foot rise in equipotential due to recharge would be extremely difficult to isolate and quantify.

Currently the AOHI as defined in the statutes is interpreted from three perspectives for regulatory purposes. The first perspective is called the maximum AOHI, the second, the minimum AOHI, and finally an annual calculated AOHI. The maximum AOHI analysis is directed toward protecting land and other water users (statutory requirement) under a maximum storage scenario. This assumes the permittee will recharge the total volume of water the facility is permitted for and not recover water during the life of the permit. The minimum AOHI provides the permittee with a given area where stored water can be recovered with a minimum aquifer cut (water recharged that may not be recovered). At the present time it is defined as the half mile radius around the spreading basin(s) or injection well(s) center(s). The annual calculated AOHI is determined in conjunction with the permittee’s storage account each year. This determination is made after receipt of the permittee’s annual report. The amount of water applied to, or injected into, and recovered from the aquifer is evaluated and confirmed. This AOHI calculation is updated each year based upon the net change in storage from the beginning of the project through the end of the most recent annual report period.
Evaluation Technique

The AOHI was evaluated using both analytical and numerical solutions. Each incorporated several different combinations of aquifer parameters and volumes of water recharged. Aquifer parameters included specific yield values that were varied from .05, .10, .15, to .20 and hydraulic conductivity values of 3.8, 19, 38, and 95.5 ft/day. Volumes of water recharged were varied from 365, 912.5, 10,950, 32,850, to 91,250 ac-ft/year. The ranges of specific yield, hydraulic conductivity, and volume recharged were selected to represent the variations in system parameters from recharge project applications that have been submitted to the Department. Fracture flow systems or multiple boundary systems were not addressed at this time. These simulations are intended to provide the Department with an idea of what can be expected for various recharge projects throughout the state. Both analytical and numerical solutions are discussed below.

Analytical solutions offer a relatively easy to obtain first approximation of a physical phenomenon. The assumptions made for the Hantush solution include instantaneous arrival to an isotropic, homogeneous, single layer aquifer with an impermeable bottom (Hantush, 1967). Ninety-six analytical simulations were performed using the computer program GRAMP (Groundwater Recharge and Mounding Program) developed by Singhofen (1983). This gives a generic overview of the sensitivity of the Hantush solution (which is used in the GRAMP and Pit and Well programs) to specific yield, hydraulic conductivity, quantity of water recharged, and growth of the recharge mound in time and space. The simulations were solved at the end of one, five, ten, fifteen, and twenty years.

The AOHI was also evaluated using the U.S. Geological Survey’s numerical groundwater flow model, MODFLOW, by McDonald & Harbaugh (1984). This allowed another approximation of an "ideal aquifer’s" response to artificial recharge. The comparison between analytical and numerical results using the same representation of an aquifer including aquifer characteristics, recharge quantity, distance, and time parameters should give the Department a better understanding of the possible variability in results due to the respective model solution. Numerical simulations incorporating the exact same parameters used in the analytical simulations were made for comparison as shown in figures 1, 2, and 3.

Discussion of Modeling Results

Figures 1 through 6 depict one half of a cross-sectional view through simulations of recharge (independent of all other stresses). Both the analytical and the numerical solutions assumed full symmetry. Only half of the mirror plane is shown for clarity. The model runs are projected on a flat water table representing only the impacts due to recharge.
RECHARGE AFTER 1, 5, 10, 15, 20 YEARS
3,650 AC-FT / YR. UNCONFINED UNIT 350'
K=95 FT./DAY SY=.10 (ANALYTICAL MODEL)

FIGURE 1
RECHARGE AFTER 1, 5, 10, 15, 20 YEARS
3,650 AC-FT/YR. UNCONFINED UNIT 350'
K=95 FT/DAY SY=.10 (NUMERICAL MODEL)

RISE IN WATER LEVEL FROM RECHARGE - FEET

DISTANCE FROM BASIN CENTER (0) IN MILES

SINGLE LAYER SIMULATION - MODFLOW
BASIN DIMENSIONS 600' X 600'
RECHARGE RATE 1 FT./DAY

FIGURE 2
RECHARGE AFTER 5 YEARS, VOLUMES AC-FT/yr
SENSITIVITY ANALYSIS ON VOLUME RECHARGED
V1=365, V2=912, V3=3650, V4=32850, V5=91250

RISE IN WATER LEVEL FROM RECHARGE - FEET

DISTANCE FROM BASIN CENTER (0) IN MILES

SINGLE LAYER SIMULATION - MODFLOW
BASIN DIMENSIONS AND RECHARGE RATES VARY
UNCONFINED UNIT 350' SPECIFIC YIELD =.10

FIGURE 4
RECHARGE AFTER 5 YEARS 3,650 AC-FT/YEAR
SENSITIVITY ANALYSIS - 4 SPECIFIC YIELDS
SY1 = .05, SY2 = .10, SY3 = .15, SY4 = .20

RISE IN WATER LEVEL FROM RECHARGE - FEET

DISTANCE FROM BASIN CENTER (0) IN MILES

SINGLE LAYER SIMULATION - MODFLOW
BASIN DIMENSIONS 660' X 660' RR=1 FT/DAY
UNCONFINED UNIT 350' K = 95.5 FT./DAY

FIGURE 5
RECHARGE AFTER 5 YEARS 3,650 AC-FT/YEAR
SENSITIVITY ANALYSIS ON HYD. CONDUCTIVITY
K1=3.8, K2=19, K3=38, K4=95.5 (FT/DAY)

--- K = 3.8 FT/DAY
----- K = 19 FT/DAY
------- K = 38 FT/DAY
---------- K = 95.5 FT/DAY
------------ 1' RISE IN W.L.

RISE IN WATER LEVEL FROM RECHARGE - FEET

DISTANCE FROM BASIN CENTER (0) IN MILES

SINGLE LAYER SIMULATION - MODFLOW
BASIN DIMENSIONS 660' X 660' RR=1 FT/DAY
UNCONFINED UNIT 360' SPECIFIC YIELD = .10

FIGURE 6
Figure 1 displays the results of an analytical solution using the GRAMP program. Figure 2 shows the results of a numerical solution using the MODFLOW code. Both solutions used the same hydraulic conductivity, specific yield, quantity of water recharged, aquifer thickness, recharge rate, and basin dimensions. The intersection of the horizontal one foot rise in water level line with any given stress period’s slope (depicting the rise in water level from recharge) would be the outer limit of the AOHI as currently defined by the Department. A comparison of figures 1 and 2 shows the maximum mound height varies between 5 to 13 feet, respectively, after 20 years. The areal extent of the one foot rise in water level created by the project varies from about 7 miles in figure 1, to about 9.5 miles in figure 2, after 20 years. Figure 3 displays five different solutions using the same aquifer parameters at the end of five years of recharge. In addition to the previous solutions depicted in figures 1 and 2, a two layer simulation was run (confined system under an unconfined system; vertical conductivity of the two systems being set at one one-hundredth of horizontal conductivity). Another analytical solution used was the Pit and Well program (Sunada et. al., 1985) that solves for both the well function and the Hantush equation. Figure 3 demonstrates that the Pit and Well, Hantush, and the MODFLOW single layer solutions are almost identical in this particular case. The Pit and Well solution is also very similar to the other two previously mentioned solutions with the exception that the inverted cone due to wellbore injection is much steeper at the origin of recharge. The two layer MODFLOW simulation reflects vertical leakage into the confined system thereby lowering the heads in the unconfined system in relation to the unconfined system in the MODFLOW single layer analysis. The vertical leakage is reflected in the higher head in the underlying confined system.

Figure 4 reflects the effects of water levels on five different annual recharge volumes ranging from 365 to 91,250 ac-ft per year. Figure 5 demonstrates the sensitivity of specific yield to one particular set of ideal conditions. Figure 6 shows the relationship between the calculated shape of recharge mounds to different hydraulic conductivities. The MODFLOW single layer simulation was used for the analysis presented in figures 4 through 6. Figures 1 through 6 merely present an extract of the total modeling work performed. The Department does not endorse any particular model, nor has it had the opportunity to calibrate any of the above simulations to an actual site.

Impact of Evaluation

A table summarizing the results is being compiled by the Department to illustrate the potential variation resulting from different models and different parameters. About 125 numerical and analytical solutions were completed for this study. Space does not permit publishing the compiled data in this article.
The simulations made represent ideal conditions rarely if ever encountered in the physical system. Nonetheless, they are appropriate tools for first approximations. The data that has been compiled by the Department can be used to better assist potential applicants prior to actual recharge testing or project operation by providing a range of the potential impacts from recharge. The Department has concluded that the current definition of the AOHI encompasses the extent of the migration of stored water pursuant to a recharge project in laterally extensive aquifers (refer to table 2). ADWR will remain flexible in determining the AOHI in site specific cases such as multiple boundary systems, fracture flow systems, multi-layer systems or sites in which monitoring demonstrates that the Theis (1935) or Hantush (1967) assumptions are not appropriate.

RECOVERABLE AMOUNT OF WATER THAT HAS REACHED THE AQUIFER

Arizona Revised Statute 45-809.E states "...‘recoverable amount’ means the amount of water, as determined by the director, that has reached the aquifer." This statement has been interpreted by the Department to mean water recharged pursuant to a recharge project cannot be withdrawn until it has reached the water table and can physically be recovered by a well. Water may be recovered any time after it has been recharged by an injection well, if the well is perforated below the water table. However, in the case of recharging water using spreading basins, recovery of the stored water is restricted until after the water has percolated through the unsaturated zone and reached the water table. No water will be debited from the storage accounts that was applied to establish a wetting condition to facilitate recharge since that water will eventually arrive to the aquifer system.

To determine when stored water becomes recoverable several equations were developed. Rather than require extensive vadose zone monitoring these equations utilize a Green and Ampt infiltration approach to flow and estimate the condition of the unsaturated zone at the site prior to the start of the recharge project (see table 1, below). Equation (1) listed below requires solving for the \( \Theta w \) term which represents the amount of water (as a percentage of total bulk volume) that is required to wet the vadose zone to a condition that is transmissive for recharge.

\[
\Theta w = n \times 0.85 \tag{1}
\]

Porosity \( n \) is multiplied by a factor of between 80 to 90 percent that is required before a transmissive zone is developed in the unsaturated zone. This factor is an estimate (Booher, 1991) and should not considered as representative of all sites. Site specific monitoring is encouraged, however estimated parameters are acceptable.
Once the wetting condition water content ($\Theta_w$) is known, then equation (2), shown below, can be solved for ($d$) which represents the depth of water that needs to be applied to the spreading basin for the initial application of water to reach the water table. To arrive at ($d$) the difference between the residual water content ($\Theta_i$) (see table 1) and $\Theta_w$ is multiplied by the thickness of the unsaturated zone.

$$d = (\Theta_w - \Theta_i) \times DTW$$  \hspace{1cm} (2)

Where:

- $d$ = feet of water (vertical measure) needed to be applied at the surface to create a wetting condition (transmissive zone) from ground surface to the water table,
- $\Theta_i$ = residual water content or pre-recharge water content (as a percentage of total bulk volume). (The residual water contents that can be applied with this equation are provided in table 1 below)
- $DTW$ = depth to the water table (feet).

The initial water contents ($\Theta_i$) of typical soils and sediments found in Arizona are provided in table 1 below. Again, these are estimates and are not intended to supersede site specific monitoring.

<table>
<thead>
<tr>
<th>Type of Sediment</th>
<th>No water applied for a year or more</th>
<th>Water applied between a month and a year</th>
<th>Water applied within the last month</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sandy/ Gravelly</td>
<td>.05</td>
<td>.075</td>
<td>.10</td>
</tr>
<tr>
<td>&gt;50% fine-grained sands, silts, and clays</td>
<td>.10</td>
<td>.125</td>
<td>.15</td>
</tr>
</tbody>
</table>

* Bouwer, 1990, personal communication
Finally, the amount of time that it takes for the initial application of water to reach the water table (in order for it to become "recoverable") can be solved using equation (3) as shown below (Wilson, 1989).

\[ t = \frac{d}{r} \]  

(3)

Where:

- \( t \) = the time in days for the initial application of water to reach the water table (in order for it to become "recoverable"),
- \( r \) = rate of vertical infiltration in feet per day; this rate should be calculated from field measurements and should take into account both wet and dry cycles (it should approach hydraulic loading rates),
- \( d \) = as defined above.

Equations (1), (2), and (3) as discussed above are governed by the following limiting assumptions:

* Homogeneous conditions exist between the land surface and the water table;
* infiltration is represented by vertical piston movement (Green and Ampt infiltration);
* dry periods are not so long as to significantly drain the vadose zone below a wetting condition \( (\Theta_w) \);
* Infiltration rates are calculated from field measurements and should take into account both wet and dry cycles (better representations of infiltration rates are determined using long term cycles that approach hydraulic loading rates);
* Antecedent moisture \( (\Theta_i; \) pre-recharge moisture) conditions in the vadose zone are approximated (see table 1).

These assumptions were made to simplify the problem of estimating the time that it would take for water recharged in any given area to reach the water table. Unless otherwise determined by site-specific monitoring, this method will be used to determine when water recharged becomes "recoverable." Once this calculation has been made by the Department a special condition is added to the recovery well permit limiting the recovery of water until the necessary "arrival" conditions have been met.

UNREASONABLE HARM

The Department’s primary focus concerning unreasonable harms lies in two areas. The first is water logging due to lack of subsurface storage space or the inability of stored water to dissipate faster than the application rate. The second area of concern for the Department is the accelerated migration of contaminated or poor quality groundwater which may adversely impact land and other water users.
Arizona Revised Statute Section 45-804.B states that "The director may issue a permit to operate an underground storage and recovery project if the director determines that all of the following apply:...." This section lists several items, one of which states that "the project will not cause unreasonable harm to land and other water users within the area of hydrologic impact of the project."

Overview

Determining unreasonable harm to land and other water users is a very complex and difficult task. The size of the project, source water quality, aquifer characteristics, past land use practices and ambient groundwater conditions all play an important part in determining unreasonable harm to land and other water users. An additional complication arises out of the fact that recharge permits are approved by the Department long before any of the possible impacts of the recharge project are observed. The Department therefore must review and regulate unreasonable harm by applying the following two step methodology.

The first step involves reviewing the initial application, including site layout and design, adjacent land use practices, operational plans, present hydrologic conditions, source water characteristics, impacts of the proposed recharge water and a proposed monitoring plan. This initial review gives the Department the necessary foundation to establish a first approximation of the possible impacts of the Recharge project. If data deficiencies in the hydrologic report are observed they are augmented in the review process. A conceptual model of the project is formulated. The Department is then able to determine if the proposed monitoring plan appears to meet all the legal requirements of the recharge statutes.

Secondly, if the conceptual model of the project is feasible and the proposed project meets all of the initial statutory requirements then the permit is issued with special conditions attached. The implementation of the special conditions (primarily monitoring and reporting requirements) over the life of the permit should verify the accuracy of the conceptual model. The director has the authority to modify the conditions of the permit if monitoring demonstrates that modifications are necessary (ARS Section 45-804.H).

Conceptual Analysis

The analysis concerning unreasonable harm relating to the lack of subsurface storage space is primarily reviewed in the mounding analysis determination. As seen in figures 4 and 6 the volume of water applied and hydraulic conductivity appear to have the greatest impact on the maximum mound height directly under the project. Pilot recharge projects offer a good opportunity to evaluate the response of the aquifer to hydraulic loading rates.
The conceptual analysis of migration was conducted using a particle tracking model PATH3D (Zheng, 1988). Ten PATH3D simulations were made representing a broad range of parameters that might be expected in Arizona recharge projects. PATH3D operates using the head output files from MODFLOW. The model calculates particle movement by advection only. Dispersion, adsorption and diffusion calculations are not employed. The MODFLOW runs that were utilized were single layer runs, subsequently no downward gradient due to leakage into a underlying system was applied to the solution (Perry, 1991). Thirty five particles were tracked in each simulation (See figures 7-9). The originating particle locations ranged from one foot from the basin center to approximately twenty miles from the basin center. The x-axis of figures 7, 8, and 9 list the origin of the generic particle at time zero. Five bars for each particle represent the total distances traveled for any given particle over the five stress periods. The stress periods being one year, five years, ten years, fifteen years, and twenty years. In each set of 5 bars the 5th bar is the bar representing 20 years of migration and is the tallest of the set.

The PATH3D results for the one layer simulations showed no migration in the downward vertical direction (Z) and negligent migration in transverse direction (Y). These results were expected in a single layer simulation symmetrical in the X, Y plane since the particles originated in the same Y location. The figures represent longitudinal (X) migration only. For example, a particle originating near the basin center in figure 7 might migrate approximately 140 feet after 5 years, over 1,100 feet in 10 years, 1600-1700 feet in 15 years and close to 2,100 feet in 20 years while practically no travel is shown for the one year stress period. Figure 7 also shows that particles originating over four miles from the basin center may have little movement strictly due to advective transport from recharge. Although figures 7, 8, and 9 are only first approximations, they do offer some perspective of possible particle migration due to advective transport.

An interesting observation was that when hydraulic conductivity and effective porosity are both low, the resulting particle migration can be relatively great even when the volume recharged is relatively small (figure 7). As expected large volumes of water recharged have a substantial effect on particle migration (figure 9).

Because of the statutory definition of AOHI, referring to the migration of stored water, table 2 (below) provides evidence that the Department is, in fact, encouraging recharge in terms of aquifer cut. Table 2 provides a comparison between a one foot rise in water level and average particle migration after 20 years of recharge. Although this table reflects a wide discrepancy between a calculated average migration of a particle and a calculated one foot rise in water level, it must be noted that particle movement due to the recharge event probably continues long past twenty years and narrows the differences between the distances, even if recharge stops.
PARTICLE MIGRATION 1, 5, 10, 15, 20 YEARS
365 AC-FT / YR. UNCONFINED UNIT 350'
K=3.8 FT./DAY EFFECTIVE POROSITY = .05

AVG MIGRATION - FEET

PARTICLE ORIGIN FROM BASIN CENTER (0) MI

FIGURE 7

PATH 3 D PARTICLE TRACKING MODEL
5 STRESS PERIODS PER PARTICLE ORIGIN
EACH STRESS PERIOD ACCUMULATIVE DISTANCE
PARTICLE MIGRATION 1, 5, 10, 15, 20 YEARS
R.R.1' / DA = 3,650 AC-FT / YR. 1 LAYER
K = 95 FT./DAY EFFECTIVE POROSITY = .10

![Graph of Particle Migration](image)

_PATH 3 D PARTICLE TRACKING MODEL_
5 STRESS PERIODS PER PARTICLE ORIGIN
EACH STRESS PERIOD ACCUMULATIVE DISTANCE

FIGURE 8
PARTICLE MIGRATION 1,5,10,15,20 YEARS
RECHARGE 91,250 AC-FT/YR. 1 LAYER
K=95 FT./DAY EFFECTIVE POROSITY = .10

FIGURE 9

PATH 3 D PARTICLE TRACKING MODEL
5 STRESS PERIODS PER PARTICLE ORIGIN
EACH STRESS PERIOD ACCUMULATIVE DISTANCE
Table 2
Particle Migration Analysis

<table>
<thead>
<tr>
<th>Volume Recharged (Ac-Ft/yr)</th>
<th>Effective Porosity</th>
<th>Hydraulic Conductivity (Ft/Day)</th>
<th>Distance From Basin Center (Ft)</th>
<th>Average Particle Migration From Basin Center (Ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>3,650</td>
<td>.10</td>
<td>38</td>
<td>51,810</td>
<td>5,189</td>
</tr>
<tr>
<td>3,650</td>
<td>.10</td>
<td>19</td>
<td>32,911</td>
<td>5,124</td>
</tr>
<tr>
<td>3,650</td>
<td>.10</td>
<td>95.5</td>
<td>51,810</td>
<td>5,233</td>
</tr>
<tr>
<td>3,650</td>
<td>.20</td>
<td>95.5</td>
<td>37,290</td>
<td>3,596</td>
</tr>
<tr>
<td>3,650</td>
<td>.15</td>
<td>95.5</td>
<td>42,570</td>
<td>4,224</td>
</tr>
<tr>
<td>91,250</td>
<td>.20</td>
<td>95.5</td>
<td>72,307</td>
<td>15,393</td>
</tr>
<tr>
<td>91,250</td>
<td>.10</td>
<td>95.5</td>
<td>&gt;108,570</td>
<td>23,106</td>
</tr>
<tr>
<td>365</td>
<td>.05</td>
<td>3.8</td>
<td>22,321</td>
<td>2,089</td>
</tr>
<tr>
<td>3,650</td>
<td>.05</td>
<td>95.5</td>
<td>92,730</td>
<td>7,473</td>
</tr>
<tr>
<td>3,650*</td>
<td>.10</td>
<td>95.5</td>
<td>24,090</td>
<td>2,222</td>
</tr>
</tbody>
</table>

* - simulated as a 1,650 foot thick unconfined unit, all other simulations conducted as 350 foot thick unconfined units.

The Department is primarily concerned with unreasonable harm due to excessive mounding or accelerated migration of contaminated or poor quality water that would adversely impact land or other water users. The scale of a project, source water quality, aquifer characteristics, past land use practices and ambient groundwater conditions all play an important part in determining unreasonable harm to land and other water users. The Department evaluates the greatest migration and mounding potential when evaluating the maximum area of hydrologic impact during the permit review process. If groundwater conditions and past land use practices reflect possible unreasonable harm scenarios during this process, the Department may opt to review unreasonable harm by using particle tracking models or solute transport models.

SUMMARY AND CONCLUSIONS

The ADWR has defined the three following issues from a technical perspective. They are: (1) the AOHI, (2) the recoverable amount of water that has reached the aquifer, and (3) that the project will not cause unreasonable harm to land or other water users within the AOHI of the project. These issues are inherent in the administration of the recharge program in order to carry out the Departments’ goals.

A sensitivity analysis was conducted for over one-hundred and twenty-five conceptual recharge scenarios. Various parameters such as specific yield, hydraulic conductivity, aquifer thickness,
and one layer versus two layer systems, volume recharged, and
growth of the recharge mound in time, were evaluated. These
parameters were evaluated using both analytical and numerical tools
in order to evaluate variability between model results. Ten
particle tracking simulations were also conducted.

The purpose of the sensitivity analysis was for the Department
to gain additional insight as to ranges and magnitude of results
that can be expected from recharge. The modeling was not site
specific and subsequently not calibrated to any actual physical
system. Even under simple "ideal aquifer" conditions variability
of results was demonstrated.

The Department will continue to review each recharge project
on a site by site basis and employ the most appropriate method for
reviewing the permit application to meet our statutory obligation.
In addition, ADWR will continue to require monitoring and reporting
of recharge projects that are currently in operation to better
characterize on-site conditions.

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DRYWELLS: PRE-TREATMENT DESIGNS AND TECHNOLOGY

Stephen C. DeTommaso
McGuckin Drilling, Inc.
Phoenix, Arizona

INTRODUCTION

With a greater awareness of the need to address the quality of urban stormwater runoff, on-site drainage systems used for disposal have come under closer scrutiny. One such system is the drywell which has been used throughout the United States to dispose of retained or surplus surface water. Past usage of these systems concentrated on rapid disposal with little detailed attention to proper treatment of water. In addition, site design rarely, if ever, considered property usage or prudent system location. Therefore, continued usage has mandated responsible locational guidelines and improved pre-treatment designs to derive the inherent benefits from natural drainage systems.

BACKGROUND

Today's most effective stormwater management systems are based on the philosophy that the natural drainage system is a community's resource that can benefit the environment. Past management practices had been to line natural channels or use closed storm drain systems to move stormwater downstream as quickly as possible. These systems can be very effective in minimizing local flooding, but are very costly and destroy the natural environment. This method can also cause serious downstream problems by increasing peak flow rates and compounding concentrations of pollutant loading. Instead, current stormwater management practices emphasize using the natural environment to the greatest extent possible to reduce the amount of pollutants in stormwater discharges as well as to benefit flood control objectives. Newly developing areas such as those in the Southwestern United States offer the greatest potential for utilizing the full range of structural and management practices to accomplish this goal.
One of the most significant features of local natural drainage systems are detention and retention practices used to reduce the peak rate of discharge from a developed site. Retention functions by storing the excess flow generated by a storm event and then draining the resulting volume over a predetermined time period, usually 36 hours. This time frame meets health concerns, such as those for mosquito-borne encephalitis and allows adequate safety margins for repeat storm events.

Infiltration measures are used to drain retention structures by recharging stormwater into the ground. According to Corbitt (1989), "Infiltration can also decrease the cost of a conventional drainage system, improve water quality, and increase dry-weather stream flows". However, in many areas in the arid Southwest, tightly cemented soils with low permeability characteristics prevent effective surface percolation. In addition, the heavy earth-moving equipment used to create the retention basins compacts the soil into a fairly impermeable formation resulting in stored water that sometimes drains in weeks instead of hours. Therefore, drywells have been used as an effective means to drain the retained water within the required time frame.

PRIOR TECHNOLOGY

A drywell, as defined by Arizona Statute, is "a well which is a bored, drilled or driven shaft or hole whose depth is greater than its width and is designed and constructed specifically for the disposal of stormwater". Structurally, drywells can range from a simple rock-filled hole to the typical sediment-trapping models used in Arizona for over fifteen years. These later designs concentrated more in the removal of sediment and large floating debris and, rapid disposal of retained stormwater.

It should be noted here that these systems were fairly effective in containing large quantities of sediment, oils and grease within their deep settling chambers. Similar sediment traps are currently recommended for use on drainage structures to remove pavement pollutants. For instance, a new 2.2 mile State Department of Highways bridge proposed near Charleston, S.C. will utilize gutter pans to trap pavement runoff from storm events. The pans were specifically designed to remove heavy metals and hydrocarbons before the runoff was allowed to drain into the surrounding coastal waters. By utilizing this method, water quality concerns of Federal and State environmental agencies were satisfied and adjacent fragile oyster and clam beds were protected.
Studies performed on the sediment deposited in deep settling facilities have indicated effective removal of solids, oils and grease. It had been noted by Nix (1988) that solids in runoff from urban surfaces tend to be dominated by particles with relatively high specific gravities and thus, if large enough, are readily settleable. Therefore, by increasing the settling structure depth relative to the point of discharge, higher removal percentages are achieved. In general, the sediment found in the drywell settling chamber is primarily composed of fine sand and silt. Sorbed to the surfaces of these particles are a variety of runoff borne compounds including petroleum based organics. In turn, the oils and grease sequester semivolatile and hydrophobic organics, undissolved phosphates and heavy metals. Trace metals present in urban runoff are of low solubility and movement from deep chambers would be considered to be minimal. In addition, the higher organic matter content present in the oil and grease laden sediment provides relatively high cation exchange capacities resulting in effective accumulation of heavy metals.

During a study of eight "worst case" Tucson area drywells installed in the early 1980's, settling chamber sediment was analyzed for content. The well design included a perforated settling chamber with a free-draining bottom. A single permeable geotextile fabric layer separated the deposited sediment from the washed rock backfill below. At one of the locations, a multi-family residential site, a high of 2900 mg/kg of oil and grease was measured in the sediment. An adjacent soil profile 80' deep showed the highest level of oil and grease was 2 mg/kg. Therefore, containment in deep settling chambers or traps has been found to be effective methodology for removing a high percentage of many constituents present in urban stormwater runoff.

IMPROVEMENTS REQUIRED

The same studies analyzing sediment content also indicated that improvements in existing designs were necessary. Again, the primary purpose of most drywell designs was to maximize the rate of disposal of retained water. Although system design rates were calculated based on draining water within 36 hours, there was no means to provide an internal constant rate of flow except for the permeability of the surrounding soil. Since the peak rate of storm flows usually far exceeded the design rate of the drywell system, surging of these flows during the storm could reduce the effectiveness of the deep chamber. Also, petrochemical hydrocarbons floating on the water's surface could push past the overflow level, down to the gravel pack of the well.
Certain volatile organic compounds emanating from vehicle fuels and asphalt pavement, such as toluene and ethylbenzene, also tend to be mobile while in solution. Although volatilization during surface transport and in the settling chamber are effective mechanisms in reducing relative concentrations, additional treatment will enhance the performance of on-site drainage systems.

STRUCTURAL IMPROVEMENTS

To provide for improved pre-treatment of flow, a control structure has been incorporated into current designs. This structure acts as a primary interceptor to receive initial surface flows through a grated inlet. Unlike earlier designs, this chamber is non-perforated to eliminate seepage of compounds associated with "first flush" flows. The first few minutes of runoff often represents a shock load to the receiving drainage system in terms of potential pollutants for many constituents. For very fine particles and solubles this occurs very soon after the storm begins and much sooner than the peak flow. Therefore, enhanced containment of these "first flush" constituents in the interceptor prior to the surge from the peak flow will result in improved water quality.

While prior designs incorporated an open mesh screen at the point of overflow, as noted earlier, the screen did not prohibit the movement of floating petrochemicals down to the gravel pack below. Therefore, current designs utilize an outer shielding device to trap these organic compounds within the chamber. The shield is equipped with a filtered vent to prevent siphoning of water after inflow ceases. An internal screen traps fine suspended debris.

An orifice reduction provides controlled flow from the interceptor to the drywell. By decreasing the orifice diameter, more residence time will be provided in the interceptor and the retention basin and, for a fixed storage capacity, result in higher removal percentages. This controlled flow, approximately 0.20 cfs, also prevents overload of the drywell settling chamber from peak storm flows. In many prior design installations, soil-permeability had generated system flow rates in excess of 3.0 cfs. Therefore, overall performance is enhanced with the new design since overload is not only a water quality consideration, but a primary cause of system aging and failure as well.

The drywell section of this system has a solid, vented cover and only receives inflow from the interceptor connecting pipe. The present design incorporates the same shielding assembly used on
the overflow pipe in the interceptor to trap residual constituents. The deep settling chamber is retained and now provides for efficient treatment of inflow under controlled flow rates.

The drywell chamber is also non-perforated and has a solid bottom to prevent seepage of trapped constituents. A solid drainage pipe is now used to carry water to the well screen below. Prior designs had incorporated a perforated pipe which caused a downward migration of fines and chamber constituents into the well screen as water receded through the rock backfill. To aid in natural soil absorption, the size of the rock backfill used in the current design has been reduced from 1½" to ¼" minus. Approximate bedding depth around the well screen is 24".

Figure 1: MaxWell® Plus System
ABSORBENT TECHNOLOGY

In addition to structural controls that provide improved pretreatment of flow, new and unique water purification processes have recently become available. One of the most effective has been the manufacture of a new breed of absorbent materials. These recently developed compounds are effective for absorbing a wide range of organic materials from transportation fluids to aromatic solvents.

One of the most successful of these products is an absorbent called Imbiber Beads. These beads are tiny plastic spheres composed of lightly crosslinked methyl acrylate polymer chains. Because of the crosslinking, the beads display markedly unusual behavior with organic solvents. Most common plastics dissolve readily in an appropriate solvent. Imbiber Beads, however, do not dissolve but instead swell and expand up to 2700%.

The absorbent beads have several significant features that distinguish them over any other "sorbents" currently available. First, the beads are hydrophobic and will not even partially absorb water or brine solutions. As a result, their full effectiveness can be realized even when fully submerged in water. Second, organic liquids are "captured" by the beads and absorbed so that the entire mass of the polymer is effective rather than the surface only. There is no competition for "active sites" as in adsorption processes.

Additionally, liquids are entrapped in the molecular network and cannot be squeezed from the beads. A fully imbibed bead can be cut in half and no "contained" fluid will escape. The fluids imbibed by the beads span a wide range of organic materials including:

- Transportation fluids such as gasoline; No. 1, 2 and 3 fuel oils; jet fuels; and diesel fuels.
- Chlorinated solvents like carbon tetrachloride, methyl chloroform, trichlorobenzene and PCB's.
- Aromatic solvents such as benzene, toluene, xylene, ethylbenzene, cumene, styrene and methylnaphthalene.
- Many polar compounds, including methylisobutyl ketone, tetrahydrofuran and ethyl acrylate.

In addition, as the beads increase in volume by imbibation of organic fluid, the affinity for many other organics will increase. This is because the organic absorbed in the bead will act as a co-sorbent for other organics. Many organic pesticides are prepared with a xylene base and react accordingly to be effectively extracted from water as well.
In recently installed drainage systems, the beads are contained in floating blankets composed of a sturdy, porous polyolefin fabric. An oleophilic wicking agent with a high surface area is mixed with the beads. This material holds the beads apart, creates open channels for fluid flow and allows spaces for Imbiber Bead expansion. The wicking agent also tends to coalesce suspended microdroplets of organic fluid on its surface and directs the fluid to the beads for absorption.

Current drainage systems contain approximately 880 square inches of blanket surface area in each settling chamber. Each blanket has a capacity of over 6 quarts of fluid and can remain submerged in water indefinitely. When used in conjunction with the new containment structures in the drainage system settling chambers, the Imbiber blankets react instantly to remove runoff constituents on contact. They will also continue to wick residual compounds remaining in the settling chamber after inflow ceases even absorbing "rainbow" sheens 1 molecule thick.

INDUSTRIAL DRAINAGE

One of the most challenging site drainage problems today is proper treatment of runoff from industrial areas where organic materials are used, handled or stored. This has been an area for concern that has demanded attention from industry, regulators and the environmental community. The problem is one of a need for prudent site engineering as well as flow management of surface runoff.

In older installations, drainage from such handling areas was treated as just another normal contributory area that added to the total runoff of a site. Current awareness as to the potential for improper handling and drainage of chemicals used in these areas dictates such areas be isolated and treated separate from all other site drainage. Once separated, the flows can be directed to a holding tank or retention facility lined with an impervious polymer membrane to prevent seepage. Unfortunately, any chemical or physical barrier will also hold back water. Thus, some means must be provided to allow water, but not chemical compounds, to escape.

Now, gravity drainage systems incorporating Imbiber Beads have been utilized to process and dispose of stored surface water from such industrial areas and, for example, service stations. Under normal runoff conditions, water will flow through the system and drain without restriction. If a leak or massive spill occurs within the contributory drainage area, the beads will be activated and instantly swell to seal the unit against any further flow of product and/or water.
Industrial drainage systems such as the Envibro® System are typically installed in a retention basin where the velocity of inflow is reduced and large solids are deposited. As water builds up in the basin, runoff spills over into a primary collector chamber with up to 1000 gallons of effective capacity. Trash, leaves and floating debris are retained in an easily accessible, deep debris basket at the inlet grate. Oils and other petroleum products are retained in this chamber utilizing the American Petroleum Institute General Standard governing the separation of petroleum from water by gravity differentials. Sludge or heavy solids settle to the bottom for easy removal.

The second step in this process begins in the collector chamber as influent is directed through a screened overflow pipe to a stationary tank. A flow regulator in the tank's first compartment slows internal velocity to enhance separation of any residual fluids and solids. A floating Imbiber blanket aids in wicking petrochemical and organic products from the water.

The next compartment filters fine particles and silts through a high capacity fabric mesh. The typical system provides over 1800 square inches of filter surface area. Larger capacities are available for more demanding drainage requirements. A service opening above the tank's compartment permits periodic removal of retained liquids or fines.

Final processing is achieved in the third compartment through a unique drain field utilizing a bed of Imbiber Beads. Under normal operations, water passes through the drain field. However, when contacted by an organic liquid, the beads instantly begin to absorb the liquid and expand. Because of their spherical structure, a packed bed of beads in the drain field has approximately 30% void space available for fluid flow. Upon contact by an organic liquid, the beads will swell 27 times their volume, rapidly filling the void space causing the drain to function as a valve to prevent further flow. The rate of flow through a gravity unit will depend upon the depth of the bead bed and head of water. The system can also accommodate a submersible pump to provide uniform flow rates under a variety of conditions.

The fully treated water is finally discharged into a drywell specifically constructed to provide controlled flow and to allow for inspection. The inspection chamber is non-perforated and has been sealed to prevent lateral seepage.

Under normal operating conditions, this industrial drainage system requires limited maintenance. The filter assembly and drain field are easily accessible and can be cleansed by
Figure 2: Envibro® Drainage System
FIGURE 3 - IMBIBER FLOW RATES THROUGH GRAVITY DRAINS
removing them from the tank and simply hosing them down to remove silt and normal fine debris. This cleaning procedure would be recommended following heavy rain-falls or high-loading of the system. Should a spill or leak occur, replacement of activated Imbiber Beads is normally required. For normal operation the beads can be expected to have a long life.

Such drainage systems used in industrial applications will prevent the flow of organic liquids under a broad range of conditions. No chemical or external mechanical actions which can fail in the "open" position are necessary. The physical swelling of the Imbiber Beads in the drain field is sufficient to stop flow. This system has State approval from the Arizona Department of Environmental Quality as control technology for problem industrial drainage areas.

SUMMARY

On-site drainage and infiltration systems, such as drywells, have played an important role in numerous municipal stormwater management plans. Early designs of these systems, which in their day were viewed as state-of-the-art, were effective in quickly disposing of retained water. However these designs required improvement to further mitigate the impact of water-borne constituents and meet today's standards.

As a result of current research and empirical data, specific areas for drainage system improvement can be identified. Prudent site engineering and locational standards should be an integral part of any drainage plan. Increased structural controls to provide enhanced containment of sediment and management of flow rates will improve water quality characteristics and system performance as well. Recent developments such as advanced absorbent technology now provide new potential for cost-effective on-site treatment systems. As the dynamic relationship between stormwater management, hydrogeology and new technology are explored, additional advances will be realized that will benefit both flood control and utilization of stormwater as an invaluable resource.
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TWENTY-FIRST CENTURY METHODS OF CLEANING DEEP RECHARGE LAKES

James A. Goodrich and Allan D. Flowers
Orange County Water District, Fountain Valley, California

ABSTRACT

Every year, the Orange County Water District percolates between 180,000 and 300,000 acre-feet of water diverted from the Santa Ana River and imported sources into deep and shallow lakes on 1,600 acres of land. Demands continue to grow on the groundwater basin while Santa Ana River flow increases due to upstream development. Land availability limits further expansion of recharge facilities. To meet the demands on the groundwater basin, the District initiated a program in 1987 to enhance the percolation capacity of the existing recharge facilities, particularly the deep recharge lakes. Initial work by the District's Bioresource Department focused on the nature of deep lake clogging and concluded that a combination of microorganisms and fine-grained sediment is responsible for rapid clogging in the uppermost few centimeters of the lake bottom sediments. This conclusion led the Operations Department to experiment with new methods of cleaning the clogged sediments. Initial operational research successfully applied dredging and sand washing technologies, which are being combined into a prototype cleaning mechanism which operates in a full lake. The mechanism separates sand from silt and clay, redeposits the sand, and sends the fine-grained material to dewatering ponds for subsequent disposal. This new technology not only maintains higher average percolation rates and keeps the recharge lakes in service longer, but greatly reduces material handling and operational costs.

INTRODUCTION

The Orange County Water District (OCWD) was formed by an act of the California State Legislature in 1933 to protect the water rights to the Santa Ana River for Orange County area landowners. In 1955, the District's enabling legislation was modified to expand OCWD's groundwater management role. Today, OCWD manages one of the largest urban groundwater recharge systems in the world. Its scope of responsibility covers groundwater replenishment, wastewater reclamation, sea water intrusion control, and water quality management.

The Orange County Groundwater Basin covers an area of about 330 square miles and lies within the larger Southern California
Coastal Plain (Figure 1). The California Department of Water Resources estimated that 40 million acre-feet of fresh water are stored in the alluvial sediments of the Orange County Groundwater Basin, of which about 1 million acre-feet are usable. Currently, over 500 wells operate in the District, of which about 200 are large municipal wells. In 1989, these wells extract about 275,000 acre-feet from the basin.

![Map of Orange County Water District](https://example.com/map.png)

Figure 1. Orange County Groundwater Basin is located in the eastern portion of the Southern California Coastal Plain.

**The Santa Ana River**

The Santa Ana River is the primary source of recharge for the groundwater basin. It drains an area of over 2,780 square miles and is the largest perennial stream in southern California. The natural safe yield of the basin is about 60,000 acre-feet per year. However, a combination of increased base flow in the river due to upstream urbanization and the development of large-scale artificial recharge systems along the Santa Ana River by OCWD have increased the yield of the basin to over 200,000 acre-feet per year. The annual base flow in the river has increased from about 40,000 acre-feet prior to any significant upstream development to over 120,000 acre-feet today. Continued upstream development will increase base flow to over 200,000 acre-feet per year by the year 2010, as illustrated on Figure 2. Figure 2 also shows the change in the source of base flow from natural runoff from inland mountains prior
Figure 3. OCWD recharge facilities cover an area of 1,600 along the Santa Ana River.

Figure 4. Aerial view of the Santa Ana River with typical "T" sand levees and adjacent deep recharge lakes.
Figure 2. Natural base flow in the Santa Ana River has steadily increased and replaced by waters of wastewater origin.

to 1940 to primarily waters of wastewater origin in recent years. It is the increase in wastewater discharges into the Santa Ana River upstream of Orange County which are causing the steady increase in base flow in the river.

In recent years, storm flow in the Santa Ana River varied dramatically from less than 1,000 acre-feet per year (1960-61) to over 400,000 acre-feet per year (1979-80). Unfortunately, in the semi-arid environment of southern California, rainfall tends to come in short-duration, high-intensity storms which creates flash-like runoff events; this makes capturing storm flow for recharge challenging.

OCWD Recharge Facilities

OCWD groundwater replenishment facilities are located along the Santa Ana River, as shown on Figure 3. Of the District's 1,600 acres of land along and in the river, about 1,000 acres are covered by water. The system includes about six miles of river (380 acres), seven deep lakes (547 acres), and four shallow lakes (87 acres). An aerial photograph of typical deep lakes and the river is shown on Figure 4. The river system contains temporary "T" and "L" sand levees which distributes the flow across the entire river bed to allow more efficient percolation; after these levees wash out during major storms, OCWD must reconstruct them. Water from the river or imported sources is diverted into the deep and shallow
lakes, which have a combined storage capacity of over 26,000 acre-feet.

Several constraints must be realized to successfully capture storm and base flows in the Santa Ana River in the future. The first constraint is that base flow in the Santa Ana River is steadily increasing. The second constraint is that land on which to build new recharge facilities is virtually nonexistent. Finally, the demand for groundwater is increasing proportionally to the rapid growth in southern California. To capture the increasing Santa Ana River flows requires that the District optimize the recharge capacity in each of its existing lakes. To accomplish this, the District has developed innovative new methods to maintain higher average percolation rates in its deep and shallow lakes. The operational methods developed over the past several years and those being planned for implementation in the next few years will be discussed in the following pages.

PERCOLATION RATE REDUCTION

High nutrient and sediment loads in the Santa Ana River can cause rapid percolation rate reductions in the shallow and deep recharge lakes operated by the District. Because the base flow in the Santa Ana River is primarily of wastewater origin, chemical constituents, such as total nitrogen and phosphates, are generally high. Total nitrogen ranges from 6 to 10 mg/L and phosphate ranges from 2 to 4 mg/L. The sediment load in the river varies from 10 mg/L to over 1,500 mg/L, depending on the flow conditions. Flows during and immediately after storms will yield the higher sediment loads, whereas, during summer low flow conditions, the average sediment load ranges from about 25 mg/L to 45 mg/L. If water is stored for any length of time behind the upstream flood control reservoir, sediment will fall out of suspension, yielding a sediment load as low as 10 mg/L at the District's diversion points.

The "once a year" line on Figure 5 illustrates typical percolation rate reduction for one of the Districts deep lakes. After cleaning and during filling, the 73-acre lake could percolate about 115 cubic feet per second (cfs) (about 3 feet per day per acre (ft/d/ac)) for several weeks. Once filled, percolation rates drop to about 40 cfs (1 ft/d/ac). From that point, the percolation rate decreases linearly to near zero over the course of the year.

In 1987, the District undertook a broad technical investigation to accurately determine: 1) the cause of percolation reduction in deep recharge lakes, and 2) identify methods to prolong high percolation rates. The study team consisted of in-house specialists in microbiology, hydrology, geology, and operations, as well as outside experts in recharge hydraulics, facility operations, and limnology.
Figure 5. Percolation rates steadily decrease due to clogging; twice per year cleaning increases water recharged by 40%.

Traditional thinking on percolation rate reduction in deep recharge lakes suggested that sediment falling out of suspension was the primary cause of clogging in lake bottoms, with secondary consideration given to accumulation of microorganisms. However, initial research at District facilities indicated a much stronger clogging effect by microorganisms than previously believed.

Research conducted by the District's study team found that the high nutrient load in the river's base flow provides an environment which encourages rapid growth of microorganisms. Diatoms and bacteria, and to a lesser extent, algae, were found to be the primary organisms contributing to lake bottom clogging. However, this research shed new light on the interrelationship between microorganism and suspended inorganic materials and their impact on the clogging of deep recharge lakes. The following paragraphs summarize some of the findings of the research team. A detailed discussion on the dynamics and kinetics of this interrelationship is provided by Gordon, et al. (1991).

Diatoms are probably the most important organism effecting clogging because they can block the passage of water between sediment grains when they are either alive or dead. They grow on the lake bottom, feeding on the abundant nutrient supply available in the lake water. In addition, they grow and reproduce in the
presence of light, generally when the water depth over the bottom is less than 20 or 30 feet; they do not survive in the deeper parts of the District's lakes because of insufficient light.

One of the most important factors about diatoms that impact clogging is the biopolymer they secrete in order to move from grain to grain. As shown Figure 6, these secretions form a netting between sand grains which effectively capture inorganic (silt and clay) and organic (bacteria, algae, etc.) materials as they settle out of the water column. Because diatoms reproduce quickly, the biopolymer net that they produce is established within a few days of initiating basin filling. This is probably the reason that percolation rates quickly fall from over 10 ft/d/ac to about 3 ft/d/ac in the first few weeks. As the basin continues to fill, the increased pressure head on the bottom materials maintains can overcome the initial clogging, but as material accumulates on the bottom, percolation rates steadily decline.

Figure 6. Electron Micrograph showing diatoms and the biopolymer secretions they use to move from particle to particle.

One positive aspects of the development of the biopolymer net is that potentially clogging materials are trapped in the upper few inches of the lake bottom. Without the biopolymers, the clogging particles could easily migrate much deeper into the sediment column, making cleaning operations difficult and eventually leading to irreversible percolation reduction. This important finding lead the research team to experiment with new methods maintaining and operating recharge lakes.
Figure 7. a) Dozers & scrapers cleaning deep lake. b) Sand wash plant separating sand and silt. c) 200 cfs pump station used for rapid draining of deep lake. d) Dredge used to clean sedimentation basins.
EVOLUTION OF DEEP LAKE CLEANING METHODS

The methods used to clean and operate the deep recharge lakes at the District have evolved over the past two decades. Originally, lakes were drained occasionally and bulldozers were used to scarify the upper foot of lake bottom; material was not removed. This procedure was later discovered to have long-term degrading effects on percolation rates because fine-grained materials (clay, silt, diatom casts, etc.) were moved deeper and deeper into the soil column, which eventually lowers the vertical hydraulic conductivity in the upper several feet of the soil column beneath the lake bed.

Upon realizing the long-term impacts caused by the original cleaning methods, the district used dozers and scrapers (Figure 7a) to remove the upper 6 to 12 inches of soil from the lake bottom. Though this method worked well in stopping the slow degradation of percolation rates in the deep lakes, two new problems arose. First, the areal dimensions of the basins were slowly growing as material was removed. Second, disposal of the removed material was difficult and time consuming.

Further research into the composition of the clogging layer on the lake bottoms revealed that only about the upper 3 inches of the soil column contained clogging material and that this material only accounted for about 5% of the total bulk volume of sediment in that 3-inch column. This discovery led to the next evolution of basin maintenance: the removed material could be separated by grain size using a standard sand wash plant (Figure 7b). The finer-grained material, consisting of only about 5% of the total volume, could be stockpiled and discarded more easily. The remaining 95% of the material removed could then be placed back into the lake during the next draining and cleaning cycle.

The new cleaning methods, by which clogging material is removed, though effective for restoring percolation rates, take longer to complete than earlier methods. Most of the time required to complete a dewatering, cleaning, and filling cycle was spent during the dewatering period. Only on of the District's deep recharge lakes had permanent pumpout facilities, and this system took well over a month to drain a lake. Other lakes required portable pumpout systems, which took even longer to drain a lake. To mitigate this problem, the District designed and constructed high capacity dewatering systems (Figure 7c) that can drain a deep lake in 6 to 10 days. These new facilities have shortened the cleaning cycle from 2 months to about 2 weeks. By being able to drain and clean the lakes more quickly, they can be cleaned at least 2 times per year. This capability has resulted in recharge lakes that are in service longer during the year and have higher average percolation rates; Figure 5 illustrates that with twice per year cleaning, total recharge for one of the deep lakes has increased over 40% per year.
SUSPENDED SEDIMENT CONTROL

Because the majority of the fine-grained clogging material consists of suspended sediment introduced from the river, controlling its concentration would lead to a further reduction in material handling. The District has several large, shallow basins immediately downstream of the river diversion that are dedicated as sedimentation basins. To accelerate the sedimentation process, an alum-based liquid polymer is injected at the diversion gates, where the water is turbulent to aid mixing. This sedimentation process works most efficiently with flows containing high sediment load (e.g., greater than 100 mg/L). As the sediment load drops below 25 mg/L, the effectiveness of adding polymer is significantly reduced. Using this method, the suspended sediment load of the river water is reduced to about 10 mg/L by the time the water reaches the deep lakes.

Though the use of polymers and sedimentation basins worked well to remove suspended sediments from the river water, the method caused an operational problem whereby fine-grained inorganic and organic materials quickly accumulated in the basins. To remove this material, the basins had to be drained. While the sedimentation basins were down for cleaning, all downstream deep basins were deprived of river water for percolation.

Removing the settled material became a logistical nightmare for the District's heavy equipment. Dozers, including swamp cats, became easily bogged down in the mud and, because of the high water content of the material, the dozer blades could not push the wet material very far. As a result, the cleaning process took several months, during which time, water could not diverted to lakes for recharge.

To solve the logistical problem of removing fine-grained materials from the sedimentation basins, the District purchased a portable dredge (Figure 7d). The dredge overcame both problems by: 1) operating in full basins so that river water could be diverted, uninterrupted, to lakes during dredging operations, and 2) keeping the basins free of fine-grained material so that heavy equipment would not be necessary for cleaning. During dredging operations, silt is pumped by the dredge to nearby shallow dewatering ponds. Once dry, the silt and clay can be easily handled by conventional heavy equipment for disposal.

FUTURE DEEP LAKE CLEANING METHODS

Realizing two problems with the deep lake recharge operations provide an opportunity to further evolve effectiveness of the cleaning process. First, percolation rates continue to decline, as shown of Figure 5, even with twice per year cleaning. Ideally, cleaning should be done monthly to maintain highest percolation rates. Secondly, when cleaning operations do occur, draining the lakes is expensive, and during the cleaning process, the lakes are
unavailable for recharge activities.

A solution conceived by the District's operations staff was to design and construct a permanent cleaning device on the bottom of the lake with the active mechanisms patterned after the District's portable dredge and sand wash plant. One variation of the system concept is illustrated on Figure 8. In this concept, several enlarged cutter heads, based on those used by the dredge, are attached to a long boom that pivots around a center hub. The upper 1 to 2 inches of material would be gently vacuumed off the lake bottom, after which, coarse- and fine-grained material would be separated by the sand wash system mounted on the cutter head chassis, with the coarse-grained material redeposited behind the cutter head and the fine-grained material sent to shore for drying and disposal. The cutters, submersible pump, and sand wash system would be hydraulically driven, using vegetable oil as the hydraulic fluid to avoid potential contamination of the recharge water.

This type of cleaning system has several deficiencies. First, if the system is as large as shown on Figure 8, it must be permanently installed in the basin, which means that many units, at considerable capital and O&M cost, will be required in the District's larger basins. In addition, permanently installed units, as described above, would miss areas of the basin because of their circular cleaning pattern. Small units could be used that could be moved around the lake. However, at a size that would be efficient, they would be difficult to move from place to place.

The ideal cleaning system to mitigate these problems would be a computer-guided, free-moving tracked vehicle, containing a cutter head, sand wash system, and discharge pump, that would move around the lake bottom in a pre-programmed manner. In other words, it would resemble a giant pool sweep. Systems similar to this were envisioned in the 1960s for sea floor mining (e.g., magnesium nodules). Currently, the automated control technology for this type of vehicle is not readily available to the public. If the District follows this line of development, the first prototypes would probably be manually controlled and operated.

The District is currently in the conceptual planning stages of developing the in-situ lake bottom cleaning devices discussed above. Prototypes are planned for construction and testing later in 1991. A full scale system could be operational by 1993.

CONCLUSIONS

Accumulation of fine-grained clastic material and biomass rapidly degrades the percolation rates in deep recharge lakes. Using traditional periodic cleaning methods to remove these clogging agents helps maintain higher average percolation rates, but the costs associated with recharge lake draining are high. Using existing allied technologies, new in-situ cleaning mechanisms are under development at OCWD which will maintain peak percolation
RENEWABLE URBAN WATER SUPPLIES
NOGALES AND THE MICROBASINS OF THE SANTA CRUZ RIVER
A CASE OF NATURAL WATER BANKING

Leonard C. Halpenny
Philip C. Halpenny
Water Development Corporation
Tucson, Arizona

ABSTRACT

A series of four sub-basins extend along the Santa Cruz River from the border. Low-permeability formations surround the shoestring aquifer of alluvial deposits. In places the bedrock intrudes into the aquifer, forming pockets of alluvium (at one bedrock boundary (Guevavi Narrows) flow is perennial, with an associated riparian habitat). These cells are periodically recharged by river flow, and in periods of no flow they constitute storage reservoirs which can be utilized as water supply sources. On a statistical basis these pocket basins constitute a reliable supply. The recharge characteristics of these basins allow them to refill quickly when flow occurs.

The City of Nogales has partially utilized one of these microbasins as a main source of supply from 1914. Further utilization of these basins can provide an increased supply to accommodate future urban growth while constituting a case of "safe-yield".

The alternative which may be considered by the City is an injection–recharge wellfield, possibly coupled with an "exchange" provision under the Augmentation Authority which would allow groundwater mining to satisfy safe yield requirements.

INTRODUCTION

In 1691 Father Kino established the first mission in Arizona at a site on the upper Santa Cruz River at a place known as Ku Vaxia, "Big Spring" or "Big Water". The site was chosen because the water flowed at that place even when the river was dry in other reaches, and as a result diversion dams had been built and irrigated fields downstream supported a large population. Kino chose the site both because of the dependability of the water supply and the number of potential converts in the area, but they were there because of the water.
We had known about the mission for many years out of interest in the history of Arizona, but we little imagined in October of 1989 that the hydrogeological character of the mission was to lead us into an extensive discussion with the Arizona Department of Water Resources about what constitutes "water" in Arizona, and incidentally led us to identify an interesting system of natural recharge which will constitute a renewable source of water for the future growth of Nogales.

In 1963 Water Development Corporation was retained by the International Boundary and Water Commission to identify areas from which the City of Nogales could derive a future water supply. A number of possibilities were developed, but for various reasons none were utilized, and with the passage of time these possible sites became unusable—for example, one suggestion was to develop a well field on the site where the current wastewater treatment plant is now located.

In 1989, on behalf of a client who owned a large ranch based on the Santa Cruz (Guevavi Ranch), negotiations were entered into with the City to develop the bottomland of the ranch as a future water supply for the City (see location map, Figure 1, and map of ranch, Figure 2). The negotiations also included the Department of Water Resources (DWR) and the Tucson Active Management Area (TAMA) office, because the City wanted to obtain as part of the acquisition, an assured water supply under the provisions of the Groundwater Management Act of 1980. The City was also under pressure to decide whether to contract for the CAP allocation they had been given, which was economically unusable because of pipeline costs. That debate was whether the Nogales CAP allocation could be "traded" in some form to acquire an Assured status even if the physical water could not be used (for general background, see Arizona Department of Water Resources, 1989, Santa Cruz County Water Issues Report: TAMA).

The topic today therefore is partly physical hydrology with a concern for "wet" water and legal hydrology which concerns "dry" water, meeting the paper requirements of the Groundwater Management Act of 1980.

The problem simply put by TAMA was that a Certificate (Designation) of Assured Supply could be granted only on the basis of utilization of "groundwater", since DWR had no position to rule on surface waters. However, the major policy goal of DWR is "safe yield" by 2025, which means that groundwater extraction is allowable only if replenishment of the aquifer takes place, presumably by recharge of water from outside the basin. Alternatively, groundwater mining can be replaced with utilization of augmented water, either from a surface or groundwater source from outside the area.

As a reliable surface water source the Santa Cruz has a major problem: it is often dry.
Figure 1. Location map of microbasins on the upper Santa Cruz River.
Source: D.G.E.T.N., Secretaría de Planeación y Presupuesto 1979
Sheet: Nogales H12B31
Scale: 1 : 50,000 (metric)
Therefore, right at the beginning of the negotiations we were put on the spot. Both the City and TAMA said "Show us" and in demonstrating why we thought the Santa Cruz could solve "wet" and "dry" problems we found some interesting aspects which made it a special case of recharge. We started with one fact we did know—the City had relied mainly upon the Santa Cruz from 1914 until the Meadow Hills wellfield was developed in the 1970's.

"WET" WATER

Geology

Frank Simons of the Geological Survey produced a geological map of the area in 1974 (see Figure 2). Beginning at the border both sides of the river are dominated by the hydrologically infamous Nogales Formation, which can be considered hydrologic bedrock. An east-west fault intrudes on the west side at the Patagonia Highway (Highway 82) bridge, north of which is a block of quartz monzonite. This block constrains the west (south) bank of the river until it breaks out into the larger alluvial valley at Calabasas (Rio Rico).

On the east side the map shows outcrops of Nogales Formation along the river distributed in Older Alluvium. In 1989 we supervised construction of a 500 foot well at the west end of the Nogales Airport and encountered Nogales Formation throughout the section. There is a northeast-trending fault along the lines of Burro and Guebabi Canyons. In 1982 we did extensive drilling at the head of Guebabi Wash and identified an aquifer of reasonable productivity, so it is possible that the Older Alluvium north and east of the northeast fault may be productive. Along the river however, Nogales Formation again dominates just north of Guebabi Wash and constrains the river until it breaks out at Calabasas.

The investigative work in 1963 consisted of having Walt Heinrichs run geophysical lines across the river at various points, followed up by test drilling. The conclusions then were that the valley of the river consisted of alluvial basins on the order of 100 feet deep, enveloped and floored by hydrologic bedrock. The basins were separated one from another by the outcrops of Nogales Formation on the east side which, associated with shallow bedrock at each location, constrained hydraulic conductivity between the basins and made them each semi-separate. In the 1989-1990 discussions we preferred to use the term "microbasin" rather than "sub-basin" to describe these, because separate sub-basins as used in DWR terminology now can have full hydraulic connectivity as for example the Upper Santa Cruz and Lower Santa Cruz Sub-basins. Four of these microbasins can be isolated: (1) Buena Vista, from the border to the outcrop at the border of the Maria Santissima Del Carmen Grant; (2) Yerba Buena (Kino Springs), which extends to the narrows just south of the Highway Bridge; (3) the "Highway 82" segment, to use Frank Putnam's terminology; and (4) Guevavi, which begins at the outcrop (not shown on the Simons map) at the Mission and extends to Eagan Narrows, just downstream from the Pendleton-River Road Bridge (compare Figure 1, the location map, with Figure 3, the geologic map).
Figure 3. Portion of geologic map showing hydrologic bedrock units.

All of these constrictions have been considered as damsites in the past. The Corps of Engineers considered Guevavi and Eagan Narrows—luckily for the survival of the mission dams were not built. Yerba Buena was planned as an agricultural water-supply dam in 1893 by George Westinghouse who envisioned a private real estate development along the river. We also recommended Yerba Buena as a damsite to the Boundary Commission in 1963.

In the litigious arena of 1990 dams are out—a flood of downstream protests would result.

Storage

The investigation in 1963 determined that the alluvial fill in the microbasins is highly permeable. Water in storage is easily extracted (coefficients of storage conservatively estimated and accepted by DWR of 17--20 percent) by wells and conversely, recharge from flow in the river is rapid and voluminous. The 1963 report contained a hydrograph of water levels at the City of Nogales Pumping Plant at the Highway 82 Bridge, which shows the relation between streamflow at the border and the depth to water table at the gallery (Figure 4). Small spurts of runoff refill the local microbasin rapidly.

The quantities in storage reflect that the basins become larger in the downstream direction.

<table>
<thead>
<tr>
<th>Microbasin</th>
<th>Recoverable Water in Storage (acre-feet)</th>
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<tbody>
<tr>
<td>Buena Vista</td>
<td>2,570</td>
</tr>
<tr>
<td>Yerba Buena (Kino Springs)</td>
<td>3,515</td>
</tr>
<tr>
<td>Highway 82</td>
<td>7,085</td>
</tr>
<tr>
<td>Guevavi</td>
<td>12,800</td>
</tr>
<tr>
<td>Total</td>
<td>26,140</td>
</tr>
</tbody>
</table>

Source: Table 1a, Putnam et. al. unnumbered page. Guevavi: Water Development Corporation

Reliability

At this point in the dialogue we had established that there was water in storage in the alluvium. However, these microbasins did not constitute major aquifers with vast amounts of groundwater in storage which neither could be mined for "wet" water for 100 years nor analyzed for 100 years of withdrawal using variants of the Theis equation, as is required in demonstrating proof of a 100-year supply under DWR regulations. Dams were out as a method of storing variable inflow, but what existed were natural semi-dams filled with a highly permeable aquifer—we decided these microbasins could function as dams with the added advantage that there was no evaporation from the water in storage.
Figure 3. — Relation between depth to water table at City of Nogales Pumping Plant and discharge of Santa Cruz River.

Figure 4. Hydrograph from 1963 Water Development Corporation report.
We were however faced with a very embarrassing fact. The investigation in 1963 had been sparked by a massive water supply crisis. The City Pumping Plant on the river at the Highway Bridge had gone dry. It was necessary to evaluate the question of the reliability of a supply stored in the alluvium of the river.

The Pumping Plant had been constructed in 1914 as a French Drain at 54 feet below the bed of the river, with a 12-inch pipeline running 6 miles into the city. One of the characteristics of the bedrock constraints of the microbasins is that underflow is brought to the surface at each constriction so the river flows at these sites while the centers of the basins may be dry. Presumably the Pumping Plant was constructed at that site because "that is where the water is" and it was, appropriately, the site of diversion dams for irrigation in that microbasin. The appropriate place for a wellfield would have been in the center of the microbasin, not at the bedrock narrows.

The immediate response to the crisis was to add another level to the Drain, at 94 feet. The next step was to drill wells. The problem was that the site was extremely limited so only two wells could be drilled on the property. To avoid well interference problems it was necessary to lease a well site from Stewart Granger, who owned Yerba Buena.

The problem of the Pumping Plant site is clear: "Historically, the City of Nogales has had severe problems meeting its water needs when the depletion of water in the Younger Alluvium of the Highway 82 pocket exceeds 3000 acre-feet [citation of the Halpenny 1964 (sic. = 1963) report] in spite of the fact that an additional 4000 acre-feet remains in storage in the sub-area (Putnam, et. al. p. 86, emphasis added)." Putnam went on to determine total demand in that microbasin for 1983 as 3328 acre-feet "which, given the water in storage in this sub-area of about 7085 acre-feet, gives about a two year supply relying on water in storage alone. This period assumes no inflow to the pocket and no natural uses of water (ibid. p. 85)."

The question of reliability then becomes a question of recharge by streamflow on an approximately two-year cycle. Our 1964 report, the Harshbarger report (1970) and the report by Putnam et. al. (1983) analyze precipitation and streamflow records thoroughly. The replenishment regime is complex and interesting: recharge can occur during dry years which have many dry months but do have some floods (Putnam et. al. p. 26). Alternatively, stress on the storage system may be less even in the driest year if runoff is evenly distributed in small amounts throughout the year (ibid. p. 74).
In the natural system there was sufficient supply on a reliable
if intermittent basis to recharge the microbasins as demand
depleted the quantity in storage. But the Santa Cruz system was
not natural. The City of Nogales, Sonora had constructed a
wellfield at Buena Vista Viejo and infiltration galleries (similar
to the Pumping Plant system) upstream, linked with a long aqueduct
(see Figure 1). The question was whether expansion of this system
would deplete flows into the U. S. part of the Santa Cruz system.

Inquiries were made, and it was determined that at the end of
1989 and in 1990 the City of Nogales, Sonora was in the process of
developing a wellfield at Agua Zarca southwest of the city, and
over a drainage divide which separates Santa Cruz drainage from
that of the Magdalena River system. The reason for this switch was
that the basin upstream from Buena Vista had reached the full
extent of development and additional wells had begun to interfere
with each other. The basin simply did not have much capacity for
storage.

The reason, given the framework of understanding we were
developing, was simple: each of the microbasins in succession
upstream was progressively smaller in storage capacity. Therefore
there would be no further depletion and what supply existed in 1989
would continue to exist. Moreover, the Sonoran wellfield was
separated from the U. S. Buena Vista microbasin by another
microbasin bedrock constriction, marked on Figure 1 just south of
the Boundary.

"DRY" WATER

The same threat posed by possible Mexican development to the
future reliability of the river as a recharge system also existed
on the U. S side in the basins upstream from Guevavi which might
also develop greater withdrawals in the future. In analyzing
future demand, we began to encounter the legal aspects of the use
of water. We no longer just had to concern ourselves with physical
facts.

Prior to the 1983 flood there had been possibly just two surface
diversions from the river: Guevavi Ranch had one of these. Since
the river was intermittent and ephemeral, water supply from the
river had been through the mechanism of wells which tapped the
water in storage. When registration of wells was intensified by
the requirements of the Groundwater Management Act of 1980, well
owners filed groundwater Grandfather claims on their wells and
irrigated acreage. Many had previously filed surface water claims,
particularly those who had been in the area and knew about old
diversions. When the notices were sent out requiring registration
under the General Adjudication of Maricopa Superior Court, these
and others also filed surface water claims on their wells. The
same wells, and the same water was being claimed twice, under two
separate systems.
This was not necessarily disadvantageous since it appeared by this double-dipping that an owner had twice as much water as actually existed. The City of Nogales purchased some of the water rights and water supply from the entity which owned Kino Springs, and to the dismay of the officials and the anger of the public it turned out that under the deal the actual physical water had to remain on the property under the first set of rights, and the City was left holding the second set of rights for the same water: this truly was a case of "dry" water.

DWR had been forced to decide for itself the character of the water in the upper Santa Cruz, in a decision interesting because it particularly identified the character of the microbasins. Carl Reinhard at Buena Vista had drilled a well near the river to support a substantial residential development which was being planned. Downstream owners complained and DWR held a Hearing (see References) at which it was determined that the water being tapped was appropriable surface water, that there was no unappropriated supply available, and that downstream rightsholders would be damaged.

Reinhard sued on appeal, and in preparation for the Rehearing the DWR report on the upper Santa Cruz was prepared (Putnam, et. al., 1983). The investigation revealed that because of the bedrock constrictions between the microbasins, withdrawals in an upstream microbasin would only partially affect recharge downstream because the flows move through the reaches of the river even as the basins are being recharged, and the cones from the wells are prevented from crossing the barriers.

In evaluating the current and future demands which might be placed on the upstream microbasins we decided that under the Adjudication the "double-dipping" of concurrent surface and groundwater Grandfathered claims would end, and one or the other would prevail. Because of the geology and the supportive findings of the Buena Vista Decision, we would suppose that the inner valley will be held to be surface water.

Surface water claims are of two types, those based on pre-1919 use and those filed on subsequent use. Pre-1919 claims have equal priority while those filed thereafter are assigned priority dates. Since the pre-1919 claims have general priority over the later claims these will be allocated first.
It is interesting to compare the pre-1919 claims with the capacities of the microbasins and with pumpage under Groundwater Grandfathered Rights:

<table>
<thead>
<tr>
<th>Microbasin</th>
<th>Recoverable Water in Storage (acre-feet)</th>
<th>Pre-1919 Claims (ac-ft)</th>
<th>1988 Pumpage (ac-ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Buena Vista</td>
<td>2,570</td>
<td>304.00</td>
<td>101.74</td>
</tr>
<tr>
<td>Yerba Buena (Kino Springs)</td>
<td>3,515</td>
<td>2,490.31</td>
<td>735.74</td>
</tr>
<tr>
<td>Highway 82</td>
<td>7,085</td>
<td>1,821.23</td>
<td>1,390.58</td>
</tr>
<tr>
<td>Guevavi</td>
<td>12,800</td>
<td>1,005.85</td>
<td>820.20</td>
</tr>
<tr>
<td>Total</td>
<td>26,140</td>
<td>5,621.36</td>
<td>3,048.26</td>
</tr>
</tbody>
</table>

Source: Table 1a, Putnam et al. unnumbered page.
Guevavi: Water Development Corporation
Water rights and pumpage: DWR

There have been some extravagant claims made in the post-1919 filings: Yerba Buena Utilities Company filed for 13,750 acre-feet.

The Guevavi Strategy

Bruce Babbitt was hired by Ralph Wingfield, the owner of the ranch, to define the legal aspects of the water rights. Familiar with hydrological concepts and because his degree in geophysics created a particular interest in the 1963 work, he was able to quickly decide to consolidate the water rights at Guevavi under pre-1919 surface water claims, dispense with double-dipping and proceed with a surface water strategy (there had been continuous cultivation at Guevavi since at least 1876 when the township plat and survey notes showed fields at ‘‘Benedict’s, the owner at the time—see Figure 5. In the 1899 homestead filing a well and artificial ponds were listed as improvements. The lakes are on the USGS 1905 30-minute quad These facts laid a strong basis for the pre-1919 claim). The concept was that there were no surface water claims between Guevavi and the Nogales Wastewater Treatment Plant at Calabasas, and there were no other claims within the Guevavi microbasin. The City could purchase not only the 1,005.85 acre-feet of pre-1919 claims but also file for an additional appropriation which, on the basis of storage and the probability of recharge we determined could safely be 3,000 acre-ft.

Protests would ensue from those downstream from the WWTP but the City would be in a position to point out that discharge from the WWTP mitigated any losses caused by upstream municipal pumping. As a final key to the surface water strategy we found that the City had a 4,200 acre-foot Permit for withdrawals from Sonoita Creek, which could be transferred to Guevavi, so that there in fact would be no need to even request an additional appropriation.
Figure 5. Original township plat showing cultivated field at site of Guevavi Ranch
The Injection-Recharge Wellfield, the CAP and the "Exchange"

While we were concerned with the specifics of the Guevavi situation, around us raged a much larger debate centered in Tucson. Since Nogales could not physically use the CAP allocation, Nogales officials were wondering if they should sign the final contracts. The uproar in Tucson was tremendous, because there was some question that if Nogales did not contract for CAP, the allocation would revert to the statewide pot. Sawara was vociferous: "...THE CAP ALLOCATIONS CURRENTLY WITHIN THE TAMA MUST BE REALLOCATED TO TAMA USERS" (Water Words, Volume 8, No. 1, January/February 1990, p. 4: emphasis in original). The Nogales situation figured prominently in discussions establishing the Tucson Augmentation Authority, since it was pointed to as a perfect case where the exchange possibilities afforded by the Augmentation Authority could be utilized.

The "exchange" was discussed from two aspects. By one theory, Nogales could relinquish its CAP allocation to another party in TAMA and be given the right to mine groundwater in like quantity and therefore meet the requirements of safe yield, just as if it were using CAP water. The concept was that safe yield applied to the water accounts of TAMA as a whole, and local groundwater mining is allowable if TAMA as a whole is in compliance.

The second theory was that Nogales could take the effluent currently being discharged into the Santa Cruz, develop a recharge injection wellfield, and gain credit in this way in order to mine groundwater to meet safe yield requirements. The link of this alternative to the CAP exchange with other parties in TAMA was less clear.

The basis of these thoughts was that Nogales had no surface water supplies and so could obtain Assurance of a 100-year water supply only on the basis of groundwater. And since groundwater mining had to be discontinued to meet safe yield, a method had to be found to allow Nogales to survive on groundwater beyond 2025. A major hydrological investigation contract was let by the City to an engineering firm, the task of which was to identify areas of groundwater potential.

Shortly thereafter another hydrological investigation contract was let, this one by CAWCD. The purpose of this contract was to determine a feasible site near the Pima Mine Road terminus of the CAP pipeline for recharge of the Nogales CAP allotment, and subsequent recovery by users in the Tucson Basin.

The Park: instream riparian areas

Faced with what seemed to be insurmountable political pressure by the users in TAMA, Guevavi as a solution of both the "wet" and "dry" problems of a future water supply for the City appeared increasingly less probable. However a completely new aspect of the microbasin environment surfaced in dramatic form.
In May, 1990 KVOA-TV did a major series of programs (five full newscasts) on riparian areas primarily along the San Pedro and focused on the BLM Little Boquillas area. The introductory segment however discussed the Santa Cruz, and showed the perennial flow at Guevavi Mission Narrows.

Throughout the discussion of the possibility of Nogales acquiring the water supply, the question remained as to what to do with the land, and the answer was obvious: it would become a riparian park, either under City, County or State Administration. The State Parks Administration was already involved in developing a linear park along the river at Tubac, sustained by the flow of effluent from the Nogales WWTP (the potential injection recharge project discussed above would obviously impact the entire riparian reach of the river).

Guevavi as a park had particularly useful aspects. The Mission site and the smaller sub-mission at Calabasas (on the hill above the WWTP) were incorporated into the administration of Tumacácori National Monument in the summer of 1990. If Guevavi were to become a park, and a trail system established along the undevelopable bottom through the Rio Rico property, the two mission sites could be linked by a trail system along the river of a conveniently hikable distance with access throughout.

A conference was held at Guevavi with development officers from State Parks, the manager from Rio Rico, and representatives from the County. At that time there were two imponderable events which were awaited—the Heritage Fund initiative was on the November ballot, and nothing could be decided until the City had made a decision.

It is important to keep in mind that the City could make withdrawals downstream from the Guevavi Mission Narrows without affecting flow at the upper boundary.

Conclusion

On December 31, 1990 the Nogales City Council voted to purchase about 490 acres of the bottomland of the ranch, with attendant water rights. The sale was to become effective on March 31, 1991. At a public hearing before the vote, every council member declared that they were voting in favor because of the dual aspect of acquiring both a park and a water supply.

The geohydrological environment of the microbasins provides at Guevavi a "recharge" system in which effluent is credited for high-quality river water, with the result that riparian areas both at Guevavi and downstream from the WWTP are maintained.
REFERENCES


TESTING OF A SALINE AQUIFER FOR AQUIFER STORAGE
RECOVERY POTENTIAL

Sean T. Skehan, Albert Muniz, Peter J. Kwiatkowski
and Kevin M. Bral

ABSTRACT

An investigation was conducted in the Florida Keys to test the feasibility of Aquifer Storage Recovery (ASR), the underground storage of fresh water, in a saline aquifer. The test site is located in Marathon, the approximate midpoint of the Florida Keys. Potable water supply throughout the Florida Keys is provided by a mainland well field near Miami and is conveyed through a 130-mile pipeline from the well field to Key West. The ongoing investigation focuses on the feasibility of obtaining potable water from the Keys distribution system and storing it in a confined aquifer which contains essentially seawater. Using ASR in the Keys could provide a reserve of fresh water to meet emergency or seasonal demands. During construction of a test well, continuous core samples identified an unconsolidated sand aquifer, bounded above and below by confining units. Using core analytical data, an ASR well was designed and constructed. Results of subsequent hydrologic testing and the performance of four cycles of injection, storage and recovery indicate that high recovery efficiencies of potable water are possible.

INTRODUCTION

Background

Noted for its resort areas and sport fishing, the Florida Keys are home to approximately 80,000 permanent residents and 65,000 seasonal residents. With the exception of small fresh water lenses on Big Pine Key and Key West, potable water is supplied through a 120-mile long transmission line from the Florida Keys Aqueduct Authority (FKAA) well field, located in Florida City, to Key West (see Figure 1).

Along the transmission line, there are several pump stations and 30 million gallons (mg) of aboveground storage owned and operated
by the FKAA. Aboveground storage capacity is limited by the availability and high cost of land in the Keys. To meet seasonal and daily peaks in water demand, as well as emergency demands, treated drinking water must be stored. The current storage volume is too limited to provide a source of potable water should an emergency occur. To address this storage need, Aquifer Storage/Recovery (ASR) is being evaluated as a cost-effective solution to system storage needs.

In 1991, the projected average annual daily flow to the FKAA service area is 13.5 million gallons per day (mgd). Since the pipeline from mainland Florida was constructed over 40 years ago, occasional line breaks have impaired its ability to convey water for many days. Because the pipeline crosses 42 bridges in its route, it is especially susceptible to damage by hurricanes.

Principal criteria dictating ASR feasibility and use for the FKAA are: (1) seasonal variations in water supply and/or demand (minimum ratio of maximum day:average day demand is about 1.3:1), (2) useful recovery capacity exceeding 1 mgd, and (3) suitable hydrogeologic conditions for storage and recovery.

To use ASR in the Keys, a thin, well-confined, moderately permeable storage zone was deemed necessary. This would prevent extensive mixing between native water (seawater) and injected water, thereby yielding high recovery percentages. Recovery percentages play an important role in the feasibility of using ASR in the Keys because of the high unit costs for water. Higher recovery percentages will help maintain unit costs.

Based on the findings of a literature search conducted during Phase I of this project (CH2M HILL, 1987), favorable hydrogeologic conditions for ASR were thought to exist at Marathon, Florida. The Marathon pump station was selected by the FKAA as the site for the ASR investigation. Located on Vaca Key the island is the approximate mid-point of a chain of small coralline limestone islands extending south from Miami to Key West.

The second phase of work consisted of constructing a 10-inch-diameter test well (OW-2) to 550 feet below land surface (bgs) to identify a suitable ASR zone. Native water quality and hydrologic data were also obtained. Results of this investigation identified a thin, semiconfined, unconsolidated sand aquifer from approximately 390 to 435 feet bgs. Water quality of this aquifer indicated chloride and conductivity concentrations as high as 20,800 milligrams per liter (mg/l) and 49,000 micro mhos per centimeter (μmhos/cm), respectively. Hydrologic data indicated an average specific capacity of 3.9 gallons per minute per foot (gpm/ft). Based on the findings of Phase II, design and construction of an ASR system was implemented in Phase III of this project.
Scope

In the third and current phase of the ASR investigation at Marathon, a 4-inch-diameter observation well (OW-1) and a 16-inch-diameter ASR well (ASR-1) were constructed in early 1990 to conduct a series of injection/recovery cycles to determine ASR feasibility. This paper presents hydrogeologic data collected during drilling, discusses aquifer characteristics and water quality, and presents the results of four cycles of injection, storage, and recovery.

Construction Details and Hydrogeologic Conditions

Construction

Phase III construction commenced with the drilling of OW-1. Based on information from Phase II drilling, continuous coring was conducted from 350 to 450 feet b.s. Information gathered during the coring was used to more accurately define characteristics of the storage interval and confirm confinement above and below the target ASR zone. Further details regarding coring are presented in the Coring section of this paper. Following coring completion, geophysical logging was conducted from land surface to a depth of 450 feet b.s. Logs included natural gamma ray, electric, and caliper. Using core data and geophysical logs, OW-1 was constructed with a casing interval of 0 to 388 feet b.s. and a screen interval of 388 to 428 feet b.s. The well was constructed with 4-inch-diameter, Schedule 80 polyvinyl chloride (PVC) casing and 40 feet of 0.025-inch slot PVC well screen. A 20/30 gravel pack was installed around the screen from 430 to 356 feet b.s., with the remainder of the annular space cemented to land surface. Based on the results of core analyses (see Coring section), the screened interval of OW-1 was divided into three intervals (top, middle, and bottom) with a tubing and packer apparatus. This apparatus was installed so that discrete water samples could be collected to determine the amount of mixing taking place in the storage interval. The intervals are 387 to 405 feet b.s. (top), 405 to 418 feet b.s. (middle), and 418 to 428 feet b.s. (bottom).

Following the completion of OW-1, a pilot hole for ASR-1 was drilled to a depth of 435 feet b.s. Geophysical logs were again performed on ASR-1 and correlated to logs performed at OW-1 and OW-2 (Phase II). It was determined from this correlation that the depths of lithologic contacts did not vary appreciably across the site. ASR-1 was constructed with casing and screen intervals similar to OW-1 using a 16-inch-diameter, schedule 80 PVC casing and a 70-foot, 10-inch-diameter, stainless steel well screen assembly. The screen assembly consists of 25 feet of riser pipe connected to 40 feet of 0.025-inch slot screen, followed by 5 feet of tailpipe. ASR-1 is located 126 feet from OW-1 and 258 feet from OW-2.
After construction of ASR-1, a mechanical piping system was installed to convey water from the distribution system to ASR-1 during recharge cycles and from ASR-1 to a shallow drainage well during recovery cycles. When the results of testing indicate that water can be recovered back to the distribution system, drainage well use will be discontinued.

Lithostratigraphic Description

The lithostratigraphic description is based on evaluation of previously reported data (CH2M HILL, 1987; CH2M HILL, 1989), examination of drilling cuttings and core samples from the installation of ASR-1 and OW-1, and the results of geophysical logs that were run during well construction. Strata encountered at this site range in age from Miocene to more recent Pleistocene deposits. Figure 2 presents the major stratigraphic units encountered while drilling ASR-1 and OW-1, as well as brief lithologic descriptions and a natural gamma ray log.

Coring

Coring at OW-1 was conducted from 350 to 450 feet bgs through a 4-inch-diameter temporary casing. The cores indicated an interval of moderately-well consolidated sandstone with a calcareous clayey matrix with some fossilization from 350 through 387 feet bgs. Below 387 feet bgs, an unconsolidated medium-to-very-coarse sand extended to a depth of 428 feet bgs. This sand was identified as being favorable for ASR because of the lack of clay throughout this interval. At 428 feet bgs, a thin layer (approximately 1 foot thick) of highly plastic clay was encountered. This layer defined the lower limit of the ASR interval. The interval from 428 to 450 feet bgs was characterized by thin lenses of interbedded clay and layers of unconsolidated quartz sands.

The deposition of the ASR interval appears to have occurred in three zones of sorted material: a very fine- to medium-grained, poorly-sorted quartz sand from 387 to 411 feet bgs, a fine-grained to gravelly, well-sorted quartz sand from 411 to 420 feet bgs, and a predominately medium-grained, poorly-sorted sand from 420 to 428 feet bgs.

Three samples (400 to 405.5 feet bgs, 413.5 to 416.5 feet bgs, and 417.4 to 420.4 feet bgs) were selected from the cored interval for select laboratory parameters. These samples were analyzed to more precisely determine plugging potential in the storage interval. These analyses included X-ray mineralogy, acid insoluble residue analysis, sieve analysis, porosity, permeability, grain density, specific gravity, cation exchange capacity (CEC), scanning electron microscope (SEM) analysis, thin section analysis, energy dispersive chemical analysis, and core photographs with descriptions.
<table>
<thead>
<tr>
<th>Lithologic Description</th>
<th>Geologic Age</th>
<th>Formation Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIMESTONE, very pale orange to white abundant coraline structure, hard and cavernous</td>
<td>Pleistocene</td>
<td>Key Largo Limestone</td>
</tr>
<tr>
<td>LIMESTONE, very pale orange to white chalky, porous, dense, with some shell shards</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SAND, white, poorly consolidated, angular to subrounded, interbedded limestone lenses</td>
<td>Pliocene</td>
<td>Tamiami Formation</td>
</tr>
<tr>
<td>SANDSTONE, light olive moderately consolidated, subangular to rounded</td>
<td></td>
<td></td>
</tr>
<tr>
<td>SAND, yellowish to light olive-gray unconsolidated, fine to very coarse grained quartz, well graded, round to well rounded grains with interbedded clay lenses below 428 feet</td>
<td>Miocene</td>
<td>Hawthorn Formation</td>
</tr>
</tbody>
</table>

**LEGEND**

- **Limestone**
- **Silty Sand**
- **Clay**
- **Quartz Sand**
- **Calcaceous Sandstone**
- **Gravel**

**FIGURE 2**

Typical Lithostratigraphic Description
The results of these analyses indicated a predominately quartz mineralogy with only minor-to-trace amounts of clay minerals. Fine-grained quartz and carbonate grains were present in significant amounts in two samples, 29.1 percent in the sample from 405-405.5 feet bs and 24.4 percent in the sample from 420-420.4 feet bs. The sample from 416-416.5 feet bs had 5.4 percent very fine sand and silt. The carbonate in each of the samples was concentrated in the fine, particle-sized fraction and appears to be recrystallized shell fragments. Average porosity of the three samples was approximately 31 percent, while average horizontal permeability was 21 ft/day. These data were considered to be favorable for ASR.

Aquifer Characteristics

An aquifer test was conducted at ASR-1 to determine aquifer characteristics at the site. Water level measurements were obtained at OW-1 and OW-2 and the data were analyzed for aquifer parameters using the Walton (1961) Method for unsteady state leaky aquifers. Drawdown data in the pumping well (ASR-1) were also collected to estimate a well loss coefficient and determine the well’s specific capacity. Tidal effects were taken into consideration and found to have minimal impact.

Similar aquifer parameters were obtained for the two observation well data sets. The aquifer parameters were then used to calculate the drawdown that would be observed in the pumping well, neglecting well losses. Based on these results, the well losses were estimated and a well loss coefficient calculated. The calculated aquifer parameters reasonably reproduce the observed behavior in the observation wells and the pumping well. Results of the aquifer test are presented below. The storage coefficient obtained is consistent with that of a leaky confined aquifer (Freeze and Cherry, 1979).

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Well OW-1</th>
<th>Well OW-2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Transmissivity (gpd/ft)</td>
<td>10,050-4</td>
<td>13,020-4</td>
</tr>
<tr>
<td>Storage Coefficient (dimensionless)</td>
<td>1.7 x 10^{-3}</td>
<td>2.1 x 10^{-3}</td>
</tr>
<tr>
<td>Leakance (day^{-1})</td>
<td>3.4 x 10^{-3}</td>
<td>4.2 x 10^{-3}</td>
</tr>
<tr>
<td>Specific Capacity (gpm/ft)</td>
<td>3.8</td>
<td>3.9</td>
</tr>
<tr>
<td>Well Loss Coefficient (ft/gpm^{2})</td>
<td>9.6 x 10^{-4}</td>
<td></td>
</tr>
</tbody>
</table>

Water Chemistry

Water samples of both native and injected waters were analyzed for organics, inorganics, and metals to determine if potential plugging problems might occur. It was determined that the recharge water is alkaline with a pH of approximately 9.5 and low in total dissolved solids (TDS) with a value of 397 mg/l. The pH is important because it controls much of the water chemistry,
particularly the precipitation of the carbonate minerals. Based on the concentration of the major ions, the water is classified as a calcium, sodium, sulfate, chloride type.

Native groundwater chemistry is considered to be seawater, with an oxidation reduction potential (Eh) of +100 mv (a slightly oxidizing condition). The dominant water chemistry is sodium chloride at a near neutral pH of 7.1. Trace amounts of silica and aluminum as well as major ions like calcium and bicarbonate are also present. A summary of the major ions and general parameters is presented in Table 1.

Considering the water chemistry of the recharge and native waters, mixture of the two should not create a problem within the storage zone. If the pH remained elevated above 9.5, the concentration of the silica or aluminum in the groundwater could result in the precipitation of clays within the aquifer. However, with porosity values of 0.31, plugging in the storage interval is not considered a problem.

**Cycle Testing**

Recovery efficiency is a measure of the success of a cycle of injection, storage, and recovery. For this project, the measure of efficiency is expressed as the percentage of recovery in relationship to the chloride concentration of the recovered waters. The drinking water standard of 250 mg/l chloride was used to define usable (potable) water. Water samples were collected on a regular basis throughout each cycle of injection, storage, and recovery. For the purposes of this study, a cycle test is defined as the data collected during a single sequence of injection, storage, and recovery.

Recharge water (potable) was conveyed from the distribution line to ASR-1 by means of a 6-inch-diameter pipeline during four cycles of injection, storage, and recovery. System pressure, typically 50 pounds per square inch (psi), was used as the driving force of injection. Specific capacity during recovery has improved from approximately 2.5 gpm per foot (Cycle 1) to approximately 3.8 gpm per foot (Cycle 4). Improved capacity may be due to the break down of residual drilling mud over successive cycles of injection and recovery. Typical injection and recovery flow rates were from 150 to 200 gallons per minute (gpm).
<table>
<thead>
<tr>
<th>Constituent</th>
<th>Water (mg/l)</th>
<th>Native Water (mg/l)</th>
</tr>
</thead>
<tbody>
<tr>
<td>pH</td>
<td>10.3</td>
<td>7.10</td>
</tr>
<tr>
<td>Carbonate Alkalinity</td>
<td>19.0</td>
<td>&lt;1</td>
</tr>
<tr>
<td>Bicarbonate Alkalinity</td>
<td>23.1</td>
<td>120</td>
</tr>
<tr>
<td>Conductivity (μmhos/cm)</td>
<td>397</td>
<td>49,000</td>
</tr>
<tr>
<td>Carbonate Hardness</td>
<td>110</td>
<td>1,390</td>
</tr>
<tr>
<td>Non-carbonate Hardness</td>
<td>95.0</td>
<td>6,480</td>
</tr>
<tr>
<td>Turbidity (NTU)</td>
<td>&lt;0.2</td>
<td>0.5</td>
</tr>
<tr>
<td>Total Dissolved Solids</td>
<td>212</td>
<td>38,900</td>
</tr>
<tr>
<td>Total Suspended Solids</td>
<td>&lt;1.0</td>
<td>4.2</td>
</tr>
<tr>
<td>Calcium</td>
<td>33.8</td>
<td>398</td>
</tr>
<tr>
<td>Magnesium</td>
<td>3.75</td>
<td>1,350</td>
</tr>
<tr>
<td>Sodium</td>
<td>20</td>
<td>10,500</td>
</tr>
<tr>
<td>Potassium</td>
<td>11.4</td>
<td>385</td>
</tr>
<tr>
<td>Silica</td>
<td>4.7</td>
<td>6.4</td>
</tr>
<tr>
<td>Aluminum</td>
<td>&lt;0.5</td>
<td>&lt;0.5</td>
</tr>
<tr>
<td>Iron</td>
<td>0.05</td>
<td>&lt;0.02</td>
</tr>
<tr>
<td>Chloride</td>
<td>41.8</td>
<td>20,800</td>
</tr>
<tr>
<td>Fluoride</td>
<td>0.80</td>
<td>0.84</td>
</tr>
<tr>
<td>Sulfate</td>
<td>91.1</td>
<td>2,910</td>
</tr>
<tr>
<td>Nitrate and Nitrite</td>
<td>&lt;0.02</td>
<td>&lt;0.02</td>
</tr>
</tbody>
</table>
ASR Well

To better understand the mixing properties of the recharge water with the native water in the storage interval, recovery efficiency curves from each cycle are plotted in Figure 3. These curves compare the percent volume recovered along the x-axis (volume recovered/total volume injected) with chloride concentrations along the y-axis. For Cycle 1, it was determined that at least 100 percent of the volume of injected water would be recovered. This was equivalent to a total of 5,132,960 gallons of water, representing 113 percent of the volume injected. As shown in Figure 3, approximately 33 percent of the volume stored was recovered before chlorides reached 250 mg/l. At the conclusion of recovery, the chloride concentration (16,200 mg/l) had returned to near background levels (21,000 mg/l). Subsequently, three additional cycles of testing were performed. Table 2 provides a summary of cycle test data showing that cycle efficiency increased from 33 percent in Cycle 1 to 70 percent in Cycle 4. Cycle 2 recovery efficiency was somewhat lower than that observed in Cycle 1 due to a 34-day storage period taking place after Cycle 2 injection.

Observation Well

During injection and recovery, Well OW-1 was constantly pumped at about 2 gpm and samples were periodically taken from each of the three different monitor zones for analysis. The purpose of this sampling was to observe changes in water quality within the aquifer away from the ASR well. Data from these analyses compare chloride concentrations along the Y-axis to the period of injection and recovery, expressed in days along the X-axis (see Figures 4 through 7). Data from Cycle 1 suggests that the salt/fresh water interface was almost vertical as it reached OW-1. Figure 4 shows that a mixing period of about 10 days occurred between recharge and native waters. This was followed by a gradual decrease in chloride concentration. During Cycle 1 complete freshening of the storage interval at CW-1 was not observed. In addition, density stratification was not apparent.

Figure 5 shows the chloride concentrations versus time during Cycle 2. This figure shows that during injection, a mixing period of about 10 days occurred, followed by gradual improvement of water quality with an increase in volume injected. After 22 days of injection, it appears that the bottom zone showed a general trend of having the lowest chloride concentrations (a minimum of 680 mg/l) while upper zone chloride concentrations are consistently elevated above the middle zone. Recovery for Cycle 2 took place over 12 days. Consistent chloride concentrations were observed during the first four days across the storage interval. Thereafter, it appears that some density stratification may have taken place. For example, the bottom zone exhibits chloride concentrations elevated above the top and middle zones while the top and middle zones have similar chloride concentrations.
<table>
<thead>
<tr>
<th>Cycle</th>
<th>Number of Days Inj./Rec (Days)</th>
<th>Volume Water Injected (Vi) (gallons)</th>
<th>Volume Water Recovered (Vr) (gallons)</th>
<th>Storage Time (Days)</th>
<th>Average Injection/Recovery Rate (gallons per minute)</th>
<th>Recovery Efficiency (Vr/Vi) (%) Chorides = 250 mg/l</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>18/20</td>
<td>4,514,430</td>
<td>5,132,960</td>
<td>0</td>
<td>200/200</td>
<td>33%</td>
</tr>
<tr>
<td>2</td>
<td>44/12</td>
<td>9,698,620</td>
<td>3,457,820</td>
<td>24</td>
<td>153/200</td>
<td>28%</td>
</tr>
<tr>
<td>3</td>
<td>28/17</td>
<td>5,322,330</td>
<td>4,181,600</td>
<td>0</td>
<td>132/171</td>
<td>67%</td>
</tr>
<tr>
<td>4</td>
<td>15/10</td>
<td>3,623,000</td>
<td>2,751,580</td>
<td>0</td>
<td>167/191</td>
<td>76%</td>
</tr>
</tbody>
</table>
FIGURE 4
Cycle 1 Chloride Concentration vs Time
at OW - 1 During Injection and Recovery

FIGURE 5
Cycle 2 Chloride Concentration vs Time
at OW - 1 During Injection and Recovery
FIGURE 6
Cycle 3 Chloride Concentration vs Time at OW - 1 During Injection and Recovery

FIGURE 7
Cycle 4 Chloride Concentration vs Time at OW - 1 During Injection and Recovery
Cycle 3 injection data clearly indicates the top zone having the highest chloride levels throughout the 28-day injection period. During this same period, the middle zone generally exhibited chloride concentrations lower than the bottom. Cycle 3 recovery data at OW-1 see (see Figure 6) exhibited a mixing period of approximately 8 days where chloride levels were similar for each zone. Thereafter, chloride levels in the lower zone again became more elevated than those observed for the upper and middle zones, indicating some density stratification.

Injection and recovery for Cycle 4 (see Figure 7) occurred over a 24-day period. Consistent with results from Cycle 3, bottom zone chloride concentrations are consistently elevated above the middle and top zones during injection and recovery. Based on this data, it appears that some density stratification has taken place.

Summary and Conclusion

ASR investigations conducted at Marathon, Florida, have shown that a thin, well confined aquifer of unconsolidated sand with a saline water composition is present at the site. Aquifer characteristics are conducive to storage of potable water with minimal mixing of seawater, thereby yielding high recovery percentages. Based on geochemical analysis, chemical characteristics of the injected water and native seawater should not create a plugging problem within the aquifer. Four cycles of testing have been conducted using chloride concentrations of 250 mg/l to measure efficiency of recovery. With the exception of Cycle 2, recovery efficiencies have progressively improved from 33 to 70 percent. The results of chloride analyses conducted on water samples from the multi-zone observation well have shown progressive improvement in water quality over time. This chloride data has also shown that the effects of density stratification have been minimized because of the designed construction. Future cycle testing with greater storage periods will be conducted to determine the long-term effects of density stratification within the aquifer.

Results of the investigations have successfully demonstrated ASR as a cost-effective means of storing water in the Florida Keys. Additional testing at the Marathon site and in Key West will determine the extent to which ASR can be implemented.

ACKNOWLEDGEMENTS

This project is being funded by the Florida Keys Aqueduct Authority (FKAA) as part of a program to provide emergency freshwater resources to residents of the Florida Keys. The authors wish to express their appreciation to Thomas E. Hartye, Director of Engineering of the FKAA for support of this project.
KERRVILLE, TEXAS--A CASE STUDY FOR AQUIFER STORAGE RECOVERY


Kerrville, Texas, is faced with the need to expand a 5-million gallons per day (mgd) water treatment plant and possibly a new off-channel storage reservoir to meet projected water demands. Recent feasibility studies for the Upper Guadalupe River Authority, the plant's owner, shows that significant cost savings are possible by installing aquifer storage recovery (ASR) wells to supplement existing facilities.

ASR is the storage of treated drinking water in a suitable aquifer during "wet" months, and then recovering it in "dry" months of the year to meet peak water demands. This differs from most other aquifer recharge methods in that treated drinking water is recharged, and that the same well is used for both recharge and recovery.

The proposed implementation plan is to first construct a prototype ASR well and extensively test it before constructing the required two to three production wells. The 16-inch-diameter by 600-foot-deep prototype well would be put into service as one of the required production wells after it proves out. The formation to be recharged, the Hosston-Sligo formation, is composed of conglomerate, sand, limestone, and shale. The proposed program would be the first application of ASR in Texas.
ABSTRACT

Title: Recharge Permitting in the Wake of H.B. 2612: Redefining DEQ and DWR Roles in the Cooperative Process.

Authors: Greg Bushner, Hydrologist
          AZ. Department of Water Resources

          Jim DuBois, Hydrologist
          AZ. Department of Environmental Quality

Recent legislation established exemptions from Arizona’s Aquifer Protection Permit Program for Recharge and USR Projects not using effluent source water. Instead, aquifer protection is assured by revisions to Title 45 giving DEQ added responsibilities for water quality review and coordination with DWR in setting permit conditions prescribing monitoring. The two water agencies have adopted a modified review process to meet the objectives of H.B. 2612.

The permit process will follow the steps and time frames outlined in Title 45. The applicant attends a pre-application and fee can be filed with a supporting Hydrologic Study addressing both water quantity and water quality concerns. Technical staff from both departments will evaluate application materials for completeness and correctness. A single Recharge or USR permit will be issued by DWR which sets specific permit conditions jointly agreed upon by DWR and DEQ.
HYDROGEOCHEMISTRY AND CHEMICAL COMPOSITIONAL CHANGES OF GROUND WATER FROM A DEEP WELL RECHARGE OPERATION USING RIVER WATER SUBJECTED TO LIMITED ON-SITE TREATMENT

Mario R. Lluria, Timothy L. Gorey and Robert B. Mack
Salt River Project, Phoenix, Arizona

ABSTRACT

The Salt River Project carried out a deep well recharge test of 60 days duration. A blend of Salt and Verde River water was filtered on-site using a microscreen rotating filter and disinfected by chlorine gas before injection. The two major goals of this project were to test the efficacy of an innovative injection system and to assess the feasibility of using a very limited on-site treatment of the raw water supply.

Baseline chemical and bacteriological characteristics were determined for the ground water and the canal water used for recharge to establish the types of changes, and their level, caused by the recharge operation. Sampling was carried out before and during injection, before and after treatment, in the recharge well and in the observation well. This permitted a complete pathway and real-time determination of changes in the injected water, and in the ground water.

The analysis of all the water quality parameters indicated that the filtration technique used in combination with the artificial recharge methodology employed was successful.

INTRODUCTION

During the spring of 1989, the Salt River Project (SRP) carried out a 60-day artificial ground water recharge test at a large capacity irrigation well (30E 5.9N). This project was selected and funded by the Research and Development Department and supported by several other divisions of the SRP, with the lead responsibility and management under the Water Quality and Geohydrology Department. One of the principal motives to undertake this research was the future availability of excess Colorado River (CAP) water. This water would become available in the SRP canal system at the completion of the construction of the CAP-SRP Interconnection Facility (CSIF). The CAP water would commingle with the Salt and Verde Rivers’ (SRP) water with only limited changes in the chemical composition of the resulting water (Camp, Dresser and McKee, 1981). Using raw CAP/SRP water or SRP water for direct well ground water recharge would require treatment for both operational and environmental reasons.

Entrained air, suspended particulates, and micro-organisms would have to be suppressed or reduced to sustain the recharge rate and avoid permanent damage to the aquifer near the borehole from pore clogging. Establishing the hydrogeochemistry of the aquifer system was also critical.
to prevent chemical reactions triggered by abrupt changes of physical-chemical conditions in the zone of the surface water/ground water mixing front of the aquifer in the vicinity of the recharge well. Precipitation of carbonates of calcium and magnesium due to a rapid decrease in the partial pressure of carbon dioxide (PCO₂), or of calcium sulfate from the higher contribution of the sulfate anion from the CAP water, were potential concerns. Ion exchange reactions of chemical species of the recharge water with others of the silicate minerals, clays in particular, of the aquifer clasts was a consideration for potential decrease of the effective porosity. The possible formation of trihalomethanes (THM’s) from chlorine disinfection of the surface water containing precursors had to be controlled and mitigated to avert exceeding its maximum contaminant level (MCL) in the ground water.

The recharge system unit employed had to provide an adequate flow rate that could make the operation cost-effective and free of entrained air particles. The on-site treatment unit would effectively eliminate particulates, both mineral and organic, and adequately disinfect the surface water.

**TECHNICAL BACKGROUND**

The well selected for the recharge test is located in the city of Mesa on the SRP’s Consolidated Canal (Figure 1). This well is 244 meters (800 feet) in depth with 61 centimeter (24-inch) casing to 190 meters (622 feet) reduced to 51 centimeters (20 inches) from this depth to the bottom. Its mean pumping capacity is 15,785 cubic meters per day (2900 gallons per minute). An observation well 10 centimeters (4 inches) in diameter, of equal depth and screened for the same interval as the recharge well was drilled at a distance of 26 meters (85 feet).

**Water Source**

SRP water, a blend of Salt River and Verde River water was conveyed from the SRP’s Eastern Canal to the recharge well site via a buried 20-centimeter (8-inch) PVC pipe.

**Injection System Unit**

Four PVC pipes ranging from 4 centimeters (1.5 inches) to 8 centimeters (3 inches) in inner diameter, strapped to the pump column and immersed 10 meters (30 feet) below mean static water level form the recharge water delivery system. The four pipes have manually operated valves and a vacuum gauge to measure the pressure and avert cavitation. They are connected to a manifold preceded by an on-line flow meter in the 10 centimeter (4-inch) supply line from the treatment unit.

**Treatment Unit**

Two elements comprise the treatment unit: a rotating microscreen drum filter and a disinfection unit. During the 60-day test 10, 21, 35 and 100 micron screens were used to study changes in water quality of the filtrate and to determine the resulting variations in flow rate. The disinfection was done using chlorine gas bubbled into the effluent of the microscreen. Chlorine dosage was decreased from 7 milligrams per liter at the start of the test to 1.5 milligrams per liter near its completion.
Figure 1. Location map of recharge well 30E-5.9N
SITE CHARACTERISTICS

Geology

The recharge well penetrates an alluvial aquifer typical of the intermontane basins of the Basin and Range Physiographic Province of the Southern Cordillera which has excellent potential for artificial ground water recharge (Lluria, 1987). The units are predominantly unconsolidated, coarse, clastic sediments deposited in a high energy depositional environment. The drillers log from the observation well indicate a gradual fining of the material with depth. The gamma, spontaneous potential and resistivity surveys carried out in the observation well indicate a change near 168 meters (550 feet) of depth with the lower 68 meters (250 feet) being more consolidated and less permeable (Figure 2).

Aquifer Characteristics

The coarse nature of the sediments in the area produce favorable hydraulic conditions for accepting recharge water. Based on a twenty-four hour pumping test conducted prior to operation, a transmissivity of 3,105 cubic meters per meter per day (250,000 gallons per day per foot) was obtained for the area. This is consistent with available data for this area. A prior study (Salt River Project, 1988) estimated the average specific yield for the area at 15% to 20% and the effective porosity was estimated at 25% to 35% percent. At the time of the injection test, depth to water was 70 meters (230 feet) below ground level. The section of the aquifer system penetrated by the test well indicates unconfined conditions.

HYDRAULIC IMPACTS

The hydraulic impact area for the injection test was predictably small. Prior to operation, a maximum impact area based on an injection rate of 63 liters per second (1000 gallons per minute) was calculated to have a radius of approximately 580 meters (1900 feet). During the test, the maximum injection rate was 19 liters per second (300 gallons per minute). The injection well had a maximum water level rise of 1.5 meters (5 feet), while the monitor well, 26 meters (85 feet) away, showed no measurable water level rise. The small radius of impact was probably due to the coarse nature of the upper sands, gravel and boulders present in the upper portion of the aquifer, which impart a very high permeability and results in a rapid dissipation of the hydraulic cone of impression.

PERFORMANCE OF THE INJECTION SYSTEM UNIT

The injection unit, consisting of the four drop pipes individually valved, was purposely over-designed for this test. The reason was to create a well recharge test station at this well where larger injection rates could be accommodated using the existing equipment in future tests. The valve system was successful in controlling air entrainment at the lower flow rates and avoiding possible cavitation in the injection pipes.
Figure 2
Geologic Cross Section of the Injection Well Site

PRODUCTION WELL
30E-5, SN

SAND

BOULDERS CEMENTED

95'

SANDY CLAY & BOULDERS

180'

SAND & BOULDERS

360'

CEMENTED SAND & ROCK

515'

HARD CLAY & ROCK WITH CEMENTED STREAMS

650'

CEMENTED ROCK

800'

85 FEET

MONITOR WELL
RCNWGR4

SILTY SANDY GRAVEL WITH BOULDERS

150'

SANDY GRAVEL WITH BOULDERS

400'

WELL-ROUNDED FINE MEDIUM & COARSE SAND WITH INTERLAYERED THIN CLAY BEDS

600'

RESISTANCE (OMG/M)

50 60

SPONTANEOUS POTENTIAL (MILLIVOLTS)

50 250

NATURAL GAMMA (API)

50 250

50 250
AQUIFER SYSTEM GEOCHEMISTRY

In examining the chemistry of the aquifer system, two major components have to be considered: the rock matrix and the ground water. The matrix is static and the ground water is dynamic, but the chemical composition of both vary with time because of the influence they may have on each other. The third important element is the group of ambient factors: temperature, hydrostatic pressure and organism activity. Parameters such as electrical conductivity, dissolved oxygen, pH and Eh (oxidation-reduction potential) are considered in this study as part of this group.

Matrix Geochemistry

The clasts of the sedimentary units that form the alluvial aquifer of this area are mainly fragments from intrusive and metamorphic rocks of Precambrian age. The intrusive rocks are predominantly felsic ranging from granites to granodiorites, with quartz (SiO₂), potassium feldspar (KAlSi₃O₈), sodic to calcic plagioclase (NaAlSi₃O₈-CaAl₂Si₂O₈), and muscovite [KAl₂(AlSi₃)O₁₀(OH)₂] as their principal mineral components. The metamorphic rocks include quartzite, metarhyolite, metaoandesite, gneiss and schist. In addition to the minerals listed above these rocks contain ferromagnesians, mainly biotite [K (Mg, Fe)₃ (Al, Fe) Si₃O₁₀(OH)₂], hornblende and pyroxene.

Dissolution, decomposition and hydration of the aquifer matrix minerals contribute to the geochemical properties of ground water in addition to those provided by the natural recharge water. Contact time as well as contact area is important in the interaction between rock and matrix (Matthess, 1982). The most common major ions released from the matrix of the aquifer of this area would be: K⁺, Na⁺, Ca+++, Mg++ and Fe++. The quantities added to the ground water would depend on the saturation index (SI) of the mineral in the ground water under the prevailing ambient conditions, predominantly the pH and Eh (Siegel, 1974). Temperature and pressure have a less influential role in the dissolution process.

Ground Water Geochemistry

Ground water in the area of the recharge well is of the sodium-bicarbonate chloride type (Smith and others, 1982). Moving south away from the Salt River the type changes to sodium chloride. These changes are noticeable in the concentration of total dissolved solids, sulfate, nitrate, calcium, magnesium and other major constituents, and reflect the impact of natural recharge in the Salt River on the chemical composition of the ground water (LLuria and others, 1989). Smith (1986) observed an increase with time in the concentration of sulfate and chloride in the ground water of the upper part of the aquifer of the area of the well recharge test based on historical chemical analysis data.

The baseline chemical composition of the ground water of the area of the recharge test well was determined from water samples from 21 wells within a radius of five miles encircling this well (Table 1). The linear correlation coefficients indicate excellent positive correlation of the concentration of most constituents with TDS, except for flouride (r = -0.41). This behavior for flouride, with its higher concentration near the Salt River suggests more restricted geochemical mobility due to adsorption or ion exchange with the aquifer matrix minerals or chemical precipitation as calcium flouride at higher salinity. The chemistry of the ground water in the zone of impact of the recharge test was established by sampling the test well and the monitoring well preceding the injection (Table 2 and Table 3).
**Table 1**

Selected major ground water constituents of the aquifer of the recharge well test area: statistical analysis of their concentration*

<table>
<thead>
<tr>
<th></th>
<th>Mean (x̄)</th>
<th>Std. Dev. (s)</th>
<th>Range</th>
<th>Correlation Coefficient with TDS (r)</th>
<th>Coefficient Variation (c)</th>
</tr>
</thead>
<tbody>
<tr>
<td>TDS</td>
<td>605</td>
<td>166</td>
<td>385-990</td>
<td>-</td>
<td>0.27</td>
</tr>
<tr>
<td>Hardness**</td>
<td>170</td>
<td>82</td>
<td>77-378</td>
<td>0.97</td>
<td>0.48</td>
</tr>
<tr>
<td>Nitrate</td>
<td>12</td>
<td>12</td>
<td>4-39</td>
<td>0.90</td>
<td>1.0</td>
</tr>
<tr>
<td>Sulfate</td>
<td>58</td>
<td>24</td>
<td>40-117</td>
<td>0.80</td>
<td>0.41</td>
</tr>
<tr>
<td>Fluoride</td>
<td>0.49</td>
<td>0.50</td>
<td>0.2-2.4</td>
<td>-0.41</td>
<td>1.02</td>
</tr>
<tr>
<td>pH</td>
<td>8.1</td>
<td>0.2</td>
<td>7.7-8.2</td>
<td>-</td>
<td>0.02</td>
</tr>
</tbody>
</table>

* Concentration in milligrams/liter
** Hardness as CaCO₃ concentration

**SOURCE WATER QUALITY**

The injection test was undertaken before the completion of the CAP-SRP Interconnection Facility and only SRP canal water was used. The surface water was sampled on a monthly basis during one year before the test (Gorey and others, 1989). Analysis of the surface water indicated a lower TDS than in ground water reflecting the lower concentration of some inorganic constituents, especially sodium chloride (Table 4). Of concern were particulates, both mineral and organic, and bacteria that could cause a reduction of the permeability of the aquifer near the injection well. Algae, fecal coliform, fecal streptococcus and total suspended solids were monitored.

The organic load (NOM), especially humic and fulvic acids content, was carefully examined for their role as trihalomethanes (THM’s) precursors during chlorination. The extent of THM formation (Singer, 1989) is a function of pH, contact time (t), temperature (T), free chlorine (Cl₂), bromide concentration (Br⁻) and precursors (NOM as determined by TOC) and is readily visualized by the empirical relation developed by Amy, Chadik and Chowdhury(1987):

\[ TTHM = A[(UV-254)(TOC)]^B (Cl₂)^C (t)^D (T)^E (pH-2.6)F (Br⁻=1)^G \]

A through F are parameters determined experimentally, and UV-254 refers to the ultraviolet absorbance of the water sample at a wave length of 254 nanometers. During the injection test, the TOC of the influent was carefully monitored, and the chlorine dosage regulated to minimize the THM formation, while accomplishing adequate disinfection.
### Table 2

Concentration of cations in ground water from the test well and the monitor well before the injection test.

<table>
<thead>
<tr>
<th>Cation</th>
<th>Concentration (mg/l)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>Range</td>
</tr>
<tr>
<td>Na⁺</td>
<td>181</td>
<td>177-188</td>
</tr>
<tr>
<td>K⁺</td>
<td>4.2</td>
<td>3.7-4.6</td>
</tr>
<tr>
<td>Ag⁺</td>
<td>-0.0001</td>
<td>-</td>
</tr>
<tr>
<td>Ca²⁺</td>
<td>63</td>
<td>60-68</td>
</tr>
<tr>
<td>Mg²⁺</td>
<td>23</td>
<td>20.27</td>
</tr>
<tr>
<td>Ba²⁺</td>
<td>0.074</td>
<td>0.068-0.080</td>
</tr>
<tr>
<td>Fe²⁺</td>
<td>0.90</td>
<td>0.45-1.14</td>
</tr>
<tr>
<td>Mn²⁺</td>
<td>0.72</td>
<td>0.054-1.4</td>
</tr>
<tr>
<td>Zn²⁺</td>
<td>0.086</td>
<td>0.019-0.215</td>
</tr>
<tr>
<td>Cu²⁺</td>
<td>0.019</td>
<td>0.016-0.022</td>
</tr>
<tr>
<td>Cd²⁺</td>
<td>0.0016</td>
<td>0.0002-0.003</td>
</tr>
<tr>
<td>Pb²⁺</td>
<td>0.003</td>
<td>0.002-0.004</td>
</tr>
<tr>
<td>Hg²⁺</td>
<td>-0.0002</td>
<td>-</td>
</tr>
<tr>
<td>Total Cr</td>
<td>0.015</td>
<td>0.010-0.019</td>
</tr>
<tr>
<td>As³⁺</td>
<td>-0.005</td>
<td>-</td>
</tr>
</tbody>
</table>

### Table 3

Concentration of anions and other parameters in ground water from the test well and the monitor well before the injection test.

<table>
<thead>
<tr>
<th>Anion</th>
<th>Concentration (mg/l)</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean</td>
<td>Range</td>
</tr>
<tr>
<td>HCO₃⁻</td>
<td>222</td>
<td>214-226</td>
</tr>
<tr>
<td>CO₃²⁻</td>
<td>0</td>
<td>-</td>
</tr>
<tr>
<td>Cl⁻</td>
<td>282</td>
<td>270-303</td>
</tr>
<tr>
<td>F⁻</td>
<td>0.2</td>
<td>-</td>
</tr>
<tr>
<td>SO₄²⁻</td>
<td>48.6</td>
<td>47.3-49.5</td>
</tr>
<tr>
<td>NO₃⁻</td>
<td>13.8</td>
<td>9.5-17.5</td>
</tr>
<tr>
<td>Alkalinity (CaCO₃)</td>
<td>182</td>
<td>175-185</td>
</tr>
<tr>
<td>Hardness (CaCO₃)</td>
<td>253</td>
<td>233-280</td>
</tr>
<tr>
<td>COD*</td>
<td>6.8</td>
<td>2.8-10.1</td>
</tr>
<tr>
<td>Temperature (°C)</td>
<td>22</td>
<td>20-23</td>
</tr>
<tr>
<td>TDS</td>
<td>813</td>
<td>787-851</td>
</tr>
<tr>
<td>pH</td>
<td>7.9</td>
<td>7.7-80</td>
</tr>
</tbody>
</table>

*COD is chemical oxygen demand
TREATED WATER QUALITY

Filtration and chlorination were the two operations of the on-site water treatment pre-injection system. Two parameters were measured to determine changes in the particulate load of the surface water: total suspended solids (TSS) and turbidity. These were determined for the influent and effluent of the microscreen filter. For chlorination, the THM concentration was determined in the effluent only, as the level in the raw canal water is below 2 micrograms per liter. Table 5 shows the ratios of TSS and turbidity for the inflow and outflow water from the microscreen filter for the 21, 35 and 100 micron screens. The results indicate that this filtration system is successful in reducing the particulate load, at least to one half of that of the inflow concentration. The reduction in turbidity is much less.

Chlorination produced THM concentrations for the water being recharged that ranged from 1.4 to 70 micrograms per liter. Free residual chlorine varied from 7 to 1.5 milligrams per liter.

GROUND WATER QUALITY IMPACTS

Effects on the quality of the ground water were investigated during the injection and subsequent pumping test by sampling, both the recharge well and the monitor well. Comparison of the chemical composition of the ground water (Tables 1, 2 and 3) with the SRP canal water (Table 4) denotes their general compatibility. The very small differences in pH, Eh, temperature, and ionic strength of the surface and ground water are not conducive to any precipitation of chemical compounds in the zone of mixing. The same holds true for the interaction between the recharged water and the aquifer minerals. Although minor desorption could occur because of the dilution in the ground water of some species by the mixing with the surface water, no major changes in hydrolysis, dissolution or ion exchange with the matrix would occur. The short duration of the test also did not allow for much mixing of the recharged water and the ground water. Similar conditions had been observed in a well recharge test in Las Vegas (Brothers and Katzer, 1989). Figures 3 and 4 show this effect on the electrical conductivity and the bicarbonate concentration. These effects were also observed in the increase in the concentration of Na+, Cl-, Ca²⁺, Mg²⁺, NO₃⁻, and alkalinity of the pumped water with time. With the exception of Na⁺ and Cl⁻, these increases were small as expected from the smaller differences in the concentration of these chemical species in the surface water and the ground water. The volume of water injected was 51,500 cubic meters (14 million gallons). The pumping rate after injection was 183 liters per second (2,900 gallons per minute). This rate would have extracted the volume of injected water in approximately 80 hours, at which time the chemical composition of the pumped water would have returned totally to that of the ground water in lieu of any mixing. However, the chemical composition vs. time plots for most species indicate that total recovery to ground water chemical composition was accomplished in the range of 160 to 200 hours.

Of interest was the much lower residence time of TOC concentration which decreased to background level in approximately 70 hours. This reflects the lower mobility of the organic compounds in the ground water after the injection. In contrast, the THM level recovered 200 hours after the start of pumping, although it had decreased to 70% in 80 hours.

The trihalomethanes were used as tracers to determine the horizontal spreading velocity of the recharge front. Based on the arrival time at the monitor well, the estimated velocity was 0.9 meters per day (2.3 feet per day). Depth samples taken in the monitor well at the completion of the injection test indicated that the THM concentration varied with depth. The higher concentration
Figure 3
Changes in conductivity of recovered water with pumping time succeeding injection test.

Figure 4
Changes in bicarbonate concentration of recovered water with pumping succeeding injection test.
was from 137 meters (450 feet) to 168 meters (550 feet). This difference could reflect the changes in permeability, with the lower concentration in the finer, grain-size sediments in the lower part of the well. The lower THM content in the ground water in the upper portion may indicate loss of THM near the phreatic surface by volatility.

| Table 4 |
|----------------------|------------------|
| Mean concentration of selected SRP canal water constituents in the South Canal upstream of the recharge test well. Data from Salt River Project (1989). |
| **Constituent** | **Concentration (mg/l)** |
| CATIONS | |
| Na⁺ | 94 |
| Ca²⁺ | 47 |
| Mg²⁺ | 21 |
| ANIONS | |
| HCO₃⁻ | 224 |
| Cl⁻ | 129 |
| NO₃⁻ | 33 |
| SO₄²⁻ | 53 |
| Temperature (°C) | 17 |
| TDS | 419 |
| Turbidity (NTU) | 2.3 |
| TSS | 9.5 |
| pH | 8.2 |

**PERFORMANCE OF TREATMENT UNIT**

The microscreen filtration was successful in removing the particulate load, both organic and mineral. All size screens employed from 10 to 100 microns, reducing the TSS to one half of the influent concentration (Table 5). The 35 micron screen removed 21% or more than the 100 micron screen, and 13% more than the 21 micron screen. Measurements of the TSS with pumping time after the injection indicated that no measurable particulates were recovered, suggesting that few were introduced during the injection. Production of ground water for an extended period of time did not show any decrease in the flow rate. This corroborates that no impact to the permeability of the aquifer near the well had occurred as a result of the injection.

The disinfection by chlorination demonstrated its efficacy. The fecal coliform and fecal streptococcus determinations in the recovered water indicated levels for both of less than 2 MPN/100 milliliters. The canal water had levels of fecal coliform in excess of 250 MPN/100 milliliters, and fecal streptococcus in excess of 80 MPN/100 milliliters. Reduction factors exceeding 100 were observed for the diatoms, nonmotile and motile algae.
Table 5

Effect of microscreen filtration on particulates of raw SRP canal water. Measuring parameters are total suspended solids (TSS) and turbidity (TY). Suffix i denotes influent and e effluent.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Mean</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>For 21 Micron Screen</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TSS_i / TSS_e</td>
<td>2.0</td>
<td>1.5-2.5</td>
</tr>
<tr>
<td>TY_i / TY_e</td>
<td>1.2</td>
<td>1.0-1.6</td>
</tr>
<tr>
<td>For 35 Micron Screen</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TSS_i / TSS_e</td>
<td>2.3</td>
<td>1.3-3.5</td>
</tr>
<tr>
<td>TY_i / TY_e</td>
<td>1.2</td>
<td>1.0-1.3</td>
</tr>
<tr>
<td>For 100 Micron Screen</td>
<td></td>
<td></td>
</tr>
<tr>
<td>TSS_i / TSS_e</td>
<td>1.8</td>
<td>-</td>
</tr>
<tr>
<td>TY_i / TY_e</td>
<td>1.0</td>
<td>-</td>
</tr>
</tbody>
</table>

The maximum flow rate obtained for each screen size is shown below:

- 10 microns -- 5 liters per second (85 gallons per minute)
- 21 microns -- 15 liters per second (238 gallons per minute)
- 35 microns -- 16.5 liters per second (260 gallons per minute)
- 100 microns -- 18.3 liters per second (290 gallons per minute)

There was an increase in flow rate with the increase in the openings of the microscreens (28% from the 10 to the 100 micron screen). However, the flow rate was well under the expected flow rate which was desired in the range of 31 liters per second (500 gallons per minute) to 63 liters per second (1,000 gallons per minute).

**SUMMARY OF RESULTS**

The performance of the injection system unit was very satisfactory and will be employed in subsequent higher rate and longer duration recharge tests. The treatment unit more than surpassed the water quality goals for which it was employed. However, the flow rate of the recharge water that it was able to filter was 40% lower than the expected lower range. No negative water quality impacts to the aquifer system ensued from the well recharge operation. There was no reduction in the permeability of the sediments near the borehole and consequently no impacts to the operational function of the well. All aspects of the recharge test demonstrated the feasibility of using the procedures and methodology for a larger scale artificial ground water recharge well operation. A larger surface area rotating drum microscreen filter or a modified filtration system that could accommodate flow rates of a maximum of 100 liters per second (1500 gallons per minute) will be necessary.
ACKNOWLEDGEMENTS

The authors would like to give recognition to many colleagues of the Salt River Project who assisted and contributed in this project. Special appreciation is given to members of the Research and Development Department, Water Civil Engineering Department and Southside Water Service Center for their contributions.

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RESULTS AND SIGNIFICANCE OF AN UNSATURATED-ZONE TRACER TEST AT AN ARTIFICIAL-RECHARGE BASIN, TUCSON, ARIZONA


ABSTRACT

Results from a tracer test at an artificial-recharge basin in Tucson, Arizona, indicates that movement of soil moisture through the unsaturated poorly sorted horizontally stratified alluvial deposits takes place along preferential-flow paths. A bromide-laced pulse of reclaimed wastewater was monitored using pressure-vacuum lysimeters installed at depths ranging from 11 to 45 feet below the bottom of a surficial recharge basin. Tracer-breakthrough curves do not indicate a consistent relation between maximum tracer concentration and depth or between time of tracer breakthrough and depth. Apparent dispersion, as indicated by the slope of the rising tracer-breakthrough curve, shows no apparent relation with depth. Velocity of soil moisture, computed from time of first tracer arrival, ranged from 1.9 to 9.0 feet per day.

Preferential flow decreases interaction between recharge water and solid-phase materials in the unsaturated zone compared with flow as a uniform wetting front. Because fewer reaction sites are available to the infiltrating water under preferential-flow conditions, chemical compounds that otherwise would be sorbed or degraded can cause ground-water contamination.
ABSTRACT

Gravity response to storage change in the vicinity of infiltration basins

by


and

Michael Hatch, University of Arizona Geohydrology Group, Tucson, Az.

Gravity measurements were made over a period of several months at a City of Tucson demonstration-recharge facility for the purpose of determining the utility of precision-gravity measurements for estimating subsurface storage change. The test facility consists of two infiltration basins filled with water from a well about 1,300 feet from the basins. The well was pumped at a constant rate of 500 gallons per minute. Relative gravity measurements were made at seven stations—one between the basins, three less than 600 feet from the basins, two more than 0.75 mile from the basins, and one near the pumping well. Standard error of the measurements was generally ±3 to ±7 microgal, equivalent to about ±0.25 to ±0.50 foot of water.

Measured gravity changes are consistent with the growth of a recharge mound beneath the basins. Gravity change relative to the station near the pumped well included increases of 85 microgal between basins, 40 microgal at stations near the basins, and 10 to 20 microgal at distant stations. Most change between the basins occurred after a few days of infiltration. Changes near the basins occurred within 2-1/2 months after the beginning of infiltration.

The test and data collection are continuing. Further analysis may include modeling of gravity changes to estimate the spatial distribution of storage change and specific yield.
VADESE ZONE AND SATURATED ZONE FLOW MODELING
TUCSON RECHARGE FEASIBILITY ASSESSMENT PROJECT
PIMA COUNTY, ARIZONA

By Laura J. Strauss and Errol L. Montgomery
Errol L. Montgomery & Associates, Inc., Tucson, Arizona

ABSTRACT

A two-dimensional finite-difference flow model for variably-saturated conditions was used to model artificial recharge by surface spreading into a two-layer aquifer system. The two layers correspond to the recent alluvium and the Fort Lowell hydrogeologic unit. A conceptual model was developed to assess sensitivity of results to aquifer geometry and hydraulic parameters. A site-specific model was developed for a proposed recharge site on Pantano Wash, near the east margin of the Tucson basin, to evaluate recharge rates and dynamics of groundwater mounding that may result from recharge at the site. Results of model simulations indicate: 1) a perched groundwater mound may develop at the interface between the alluvium and the Fort Lowell; 2) height of groundwater mounding is sensitive to thickness and hydraulic conductivity of the alluvium; 3) except when hydraulic conductivity of the alluvium is small, infiltration rates would be controlled chiefly by hydraulic conductivity of the skin which is expected to develop in the uppermost part of the alluvium at the recharge site; and 4) magnitude of lateral groundwater movement and height of the groundwater mound are sensitive to the contrast in vertical hydraulic conductivity between the alluvium and the Fort Lowell.

INTRODUCTION

Recharge by surface methods is being investigated by Tucson Water as part of the Tucson Recharge Feasibility assessment project. Vadose and saturated zone flow modeling was conducted to evaluate groundwater mounding and feasible recharge rates resulting from recharge by surface methods. Objectives of the modeling investigation were: 1) To assess the sensitivity of model results to aquifer geometry and aquifer hydraulic parameters, and 2) to predict the magnitude of recharge rates and the vertical and lateral growth of the groundwater mound which would result from sustained recharge by surface methods at Pantano Wash reach 1. Pantano Wash reach 1 is located in the eastern part of the Tucson basin.
The technical investigations described in this paper were conducted by Errol L. Montgomery & Associates, Inc., under the general supervision of R. Bruce Johnson, Chief Hydrologist, and his staff at Tucson Water. The investigations were conducted in conjunction with CH2M Hill and Dr. Gray Wilson.

MODEL DESIGN

Figure 1 is a block diagram of the field conditions which were modeled. Groundwater movement beneath the surface recharge site is assumed to be symmetric about the center axis of the recharge site. Therefore, only half the area of interest need be modeled. The modeled area is a vertical section, perpendicular to the center axis of the recharge site. The vertical section includes the recent alluvium and the Fort Lowell hydrogeologic unit (Figure 1).

The model used for the study is VS2D, a U.S.G.S. finite difference model which is designed to simulate two-dimensional groundwater movement in a variably-saturated vertical section (Lappala and others, 1987). Two grids were used in this study; the grids differ only in the depth to the interface between the recent alluvium and the Fort Lowell unit. Each grid consists of 107 columns and 96 rows, or 10,272 cells, and represents a field site which is 615 feet in the horizontal dimension and 300 feet in the vertical dimension. Three layers comprise the grid.

![Block diagram of modeled field conditions](image)

Figure 1. Block diagram of the modeled field conditions.
Layer 1

Layer 1 represents a low conductivity skin which is expected to develop in the uppermost part of the recent alluvium, immediately beneath the surface recharge site, as a result of algal growth and deposition of fines. The effect of the skin is to reduce infiltration rate. Layer 1 was assigned a one-foot vertical thickness and a 75 foot width.

Layer 2

Layer 2 represents the recent alluvium. The recent alluvium includes unconsolidated modern stream channel and floodplain deposits, and terrace deposits associated with the Jaynes and Cemetery terraces. The recent alluvium is comprised chiefly of gravel and sand; the silt and clay fraction is small. Because the recent alluvium deposits are coarse-grained and unconsolidated, they are porous and permeable and function as efficient infiltration media. The recent alluvium is not saturated in most parts of the Tucson Basin. Layer 2 was assigned a 70 foot thickness for the first grid and 40 foot thickness for the second grid.

Layer 3

Layer 3 represents the Fort Lowell Hydrogeologic unit. The Fort Lowell hydrogeologic unit comprises the Fort Lowell formation and the uppermost part of the Upper Tinaja beds. The Fort Lowell Formation and the uppermost part of the Upper Tinaja beds have similar hydrogeologic characteristics and were considered as a single unit, or layer, for the model. The Fort Lowell hydrogeologic unit, referred to herein as the Fort Lowell unit, consists of unconsolidated to weakly lithified interbedded clayey silt, sandy silt, and gravel strata. The stratified nature of the Fort Lowell unit causes the unit to have a large ratio of horizontal to vertical hydraulic conductivity, herein referred to as the anisotropy ratio. The Fort Lowell hydrogeologic unit is the principal water bearing unit in the Tucson Basin. Layer 3 was assigned a 230 foot thickness for the first grid and a 260 foot thickness for the second grid.

MODEL INPUT

Required input for the model fall into three categories: 1) unsaturated and saturated aquifer hydraulic parameters, 2) boundary conditions, and 3) initial conditions. Hydrogeologic conditions for the vadose zone and saturated zone were estimated from information for wells and exploration boreholes in the vicinity of Pantano Wash. Figure 2 is a schematic diagram which shows model grid boundary conditions and initial conditions.
Aquifer Hydraulic Parameters

Saturated Aquifer Hydraulic Parameters. The saturated aquifer hydraulic parameters required as input for the model are hydraulic conductivity, anisotropy ratio, and specific storage, where hydraulic conductivity is the horizontal hydraulic conductivity or the value which may be determined from analysis of aquifer test data. For the Fort Lowell hydrogeologic unit, these parameters were based on pumping tests conducted in the vicinity of Pantano Wash. For the model, hydraulic conductivity ranged from 100 to 400 gallons per day per square foot at 1:1 hydraulic gradient (gpd/ft²), anisotropy ratio ranged from 10:1 to 50:1, and specific storage remained constant at \(10^{-8}\) ft⁻¹.

Because the recent alluvium is not saturated in most of the Tucson Basin, hydraulic conductivity can not be obtained from standard pumping tests and has not been reported. However, the recent alluvium is coarse grained and well sorted, and hydraulic conductivity is assumed to be 5 to 10 times larger than hydraulic conductivity of the Fort Lowell unit. Therefore, for the model, hydraulic conductivity assumed for the recent alluvium ranged from 500 to 2,000 gpd/ft²; for all simulations anisotropy ratio remained constant at 10:1, and specific storage remained constant at \(10^{-8}\) ft⁻¹.
Unsaturated Hydraulic Parameters. In the vadose zone, water is held in pores by adsorptive and capillary forces, and pressure head is negative. The magnitude of pressure head is dependent chiefly on moisture content. The relationship between experimental data for pressure head and moisture content can be described by various non-linear algebraic functions. The Van Genuchten (1980) functional relationship was used to describe the relationship for this model. The unsaturated hydraulic parameters used in the Van Genuchten functional relationship are: 1) a coefficient alpha, 2) an exponent n, and 3) residual water content. These parameters uniquely describe a given soil type. Physical soil properties including bulk density, moisture content, texture, and grain-size distribution have been reported for soils in the vicinity of Pantano Wash (Sergent, Hauskins, and Beckwith, 1980; and Western Technologies, 1987). A representative soil type was selected by comparing these soil properties to soil properties for similar alluvial soils for which the van Genuchten unsaturated hydraulic parameters have been determined. A graph of moisture content (θ) versus pressure head (ψ) for which an algebraic equation has been fitted to experimental data is called a characteristic curve. The characteristic curve used for the model is shown on Figure 2 and is similar to a curve that was developed from experimental data for a stony sandy loam from Safford, Arizona (Panian, 1987).

The unsaturated hydraulic parameters required as input to the model are identical to the Van Genuchten parameters. VS2D uses alpha, exponent n, residual moisture, saturated hydraulic conductivity, and the van Genuchten functional relationship between pressure head and moisture content to compute unsaturated hydraulic conductivity. The recent alluvium and Fort Lowell unit are treated differently in the model by specifying different values for saturated hydraulic conductivity and anisotropy ratio.

Boundary Conditions

Bilateral symmetry for groundwater movement about the center axis of the surface recharge site was assumed. Vertical flow is assumed to occur directly beneath the center of the recharge site, therefore, the left side boundary could be considered a vertical streamline and was assigned a no-flow boundary condition. Horizontal flow was assumed to occur in the lowest part of the saturated zone. Therefore, the bottom boundary could be considered a horizontal streamline and was assigned a no-flow boundary condition.

The 75 foot segment on the left side of the top boundary represents the surface recharge site and was assigned a constant head boundary condition. The remainder of the top boundary is assumed to have negligible evapotranspiration or recharge and was assigned a no-flow boundary condition. It was assumed that the groundwater level at the model outflow boundary did not change and that outflow does not occur above the initial groundwater level. Therefore, a no-flow boundary condition was assigned to cells along the right-side boundary above the initial groundwater level, and a
constant-head boundary condition was assigned to cells along the right-side boundary below the initial groundwater level (Figure 2).

**Initial Conditions**

Initial conditions in the saturated zone were assigned by specifying an initial depth to groundwater of 150 feet. Hydrostatic conditions were assumed to occur below initial groundwater level (Figure 2).

Initial conditions in the vadose zone were computed by VS2D. The model code computes initial pressure head distribution in the vadose zone based on an equilibrium pressure head profile above the initial water level, using the van Genuchten functional relationship and the specified unsaturated hydraulic parameters, alpha, exponent n, and residual water content.

**CONCEPTUAL UNDERSTANDING OF MOUNDING AND INFILTRATION RATE**

Recharge to groundwater by infiltration at a surface recharge site causes groundwater level to rise and causes a groundwater mound to develop below the recharge site. The dimensions of the groundwater mound are a function of: 1) geometry of the surface recharge site, 2) rate of infiltration, 3) aquifer geometry, and 4) aquifer hydraulic parameters for the vadose and saturated zones. Parameters most pertinent to evaluation of groundwater mounding and infiltration rate include hydraulic conductivity of the three layers, anisotropy ratio of the Fort Lowell unit, and thickness of the recent alluvium.

**RESULTS**

The following is a brief review of the salient results of the modeling study.

**Effect of the Hydraulic Conductivity of the Skin**

To evaluate the effect of the skin, hydraulic conductivity of the skin ($K_{\text{skin}}$) was varied while all other hydraulic parameters remained unchanged. Hydraulic conductivity of the recent alluvium and Fort Lowell unit were 2,000 gpd/ft$^2$ and 400 gpd/ft$^2$, respectively. Anisotropy ratio for each model layer was 10:1. After about 60 days of simulated recharge, the model results indicated that the infiltration rate was about 16.5 feet/day when $K_{\text{skin}}$ was 400 gpd/ft$^2$, 9.2 feet/day where $K_{\text{skin}}$ was 200 gpd/ft$^2$, and 4.6 feet per day when $K_{\text{skin}}$ was 100 gpd/ft$^2$. Height of the top of the groundwater mound was not sufficient to intercept land surface at the recharge site. For comparison, a model simulation without considering a skin effect was conducted by specifying hydraulic conductivity of the skin equal to hydraulic conductivity of the recent alluvium. When $K_{\text{skin}}$ was 2,000 gpd/ft$^2$, infiltration rate was about 23.5 feet per day.
constant-head boundary condition was assigned to cells along the right-side boundary below the initial groundwater level (Figure 2).

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Model results were compared to results from preliminary field tests conducted by Tucson Water at recharge test pits. Comparison of results indicates that model simulations are sensitive to the hydraulic conductivity of the skin, and that reasonable computed infiltration rates result only from simulations that consider the skin effect. A skin effect was included in the model simulations to more reasonably simulate field conditions.

**Effect of Hydraulic Conductivity and Anisotropy Ratio for the Recent Alluvium and the Fort Lowell Unit**

Due to the contrast in hydraulic conductivity at the interface between the recent alluvium and the Fort Lowell unit, infiltrating water moves chiefly downward until it encounters the interface between the recent alluvium and the Fort Lowell unit. Contour plots of the zero pressure isobar for two stages of recharge are shown on Figure 3. The zero pressure isobar defines the boundary for saturated conditions. The dashed line represents initial groundwater level. The upper diagram represents conditions after four days of recharge and the lower diagram represents conditions after 21 days of recharge. The upper diagram illustrates that at the interface, chiefly lateral groundwater movement occurs in the recent alluvium and chiefly downward movement occurs in the Fort Lowell unit. Because the vertical hydraulic conductivity for the Fort Lowell unit is smaller than for the recent alluvium, the Fort Lowell unit cannot transmit water at the same rate as the recent alluvium. Groundwater accumulates at the interface and perched groundwater develops. Slowly, the wetting front moves downward within the Fort Lowell unit until it encounters the initial groundwater level. As the lateral extent of the perched groundwater becomes larger, the wetted surface at the interface becomes larger, the Fort Lowell unit is able to transmit the volume of water infiltrating from the recharge site, and the rate of lateral groundwater movement decreases substantially. Once the wetting front encounters the initial groundwater level and the Fort Lowell unit transmits the entire volume of infiltrating water, a broad groundwater mound develops, and a quasi-steady state condition may be established, shown in the lower diagram of Figure 3.

**Figure 4** is a contour plot of the zero pressure isobar after 75 days of recharge into a homogeneous unit. **Figure 4** illustrates that if no interface were present and if one homogeneous unit would occur, infiltrating water would move chiefly downward until the water encountered the initial groundwater level. Perched groundwater would not develop under these conditions.

**Height Of Groundwater Mounding.** The height of groundwater mounding is sensitive to hydraulic conductivity for the recent alluvium. Contour plots of the zero pressure isobar after 33 days of recharge for three simulations are shown on **Figure 5.** Infiltration rate is given for each simulation; the rate is represented by the letter I. Comparison of the second and third diagrams on **Figure 5** indicates that when the hydraulic conductivity for the recent alluvium is changed from 1,000 gpd/ft² to 500 gpd/ft², the
Figure 3. Zero-pressure isobar after four and 21 days of recharge into a two-layer system.

Figure 4. Zero-pressure isobar after 75 days of recharge into a homogeneous unit.
Figure 5. Zero-pressure isobar and infiltration rate after 33 days of recharge for three simulations.
decrease in hydraulic conductivity caused the groundwater mound to intercept land surface at the recharge site. Small hydraulic conductivity for the recent alluvium causes lateral and vertical movement of the infiltrating water to become slower. If lateral groundwater movement is slow due to small hydraulic conductivity of the recent alluvium, the recent alluvium will saturate in the vertical direction to accommodate the infiltrating water. Therefore, height of groundwater mounding will be larger for conditions of small hydraulic conductivity than for conditions of large hydraulic conductivity for the recent alluvium.

Figure 6 is a graph of the height of groundwater mound versus time for two different conditions of aquifer hydraulic parameters; height of the groundwater mound is measured from the initial water level. Each set of conditions was used to simulate aquifer conditions where the thickness of the recent alluvium was 40 feet and 70 feet. The number, in days, above the data curves indicates the time required for the wetting front to reach the initial groundwater level. Aquifer parameters for the two sets of conditions were similar except for the anisotropy ratio. The solid line represents conditions with anisotropy ratio of 50:1 for the Fort Lowell unit and the dashed line represents conditions with anisotropy ratio of 20:1. Figure 6 shows that an increase in anisotropy ratio for the Fort Lowell unit from 20:1 to 50:1 causes an increase in the height of the groundwater mound but does not cause the mound to intercept land surface.

![Graph of height of groundwater mound versus time](image)

**Figure 6.** Graph of height of groundwater mound versus time.
Infiltration Rate. Infiltration rate is controlled partly by hydraulic conductivity of the recent alluvium. Comparison of the second and third diagrams on Figure 5 indicates that when the hydraulic conductivity of the recent alluvium is changed from 1000 to 500 qpd/ft³, the decrease in hydraulic conductivity caused the groundwater mound to intercept land surface and the infiltration rate to decrease from 4.6 to 3.5 feet per day. For reasonable hydrogeologic conditions, hydraulic conductivity or anisotropy ratio of the Fort Lowell unit does not affect the infiltration rate.

When the groundwater mound intercepts land surface, infiltration rate becomes smaller because the pressure head gradient beneath the recharge site becomes smaller. Figure 7 is a graph of pressure head versus time for a point located ten feet below the surface recharge site and a graph of infiltration rate versus time. As pressure head below the surface recharge site becomes larger the infiltration rate becomes smaller. Figure 7 shows a lag time between the time when pressure head ten feet beneath the recharge site becomes smaller and when the infiltration rate begins to decline. The lag time occurs because the graph of pressure head represents conditions ten feet below land surface and the graph of infiltration rate represents conditions at land surface. The lag time represents the time required for pressure head conditions directly beneath the recharge site to increase to conditions similar to what is shown on the graph of pressure head ten feet below land surface.

Figure 7. Graph of pressure head versus time for a point ten feet below the recharge site, and a graph of infiltration rate versus time.
Effect of Thickness of the Recent Alluvium

The height of groundwater mounding is also sensitive to thickness of the recent alluvium. The upper two lines on Figure 6 represents simulations conducted using the grid with 40 foot thickness for the recent alluvium, and the lower two lines represent simulations conducted using the grid with 70 foot thickness for the recent alluvium. Analysis of Figure 6 indicates that if the thickness of the recent alluvium is small, the height of the groundwater mound becomes larger, and the time required for the wetting front to reach the initial water level becomes longer. Comparison of the first and second diagrams on Figure 5 illustrates that height of the groundwater mound is larger for conditions of small thickness for the recent alluvium. Small thickness for the recent alluvium causes a large distance from the initial water level to the top of the perched groundwater mound because infiltrating water accumulates at the interface between the recent alluvium and the Fort Lowell unit. Height of the perched groundwater mound, measured from the interface between the recent alluvium and the Fort Lowell unit, is similar for conditions of similar aquifer hydraulic parameters. Therefore, when perched groundwater encounters the initial groundwater level, the resulting groundwater mound is closer to land surface for conditions of small thickness for the recent alluvium, because the initial perched groundwater was closer to land surface.

Effect of a Large Vertical Hydraulic Conductivity Contrast Between the Recent Alluvium and the Fort Lowell Unit

Lateral movement of the groundwater mound is most sensitive to the contrast in vertical hydraulic conductivity between the recent alluvium and the Fort Lowell unit. Contour plots of the zero pressure isobar for two simulations that differed only in the anisotropy ratio for the Fort Lowell unit are shown in Figure 8. Anisotropy ratio for the Fort Lowell unit was 20:1 for the upper diagram and 50:1 for the lower diagram. For both simulations, horizontal hydraulic conductivity was 1,000 gpd/ft² for the recent alluvium and was 200 gpd/ft² for the Fort Lowell unit, and anisotropy ratio was 10:1 for the recent alluvium. Vertical hydraulic conductivity was 100 gpd/ft² for the recent alluvium and was four gpd/ft² for the Fort Lowell unit. Ratio of vertical hydraulic conductivity between the recent alluvium (Kv*R) and the Fort Lowell unit (Kv*FL) was 10:1 for the upper diagram and 25:1 for the lower diagram. A large contrast in vertical hydraulic conductivity between the recent alluvium and the Fort Lowell unit is caused by: 1) a large contrast in hydraulic conductivity, where hydraulic conductivity is the horizontal hydraulic conductivity value such as that which may be determined from analysis of aquifer test data, and 2) a large contrast in the anisotropy ratio between the recent alluvium and the Fort Lowell unit, as shown in Figure 8.

A large vertical hydraulic conductivity contrast causes: 1) a large lateral extent of the groundwater mound, 2) increased time required for the wetting front to encounter the initial groundwater

185
level, and 3) large height of the groundwater mound. Figure 8 illustrates these three effects. The upper diagram shows the effects when the vertical hydraulic conductivity contrast is moderate. Conditions are shown after 12 days of recharge. The lateral extent of the groundwater mound is small, the perched mound has encountered the initial groundwater level, and the top of the groundwater mound is well below land surface. The lower diagram illustrates the effects when the vertical hydraulic conductivity contrast is large. Conditions are shown after 24 days of recharge, twice the infiltration duration as in the upper diagram. The perched groundwater mound has moved far in lateral extent, the perched mound has not yet encountered the initial water level and the height of the groundwater mound is closer to land surface.

Figure 8. Zero-pressure isobar for a simulation with vertical hydraulic conductivity contrast of 10:1 (upper) and 25:1 (lower).
SUMMARY

Results of model simulations indicate that for two-layer aquifer systems, recharge by surface spreading may result in development of a groundwater mound and temporary perched groundwater. Height and lateral extent of the groundwater mound is controlled by hydraulic conductivity of the recent alluvium and Fort Lowell unit, the contrast in vertical hydraulic conductivity between the two units, and thickness for the recent alluvium. Infiltration rate is controlled by hydraulic conductivity for the skin and for the recent alluvium. If the groundwater mound does not intercept land surface at the recharge site, infiltration rate does not change; if the groundwater mound intercepts land surface at the recharge site, infiltration rate becomes smaller.

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THE 5TH SYMPOSIUM ON ARTIFICIAL RECHARGE OF GROUNDWATER

"Investigation of the Mechanism of Percolation Reduction in a Deep Recharge Basin"

Grisel Z. Gordon, Donald W. Phipps, and Harry F. Ridgway
Biotechnology Research Department
Orange County Water District, Fountain Valley, CA

ABSTRACT

The Orange County Water District operates several deep recharge facilities located adjacent to the Santa Ana River in southern California. Percolation rates within these facilities decrease rapidly during operation. A study of percolation reduction kinetics in Anaheim Lake was undertaken to (1) elucidate the physico-chemical and biological factors responsible for this reduction and (2) determine possible remedial actions. A computer-programmed percolation sensing device (PSD) was designed to measure ultra-low flow rates in experimental sediment-filled columns implanted in the actual lake bottom. Results from the PSD, whose operation was based on the principle of ion migration, indicated that (1) columns filled with native bottom material rapidly plug [90% reduction within 15 days], (2) a low-permeability crust or plugging layer rapidly develops on the bottom, (3) the crust is <4 cm thick, (4) siltation in a full basin is not the major factor in crust development, (5) sediments underlying the plugging layer support high percolation velocities, (6) columns filled with silica sand or with native bottom material overlaid with sand inhibit percolation reduction. A computer-programmed laboratory column system (LCS) was used to model the field observations, as well as identify and characterize suspended particulate matter responsible for basin plugging. The LCS studies suggested that early basin plugging events are abiotic in nature, resulting primarily from filtration and entrainment of microorganisms and inorganic colloids. Microbial processes (e.g., biopolymer synthesis) appear to be involved in subsequent stabilization of the initial plugging layer.
KERN FAN ELEMENT - FEASIBILITY OF CONJUNCTIVE USE

John R. Fielden, Dept. of Water Resources, Sacramento, CA
Terry L. Erlewine, Dept. of Water Resources, Fresno, CA

ABSTRACT

The Kern Fan Element (KFE) is a proposed conjunctive use element of the California State Water Project (SWP). The KFE will store 350,000 acre-feet (taf) of water in the Kern County Groundwater Basin and produce 50-70 taf per year of dry period yield for the SWP at a cost of about $95/af. The Department of Water Resources (DWR) has acquired 20,000 acres overlying the groundwater basin for project operation. Historical land uses include irrigated agriculture and petroleum production which raises water quality concerns. DWR has collected soil samples throughout the property and at proposed recharge sites for analysis of pesticide residues. No residues have been detected. DWR has constructed a network of multilevel monitoring wells to collect water level and quality data. Water quality monitoring has identified contamination with toxaphene, diuron, EDB and EPTC as well as areas of naturally poor quality water. Benzene contamination has been detected at one location near a leaking pipeline. A four layer groundwater model is being developed to evaluate project impacts. Initial results indicate improved water levels in adjacent areas compared to a no project alternative. The model results and regional monitoring will provide the basis for operating agreements with adjacent districts.

INTRODUCTION

The California Department of Water Resources (DWR) operates the State Water Project (SWP) to deliver water for municipal and agricultural supply to 30 contractors. DWR is obligated to ultimately deliver 4.2 million acre-feet per year to these contractors. However, the current SWP facilities can reliably deliver only about half this amount. DWR has undertaken an extensive planning program to identify facilities that can increase the yield of the SWP. Part of this planning effort focuses on developing conjunctive use or groundwater banking projects. These projects tend to be less costly than new reservoirs and do not face the opposition by environmental interests that surface water projects do.

In 1988, DWR purchased approximately 20,000 acres of land overlying the Kern County Groundwater Basin for the development of a
conjunctive use element of the Kern Water Bank (KWB), a proposed addition to the SWP. The Kern Fan Element (KFE) entails direct recharge to and extraction of water from the basin and is the focus of this paper (Figure 1). Additional elements of the KWB will involve in situ recharge of groundwater. Prior to property acquisition, DWR completed a reconnaissance level investigation (DWR, 1987) of the project. The hydrogeology of the project area was poorly understood and significant water quality concerns arising from the historic use of the area for irrigated agriculture and petroleum production were identified. DWR undertook further investigation of the project and has recently completed the feasibility investigation (DWR, 1990) which is reviewed in this paper. Implementation of the project is expected to begin in 1992 following contract negotiation with the SWP contractor in the area.

PROJECT SETTING

The project area is located in the southern San Joaquin Valley of California and overlies the Kern County Ground Water Basin. The KFE property is located on the distal portion of the large Kern River alluvial fan which drains the Sierra Nevada to the east. The modern channel of the Kern River bisects the KFE property. In most years all flow in the Kern River is diverted upstream of the project area. The project site is bordered on the west by the Elk Hills and is adjacent to the California Aqueduct which transports water from northern California.

Hydrogeology

The San Joaquin Valley in the project area is a deep, asymmetrical sedimentary basin. The basin consists of deep depocenters to the north and south of the project area separated by a basement high known as the Bakersfield Arch that generally underlies the Kern River. Most of the basin is filled with a thick sequence of marine rocks that overlie a crystalline basement. Overlying the marine rocks are a series of continental deposits. In the project area the continental rocks consist of the Plio-Pleistocene Tulare Formation, a thick sequence of water-lain sands, silts, and clays exposed on the west side of the San Joaquin Valley. Overlying the Tulare are a series of alluvial fan deposits of the Kern River and ephemeral drainages of the Elk Hills. The developed portion of the ground water basin is above the base of fresh water in the continental deposits. At the project site, the base of fresh water varies from a depth of about 2,800 feet below mean sea level on the eastern edge to about 800 feet adjacent to Elk Hills (Page, 1971).

In much of the San Joaquin Valley, the Tulare Formation contains a thick lacustrine clay known as the Corcoran or "E" Clay which separates the basin into a lower confined system and an upper unconfined to semi-confined system. Exploration drilling on the KFE property determined that no continuous fine grained deposit that could be the Corcoran Clay or an equivalent was present. This
interpretation was supported by regional geophysical investigations by Pacific Geotechnical (1990) which indicated that the Corcoran Clay or equivalent was not present in the project area.

The KFE will operate in the upper 700 feet of the aquifer system in the Tulare Formation and overlying alluvium. Drilling and water level monitoring suggest that this part of the aquifer system behaves in a semiconfined manner with increasing degrees of confinement exhibited with depth. The upper 200-250 feet of the system are dominated by thick, sheet-like sand deposits of high permeability that contain lenticular, fine-grained overbank deposits. Below this depth, fine-grained, low permeability deposits predominate. These deposits are interbedded with numerous, discrete channel sand bodies. Nevertheless, water level data suggest that while localized confinement is present sufficient lateral and vertical continuity exists in the coarser materials for the system to behave as though semiconfined.

Groundwater Quality

DWR has installed a network of 21 multicompletion or cluster wells to obtain water level and water quality information in discrete zones of the aquifer system (Figure 2). Additional information is obtained from the composite production wells on the KFE. Quarterly sampling of monitoring wells on the KFE and selected wells in the 2800-acre area is conducted for: 1) standard minerals and physical parameters; 2) selected minor and trace elements; 3) purgeable organics; 4) organic phosphorus pesticides; 5) chlorinated organic pesticides; 6) carbamate pesticides; 7) gross alpha and gross beta; and, 8) chlorophenoxy herbicides. Production wells are sampled annually for the same parameters. Water quality monitoring will continue over the life of the project but will be reduced in scope and frequency as operational experience with the project is gained.

Mineral Quality. Early monitoring has confirmed that the general mineral quality in the project area is quite good. Occasional measurements of minor or trace elements exceed present or proposed drinking water standards. Generally, these do not pose a significant impediment to project operation. Of most concern are two areas with elevated levels of arsenic. These are probably contiguous and are the northern extension of a regional area of elevated arsenic. The extent of contamination is not known. Water extracted from this area will require blending to meet MCL's. No extraction will occur in this area during the First Stage KFE.

Two areas have been identified with total dissolved solids (TDS) content of about 4,000 mg/l. Both appear to be associated with oil field brine disposal. A small area adjacent to Elk Hills contains elevated TDS that may result from either brine disposal or migration of connate water into the aquifer system. This area appears to be largely isolated from the main groundwater basin and is not a significant impediment to project operation. However, a monitoring system will be installed to detect any movement towards a nearby municipal well field. The second area of elevated TDS is
Figure 2. Monitoring Well Network

LEGEND

▲ 3 Level Monitoring Wells
● 4 Level Monitoring Wells
located approximately one quarter mile down gradient of a set of oil field sumps. Water quality has been degraded to a depth of at least 600 feet. The areal extent of contamination has not yet been determined. This contamination is located in the area of highest recharge potential on the KFE.

**Organic Quality.** Monitoring has detected residues of four pesticides in KFE groundwater. The most significant of these is ethylene dibromide (EDB) a soil fumigant widely used on the property prior to the cancellation of its registration in 1983. Contamination is present beneath several hundred acres in the northern part of the KFE and may be contiguous with an area of regional contamination to the north. The extent of contamination on the KFE is poorly defined at this time because of inadequate well distribution. Furthermore, the EDB was used much more widely than the area of contamination but wells suitable for its detection are not present in much of that area. The area of known contamination will be avoided during First Stage operations. However, if further exploration indicates widespread contamination it may be necessary to implement a treatment program as project operations expand.

The pesticides toxaphene, EPTC, and Diuron have each been detected in one well. None of these pesticides have a reported history of use on the property. EPTC is believed to have contaminated the aquifer through a faulty backflow prevention device during chemigation. Diuron contamination probably resulted from its use for weed control in an adjacent recharge facility. The source of toxaphene contamination is unknown but it may have been introduced directly into the well which lacks a surface seal or well pad. None of these contaminants are thought to threaten project operation.

During shallow exploration of a proposed recharge pond, soil contaminated with petroleum hydrocarbons was found along an existing petroleum pipeline alignment. Further exploration delineated a body of contaminated soil extending about 200 feet along the pipeline, 175 feet laterally, and to a maximum depth of about 65 feet. Low levels of benzene, ethylbenzene, toluene, and xylene were detected in groundwater. Further shallow exploration along the pipeline alignment identified several other areas with fresh contamination. DWR is working cooperatively with the pipeline owner and the water quality regulatory authority to address this problem. The potential for finding additional areas contaminated by petroleum hydrocarbons is high as an extensive pipeline network is present on the KFE.

**Soils Exploration**

DWR has completed a soils exploration program on the KFE to obtain stratigraphic information and to collect samples from the vadose zone for analysis of pesticide residues and petroleum hydrocarbons. The initial exploration consisted of 100 hundred augur borings to a depth of 50 feet throughout the KFE. Fifty seven of these
borings were sampled for Aldicarb, aldicarb sulfone, aldicarb sulfoxide, EDB, and DBCP. No residues were detected. Concurrently, shallow sampling was conducted at sites of petroleum contamination in the oil fields and associated with farming operations. Petroleum hydrocarbon levels as high as 55,000 mg/kg were found in shallow soils. However, high levels were restricted to the upper 2-3 feet of soil. The initial soil screening was encouraging as no obvious problems were identified with pesticide contamination.

Once proposed recharge pond locations were identified a more extensive soil sampling program was undertaken at each site. A total of 39 auger holes were constructed to a depth of 150 feet or to the water table where possible. These holes were sampled for those pesticide residues (Table 1) that had a history of use at the site and that exhibited a significant potential for movement to groundwater. No detectable residues were found. A monitoring well network will be installed at each pond site to detect any contaminant that may be leached.

Groundwater Model Development

Concurrently with the development of the monitoring network and the exploration program a four layer groundwater flow model is being developed. The model is based on the U. S. Geological Survey modular three-dimensional finite-difference model, MODFLOW. Aquifer parameters were initially derived from analysis of drillers', electric, and geologists' logs for wells in the area. Initially, the model is being calibrated using a two-year period of 1988-1989. Monthly groundwater pumpage and recharge were estimated using water budgets based on surface water diversions, precipitation, and water use. The model results were calibrated against monthly water level measurements for the monitoring well network and appropriate production wells. To improve the data base for modeling, DWR has installed propeller flowmeters on all active production wells on the KFE. In addition, an on site climate station will be developed to provide information on precipitation, evaporation and other parameters. Preliminary model results indicate that the First Stage can operate without significant adverse impacts on groundwater levels and that in most cases adjacent areas will experience an improvement over conditions that would result in the absence of the project (Figure 3).

<table>
<thead>
<tr>
<th>Aldicarb</th>
<th>Atrazine</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carbofuran</td>
<td>Diazinon</td>
</tr>
<tr>
<td>1,2-Dichloropropane</td>
<td>Dimethoate</td>
</tr>
<tr>
<td>Disulfoton</td>
<td>EDB</td>
</tr>
<tr>
<td>Endothall</td>
<td>Parathion</td>
</tr>
<tr>
<td>Phorate</td>
<td>Prometryn</td>
</tr>
</tbody>
</table>
Figure 3. Water Level Fluctuations

Element 221 (Rosedale–Rio Bravo WSD)
Projected Ground Water Levels
Unconfined Aquifer

Element 219 (Buena Vista WSD)
Projected Ground Water Levels
Unconfined Aquifer
PROJECT DESCRIPTION

Staged development is planned for the KFE with the First Stage limited to a maximum storage of 350,000 af. The ultimate storage capacity for the project is expected to be 1 million acre-feet. The First Stage has been designed to make maximum use of existing facilities on the KFE property and will also utilize the adjacent 2800-acre recharge area of the City of Bakersfield. The project will recharge up to 90,000 af over a six month period during years when excess flows can be delivered through the California Aqueduct. In years in which the SWP experiences shortages in its ability to deliver water, up to 75,000 af will be extracted and returned to the California Aqueduct either directly or by exchange. The project will produce 50-70 taf of dry period yield (defined over the 1928-34 critical period for SWP operations) at a cost of about $95/af. The facilities for this project are shown in Figure 4.

Recharge and Conveyance Facilities

Water to be recharged in this project will be transported from the Sacramento/San Joaquin Delta through the California Aqueduct and then the Cross Valley Canal (CVC) to the project site and the 2800-acre recharge area. Approximately 1,100 acres of recharge ponds will be constructed on the KFE. A typical pond will be 2-3 feet in depth and consist of individual cells that can be operated independently to allow periodic drying and maintenance. A monitoring well network will be installed at each recharge site to obtain water quality and water level information. The information collected will be used to evaluate the effect of low permeability layers on recharge operations and to identify any contaminants that may be mobilized during recharge. Operational monitoring will also include the quality of recharge water, percolation rates, and the biological condition of the ponds.

Recharge Water Quality. An existing program is in place to monitor pesticides and selected mineral parameters in the California Aqueduct. This program will be supplemented to include parameters required to evaluate recharge operations. Electrical conductivity, total dissolved solids (TDS), and sodium content will be measured to establish criteria that will optimize percolation rates. Turbidity and suspended solids will be monitored to establish criteria to minimize clogging of the basin soils. Nutrients (nitrogen and phosphorus compounds) will be monitored to evaluate the biological conditions in the ponds.

Local interests have expressed concern over maintaining the salt balance in the project area. In response to this concern, DWR will comply with criteria for the TDS content of recharge water. The proposed criteria limit TDS to 500 mg/l instantaneous, 440 mg/l as a thirty day average, and 300 mg/l as an annual average. These criteria will have minimal impact on project operations.
Extraction Facilities

Approximately 60 production wells are present on the KFE and are used for irrigation or are idle. Twenty-five of these wells will be reconditioned and used to extract water for delivery back to the SWP. In addition, four new wells will be constructed. A typical well is 16 inches in diameter, 700 feet deep, perforated about 250 feet and produces about 2,400 gallons per minute. Extracted water will be conveyed through a system of pipelines and open canals to the CVC or to the California Aqueduct. Water delivered to the CVC will be exchanged for water that would otherwise have been delivered from the California Aqueduct.

Produced Water Quality. The California Aqueduct and the CVC serve as sources of public water supply and as such engender concern for the quality of extracted water. DWR policy requires that groundwater returned to the aqueduct comply with all maximum contaminant levels (MCL's) that are established for drinking water. In order to comply with this policy extracted water quality will be monitored at points of return to the aqueduct or the CVC. Monitoring will be conducted on a quarterly basis for major minerals and those minor elements and organic compounds with established MCL's during periods of extraction. A confirmed measurement that exceeds 0.6 times an MCL will trigger sampling of all wells tributary to the sampling point to identify the source. Appropriate corrective action will be taken to identify and remediate the source of contamination and operational procedures will be modified to limit its spread.

Environmental Enhancement

The predominant land use on the KFE has been irrigated agriculture and petroleum production. About three fourths of the area has been farmed and much of the rest is located within existing oilfields. DWR is phasing out agricultural production over a five year period following acquisition. This phaseout will eliminate a significant potential contaminant source and provide an "overdraft correction" benefit to the groundwater basin. Irrigation on the KFE has been largely reliant on groundwater pumping and its elimination will reduce overdraft by about 26,000 af/yr.

Land not required for project facilities will be allowed to revert to natural habitat with significant benefits to threatened and endangered species present on portions of the KFE. In addition, DWR is considering the establishment of seasonal wetlands on portions of the property for use by migrating waterfowl.

SUMMARY

The KFE is a proposed groundwater storage component of the SWP. The project will add significant conjunctive use potential to the SWP allowing for more efficient use of the State's water resources and if successful will pave the way for similar projects in the
future. To insure project success, an extensive exploration, monitoring, and modeling program has been implemented. DWR is currently negotiating detailed operational contracts with KCWA which is expected to operate the project on DWR's behalf. Contracts are also being negotiated with other affected parties. The project will provide significant environmental benefits while developing new water supplies at a cost significantly less than alternatives. The hydrogeologic environment of the project, water quality problems and concerns, and institutional relations will make the KFE an operationally complex project. Nevertheless, the project has been determined to be feasible and is tentatively scheduled for implementation in 1992 following completion of the Environmental Impact Report and the operating agreement with KCWA.

REFERENCES


GRAVEL-FILLED TRENCHES FOR RECHARGING

R. J. Lutton
U.S. Army Engineer Waterways Experiment Station
Vicksburg, MS 39180-6199

ABSTRACT

Recharge capacity of a system for containing and treating contaminated ground water has been improved by addition of gravel-filled recharge trenches. Satisfactory performance was confirmed by monitoring through a 4-month start-up period and subsequently during routine operation. Trenches are 160 ft long, about 3 ft wide, and average about 16 ft deep in penetrating to the base of the unconfined aquifer. Flow rates vary trench to trench from about 2 to 40 GPM according to the variable nature of the alluvium. Water levels were monitored within trenches and downstream and upstream. Water rose during start-up as much as 11 ft adjacent to trenches. The trenches achieved the intended goals of a high rate of recharging and a favorable reversal of ground-water gradients across a key section of the slurry-wall containment. Monitoring is continuing for performance evaluation and for early detection of any deterioration such as may develop from bacterial activity or silting by fines.

INTRODUCTION

Experience in environmental restoration indicates that a water-resource management alternative to recharging in surface basins and through wells is to be found in recharging through gravel-filled trenches (Lutton 1989). A set of ten recharging trenches was installed in 1988 as an addition to a system for confining and treating contaminated ground water. The trenches have so far performed satisfactorily in their objectives of increasing recharging capacity and distributing the water systematically along the confining barrier. The set of trenches may be viewed incidentally as a pilot-scale demonstration in the context of water-resource management at a larger scale.

GENERAL DESCRIPTION

The system is located at the north boundary of the Rocky Mountain Arsenal (RMA) in Colorado. Ground water flowed
naturally northward across the north boundary within a shallow aquifer of Pleistocene sand and gravel. The aquifer lies directly on bedrock of the Denver formation at a depth of about 16 ft. Figure 1 shows the arrangement of the new trench system with respect to the existing barrier, dewatering wells, recharging wells, and monitoring wells. Also notice in Figure 1, the feature identified as a bog about 1,000 to 1,800 ft east of D Street. Prior to installation of the trenches the bog was used as a recharging basin, with substantial flow directed into the area. Much of the water to be recharged into the trenches was redirected from that previously flowing to the bog.

Dewatering wells extract water from the aquifer south of the slurry wall, i.e. upstream of the barrier. Well water is collected via three manifolds and transmitted for treatment in three granular-carbon columns. Water from the column adsorbers is filtered and stored in a sump as a means of regulating recharge.

Before the addition of trenches, treated water was pumped from the effluent sump to the recharging wells located north of the slurry wall, i.e. downstream of that barrier. Recharging wells were particularly susceptible to clogging and periodic cleaning continued to be necessary. More than half the recharging in the 1986-1987 period was actually accomplished not through wells but instead by piping water from recharging wells RW-18 through RW-21 overland to the bog located nearby.

The columns and plant have a design capacity of about 600 gallons per minute (GPM), but actual throughput has seldom exceeded about 280 GPM, the approximate limit of water acceptance by the recharging wells and bog. The distribution of treated water was changed with the installation of the trenches. Prior to the trenches only about 55 GPM could be recharged through the wells in the western half of the system. After the trenches began operating, the recharging to the same portion of the system has been as great as 200 GPM.

CONCEPT AND DESIGN

The conceptual design prepared in 1986 (Lutton 1988) consisted of gravel-filled trenches penetrating into the alluvial aquifer stratum. Recharge water was to be fed from one end longitudinally through a perforated plastic pipe near the top of the gravel prism in each trench. An impermeable membrane or filter fabric sheet was to separate the gravel prism from silty soil placed to the surface as backfill above. Fabric was also proposed to protect against lateral intrusion of silt as the water level fluctuated. Ten separate trenches were suggested to facilitate maintenance and control. Trench width was to be about 2 ft, depth was expected to average 15 ft, length was to be 100 ft, and offset from the barrier was to be 45 ft.
Figure 1. Slurry wall barrier and recharge trench system at North Boundary. Ground water flows northward.
The final specifications and drawings of the system were prepared by Morrison-Knudsen Environmental Services (MKE). Figure 1 shows the location of the ten trenches with respect to the barrier and Figure 2 is a typical cross section. Each trench included features recommended in the conceptual design: a prism of gravel of narrow size gradation, a filter fabric envelop minimizing the influx of silt, and a distribution system carrying water underground to a perforated pipe at the top of the gravel. In the foremost departure from the conceptual design, the trench lengths were increased to about 160 ft, thus allowing an extension of the trench line eastward to a point about 400 ft east of D Street. The specified gravel shown in Figure 2, is somewhat coarser and more narrowly graded than had been proposed in the conceptual design.

CONSTRUCTION

Part of the conceptual design was focused on potential problems of instability when excavating into locally saturated, cohesionless soil. A sequence of steps in construction was outlined to facilitate rapid placement of gravel and reduce the stand-up time for precariously high trench walls.

Fortunately, when MKE constructed the system in August to October 1988, they encountered no unresolvable problems. Relative stability of trench walls was at least in part due to the fact that the water level was low and seldom much above bedrock, the ultimate depth of trenching. Hence, problems with saturation and concomitant weakening of soil were minor. The use of a working bench also proved advantageous to stability. From 4 to 6 ft of the trench wall height was eliminated by excavation of this wide bench.

FLOW HISTORY

The newly completed gravel-filled trenches were started in operation on Monday, 31 October 1988 with 10 GPM going into trench 8 just west of D Street. Four more trenches were activated later in the same week, and by 14 November all trenches were receiving recharge water.

Water for the new trenches was obtained in two ways. First, the treatment plant throughput was increased above that needed to handle the flux of ground water flowing across the system toward the north boundary. Flow through the plant was increased from about 215 GPM to about 300 GPM during the first month (November). Second, the recharge flow to the bog was reduced in increments by closing outlets at recharging wells RW-16 through RW-21. By 10 November water was being piped to the bog at only one well (RW-21). All recharge to the bog was stopped on about 11 January but was restarted on 19 January as equilibrium flow was approaching in the trenches.
Figure 2. Cross section of typical trench from MKE as-built drawing.
Early in January 1989 all recharging wells west of D Street were shut off permanently. Recharging through these wells had become counterproductive at this point in time since the water table had been raised sufficiently by trench recharging.

Table 1 shows selected, calculated or metered flow rates for the trenches. Peak rates came early in December about four weeks after starting. By the middle of January the system was approaching limiting water-table levels imposed by the bottoms of manholes for accessing trench water lines. Accordingly, some trench flow rates were reduced in anticipation of a condition of more or less stabilized flow. Over 28 million gallons of water had been recharged through trenches by March 1989. Trench recharge rates were reduced further in the spring as a means of managing the system.

RISING GROUND WATER

The response of the ground water to recharging through trenches is illustrated in Figures 3 and 4. Note that the rise in water table is what is contoured, not the elevation of the

Table 1

<table>
<thead>
<tr>
<th>Trench</th>
<th>Start to 10 Feb</th>
<th>29 Nov to 7 Dec</th>
<th>17 Feb</th>
<th>3 Mar</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>6.0</td>
<td>7.1</td>
<td>3.7</td>
<td>2.6</td>
</tr>
<tr>
<td>2</td>
<td>1.2</td>
<td>2.8</td>
<td>0.0</td>
<td>0.0</td>
</tr>
<tr>
<td>3</td>
<td>2.2</td>
<td>2.5</td>
<td>0.0</td>
<td>0.6</td>
</tr>
<tr>
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<td>9.6</td>
<td>9.2</td>
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<td>9</td>
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<td>43.0</td>
<td>28.3</td>
<td>25.5</td>
</tr>
<tr>
<td>10</td>
<td>17.5</td>
<td>20.5</td>
<td>15.7</td>
<td>14.5</td>
</tr>
</tbody>
</table>

| Total  | 169.9          | 197.5           | 173.9  | 157.0 |
| Total (Meter 11) | 194.0          |                  |        |       |

208
Figure 3. Level changes (ft) in alluvial ground water from 29 October to 30 November.
water table. Also notice the proximity of the hydraulic barrier. One purpose of the trenches is to maintain a higher water table north of the barrier, sufficient to create a reversed gradient southward across it.

The initial rise in water table (Figure 3) was a rapid, direct response to recharging at the rates indicated in Table 1. The greatest rise, exceeding 6 ft in November, occurred in the high-flow trenches 7, 8, and 9 across a preferred northward-flow pathway recognized in past studies. The hydraulic conductivity is approximately $4.6 \times 10^2$ cm/s in this vicinity. The aquifer had been drained in this interval downstream of the barrier as a consequence of the low capacity of the old recharging wells.

A separate viewpoint of the rise in water table is presented in the profiles in Figure 5 along an east-west line located between the trenches and the slurry wall. The water level in October predated trench recharging while that in March is four months after initiation. Notice the decline near the bog where surface recharging was reduced.

**PERFORMANCE**

Several important findings resulted from initial studies. The trenches functioned satisfactorily during the first few months, and with continued operation, the trenches are becoming important as a prototype of a promising method of recharging.

**Gradient Reversal**

Ground water was raised as much as 11 ft by trench recharging during start-up. This change reversed the ground-water gradient in alluvium across the slurry wall along the entire trenched interval. The head difference locally reached as much as 6 ft before the flow rates were reduced late in start-up.

**Trench Capacity and Control**

The system of ten recharge trenches accepted water at rates up to 200 GPM. Recharge flow was gradually reduced to 160 GPM at the end of start-up as equilibrium approached. Equilibrium flow may be even lower depending on the position of the water table desired. Trenches have been clearly shown to be useful in controlling water distribution within their area of influence.

**Constructability**

The construction of deep gravel-filled trenches for recharging is potentially difficult due to possible instability of trench walls in aquifer strata. However, trench construction was successful at RMA, where the water table in the aquifer had been lowered for a long period and the soils had gained strength sufficient to remain stable. Construction should be relatively
Figure 5. Rise and decline of water table associated with recharging through trenches and the bog during first four months.
easy where the water table is initially below trench bottoms, as in alluvial basins of semiarid regions.

**Plugging of Pores**

A possible limitation on trench recharging involves the accumulation of particulates or bacterial growth. Dissolved or particulate matter and bacterial action would potentially plug the voids within or immediately adjacent to the trenches. The occasional movement of carbon fines into the RMA recharging wells has caused concern in the past but is no longer considered a serious problem. The trenches functioned during the short-term period of start-up without recognizable signs of deterioration in recharging. Nevertheless, the degree of long-term plugging remains an open question that will be clarified in continuing long-term studies.

**Cost**

Costs were compared while developing the conceptual design to those for recharging through surface basins and through wells. It was concluded that trench recharging is less costly per GPM in this application.

**Advantages/Disadvantages**

Recharging through gravel-filled trenches appears to offer the following advantages:
- Low water loss by evaporation, particularly in semi-arid regions
- Easily replaceable

At the same time there are disadvantages as follows:
- Clogging directly shortens service life
- Unclogging is presently not feasible

**REFERENCES**


THE RILLITO CREEK RECHARGE PROJECT: GOALS AND STATUS

J. Craig Tinney
Pima County Flood Control District
Tucson, Arizona

ABSTRACT

The Rillito Creek Recharge Project was initiated formally in 1987. The project goals exceed water supply augmentation. Agencies from three governments have agreed to pursue cooperatively a recharge demonstration project along the Rillito Creek. The overall project is undertaken as a cooperative effort of the Pima County Flood Control District, the Arizona Department of Water Resources, and Tucson Water.

Although all cooperators want to augment groundwater recharge, a range of goals reflects the mission of each cooperator agency. Feasibility and design tasks for the project explore: the enhancement of benefits generated by capital improvement expenditures for flood control; riparian habitat preservation; site specific hydro-geological data requirements; assessments of the impacts of landfills on recharge waters; the development of transferable technology applicable to other recharge projects in Arizona; and opportunities to initiate conjunctive management of surface and groundwater resources. These goals have been adopted into feasibility and design studies for the Rillito Creek Project.

The Phase A feasibility study is completed. No constraints were detected that would compromise the feasibility of a demonstration-scale recharge project. Design studies are expected to begin in the summer of 1991.

The objective of this presentation is to provide a status report for the project and lend insight into planning and administration of a multiple goal project.

INTRODUCTION

The Rillito Creek Groundwater Recharge Demonstration Project (the Project) is a cooperative effort of the Pima County Flood
Control District (the District), Tucson Water, and the Arizona Department of Water Resources (ADWR). Additionally, federal participation for funding is sought for specific Project elements. Each cooperating entity has institutional missions that are distinct and which translate into different technical and institutional project goals. The results of initial feasibility studies serve to focus the Project, too.

Technical considerations for the Rillito Creek Project include assessments of the quantity and quality aspects associated with local hydrogeologic conditions and an evaluation of the surface water system as a source of recharge water. Institutional considerations include establishment of inter-agency management of water resource development, the appropriability of streamflow, and an assessment of the liabilities associated with an urban artificial groundwater recharge project.

While some local experience exists in hydrogeologic assessment of groundwater recharge, no local experience exists for the capturing of stormwaters as a source of groundwater recharge augmentation. Federal participation is sought to initiate Project research, a clear understanding of watershed yield in terms of peak flow and overall discharge is necessary for the design of in-channel structures to ensure structural integrity. Further, advanced surface water quality information is required to insure that recharged water does not degrade existing groundwater. Because this project exploits a unique water resource, the costs associated with initiation requirements such as information development are beyond the resources of local entities.

The initial feasibility study for the Rillito Project did not uncover constraints which restrict recharge but did raise concerns that serve to focus Project goals. Legal entitlement of surface water, landfill locations, the quality of urban stormwater runoff, the use of multiple water sources with varying qualities, and limited storage capacity are issues that focus Project design and operation.

**RILLITO PROJECT PARTICIPANTS AND GOALS**

Table 1 presents the institutional participation in the Rillito Recharge Project.

**The Cooperators**

The Rillito Recharge Project is being pursued cooperatively as an effort of state, county, and city governments (informally
Table 1
Rillito Recharge Project: Institutional Participation by Local, State, and Federal Agencies

Rillito Fund the "Cooperators"—Intergovernmental Agreement:
Pima County Flood Control District
Arizona Department of Water Resources
Tucson Water

Rillito Project Management Committee, the "RPMC"—Charter:
Pima County Flood Control District
Arizona Department of Water Resources
Tucson Water
University of Arizona
Arizona Department of Environmental Quality
Geological Survey
Private Consultant

High Plains Groundwater Demonstration Program—Federal Intergovernmental Agreement:
Pima County Flood Control District
Bureau of Reclamation
EPA
US Fish and Wildlife Service
USGS

referred to as the "Cooperators"). The Arizona Department of Water Resources/Tucson Active Management Area, the Pima County Flood Control District, and Tucson Water established the Rillito Fund through Intergovernmental Agreement. The Cooperators' agencies provide funding participation and guide Fund disbursement. Rillito Fund expenditures are for: monitoring services provided by the U.S.G.S., limited public participation materials, limited monitoring equipment purchase, and feasibility/pre-design studies performed by professional consulting engineer firms.

The Rillito Project Management Committee

Technical oversight is provided by the Rillito Project Management Committee (RPMC) which was formed in 1987 to advise the Pima County Flood Control District, the City of Tucson (represented by Tucson Water), and the Arizona Department of Water Resources on the development of the Rillito Recharge Project. Membership is
established through a charter that includes: the Arizona Department of Water Resources, Tucson Water, Pima County Flood Control District, Arizona Department of environmental Quality, United States Geological Survey, University of Arizona, and a private consultant under contract for feasibility studies. The purpose of the RPMC is to advise the cooperators in the development of feasibility and other supporting studies for the Rillito Recharge Project. The responsibilities of the RPMC are established in the "Cooperators" intergovernmental agreement.

Federal Participation

Financial aid was requested from the Bureau of Reclamation through the High Plains Demonstration Program Act of 1983 by the Pima County Flood Control District to realize the watershed and environmental monitoring program required for the Rillito Creek Groundwater Recharge / Flood Storage / Natural Riverine Preservation Project. A proposal on May 30, 1986 was transmitted to the Bureau for inclusion into the program. The proposal outlined a program that would provide information about existing environmental conditions and a method for watershed monitoring that would serve as a basis for the design construction of a groundwater recharge facility.

Project Goals

All of the Cooperators want to recharge the groundwater at the project area. All desire to establish flexibility in local water resources management. Besides these obvious goals, participation in the Project meets other more specific goals related to agency missions.

Pima County Flood Control District. Responsibility for protection from flood inundation and the erosion of urbanized watercourses is one of the missions of the District. Structural and non-structural watercourse bank protection goals require the examination of multi-purpose solutions because of the great expense associated with all alternatives.

Structural bank protection in the Rillito Creek Project site is necessary given the public demand for land along the watercourse. Additionally in the site area, the acquisition of flood prone land has been utilized, to the greatest extent possible, to act as buffer zones. On a regional context, the District maintains an early flood warning system of hydrometeorological telemetry sensors that report hydrological conditions of upland watersheds. Multi-purpose use of the assets of bank protection structures, publically-held flood prone lands, and the telemetry sensors is the goal of the District. Multiple
benefits will place public assets in higher valued uses.

**Arizona Department of Water Resources.** The Project is a step toward meeting the groundwater safe-yield withdrawal goal that is the emphasis of the Groundwater Code of the 1980 Arizona Groundwater Management Act. Active Management Areas operating under the Arizona Department of Water Resources (referred to as the ADWR) for groundwater management were established under the Code. In 1984, the Arizona State Legislature gave the ADWR authority to participate in the development of a recharge augmentation facility in the Tucson area. The Rillito Creek Recharge Project is consistent with current ADWR plans for a demonstration recharge project and will help meet the safe-yield groundwater goal.

Transferable technologies is central to the goals of ADWR since as an agency it serves to provide planning and expertise for recharge projects state-wide. Envisioned for this goal are suggestions for reference materials, legal procedures, and facility design, among other things.

**Tucson Water.** Tucson Water has established an aggressive groundwater recharge research and development effort. Increasing demands for water supplies and increasing regulatory constraints require innovation and action. While the Rillito Project is small in the scheme of municipal water supplies, it does provide an opportunity to collect performance data to assess the utility of enhancing recharge along watercourses. Study results will complement existing experience of in- and off-channel pilot recharge projects.

**PROJECT STATUS**

The development of the Rillito Recharge Project was halted twice in the fiscal year 1989-1990 because of delays with the conceptual feasibility study and because of water quality analysis issues. The feasibility study assessed constraints and implications to resource allocations associated with groundwater recharge within the Project area. Water sources, water quality, environmental conditions including a landfill analysis, legal considerations and landscape/natural attribute topics were investigated. A framework for the development of the project was the objective. Project delays associated with the feasibility study retarded the time schedule by seven months or more. Phase B activities with on design alternatives has been put to bid.

Water quality issues surfaced early on in the project. In 1986, an unidentified contaminant was detected in groundwater
samples collected in the subject area. The presence of the contaminant was not correlated with known landfills or waste disposal sites. Positive identification of the substance was not made until August 1988. Local, state and federal agencies were informed and involved in the investigations.

The investigations concluded that equipment contamination was the source of the unknown substance. A hose for extracting water from wells not equipped with pumps was presumed to be constructed of the material, teflon. Instead, the construction material was plastic based which leached a plasticizer compound, the unknown substance. Final and conclusive testing ended in January 1990.
GROUNDWATER RECHARGE FIELD TESTING ON
O'NEILL UNIT, NEBRASKA

Burnett, Roger P.
Cast, Larry D.
Kube, Michael D.
U. S. Bureau of Reclamation
Grand Island, Nebraska

ABSTRACT

The purpose of testing was to analyze groundwater recharge methods that are
 technically feasible for the O'Neill Unit. Two sets of groundwater recharge
 facilities located in north central Nebraska were operated from October 1989
to February 1990.

Four recharge methods were employed at the two separate areas to determine
their effectiveness in near surface sand and gravel deposits. The four
methods employed were:

1. Recharge Line - a perforated, subsurface, 6 and 8-inch diameter
corrugated plastic line.

2. Recharge Pit - a 100-foot long, 10-foot wide, and 5-foot deep pit.

3. Unsaturated Recharge Well - a 24-inch diameter hole with 8-inch screen
placed above the groundwater level.

4. Saturated Recharge Well - a 22-inch diameter hole with 8-inch screen
placed below the groundwater level.

The source water used for the tests was from existing irrigation wells
located approximately 1-2 miles from the recharge facilities. Local irriga-
tion wells were used because of their availability, similar water chemistry,
and their ability to provide sediment free water.

Except for minor operational problems, all four methods performed satis-
factorily. Algae growth occurred in the recharge pits which reduced the
infiltration rates.

This paper presents details of the design and operation of the facilities
and a summary of the results.
INTRODUCTION

The O'Neill Unit is located in north central Nebraska. The original concept was that of a traditional reclamation irrigation project, which consisted of a dam and storage reservoir located on the Niobrara River, canal distribution system, and surface water delivered upon demand to 77,000 acres during the irrigation season from mid-June to mid-September. This plan was found to be environmentally unacceptable by a Federal Court. One of the resulting alternatives to the dam and reservoir concept was groundwater recharge.

![Figure 1. Location of O'Neill Unit](image)

The O'Neill Unit is located within an area of extensively developed irrigated agriculture. Groundwater levels are dropping and models have shown a dewatering of the aquifer will occur within the next 20-30 years to an extent that dryland farming will be resumed. The area receives about 18 inches of precipitation annually.

Additionally, the intensive ongoing agricultural activities have caused a general deterioration of groundwater quality—primarily through increased nitrate levels. Much of the area has nitrate levels greater than 20 mg/l (Druliner, 1988). The addition of the low nitrate level recharge water would improve groundwater quality.

The water supply for the recharge project remains the Niobrara River. Direct diversion from the stream was not given lengthy consideration due to sediment-colloid content (500 to 2400 mg/l) of the stream (Druliner, 1988). This colloid content is considered to be sufficient to adversely affect the operation of the recharge facilities. Sediment free groundwater obtained from the Niobrara River alluvium is the considered source or supply.

The proposed recharge project is anticipated to be in operation 10-12 months per year. Groundwater recharge will take place only during nonirrigation months. During the irrigation season the water will be diverted or utilized as a surface supply to serve lands where there is insufficient groundwater.
Figure 2. Location of Recharge Facilities and Source Well at O'Neill Recharge Site.
It is estimated that 150 cubic feet per second, 167,000 gallons per minute, will be required for the project. The goal of the project is to raise the groundwater level and maintain it at preirrigation development levels.

Physically there are two distinct areas to the O'Neill Unit. The limits or project configuration generally being determined topographically by prominent drainages and drainage basin divides. One area is northeast of Atkinson, Nebraska, with the other area located northeast of O'Neill, Nebraska. The two areas are approximately 20 miles apart. There are project areas considered to be geologically unsuitable for groundwater recharge, and these would be served by traditional surface methods using the recharge water and groundwater withdrawn from the recharge areas.

GEOLOGY

Geologically the areas are similar in that there are Pleistocene sands and gravels underlying a thin 2-5 foot layer of residual soil. The sand and gravel thickness is 80-100 feet in both recharge areas. Depths to groundwater in the Atkinson area are typically 35 feet, and in the O'Neill area 60 feet. One difference of concern was the presence of a partial coating of bentonite (montmorillonite) on the individual grains in the upper 50 feet of the unsaturated sands and gravels at the O'Neill recharge sites. The Ogallala Unit underlies the area and has a thickness of 100-200 feet. This is the eastern limit of the occurrence of the Ogallala Unit, and it consists of variations of fine sand and silt with areas of cementation. It does provide groundwater supply, but because of its fine grained nature, the high yields required for irrigation wells are not readily obtainable. The fine grained nature and depths to the Ogallala Unit eliminated it from being utilized in the recharge program.

DESIGN CONCEPTS

Because the sand and gravel deposits occur near the surface, economical, low maintenance, and easily constructed surface and shallow subsurface methods were considered for recharge field testing. The subsurface geologic conditions were basically similar in both potential recharge areas of the project, but there were minor differences, such as the depth to groundwater, bentonite coatings on sand grains, and grain size differences. The decision was made to conduct the same field recharge methods at both sites in order to generally compare the influence of these profile differences. The selected methods were:

1. Recharge Line - a 1,000 foot long, perforated 6 to 8-inch diameter corrugated plastic drainpipe, 4 to 6 feet deep within a gravel envelope.

2. Recharge Pit - an excavated pit with dimensions of 100 feet in length, 10 feet of bottom width, 5 feet in depth and 2 1/2:1 side slopes.

3. Unsaturated Recharge Well - a 24-inch diameter hole to within one-half foot of the groundwater table, with an 8-inch diameter PVC casing and 20 to 30 feet of screen above the water table.

224
4. Saturated Recharge Well - a 22-inch diameter hole using standard drilling methods for irrigation wells to depths of approximately 100 feet with an 8-inch diameter PVC casing, screen and gravel packed below the groundwater table.

The test period of October through February was selected because this was the time existing irrigation wells were available as a water source, little natural recharge occurred, and there was generally uniform aquifer nonuse.

To expedite and insure qualified contractors performed the work, separate specifications were prepared and awarded for the construction of each recharge method. Figure 2 shows the locations of the recharge facilities at the O'Neill sites. Similar arrangements were used for the Atkinson sites.

**OPERATION**

The groundwater recharge facilities were continually operated from October 1989 to February 1990 for a total of 126 days. During this period, a total of 351 acre-feet of recharge was achieved in the O'Neill area and 476 acre-feet of recharge was achieved in the Atkinson area.

A summary of the recharge data collected from each site is as follows:

<table>
<thead>
<tr>
<th></th>
<th>Atkinson Area</th>
<th>O'Neill Area</th>
</tr>
</thead>
<tbody>
<tr>
<td>Recharge Line</td>
<td>232 (GPM)</td>
<td>163 (GPM)</td>
</tr>
<tr>
<td>Recharge Pit</td>
<td>262</td>
<td>157</td>
</tr>
<tr>
<td>Unsaturated Well</td>
<td>117</td>
<td>66</td>
</tr>
<tr>
<td>Saturated Well</td>
<td>134</td>
<td>301</td>
</tr>
<tr>
<td>Recharge Quantity (Ac-Ft)</td>
<td>137</td>
<td>82</td>
</tr>
<tr>
<td>Observation Well Response (Ft)</td>
<td>2.5</td>
<td>4.2</td>
</tr>
</tbody>
</table>

(a) Average Flow Rate, Gallons Per Minute (GPM)
(b) Observation Well Radius Distance from Recharge Point in Feet

Manholes were constructed at each recharge site and contained a calibrated 4-inch inline flow meter that recorded instantaneous flows and accumulated flows. A butterfly valve was installed immediately downstream of each flow meter to control flow into the recharge facility.

Eight to 10 observation wells were installed at each recharge method site to monitor the effects of the recharge water. All wells were 2 inches in diameter, except one well at each site was 5 inches in diameter. Water level measurements were made on a daily basis during recharging and then
intermittently for 14 days following termination of testing. Groundwater samples were taken from each of the 5-inch wells on a monthly basis by the U. S. Geological Survey to monitor groundwater quality changes.

Recharge rates, based on available water supply and field hydraulic tests, were assigned to each recharge facility prior to start-up. Only minor adjustments to flows were made at each site during the test period; except for the unsaturated recharge well at the O'Neil site which did not perform as anticipated. A flow reduction of 70 percent was implemented at this facility in the initial stages of the test. The recharge well did not accept water as anticipated from initial permeability testing and other area aquifer tests.

Figure 3 shows the accumulative lowering of the groundwater table in the project area over a six year period, and the rebound that occurs annually between irrigation seasons. The recharge research occurred during the 1989-1990 period of maximum rebound. The value or amount of rebound during the test period was determined by utilizing historic data and interpolating it to reflect daily rebound amounts. The daily rebound amounts were then subtracted from the observation well readings during the recharge testing. These adjusted readings show the change in the groundwater table that were the result of artificial recharge and are illustrated in Figure 4.

Observation well response to the introduction of water in the two types of recharge wells was instantaneous due to the high permeability of the aquifer and the direct contact of the recharge water with the groundwater of the aquifer. The response of the near surface methods, recharge line and pits was evident at the observation wells on the first day of recharge.

Transiometers

Additional monitoring at the recharge pit and line sites was conducted by use of soil moisture tension measuring devices at certain levels below the pits and recharge lines. This was accomplished by the use of a relatively new instrument called a transiometer. The transiometer developed by the Water Resources Institute, South Dakota State University, Brookings, South Dakota was used. The instrument was developed to measure both positive and negative soil moisture tensions electronically (Troonen, 1985). The transiometers used at the recharge pits were placed at depths of 1, 3, 6, 9, and 15 feet below the base of the recharge pits. The role of the transiometers was to determine if the recharge water, as it moved through the soil profile to the water table, would "perch" on layers of soil that may have low vertical permeability. Perched water tables would also be a sign of potential unsuitability of the soils at the site for the type of recharge methods being tested.

The transiometers at the 1-foot depth below the bottom of the recharge pits in most cases reflected the depth of water in the pits. This may be due to the very porous and open structure of the sandy soil, which may have allowed direct head on the soil column at this shallow depth.

The transiometers at the various depths below 1-foot indicated continual soil moisture tension at the Atkinson recharge site. The transiometers at the O'Neil site did show some fluctuation between moisture tension and
Figure 3. Annual Groundwater Changes 1984-1990.

Figure 4. Observation Well Levels Before and After Correction for Annual Recovery.
positive pressure. The positive pressure is an indication of head build-up which is a result of water table perching.

**Source Water**

Sediment-free groundwater from the existing irrigation wells was used as the recharge water. It was economically impossible to use groundwater from the potential project source, the Niobrara River alluvium. The source water from the wells was considered compatible both chemically and quality-wise with the groundwater into which it was introduced. This was an issue of concern by the U.S. Environmental Protection Agency and the Nebraska State Department of Environmental Control, since the saturated recharge wells created a situation of direct contact between the source water and the groundwater of the receiving saturated aquifer.

The source wells were located 3/4 mile from the closest recharge site and the pumped water was conveyed to the recharge sites via PVC and aluminum pipe. The pipe was placed above ground for approximately 80 percent of the distribution system. The irrigation wells were electric powered to minimize maintenance problems. A flow meter and control valve were located immediately downstream of each source well to provide measurement of flow and to regulate flow.

The chemical compatibility of the recharge water found in the Niobrara River alluvium and the project land's groundwater is a concern that is presently being studied. Computer modeling of the two mixed waters is being conducted to determine if precipitates would form during recharge which would plug the aquifer at the points of recharge.

**RESULTS**

**Recharge Line**

The recharge line was installed using a wheel trencher with laser guided grade control. The excavated trench was 20 inches wide, 4 to 6 feet deep, and had 4 inches of washed, graded sand and gravel placed around the perforated, corrugated polyethylene pipe. The corrugated pipe was drainage tubing manufactured by Hancor, Inc. with 3/8-inch diameter holes spaced around the circumference of the tubing. The holes provided 5 to 6 square inches of open area per linear foot of pipe length. The tubing was covered with filter fabric to prevent finer grains of the envelope material from falling into the pipe. The installed depths were sufficient to be within the Pleistocene sand and gravel deposits. At the Atkinson site the water introduction was via a PVC manhole located midpoint in the line. The line was constructed with a negative slope of .002 ft/ft away from the manhole. At the O'Neill site recharge water entered through a manhole at the southern end of the line. The recharge line sloped away from the manhole at a negative grade of .004 ft/ft. Standpipes were installed at 250-foot intervals in the line for water level measurements within the line.

**O'Neill Site.** Figures 5 and 6 show the performance of the O'Neill recharge line. Recharge effects on the initial groundwater levels were recorded on
Figure 5. Inflow Rate at O'Neil Recharge Line.

Figure 6. Elevation of Water in Manhole at O'Neil Recharge Line.
the first day of recharge. A stabilized recharge water level (beginning at 2000 hours) with a constant inflow rate was achieved. Water elevations or head at all risers along the line were similar to those at the manhole. Initial depth to groundwater was approximately 74 feet. Four observation wells adjacent to the line were completed within the unsaturated zone to detect perching of the downward percolating water. Depths of these wells were based on site investigation, which revealed zones or layers which may have slow vertical permeability. Wells completed at depths of 33 and 40 feet did show perching. At the 40-foot depth, perching began three days after testing began and a maximum buildup of 3.5 feet occurred in the early stages, but began declining and at 1000 hours had disappeared. At the 33-foot depth, buildup was observed after 14 days of testing and a rise of 2 feet was recorded in the well. This stayed at a constant level throughout the test period indicating the lateral extent of the restricting layer was limited, or the permeability was not much less than that of the surrounding area.

Transiometers were installed at the gallery line at depths of 7, 10, 15, 20, 30, and 40 feet. Positive readings of soil moisture, indicating zones restricting the downward movement of the recharge water, were noted: at 15 feet beginning after 160 hours of recharge, at 30 feet beginning at 600 hours, and at 40 feet beginning at 190 hours. Observation wells completed to depths of 33 and 40 feet recorded free water after 340 and 72 hours respectively. A thin clay layer and the bentonite coated sand grains were considered to be the restrictive barriers.

Atkinson Site. The initial recharge rate was 260 gpm, and at this rate a continued rise of water level in the manhole occurred. The rate was decreased to 240 gpm at 860 hours, and then to 230 gpm at 1600 hours as the rise of water in the manhole continued. At 2600 hours the water level in the manhole stabilized at the 230 gpm flow. The stabilized level was 2.3 feet above invert of the 8-inch recharge line. Initial depth to groundwater was 35 feet and at the end of the test period a total groundwater rise of 2.5 feet was recorded of which 0.7 feet is attributed to recharge with the remaining 1.8 feet attributed to regional groundwater rebound. An observation well completed at 20 feet of depth did not record perched water. Another observation well completed at 30 feet of depth recorded perched water on the third day of recharging, and the total rise of 0.7 feet occurred by the tenth day.

Transiometers were installed for the monitoring of this recharge line at depths below ground of 5, 6, 8, 15, 23.5, and 25 feet. Soil moisture measurements indicated continuous moisture tension. This indicated zones of slower vertical permeability were not present to produce groundwater perching in the unsaturated zone below the recharge line.

The transiometers were installed near the center of the 1000-foot long recharge line. The observation well which indicated a perched water table was installed 250 feet away from the transiometer sites. The variability of the soil profile between the transiometers and the observation well is the only explanation why one site shows perched groundwater and the other site does not.
Recharge Pits

The recharge pits were excavated with a paddle scraper to within 2 feet of final grade. A backhoe, placed on the outside limits of the pit, was used for the remaining excavation to minimize compaction and disturbance of the bottom of the pits. After the pits were excavated, the area received approximately 2 inches of rain and the 2 to 3-foot thick soil zone suffered erosion resulting in silt and clay being deposited on the pit floors. The pits were cleaned by a Gradall excavator prior to the start-up of the test. With the pipe outlet placed near the pit floor, water was introduced at one end. To minimize problems associated with air entrainment and potential frost problems during cold periods, the outlet was submerged in the pit water during the recharge periods. To provide for operational flexibility, a second pit was excavated at the site to handle excess water from the source well. However, the second pit was never utilized for this purpose. The incoming water had sufficient temperature and the seepage loss rate was great enough to prevent water temperatures from dropping below 32° Fahrenheit and icing over. Wind chills and air temperatures were often well below 0° F.

The use of high nitrate recharge water provided for accelerated growth of filamentous algae at both recharge sites. This resulted in a permeability reduction. To maintain a constant recharge rate, water depths gradually increased, thus increasing the wetted perimeter and adding uninfested surface area. The nitrate content of the recharge water was 25 mg/l at the Atkinson site, and 6 mg/l at the O'Neill site (Druliner, 1990). Algae concentration and its growth rate were noticeably greater at the Atkinson site than at the O'Neill site. The water levels in the pits were allowed to rise until they reached levels which began to inundate the walkways of the pits at which time the systems were shut down and the pits allowed to empty for cleaning purposes. The residual algae deposits on the Atkinson pit bottom were a 1/4 to 1/2-inch layer of viscous material. The deposits at the O'Neill pit varied from paper thin to 1/16-inch thick. The pits were cleaned manually, removing the algae and about 1/4 to 1/2-inch of surface material. Visually the pits did not appear as clean as prior to initiation of the recharge program. Water was then reintroduced into the pits at the previous inflow rates. The pond depth gradually increased and after 15 days was within 0.5 foot of pre-cleaning levels. This coincided with the end of the recharge program and no additional cleaning was performed.

Algae growth occurred only in the pits as these were the only locations where the recharging water was exposed to sunlight. Nitrate levels in the groundwater from the Niobrara River alluvium are currently less than 1 mg/l (Druliner, 1988), and these levels are not expected to increase. These low concentrations should reduce the algae problem in pits, should this recharge method be utilized for the proposed project.

Ring permeameter tests were conducted in the bottom of each recharge pit prior to their use, at the time of algae infestation prior to cleaning, and immediately after cleaning.

Table 2 provides the test result values.
<table>
<thead>
<tr>
<th>Site</th>
<th>Pre-Test</th>
<th>Algae Infested</th>
<th>Cleaned</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atkinson</td>
<td>16</td>
<td>3</td>
<td>12</td>
</tr>
<tr>
<td>O'Neill</td>
<td>28</td>
<td>0.25</td>
<td>24</td>
</tr>
</tbody>
</table>

During the first week of operation, the calculated loss rates at the Atkinson and O'Neill sites were 18 inches and 17 inches per hour respectively.

Algae conditions were noticeably more severe at the Atkinson site than at the O'Neill site. It is speculated that the bentonitic coatings on the sand grains were the primary agent in permeability reduction at the O'Neill site, although it was not a factor in the 12-hour long ring permeameter tests nor the initial time period of the testing.

Figure 7 shows the relationship between algae growth at the Atkinson recharge pit, which was first noted at 900 hours into the testing, and the head increase necessary to compensate for the decreasing infiltration rate. Figure 8 illustrates that the inflow rate of recharge water into the pit remained constant.

**Unsaturated Recharge Wells**

Unsaturated recharge wells were excavated by bucket auger to within 2 feet of the water table. A bucket auger was used to minimize sidewall disturbance of the hole. The Atkinson well contained 20 feet of screen and the O'Neill well had 30 feet. Spiral wrap 8-inch diameter PVC screen having a 0.040 inch slot size and about 75 square inches of opening per linear foot of screen was used. Washed road gravel was used as backfill around the screen. No well development was performed. Injection tubing was placed in the center of the casing and sized to insure a full flow of water in the tube. The tubes were used to discharge the recharge water at the bottom of the well to eliminate cascading water and reduce the amount of entrapped air (Smith, 1980).

**Atkinson Site.** The unsaturated well had an initial recharge rate of 12 gpm per foot of head with a rise of 1.4 feet measured in the observation well located 12 feet away. After 3000 hours of operation, the recharge rate was 7.8 gpm per foot of head with a total rise of 3.7 feet measured in the same observation well. During the final 300 hours of the test, the inflow rate was doubled and this resulted in a recharge rate of 7.5 gpm per foot of head.

The direct relationship between head of water in the well column and the inflow rate is illustrated by the parallel changes, shown in Figure 9, that occurred with the increase of inflow for the final 300 hours. An increase in head did not increase the recharge rate. The accumulated rise of the observation well was 7.1 feet. It is speculated the general decline in recharge rate per foot of head is attributed to entrapped air within the unsaturated portion of the aquifer and the encroachment of the groundwater mound onto the screened interval. Figure 10 displays recharge rates of this well.
Figure 7. Inflow Rate at Atkinson Recharge Pit.

Figure 8. Infiltration Rate and Water Depth at Atkinson Recharge Pit.
Figure 9. Inflow Rate at Atkinson Unsaturated Recharge Well.

Figure 10. Elevation of Water in Well Column at Atkinson Unsaturated Recharge Well.
O'Neill Site. The well at this site did not recharge as anticipated. Bentonite coatings were found on sand grains in this area, and it is thought the augering performed during well construction caused a sealing of the bore hole walls. The recharge rate during the first sustained time period was 1 gpm per foot of head in the well column. At the 1900-hour time period, the rate was 2 gpm per foot of head. The original low rate is considered primarily to be the result of the original augering causing a sealing of the hole side wall by the smearing of the bentonite coatings on the sand particles. The "improvement" noted during the test period was the result of the many filling and emptying cycles associated with the numerous shutdowns experienced by the O'Neill source well.

Saturated Recharge Wells

The saturated recharge wells were drilled by reverse circulation rotary methods using an 8-inch PVC spiral screen with 0.040 inch slot installed within saturated Pleistocene sand and gravel. Forty feet of screen was installed at the Atkinson site and 35 feet at the O'Neill site. The top of the screen was at the groundwater level. A washed and graded gravel pack material was placed around the screen. The well was developed by use of a surge block. Each well was pumped for one hour to determine a yield/drawdown relationship, post recharge pumping of the wells was not performed.

The design of the wells utilized a drop pipe or injection tube to discharge the water below the water table. The pipe was sized to maintain a full flow in the tube to eliminate cascading of water and the possibility of air entrainment (Smith, 1980).

Atkinson Site. The yield test at this saturated recharge well indicated 52 gpm/foot of drawdown. The recharge rate after 100 hours of testing was 48 gpm/foot of head in the well column, and after 3100 hours (end of test), was 21.2 gpm/foot of head. The total rise in the recharge well column was 6.7 feet and the closest observation well, which was located 10 feet away recorded a groundwater level rise of 2.9 feet of which 1.1 feet is attributed to the artificial recharge, and 1.8 feet is attributed to regional groundwater rebound.

O'Neill Site. The yield test for the recharge well produced a value of 42 gpm/foot of drawdown. After 100 hours of testing the recharge rate was 32 gpm/foot of head, after 720 hours the rate was 13.3 gpm/foot of head, and after 2100 hours, the recharge rate was 6 gpm/foot of head. Figures 11 and 12 illustrate the performance of the O'Neill saturated recharge well. The closest observation well was 12 feet from the recharge well and it recorded a total rise of 7.8 feet after 2800 hours of testing, of which 6.0 feet is attributed to artificial recharge, while the head or water column in the well rose 56.6 feet. At 2000 hours, a stable inflow and column water level was achieved and maintained until the end of the test.

Entrained air in the recharge water is considered a prime suspect in the permeability reduction. An additional factor for the well performance was the source well pumping at 90 psi with a gate valve used to restrict the flows. The water discharged at the recharge pit and the recharge line was
Figure 11. Inflow Rate at O'Neill Saturated Recharge Well.

Figure 12. Elevation of Water in Well Column at O'Neill Saturated Recharge Well.
noticeably aerated, which is thought to be a result of the combination of the high pressure and the flow around the partially closed gate valve at the well head.

SUMMARY

The groundwater recharge facilities were continually operated from October 1989 to February 1990 for a total of 126 days. During this period, a total of 351 acre-feet of recharge was achieved in the O'Neill area and 476 acre-feet of recharge was achieved in the Atkinson area.

A summary of the recharge data collected from each site is as follows:

<table>
<thead>
<tr>
<th>Table 1</th>
<th>Summary of Recharge Results, Atkinson and O'Neill Sites</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Recharge Rate (GPM) (a)</td>
</tr>
<tr>
<td>Atkinson Area</td>
<td></td>
</tr>
<tr>
<td>Recharge Line</td>
<td>232</td>
</tr>
<tr>
<td>Recharge Pit</td>
<td>262</td>
</tr>
<tr>
<td>Unsaturated Well</td>
<td>117</td>
</tr>
<tr>
<td>Saturated Well</td>
<td>134</td>
</tr>
<tr>
<td>O'Neill Area</td>
<td></td>
</tr>
<tr>
<td>Recharge Line</td>
<td>163</td>
</tr>
<tr>
<td>Recharge Pit</td>
<td>157</td>
</tr>
<tr>
<td>Unsaturated Well</td>
<td>66</td>
</tr>
<tr>
<td>Saturated Well</td>
<td>301</td>
</tr>
</tbody>
</table>

(a) Average Flow Rate, Gallons Per Minute (GPM)
(b) Observation Well Radius Distance from Recharge Point in Feet

Each of the recharge features provided some degree of success. The overall project concept for the O'Neill Unit remains to be studied for economics and large scale feasibility. The results of this testing indicate the importance of site selection based on investigations, as is the case of the bentonitic sands, and the operational problems associated with exposed water surface recharge methods. The recharge methods associated with the underground facilities, i.e. wells and recharge lines, did provide sufficient success to warrant further consideration in any future planning of this project. Questions concerning the longterm success over the designed project life (40 years) remain unanswered.

The recharge testing points out the potential operational problems that may be encountered with a large scale recharge project. If the benefits obtained can be justified economically, each method of recharge indicates a potential for large scale usage. Site specific designs will be required for each recharge facility.
ACKNOWLEDGEMENTS

The work summarized in this paper has been supported by the North Central Nebraska Reclamation District. The authors acknowledge and appreciate their support, especially from Mr. Alfred Drayton, President of the Reclamation District for his assistance and contributions toward this project.

REFERENCES


IMPLEMENTATION OF
"THE HIGH PLAINS STATES' GROUNDWATER
DEMONSTRATION PROGRAM ACT of 1983"

Richard O. Lasson, P.E.
U. S. Bureau of Reclamation, Salt Lake City, Utah

ABSTRACT

The Bureau of Reclamation (Reclamation) has been given the responsibility of implementing a series of groundwater recharge demonstration projects under Public Law 98-434, known as "The High Plains States' Groundwater Demonstration Program Act of 1983." The projects will demonstrate the effectiveness and viability of various recharge techniques. Reclamation was directed to work cooperatively with Federal, State, and local entities in gathering and documenting the resulting effects from each of the groundwater recharge demonstration projects. Proposals were received from local sponsors who would undertake activities such as testing local recharge potentials while monitoring the effects of the recharge water on the groundwater quality and quantity in order to provide a reliable water supply to meet future water demands. This paper will address the implementation of this program with emphasis on two deep-well injection projects.

INTRODUCTION

The High Plains States Groundwater Demonstration Program Act of 1983 (the Act), Public Law 98-434 (Groundwater Recharge Program), authorized and directed the Secretary of the Interior, acting through the Bureau of Reclamation, to investigate and establish demonstration projects to recharge groundwater in the High Plains States of:

- Colorado
- Kansas
- Wyoming

and the other Reclamation Act States of:

- Oklahoma
- Texas
- New Mexico
- South Dakota
- Nebraska
- Arizona
- California
- Nevada
- Utah
- Idaho
- North Dakota
- Washington
PROGRAM OBJECTIVES

The primary objective of this program is to move from the research mode on groundwater recharge into the pilot demonstration phase in order to lay the groundwork for larger operational programs in the future. Other program objectives are:

- to emphasize groundwater recharge rather than conjunctive use, conservation, and management of existing supplies;

- to use local surface water supplies to recharge nearby aquifers. Interbasin transfer of water was specifically excluded by the legislation. The underlying theme of the Congressional debate on the Act was "small and local;"

- to determine whether groundwater recharge is a more economically practical and environmentally safe way to store water for future use than surface storage;

- to address the impact of groundwater recharge on surface and underground water quality;

- to examine the institutional and legal aspects of groundwater recharge. In many situations, groundwater recharge may be technically feasible, but, legal and institutional constraints such as permits, zoning and State water laws may limit its implementability; and

- to test a wide variety of recharge technologies that may be applied to diverse geologic and hydrologic conditions.

COOPERATING AGENCIES

The Bureau of Reclamation was directed by the Act to consult with the U.S. Geological Survey (Survey) and the Environmental Protection Agency (EPA) as well as with the States in the development, evaluation and selection of participating projects. In addition, EPA was directed to provide an evaluation of the impacts on surface water and groundwater quality resulting from the various demonstrations. Although not specified in the Act, coordination has also been carried out with the U.S. Fish and Wildlife Service (FWS) and State Fish and Game agencies to assure that any adverse impacts on fish and wildlife resources will be mitigated.
PROGRAM DEVELOPMENT

This program is being carried out in two phases. Phase I was a 2-year period allocated by Congress for the planning development, and site selection of recommended projects. The Act stated that not fewer than 12 demonstration sites be selected in the high-plains states and not fewer than 9 sites be selected in the other 17 western states. Phase I culminated in December 1987 with a report to Congress recommending 21 projects for construction.

Phase II is a 5-year design, construction, operation, and evaluation phase. Reclamation will make annual reports to Congress on the progress made under Phase II and will submit a final report at the end of the program.

Funding

Congress appropriated $20 million for Phase II and authorized Reclamation to cost share with State or private entities who would pay no less 20 percent of the project costs.

Solicitation of Proposals

During Phase I, Reclamation worked with representatives appointed by the Governors of the 17 Western States to solicit and evaluate groundwater recharge proposals. After each Governor reviewed and prioritized each proposal from his state, all proposals were submitted to Reclamation.

Reclamation received a total of 41 proposals representing a wide range of types of groundwater recharge projects. Some involved using existing recharge projects and increasing the facilities for recharge and/or monitoring. Others used existing conveyance facilities to transport surface water to potential recharge sites; still others proposed using existing pits or ponds to store recharge water supplies and existing or abandoned wells to inject recharge water supplies into the aquifers.

The majority of the proposals, however, called for the construction of new facilities. These facilities included channel diversion structures, retention dikes and gates, flushable gravel filters, sediment ponds, dual-purpose (injection and extraction) wells, monitoring systems, spreading mechanisms, percolation ponds, underground barriers, and shallow dry wells. Several ways to obtain the necessary water supplies were proposed. Some proposals were based upon accumulation of snow and subsequent melting, some proposed use of excess spring runoff, and some the use of treated effluent. Exchanges of water to obtain a recharge supply were also considered. Some proposals took advantage of fluctuations in seasonal water supply or demand to obtain water for groundwater recharge demonstration purposes.

In addition to the objective of increasing aquifer supplies, some proposals would evaluate the reduction of or stabilization of
land subsidence through injection of water into underlying aquifers. Other proposals would reduce salt-water intrusion into aquifers through the use of injection wells.

**Evaluation of Proposals**

A technical evaluation process was developed jointly by Reclamation, USGS, and EPA. The evaluation process included a USGS evaluation of the hydrologic and geologic aspects of the proposals including their monitoring plans, an EPA review of the plans for monitoring and evaluating the impact on general water quality, and a Reclamation evaluation of the engineering, economic, environmental, and legal aspects of the proposals.

**Initial Screening.** Each proposal was subjected to the following initial screening criteria based on requirements specified in the Act:

- A declining water table
- An available surface water supply
- A minimum of 20 percent non-Federal cost sharing
- No serious environmental problems
- Public acceptability of the proposal
- Priority from the Governor

**Technical Evaluation.** Following initial screening, each remaining proposal received a full technical evaluation by Bureau of Reclamation regional evaluation teams with input from the USGS and the EPA. This evaluation was based on the following factors using a weighted-point system:

- Geohydrologic feasibility
- Engineering feasibility
- Cost Estimate (the quality of the estimate itself)
- Legal access
- Monitoring plan
- Rehabilitation plan (is there an acceptable plan for permanent use of the facilities).
- Cost sharing commitment
- Total cost
- Legal and institutional issues
- Environmental issues
- Uniqueness

**Environmental Evaluation.** Due to the relatively short time available to receive and evaluate proposals and to develop a recommended plan to present to Congress, there was no time available for the preparation of time-consuming, environmental impact statements (EISs). Consequently, it was imperative that only environmentally sound proposals be selected. To insure this, environmental issues was included as both a major screening factor and as one of the 11 evaluation factors. Eventhough this precaution was taken, many of the proposals still had unanswered questions regarding possible environmental impacts. Accordingly,
a separate, environmental compliance review was conducted. Each proposal was ranked according to the anticipated complexity of achieving environmental compliance requirements. The following rankings were developed for this evaluation:

- **Category 1** Proposals with a high potential for significant environmental impacts - would require preparation of an EIS.
- **Category 2** Proposals appearing to have significant impacts due to size, nature, or potential impacts on drinking water supplies - would require additional data.
- **Category 3** Proposals lacking information but would generally minimize impacts - would demonstrate no significant impacts with some additional data and consultation.
- **Category 4** Proposals for which a finding of no significant impact could be demonstrated with existing data.

**Plan Selection.** After each project was evaluated and scored according to both the technical evaluation criteria and the separate environmental evaluation, an overall plan was developed. In selecting projects to be included in the final recommended plan, the following four overall objectives were considered:

1. Technical merit
2. Responsiveness to the intent of the Act
3. Balance in types of recharge projects
4. Uniqueness
5. Environmental Clearance Requirements
6. Requirements of the Act
   - At least 12 projects in the 8 High Plains States
   - At least 9 in the remaining Reclamation Act States
7. Cost Ceiling Constraints
   - Cost ceiling of $20 million for Phase II (1983 prices)

The process for selecting projects for the recommended plan considered both quantitative and qualitative or judgmental factors. These four overall objectives had to be balanced and traded off where all four objectives could not be achieved simultaneously. Often constraints imposed by one objective had a restrictive effect on proposals that would otherwise be selected based on other objectives. The overriding objective was to select a minimum of 21 demonstration projects which were the most technically sound, environmentally safe, and which would contribute the most toward proving new and innovative groundwater recharge technology.

The proposals received ranged in cost from $80,000 to $3,262,000. While the cost ceiling of $20,000,000 was not a target, it was a
constraint. Even though some proposals were very highly rated, the cost of their inclusion could preclude a number of other desirable projects. It was not feasible to quantitatively trade off the specific technical merits of one large proposal versus a number of smaller proposals. Therefore, judgment was used in making the selections by considering the objectives of attaining a balance between recharge techniques, and consideration of institutional, geographic and hydrogeologic, and climatic settings.

In the final selection, only those projects, which could reasonably be expected to be environmentally sound at the time the final Phase I report was prepared, were selected. Consequently, some projects, which might significantly contribute to groundwater recharge information were dropped from consideration because of complex or time consuming environmental issues, even though they had a high probability of eventually receiving all environmental clearances.

In the end, 21 projects in 15 States--12 in the High Plains States and 9 in the other Western States were selected and proposed to Congress for construction. Three of the States included in the Act, North Dakota, Wyoming, and New Mexico, do not have projects in the recommended plan. North Dakota did not submit any proposals; Wyoming, a High Plains State, submitted one proposal, however, it was rated technically deficient; and New Mexico submitted a proposal which was originally included in the recommended plan, but was later withdrawn by the sponsor.

National Environmental Policy Act (NEPA) Compliance

Following final plan selection, and after consulting with EPA and the Fish and Wildlife Service, a Finding of No Significant Impact (FONSI) covering all 21 projects was prepared and approved. In this FONSI, nine projects were contingent upon special mitigation features being included in project implementation. Commitments for these features are, or will be, included in the cooperative cost-sharing agreements between the sponsors and Reclamation for these projects.

GROUNDWATER PROTECTION STRATEGY

EPA Groundwater Protection and Monitoring Policy

To assure that the Groundwater Recharge Program would protect human health and the environment and comply with all applicable regulations, EPA issued on January 26, 1989, its groundwater monitoring policy (Mlay and Cook, 1989) for the program. This policy provides guidance to project sponsors on EPA's groundwater protection goals, on determining the hydrogeologic framework, on selecting constituents for monitoring, on baseline data collection, and on monitoring frequencies throughout the program.
As explained in this policy, compliance with either one of the two following conditions will ensure that no endangerment of a USDW would occur in the Groundwater Recharge Program.

1. No endangerment would occur if constituent concentrations in the groundwater at the point of injection do not exceed the National Primary Drinking Water standards (i.e., maximum contaminant levels [MCLs]) promulgated in 40 CFR Part 141, or EPA recommended health based limits which have been peer reviewed by EPA, such as by health advisories.

2. Where such standards are already exceeded due to activities not related to the Groundwater Recharge Program, no endangerment would occur if constituents in the injectate do not exceed ambient concentrations in the groundwater.

The Groundwater Recharge Program assures compliance with this policy by monitoring the injectate and only injecting fluids which meet the above mentioned standards. In order to meet the standards, fluids may have to be treated or surface retention basins used to avoid injection of concentrated "slugs."

**Compliance with Regulations**

One concern of the Groundwater Recharge Program is to assure that no underground drinking water supplies are endangered as a result of the program. This concern is addressed through strict compliance with regulations. Under the authority of the Safe Drinking Water Act (SDWA), EPA has promulgated minimum requirements for effective Underground Injection Control (UIC) programs. The UIC programs are either carried out by the States, or directly implemented by EPA. The UIC regulations are designed to prevent endangerment of an underground source of drinking water (USDW) from underground injection. The regulations prevent endangerment of USDW from underground injection by prohibiting movement of fluid containing any contaminant into a USDW if the presence of that contaminant may cause a violation of any primary drinking water regulation or may otherwise adversely affect the health of persons. Approximately half of the demonstration sites are expected to use injection wells as part of the recharge technology employed.

Other statutory provisions relevant to the Groundwater Recharge Demonstration Program include:

- additional portions of the Safe Drinking Water Act, the Clean Water Act and the National Environmental Policy Act (NEPA),

- state Wellhead Protection Programs,
emergency powers to prevent imminent and substantial endangerment to human health from any contaminant that is likely to enter USDW,

discharges to surface waters (and to groundwaters in some very limited circumstances) under the National Pollutant Discharge Elimination System,

dredge and fill operations within navigable waters (e.g., for project construction or maintenance), and

review authority by EPA over certain Federal projects (NEPA section 309).

In administering Federal statutes, states often apply standards which are more stringent that the Federal "baseline." In addition, many states administer their own laws which cover activities not controlled by Federal laws.

### Impacts on Water Quality

Reclamation has entered into a Memorandum of Understanding (MOU) with EPA to provide an evaluation of impacts on surface and groundwater quality resulting from the groundwater recharge demonstration projects. Under the terms of the MOU, EPA will, in consultation with USGS, make maximum use of data, studies, other technical resources, and assistance available from State and local entities in conducting an evaluation of water quality impacts. EPA's final report will be included in the Secretary of the Interior's final report to Congress.

### INTEGRITY OF DATA

In order to insure that data generated on all projects in this program is of a known and generally high quality so that conclusions regarding the impacts of recharge activities on groundwater and surface water quality and so that recommendations made concerning future recharge projects can be based on sound scientific evidence, a strict quality control/quality assurance (QA/QC) program has been instituted for all data collected. In the Spring of 1989 EPA conducted QA/QC training for all project sponsors and is providing ongoing assistance in the development and review of QA/QC activities for each project. Each EPA region is authorized to approve the QA/QC programs of each project sponsor. They are also conducting audits of the laboratories performing the analytic work to assure that all analysis and testing is performed in accordance with established standards in order to meet the specific needs of the program.
ECONOMIC AND INSTITUTIONAL STUDY

Reclamation was directed by the Act to contract with the States to study ways to "...identify and evaluate alternative means by which the costs of groundwater recharge projects could be allocated among the beneficiaries of the projects...and identify and evaluate the economic feasibility of and the legal authority for utilizing groundwater recharge..." in water development projects. To accomplish this task, Reclamation entered into a cooperative agreement with the Western States Water Council to study the economic and institutional aspects of groundwater recharge. The Council, whose members are appointed by the Governors of the seventeen Western States, provides a forum for the states to operate in the evaluation of water resource development and management issues from a State perspective. The Council, which uses an effective organization of State committees, is uniquely suited to conduct this study and to bring a local and State perspective to the study.

The study is composed of two parts:

Part one is devoted to an overview of methodologies for evaluating the benefits of groundwater recharge, the identification of cost allocation and cost sharing procedures, and a state-by-state summary of the legal and institutional settings for groundwater recharge. This study was completed by the Council in 1990.

Part two will be a series of case studies of a select number of the 21 demonstration projects. Each case study will address project benefits, costs, cost sharing, and legal and institutional aspects.

APPROPRIATION CEILING

As mentioned previously, the cost ceiling specified in the Act for Phase II was $20,000,000 based on October, 1983 price levels. In the Phase I Report to Congress, the cost of the recommended plan to construct 21 projects was estimated at $18,520,400 based on preliminary plans and cost estimates submitted by local sponsors and September 1986 price levels. This was within the authorized $20 million cost ceiling and even allowed for contingencies for future cost escalation due to inflation or other changes in program costs.

However, during the detailed planning for the demonstration projects, extensive environmental monitoring systems, QA/QC programs, and environmental protection features were added to the program in order to comply with groundwater protection goals and data quality objectives. Environmental monitoring greatly increased the cost of the program for such activities as: installing numerous systems of monitoring wells; gathering of baseline data to be certain demonstration project results are
reliable; and, testing for the presence of classic organic and inorganic contaminants, volatile organic, and radioactive contaminants.

Furthermore, during 1989, three projects withdrew from the program and were replaced by three new projects which were significantly more expensive than the original three projects. The above factors, combined with the changes in costs due to inflation, have resulted in a current estimate of $31,000,000.

To alleviate this problem and to allow all 21 proposed projects to be completed, a process was begun in 1989 to obtain congressional approval for an appropriation ceiling increase to $31 million and to permit indexing for changes in costs due to inflation.

Since the current cost ceiling will not allow the completion of all 21 projects and since the 11 cooperative agreements currently in place total $13.7 million, only $6.3 million is available for construction of the 12 remaining projects. Therefore, some, but not all of the remaining projects can still be implemented under the $20 million ceiling. Reclamation has adopted the policy that, once started, projects should be fully funded to completion and that those projects that are first brought to the point of implementation with all clearances obtained, will be started if project completion will not cause the appropriation ceiling to be exceeded. Those not funded within the ceiling will be deferred pending the enactment of legislation to increase the appropriation ceiling.

REPORTS

During Phase II, annual reports detailing progress and preliminary findings will be submitted to Congress. The final Phase II Report to Congress will include:

- a detailed evaluation of the demonstration projects,
- the results of the economic and institutional studies,
- specific recommendations regarding the location, scope, and feasibility of operational groundwater recharge projects to be constructed and maintained by the Bureau of Reclamation,
- an evaluation of the feasibility of integrating groundwater recharge projects into existing Reclamation projects, and
- a report by the Environmental Protection Agency on the impacts of the Groundwater Recharge Program on surface and groundwater quality.
TWO GROUNDWATER RECHARGE PROJECTS

Two of the 21 projects included in the Recommended Plan are under the area of Responsibility of the Reclamation's Upper Colorado Region, headquartered in Salt Lake City, Utah. These are the Southeast Salt Lake County Recharge Project in Salt Lake City, Utah and the Hueco Bolson Recharge Project near El Paso, Texas. Both projects are among the 11 now in the implementation phase.

Southeast Salt Lake County Recharge Project

The Southeast Salt Lake County Recharge Project is sponsored by the Salt Lake County Water Conservancy District who is a municipal water supplier serving a population of over 450,000 people. This $3,130,000 project began implementation in 1990 and is being 50 percent cost-shared by the sponsor.

Project Description. The primary concept of this project is to provide temporary underground storage of water by creating a groundwater mound during the winter months that would be utilized during the summer peak demand months for municipal water. If this concept is successful in meeting peak summer demands and if it proves to be technically, economically and environmentally feasible, additional projects could be implemented by the sponsor as an alternative to enlarging the aqueduct capacity or to building additional surface storage in this heavily populated area.

The project would inject approximately 1,430 acre-feet of water annually from Deer Creek Reservoir into the east bench of the Salt Lake Valley during the low-use 6 month period of mid-October through mid-March of each year. The water will then be recovered for municipal use each year during the summer period of June through August. The water will be delivered to the injection site via the Salt Lake Aqueduct, which conveys flows up to its capacity of 170 cubic-feet per second (cfs) during the summer, but conveys only a base flow of about 20 cfs during the winter. Chlorine is currently introduced at the Salt Lake Aqueduct intake at Deer Creek Reservoir some 30 miles upstream as an oxidant for taste and odor control, and to prevent bio-fouling of the aqueduct interior.

Two injection wells, two recovery wells, and three monitoring wells will be constructed. Two of the three monitoring wells are incorporated into the two downgradient recovery wells. Other existing wells will also be used for recovery and monitoring purposes.

The project will test the injection of both treated and untreated water. One well will inject water directly from the aqueduct upstream of the Little Cottonwood Treatment Plant. Prior to injection, a small water treatment and filtration facility which will be constructed adjacent to the injection well, will reduce the particle loading of the raw water and produce treated water.
which meets drinking water standards prior to injection. The receiving aquifer has high quality water and excellent hydraulic characteristics.

The second injection well is downstream of the Little Cottonwood Treatment Plant and will inject conventionally treated drinking water. The project is now under construction and initial injection is scheduled for November 1991.

The sponsor anticipates a recovery rate of about 1,588 acre-feet annually. A recovery of about 90 percent of the physical water injected, together with additional native groundwater, is expected. Recovered water will be distributed through the existing distribution system owned by the sponsor.

Special Investigations. Due to the size of investment and the importance of this aquifer to the metropolitan Salt Lake City area, the sponsor is taking a very cautious approach to this recharge project. Project implementation is being preceded with several special studies and investigations.

- A pilot water treatment plant was constructed, and a pilot study performed to determine the optimum process and chemical dosages for treating the surface water prior to injection. That study recommended an in-line filtration process with ultra-violet radiation as the mode of disinfection.

- A preliminary hydrogeologic review and study was performed by a consultant with previous experience in artificial recharge/aquifer storage and recovery projects. This study recommended testing of a dual purpose injection/recovery well and careful attention to be given to disinfection in order to prevent microbial growth in the injection wells. A second phase of that study including a review of the first injection well drilling results and geochemical modeling to determine compatibility of surface and groundwaters in underway.

- A pilot injection test was performed by the sponsor to determine the potential for injection well plugging. The test was performed in one of the sponsor’s existing municipal production wells by inserting an injection tube. Treated water from a conventional treatment plant was injected for 30 days. The results indicated no occurrence of well plugging.

- Prior to actual injection in November 1991, the sponsor will conduct an in-house review including Reclamation, EPA, and USGS personnel to verify
that the groundwater injection plan will have no harmful effects on this important aquifer.

**Monitoring and Quality Assurance.** A comprehensive 4-year monitoring program will be conducted to determine the feasibility and efficiency of the recharge project and its effect on the receiving aquifer. Ground and surface water monitoring occurred in 1984 and 1985 to assess surface groundwater interactions and compatibilities. The two water types appear to be very similar in quality and therefore are believed to be compatible. Four existing municipal wells were sampled, and Salt Lake Aqueduct water quality samples were averaged over a 7 year time frame (1978-1985). The analyses were conducted mainly for cations and anions.

The ground and surface waters appear to meet primary and secondary drinking water standards for cation and anions. Although the average total dissolved solids concentration of the recharge water is well within secondary drinking water standards at 238 mg/l, it exceeds the average groundwater total dissolved solids of 174 mg/l.

**Permits.** The Utah Bureau of Water Pollution Control has issued an underground injection control (UIC) permit to the sponsor, based upon its project development plan. The project development plan and the monitoring and QA/QC plans have also been reviewed and accepted by the State Bureau of Water Pollution Control and the Utah Bureau of Drinking Water/Sanitation.

**Legal and Institutional Issues.** Since this project was initiated, the sponsor has experienced greater than anticipated demands in meeting institutional requirements. These are explained below:

- The State Engineer does not have existing State laws to address artificial groundwater recharge. Regulatory agencies are also struggling with how to regulate such a project. The sponsor is required to spend far greater staff time in addressing regulatory issues than originally anticipated.

- Local city zoning requirements have been strict. The dealings with the local city have required much more staff time than originally anticipated. Sandy City is currently requiring a site landscaping plan that exceeds the level of effort anticipated. Additional zoning requirements have made the acquisition of injection sites very difficult and time consuming.

- The sponsor has requested guidance from EPA on whether drinking water MCL’s for bacteria must be observed in the injectate water. This question
also pertains many other projects in the Groundwater Recharge Program.

- The Utah Bureau of Drinking Water/Sanitation may require the disinfection of recovered water, depending on water quality.

**Hueco Bolson Recharge Project**

The Hueco Bolson Recharge Project is sponsored by the El Paso Water Utilities Public Service Board (El Paso Water Utilities) who provides municipal water supplies to the City of El Paso, Texas. This $632,000 project which began implementation in 1989 is being cost-shared 20 percent by the sponsor.

**Project Description.** This project provides a unique opportunity to further the state-of-the-art of groundwater recharge by documenting the results of the Hueco Bolson Recharge Project, which the El Paso Water Utilities has been operating since 1985. The recharge system includes the 10 million gallon per day Fred Hervey Water Reclamation Plant, a pipeline system and 10 injection wells. All sewage collected in the northeast area of the city of El Paso is currently pumped to the treatment plant and then into the injection system. Injection into the Hueco Bolson, a 10 million acre-foot freshwater aquifer, occurs between existing production wells. Wells are spaced and flow controlled so that residence time in the aquifer will be at least 2 years prior to production. No treatment of produced water other than chlorination is provided. The demonstration program will document the water injection operations currently underway and will correlate the observed data with current analytical techniques available to predict performance.

**Monitoring and Quality Assurance.** The water quality injected over the study period will be monitored for suspended and dissolved constituents, both organic and inorganic. The operation of the surface facilities will be studied to document any problems with air entrainment or control which may be correlated with injection well performance. Injection well operating data will be obtained and studied to determine the plugging rate. Optimization of backwash frequency will also be an objective of the demonstration program.

Operational problems such as injection flow rate control and corrosion will be closely monitored and data from existing observation wells will be collected and analyzed. Water levels in the well bores will also be monitored and correlated with injection water quality and quantity. Water quality parameters in the aquifer will be monitored at the production wells over the project time frame to document travel time and a three-dimensional solute model of the aquifer will be calibrated with the field data.
Project Expansion. This project was recently expanded to include a special merit-funding research proposal sponsored by USGS and cost-shared by the Texas Water Development Board, El Paso Water Utilities, and Reclamation, in order to investigate the fate and movement of trihalomethane compounds (THMs), a group of carcinogens associated with wastewater injection, into the Hueco Bolson aquifer.

Before injection, wastewater from the Fred Hervey Water Treatment Plant is treated to meet or exceed EPA primary drinking water standards using a combination of techniques including: (1) primary clarification, (2) biological treatment under aerobic and anaerobic conditions with granular activated carbon (GAC), (3) lime treatment, (4) ozone disinfection, (5) pH-adjusted filtration through GAC and finally (6) chlorination (Buszka, 1990).

The final chlorination step is deemed necessary to reduce the possibility of pathogenic bacterial occurrence in the treated wastewater. This process, however, can create appreciable concentrations of THM. The EPA mandates that the sum of concentrations of all THM compounds not exceed 100 micrograms per liter (µg/L) in drinking water used for public supply. Concentrations of the sum of THM compounds produced in the wastewater during final chlorination which was used for injection at the Hueco Bolson Recharge Project during 1985 to 1988 ranged from 10 to 26 µg/L, well below the EPA standard of 100 µg/L. As of the end of 1988, groundwater analyses from the observation and production wells at the site had not detected any of the THM compounds (Buszka, 1990).

Even though these test results appear encouraging, an understanding of the biological and chemical processes affecting THM compounds after their injection into an aquifer is important in order to define their environmental persistence and in order to maintain proper management of the water supply. The objectives of this special investigation are consistent with the Groundwater Recharge Program and the results will be included in Reclamation's final report to Congress.

REFERENCES


NONUNIQUE SIMULATIONS OF THE QUALITY OF WATER RECOVERED FOLLOWING INJECTION OF FRESHWATER INTO A BRACKISH AQUIFER

Michael L. Merritt, U.S. Geological Survey, Miami, Florida

ABSTRACT

Between 1975 and 1980, demonstration testing of subsurface storage and subsequent recovery of freshwater took place near Miami, Florida. The possibility of storing wet-season excess waters in the brackish artesian formations of southeastern Florida has attracted growing local interest in subsequent years, as demand for municipal water supply has increased and options for development of new sources have diminished. Because of this interest, data collected during the 1975-80 test period were subjected to rigorous analysis that included computer simulation of processes that produced the observed changes in water quality.

Although the tests included an extensive and diverse data-collection phase, some physical and hydraulic characteristics of the aquifer and flow system needing representation for simulation purposes could not be assigned parameter values known to be quantitatively precise. These included flow-zone thickness and effective porosity and the degree of flow anisotropy. Because of such uncertainties, the aquifer test data and recovered water-quality changes were simulated in several ways using different values of these parameters with corresponding sets of other parameter values determined by calibration.

Changes in quality of recovered water generally were simulated in a calibration procedure that consisted of varying dispersivity coefficients, molecular diffusivity, and the rate of regional aquifer flow. Because the estimated hydraulic conductivity and regional flow velocity were not consistent with published estimates of the regional hydraulic gradient, an additional simulation was calibrated that assumed a different value of hydraulic conductivity. Each simulation was fully successful in matching observed data using a particular set of calibration coefficients.

Each set of parameter estimates and corresponding calibration coefficients was used for predictive estimates of the recovery efficiency that could be achieved in a period of 10 annual cycles. Results of all predictive simulations were similar, demonstrating that predictive consistency depended on the accuracy of simulation of the measured data rather than on the precise identification of aquifer characteristic values. The result helped enhance the credence of the predictive simulations.
INTRODUCTION

The population growth of the southern tip of peninsular Florida, in the extreme southeastern United States (fig. 1), has raised concerns about the future adequacy of current sources of local water supply. Ways of augmenting or conserving existing supplies are being investigated. One proposed conservation method, the storage of surplus freshwater in underground formations containing native brackish water, takes advantage of the seasonal pattern of precipitation in this semitropical region. Permeable artesian zones suitable for this purpose occur throughout the study area (fig. 1) within the Upper Floridan aquifer. Summer wet-season surplus could be stored for recovery and use during the winter dry season.

The U.S. Geological Survey and regional and local agencies cooperated in an operational feasibility study to test the concept at a test site at Hialeah, near Miami, Fla. The field study, which ran from 1974 to 1980 under the direction of F.W. Meyer (USGS, retired), provided an extensive data set that could support sophisticated computer analysis. These data were used for a simulation of the processes of injection, storage, and recovery at the test site. The model representation was calibrated to replicate the increases in salinity of the recovered water observed during the three recovery phases, thereby also replicating the observed "recovery efficiencies." Recovery efficiency is defined as the volume of potable water recovered expressed as a percentage of the volume of freshwater injected. Water generally is considered nonpotable in the United States when the chloride concentration exceeds 250 mg/L, though more saline water is used in arid regions. In this study, the volume of recovered water with chloride concentration less than 250 mg/L was used to compute recovery efficiency.

Figure 1.—Location of Florida and region of Florida where the Upper Floridan aquifer contains nonpotable water.
SITE ACTIVITIES

To determine the nature of subsurface geologic and hydrologic conditions and to provide a means for the subsurface emplacement of freshwater in suitable formations containing native brackish water, two wells were drilled in late 1974 at the Hialeah Water Treatment Plant. The well used for injection was completed with 14-inch steel casing extending to a depth of 955 feet. A nominal 12-inch hole was then drilled to a depth of 1,105 feet. The open-hole interval penetrated beds of carbonate rocks of Oligocene and Upper Eocene age that included part of the Upper Floridan aquifer. An observation well was drilled at a distance of 289 feet from the injection well, and had an open-hole interval from 953 to 1,064 feet. Water for injection was obtained by extending a suction line to a water-supply well in the surficial aquifer. The surficial aquifer is vertically separated and hydraulically isolated from the injection zone by about 800 feet of relatively impermeable clastics.

In early 1975, an aquifer test was performed in which the observation well was pumped for 100 minutes at a rate of 250 gallons per minute while pressure was measured in the injection well. The purpose was to determine the transmissivity of the injection zone, data that would be helpful in evaluating results of the operational tests. Three cycles of injection, storage, and recovery with successively larger injection volumes and longer storage periods were performed between mid-1975 and early 1980, with the following results that have been previously published (Merritt and others, 1983; Meyer, 1989):

<table>
<thead>
<tr>
<th></th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>* Quantity injected (gallons x 10^6)</td>
<td>41.9</td>
<td>85.0</td>
<td>208.0</td>
</tr>
<tr>
<td>* Storage period (days)</td>
<td>2</td>
<td>54</td>
<td>181</td>
</tr>
<tr>
<td>* Quantity of potable water recovered (gallons x 10^6)</td>
<td>13.8</td>
<td>40.7</td>
<td>80.1</td>
</tr>
<tr>
<td>* Recovery efficiency (percent)</td>
<td>32.9</td>
<td>47.8</td>
<td>38.5</td>
</tr>
</tbody>
</table>

Recovery was from the injection well by natural artesian flow. During the recovery parts of each cycle, periodic measurements of the increasing chloride concentration of the recovered water were obtained and tabulated with corresponding values of the volume of water recovered.

THE DIGITAL SIMULATOR USED IN THE STUDY

The simulator used was a version of the SWIP code (INTERA Environmental Consultants, Inc., 1979) that has been slightly modified by the author. This block-centered finite-difference code solves simultaneous coupled equations for flow and transport of solute and thermal energy. A two-fluid approach is used for solute-transport simulation in which each fluid is assigned a characteristic density value. Fluid density and viscosity vary...
spatially and temporally as a function of pressure, temperature, and fluid composition. Because absolute pressure is the independent variable of the flow equation, density-driven flow phenomena, such as buoyancy stratification, can be simulated. The code is based upon a porous media concept. In this study, however, a carbonate layer with secondary (solution) porosity was treated as an equivalent porous media. The advective and dispersive processes depicted by the model were assumed to be sufficiently correct on a spatially averaged basis at the scale of the volumes of water displaced by the injected freshwater to permit an accurate representation of the mixing processes that caused the salinity of recovered water to show a gradual increase.

MODEL CALIBRATION

The 150-foot open part of the well was discretized into 12 layers, of which the middle 6 represented the 12-foot receiving zone. The layers had a 43 x 31 lateral discretization. Some parameter assignments were based on examination of data, including descriptions of rock samples, chemical analyses of water samples, and geophysical logs. These included the receiving-zone thickness of 12 feet and porosity of 35 percent, and the density of the native water in the receiving zone. A value for the latter was chosen based on the measured chloride and dissolved-solids concentrations, 1,200 and 2,700 milligrams per liter, respectively. The assumption that a single 12-foot section of the 150-foot open-hole interval received most of the injected water was primarily based on examination of 18 flowmeter logs run during the course of the operational tests. A typical example is illustrated in figure 2. The formation porosity value was chosen based on a neutron porosity log (fig. 2) that was compensated for borehole wall irregularities.

None of the water samples collected as part of the study were considered to represent the water quality of the confining beds, although samples from 840 feet had a chloride concentration of about 2,000 milligrams per liter. On this basis, the confining beds were assumed to have chloride and dissolved-solids concentrations of about 2,700 and 6,000 milligrams per liter, respectively, more than twice that of the receiving zone. This conceptual model was considered to reflect flushing of the permeable receiving zone by fresher water from upgradient recharge areas.

Based on these parameter choices, other parameters were obtained by calibrating the model against data from the aquifer test (fig. 3). The hydraulic conductivity of the receiving zone was determined to be 800 feet per day in the horizontal plane and was arbitrarily set equal to 80 feet per day in the vertical direction. No basis was found for estimating the hydraulic conductivity of the underlying and overlying confining layers, so an arbitrary low value of 0.01 foot per day was used to represent their confining character. Also as a result of calibrating the aquifer test, rock compressibility was determined to be 0.4 x 10^-4 inverse pounds per square inch. Calibration of the 100-minute aquifer test was considered to be unaffected by the regional background particle velocity.
Figure 2.—(A) Spinner flowmeter and (B) neutron porosity logs of the injection well at Hialeah.

Figure 3.—Simulation of the February 10, 1975, aquifer test data, (A) complete test period, and (B) early time data. Data show drawdowns in the injection well as the observation well was pumped at 250 gallons per minute.
Calibration against the observed chloride increases in recovered water determined the remaining transport parameters. Longitudinal and transverse dispersivity coefficients of 65 feet in the plane of the receiving zone were needed to represent the aggregate mixing effect within the volume of aquifer in which the native water was temporarily displaced. The transverse dispersivity coefficient value in the vertical direction was about 1 percent of that in the plane of the receiving zone. The dispersion process, and other mixing processes, cause the development of a "zone of dispersion" about the theoretical injected water/native water interface. Within this zone, the chloride concentration grades from that of the injected water to that of the native water. It is the existence of this zone of dispersion that causes recovered water to exhibit a gradual rather than instantaneous increase in chloride concentration, and that reduces recovery efficiency to less than 100 percent if the native water is sufficiently saline.

Although the dispersivity parameters worked well in simulating the chloride increases of the first recovery, which began after a short (2-day) storage period, the chloride increases after the longer second (54-day) and third (181-day) storage periods were not well matched by the simulation. Alternative dispersivity coefficient values only produced disagreement with data from the first recovery. To resolve this problem, a regional background particle velocity of 260 feet per year for the aquifer was incorporated into the simulation. This did not appreciably affect the first recovery simulation, but the second and third recovery data were much better matched by assuming that downgradient advection during storage reduced recovery efficiency.

The third recovery continued for 2.5 years, until the recovered water had nearly reached the chloride concentration of the native water. The ability of the model to simulate the rate at which computed recovery chloride concentrations approached the measured ones late in this recovery cycle depended upon the chosen molecular diffusivity coefficient value. The value used for calibration was $0.2 \times 10^{-3}$ square feet per day. The calibration of recovery chloride concentration values in all three cycles is shown in figure 4 and is designated case C.

**EFFECT OF IMPRECISE DATA ON ASSIGNMENTS OF PARAMETER VALUES**

Using data acquired during well construction and during the subsequent test cycles, the processes of data interpretation and computer simulation were sufficient to generally determine some of the chemical and hydraulic parameters of the formation exposed to the open interval of the well. However, some ambiguity persisted in assigning precise numerical values to the physical and hydraulic properties of the receiving zone and its vertically adjacent confining layers. This led to some concomitant uncertainty concerning the appropriate design of the conceptual model. The set of parameter values just described was considered a "best guess" conceptual model, and the resulting model calibration (case C) is referred to as the "basic simulation."
It was considered worthwhile to make some assessment of the possible effects of incorrectly assigning some numerical values in the "best guess" model. Several parameters were selected that were considered to have the greatest potential for being in error because of measurement imprecision and that were known to have a significant effect on the transport processes modeled. Each of these parameters, in turn, was assigned a value significantly different from the "best guess" estimate but still within a possible range of variation. A separate model calibration was obtained based on each such parameter change. The resulting "alternative simulations" were as good, in the sense of "curve matching," as the "basic simulation," demonstrating the nonuniqueness of the model calibration that resulted from recognizing the imprecise nature of the measurements.
ALTERNATIVE SIMULATIONS

One parameter that may have been in error was the receiving-zone thickness that was assigned based on examination of flowmeter logs. Some of these logs and temperature and fluid-resistivity logs run during the three test cycles indicated that the receiving zone might extend deeper than estimated. Therefore, the six model layers corresponding to the receiving zone were each increased in thickness from 2 to 3.5 feet for case C-2, the first "alternative simulation." The resulting calibrations against aquifer test data and recovery chloride concentration data used substantially revised estimates for rock compressibility, horizontal hydraulic conductivity, dispersivity coefficients, regional background particle velocity, and molecular diffusivity (table 1) but were as good as the "basic simulation."

Table 1.--Physical and hydraulic parameters and calibration coefficients used in basic and alternative simulations

[Acronyms: RBPV, regional background particle velocity; DIS, dispersivity coefficients; DIF diffusivity coefficient. English units: Thickness and DIS, in feet; porosity, in percent; gradient, in feet per mile; RBPV, in feet per year; DIF, in square feet per day; compressibility, in inverse pounds per square inch; conductivity, in feet per day]

<table>
<thead>
<tr>
<th>Flow zone</th>
<th>Case</th>
<th>Thickness</th>
<th>Effective porosity</th>
<th>Regional gradient</th>
<th>RBPV</th>
<th>DIS</th>
<th>DIF</th>
<th>Molecular diffusivity</th>
<th>Rock compressibility</th>
<th>Lateral hydraulic conductivity</th>
</tr>
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<tr>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Kₓ</td>
</tr>
<tr>
<td>C</td>
<td>12</td>
<td>35</td>
<td>1.6</td>
<td>260</td>
<td>65</td>
<td>0.0002</td>
<td>0.0000400</td>
<td>800</td>
<td>800</td>
<td></td>
</tr>
<tr>
<td>C-1</td>
<td>12</td>
<td>35</td>
<td>0.4</td>
<td>260</td>
<td>65</td>
<td>0.0002</td>
<td>0.0000400</td>
<td>3,200</td>
<td>3,200</td>
<td></td>
</tr>
<tr>
<td>C-2</td>
<td>21</td>
<td>35</td>
<td>2.35</td>
<td>220</td>
<td>50</td>
<td>0.0004</td>
<td>0.0000225</td>
<td>475</td>
<td>475</td>
<td></td>
</tr>
<tr>
<td>C-3</td>
<td>12</td>
<td>20</td>
<td>1.4</td>
<td>364</td>
<td>80</td>
<td>0.00006</td>
<td>0.0000750</td>
<td>750</td>
<td>750</td>
<td></td>
</tr>
<tr>
<td>C-4</td>
<td>12</td>
<td>35</td>
<td>0.8</td>
<td>364</td>
<td>50</td>
<td>0.0002</td>
<td>0.00001000</td>
<td>2,350</td>
<td>2,350</td>
<td></td>
</tr>
</tbody>
</table>

1basic simulation; 2increase hydraulic conductivities, decrease regional gradient; 3increase flow-zone thickness; 4decrease flow-zone effective porosity; 5anisotropic permeability.

A second parameter that may have been in error was the porosity of the receiving zone. The SWIP code assumes that transport occurs within the entire volume of pores within the formation; therefore, the specified porosity is understood to be the "effective porosity," or the volume of pores through which flow occurs. In carbonate formations, however, this conceptual model requires additional modification. In a permeable flow zone, porosity occurs both as interstitial porosity and as solution-opening porosity. Virtually all flow occurs within the latter, and flow through interstitial pores is negligible. Therefore, measures of total porosity, such
as the neutron porosity log of figure 2, can be misleading, and the effective porosity in which fluid movement occurs may only be part of that indicated by the log (35 percent).

To account for this possibility, the effective porosity of the 12-foot thick receiving zone was changed to a lower value of 20 percent. The model was recalibrated (case C-3) with significant revisions to rock compressibility, hydraulic conductivity, dispersivity coefficients, regional background particle velocity, and molecular diffusivity. Agreement between computed and measured data was as good as in previous calibrations.

A third variation on the "best guess" conceptual model was to abandon the assumption that flow within the plane of the receiving zone was laterally isotropic. Where regional flow at a specific location has long occurred in a particular direction, as a point on a flow path from a recharge area to a coastal discharge region, it is possible that the solution-opening porosity has primarily developed in the flow direction and that hydraulic conductivity in the plane of the receiving zone is anisotropic.

A 10:1 ratio of hydraulic conductivities in the X- and Y-coordinate directions was assumed (case C-4) so that the highest hydraulic conductivity was in the direction of regional flow. The observation well of the aquifer test was assumed to be in the direction of lowest hydraulic conductivity, based on its location at a right angle from the regional flow direction. The aquifer test data were simulated with X- and Y-coordinate hydraulic conductivity values of 2,350 and 235 feet per day and a new value of rock compressibility (table 1). The recovery chloride data were simulated with revised dispersivity coefficients and a new value of regional background particle velocity. Molecular diffusivity, however, was unchanged from the "basic simulation."

Finally, the list of parameters that may have been in error was extended to include the (isotropic) hydraulic conductivity value determined from simulation of the aquifer test data. The "basic calibration" values of regional background particle velocity and hydraulic conductivity required a regional gradient estimate about four times that estimated on the basis of measured water levels from the Upper Floridan aquifer. It is known, furthermore, that aquifer tests in solution-riddled carbonates can yield erroneous or inconsistent results if the wells used to impose stress on the aquifer or wells used for pressure measurements are located where local hydraulic properties do not match conditions in a regional "average" sense. Therefore, the hydraulic conductivity was arbitrarily increased four times, and the regional gradient estimate was decreased to one-fourth the previously estimated value. No other parameters were changed (table 1).

Results were a calibration (case C-1) almost identical to the basic simulation (case C). Higher hydraulic conductivity can increase buoyancy stratification to a degree sufficient to reduce recovery efficiency. Although case C-1 did show a slight degree of buoyancy stratification, the effects were offset by other factors.
PREDICTIVE SIMULATIONS OF RECOVERY EFFICIENCY

The principal purpose of the simulation effort was realized by representing 10 years of injection, storage, and recovery using parameter coefficients of the "basic simulation" (C) for a prediction of the potential recovery efficiency after many cycles. Recovery efficiency increases with successive cycles because residual freshwater remains in the formation after recovery ends (Merritt, 1986). The hypothetical annual cycle represented in the simulation was injection at 150,000 cubic feet per day during a 5-month wet season, storage for 3 months, and recovery at the same rate as injection for the remaining 4 months of a dry season. Table 2 shows that recovery efficiency improved rapidly in the first three cycles and then improved more slowly to 68.55 percent after 10 cycles.

Table 2.--Improvement of recovery efficiency, in percent, with successive cycles

<table>
<thead>
<tr>
<th>Case</th>
<th>Cycle 1</th>
<th>Cycle 2</th>
<th>Cycle 3</th>
<th>Cycle 4</th>
<th>Cycle 5</th>
<th>Cycle 6</th>
<th>Cycle 7</th>
<th>Cycle 8</th>
<th>Cycle 9</th>
<th>Cycle 10</th>
</tr>
</thead>
<tbody>
<tr>
<td>C</td>
<td>40.6</td>
<td>58.4</td>
<td>63.9</td>
<td>66.3</td>
<td>67.4</td>
<td>68.0</td>
<td>68.3</td>
<td>68.4</td>
<td>68.5</td>
<td>68.55</td>
</tr>
<tr>
<td>C-2</td>
<td>41.8</td>
<td>58.4</td>
<td>63.2</td>
<td>65.1</td>
<td>65.9</td>
<td>66.3</td>
<td>--</td>
<td>--</td>
<td>--</td>
<td>--</td>
</tr>
<tr>
<td>C-3</td>
<td>42.0</td>
<td>58.1</td>
<td>62.5</td>
<td>64.6</td>
<td>65.7</td>
<td>66.4</td>
<td>66.8</td>
<td>67.0</td>
<td>67.2</td>
<td>67.2</td>
</tr>
<tr>
<td>C-4</td>
<td>40.1</td>
<td>58.9</td>
<td>65.1</td>
<td>67.8</td>
<td>69.1</td>
<td>69.7</td>
<td>70.1</td>
<td>70.2</td>
<td>70.3</td>
<td>70.3</td>
</tr>
</tbody>
</table>

What if one of the "alternative simulations" used parameter values that more accurately represented the true nature of processes occurring in the subsurface environment? How would this have changed the outcome of the predictive simulations? For a resolution of this issue, the 10-cycle simulation was repeated with coefficients of the alternative simulations (cases C-2, C-3, and C-4). Table 2 shows that the improvement of recovery efficiency and the recovery efficiency at the end of 10 cycles differed only slightly among the cases. Case C-1 was not rerun because most coefficients were the same as for the basic simulation (case C). Case C-2 was terminated by a computer system failure after six cycles.

SUMMARY AND CONCLUSIONS

Of the parametric coefficients and other data describing the Upper Floridan aquifer and its hydraulic and transport properties at the Hialeah aquifer site, several (rock compressibility, molecular diffusivity, longitudinal and transverse dispersivity, and regional background particle velocity) can be determined only through model calibration, and only three (receiving-zone thickness, effective porosity, and hydraulic conductivity) can be estimated independently using other methods of investigation. Given known values of the
three latter parameters, aquifer test data can be used to determine rock compressibility and hydraulic conductivity. Recovery chloride data from three highly disparate storage and recovery cycles provide enough information to determine the dispersivity coefficients, pore velocity, and molecular diffusivity through simulation modeling.

However, when a substantial degree of uncertainty is associated with the measurements of the three "known" parameters, as was true in this study, additional degrees of freedom characterize the determination of parametric coefficients through model calibration. By substituting substantially revised, but still justifiable, values for each of the three "known" parameters, several independent sets of parameter values are obtained, each of which can provide simulation results that match aquifer test measurements and measurements of observed chloride increases during recovery.

In this sense, the calibration of the model for this study has multiple nonunique solutions, each of which describes a unique set of transport processes occurring during the operational tests. However, every 10-year predictive simulation based on one of the sets of calibration parameters yielded nearly the same results, indicating that the outcome of the predictive simulation depended upon the ability of the calibrated models to match observed data rather than upon identifying which, if any, of the sets of coefficients used for the calibration are, in fact, the most accurate representation of actual conditions in the aquifer.

This demonstration lends credence to results of the predictive simulations of multiple-cycle recovery efficiency, which otherwise could be considered questionable to the same extent that "best guess" values of the "known" parameters could be considered questionable.

REFERENCES


COUPLING DIFFUSE AND CONDUIT FLOW MODELS TO SIMULATE INJECTION AND RECOVERY OF WATER THROUGH A SINGLE WELL

Vicente Quiñones-Aponte and Miguel A. Medina, Jr.
Duke University, Durham, North Carolina

ABSTRACT

Analytical solutions to both conduit-flow (free-flow) advective-diffusive and diffuse flow (Darcian or retarded porous media flow) advective-dispersive transport equations are modified and coupled into a model to simulate the feasibility of artificial recharge of ground water through a single well. The wellbore is used as interface to combine the results from the models of the two different aquifer units. Existing data from field experiments conducted in Arecibo, Puerto Rico are used in a case study. The aquifer is composed of two flow zones. The first zone consists of about 100 feet of alluvial deposits. The second or lower zone consists of about 50 feet of Tertiary limestone which has zones featuring karst-solution conduits. The analysis indicates that although the coupled models performed well on simulating the actual data, more information on the variability of the limestone matrix porosity and molecular diffusion coefficient are needed. For the presented case study most of the flow and transport seems to occur throughout the limestone or conduit zone.

INTRODUCTION

The problem posed in this paper is relevant to the technique of artificial recharge through a single well. In this paper a solution is proposed for injection/recovery cycles where the well is tapping various hydrologic formations within which flow types are different. Consequently, dispersion processes occurring between the two hydrogeologic units differ in rate and form.

This paper presents the case of a two-layer aquifer where the upper layer is composed of sand and clay and the lower layer of a dense consolidated rock (limestone) matrix featuring several large-size conduits. The flow type would be different along the two aquifer units because of the relatively higher ability of the secondary porosity system (conduit porosity) to conduct water.
Although it is a reasonable assumption to treat fractured and karst aquifers as porous media for regional evaluation purposes, certainly this is not the case for smaller scale studies where the local conditions are very influential. Coupling of models capable of simulating these two types of flow regimes seems to be a reasonable approach.

**Purpose and Scope**

This paper proposes a solution for the analysis of injection/recovery cycles when the injection well taps more than one hydrogeologic unit, each of which having different types of flow (porous media flow/conduit flow) which can result in different flow regimes.

Existing analytical models for porous media flow (Gerald and Collins, 1971) and conduit flow (Quiñones-Aponte, 1989) are linked together by simulating hydraulic and mixing processes along the well bore. Data from injection/recovery cycles conducted in Arecibo, Puerto Rico (Quiñones-Aponte and others, 1989) are used in a case study for testing the coupled models.

**THEORETICAL BACKGROUND**

If chemical transformations, solid phase adsorption, and radioactive decay are neglected, dispersion processes for a conservative tracer in both conduit and porous media flow systems will depend mainly on the hydrodynamic dispersion coefficient \( D_c \) (Bear and Bachmat, 1967):

\[
D_c = D_m + D_d, \tag{1}
\]

where:

- \( D_m \) = coefficient of mechanical dispersion \([L^2 T^{-1}]\), and
- \( D_d \) = coefficient of molecular diffusion \([L^2 T^{-1}]\).

Flow regimes and hydraulic characteristics of the media control the processes that define the hydrodynamic dispersion phenomena in both conduit and porous media flows.

In general, dispersion processes are different under laminar and turbulent flow conditions. In the case of conduit flow, the velocity components and the viscous forces control the dispersion processes. Taylor (1954) defined the dispersion process for laminar flow as a function of the velocity variations at a given cross section due to frictional forces. He also used Reynolds analogy to derive an expression representative of turbulent flow through a pipe. Since the Reynolds number can be used to determine the flow regime, it will also serve to determine whether laminar or turbulent dispersion is taking place for the conduit flow case. The Reynolds number (\( R_e \)) is equal to the ratio of inertial to viscous forces.
\[ Re = \frac{V d}{\nu}, \]  

where:
\[ V = \text{mean fluid velocity} \ [\text{LT}^{-1}], \]
\[ d = \text{some characteristic length} \ [\text{L}], \] and
\[ \nu = \text{fluid kinematic viscosity} \ [\text{LT}^{-1}]. \]

Since karst conduits can be considered analogous to pipes, the Poiseille equation can be used, as first approximation, to simulate conduit flow under steady laminar conditions:

\[ V = -\frac{\rho g r^2 \, dh}{8 \mu \, dl} \]  

where:
\[ \rho = \text{fluid density} \ [\text{MF}^{-3}], \]
\[ g = \text{gravitational constant} \ [\text{LT}^{-2}], \]
\[ r = \text{conduit radius} \ [\text{L}], \]
\[ \mu = \text{fluid dynamic viscosity} \ [\text{MTL}^{-2}], \] and
\[ dh/dl = \text{head loss term} \ [\text{LL}^{-1}]. \]

Laminar flow may occur in smooth pipes at Reynolds numbers up to 2,100, but for rough pipes turbulent flow begins at lower Reynolds numbers.

The implications of the Reynolds number for porous media flow is that the validity of Darcy's law is limited to Reynolds numbers smaller than 10. For Reynolds numbers between the range of 10 to 100 nonlinear laminar flow occurs, whereas for values greater than 100 turbulent flow occurs (Bear, 1979).

For porous media flow, a relation between molecular diffusion and mechanical dispersion modified by Bear and Bachmat (1967) after Saffman (1960) allows the determination of the dominant transport process. Bear and Bachmat derived an expression for the relationship between the longitudinal dispersion coefficient, the geometry of the porous media particles, the flow velocity, and the molecular diffusion coefficient. This relationship introduces the Peclet number which in its general form is:

\[ Pe = \frac{V L}{D_d}, \]

where:
\[ L = \text{some characteristic length of the porous media particles} \ [\text{L}]. \]

The Peclet number can be used to identify the dominant dispersion process for different porous-media flow conditions. Molecular diffusion occurs almost alone for Peclet numbers < 0.4 where the mean flow velocity is very slow. There is a transition zone that occurs for the range 0.4 ≤ Pe ≤ 5.0
within which the effects of mechanical dispersion can be appreciated but molecular diffusion still dominant. For Peclet numbers between $5.0 \leq P_e \leq 1,000$ mechanical dispersion will predominate but molecular diffusion can't be neglected. As the mean flow velocity is larger the effect of molecular diffusion becomes negligible ($1,000 \leq P_e \leq 10^5$). Finally for $P_e \geq 10^6$ the flow regime will be out of the Darcy's Law domain.

The Reynolds and Peclet numbers, and estimates of the fluid velocity are used as criteria to determine the dominant dispersion processes.

**Definition of Recovery Efficiency from a Single-Well**

The recovery efficiency is expressed as a percentage of the volume injected and defined by the volume of water recovered before the mixed water withdrawn fails to meet some pre-established water-quality limits. The previous definition is meaningful for practical purposes. However, in order to obtain a meaningful relationship in terms of the actual physico-chemical process leading to the mixing of the injected and native aquifer waters, the actual amount of water recovered must be defined. Such amount of water is defined as the maximum recoverable portion of the injected water. The ratio of the maximum recoverable portion of the injected water to the injected volume defines the recovery efficiency. The maximum recoverable portion of the injected water is the total amount of injected water that would be retrieved through the well if pumping continues for an infinite time period. For practical purposes, a time limit is established beyond which the proportion of freshwater in the mix is insignificant. The maximum recoverable portion of the injected water can be determined by defining a breakthrough curve (fig. 1) during the recovery pumping period. If the quality of the injected and native aquifer waters are different enough to be distinguished, changes in the concentration of conservative naturally occurring ions can be used to study the recovery efficiency. In cases where the quality of the injected and native waters cannot be easily distinguished some inert (non-reactive) tracer can be added to the injected water. The area over the breakthrough curve represents the maximum recoverable portion of the injected water (fig 1). The volume can be quantified by simple numerical integration. If the tracer concentration ($C(t)$) is defined as a function of the recovery pumping:

$$C(t) = \frac{(Q_r(t) \cdot C_t + Q_n(t) \cdot C_n)}{(Q_r(t) + Q_n(t))}, \quad (5)$$

where:

- $Q_r(t) =$ injected component of pumped water [$L^3 T^{-1}$],
- $Q_n(t) =$ native component of pumped water [$L^3 T^{-1}$],
- $C_t =$ tracer concentration in the injected water [$ML^{-3}$],

and

270
\[ C_n = \text{tracer concentration in the native water [ML}^{-3}] \].
The total pumping rate \( Q_p(t) \) is then:

\[ Q_p(t) = Q_1(t) + Q_n(t). \] (6)

![Figure 1.- Typical breakthrough curve during the recovery pumping period.](image)

By combining equations (5) and (6) and rearranging:

\[ Q_r(t) = Q_p(t) \frac{C_n - C(t)}{C_n - C_1}. \] (7)

Then the maximum recoverable portion of the injected water:

\[ V_r = \int_0^\infty Q_r(t) \, dt, \] (8)

or

\[ V_r = \frac{1}{C_n - C_1} \int_0^\infty Q_p(t) (C_n - C(t)) \, dt. \] (9)

If the recovery pumping rate is constant \( Q_p \):

\[ V_r = \frac{Q_p}{C_n - C_1} \int_0^\infty (C_n - C(t)) \, dt, \] (10)

which can be approximated by:
\[ V_r = \frac{Q_p}{C_n - C_l} \sum_{j=0}^{Lt} [(C_n - C_j) (t_{j+1} - t_j)], \]  
\[ (11) \]

where:

- \( Lt \) = time when the asymptote of the breakthrough curve is reached \([T]\) (fig. 1), and

\[ C_j = (C(t_j) + C(t_{j+1})) / 2. \]  
\[ (12) \]

Breakthrough curves are defined by the combined effects of the different dispersion processes. Spreading of solute particles due to dispersion is in general a function of the hydrodynamic dispersion coefficient which was previously described.

**Analytical Solution for a Porous Media Flow System**

A solution to the dispersion equation for injection/recovery cycles in a porous media aquifer was developed by Gelhar and Collins (1971). They obtained an approximate solution for the general longitudinal advective-dispersive equation:

\[ \frac{\partial C}{\partial t} + u \frac{\partial C}{\partial s} = \alpha u \frac{\partial^2 C}{\partial s^2} + D_a \left( \frac{\partial^2 C}{\partial s^2} - \frac{1}{u} \frac{\partial C}{\partial s} \right) \]  
\[ (13) \]

where:

- \( \alpha \) = longitudinal dispersivity \([L]\),
- \( u \) = flow velocity \([LT^{-1}]\),
- \( s \) = the arc length along the direction of the flow assuming a curvilinear coordinate path \([L]\),
- \( t \) = time \([T]\), and
- \( D_a \) = effective molecular diffusion coefficient \([L^2T^{-1}]\).

Their approximation to the solution during the recovery pumping period was made by applying the boundary layer techniques. They defined the concentration at the well by setting \( r = r_w \) and by neglecting the effects of the well bore and molecular diffusion. They expressed the solution in terms of volumetric variables:

\[ C = \frac{1}{2} \text{erfc} \left( \frac{\sqrt{16\alpha \left[ 2 - \frac{V_2}{V_1} \left( 1 - \frac{V_2}{V_1} \right) \right]^{1/2}}} {3R_1 \left( \frac{V_2}{V_1} \right)^{1/2}} \right) \]  
\[ (14) \]

where:
\( R_1 = \) total distance traveled by the injected water front \([\text{L}]\),
\( V_1 = \) total volume injected \([\text{L}^3]\), and
\( V_2 = \) volume being pumped during recovery \([\text{L}^3]\).

and

\[
erfc(\xi) = 1 - 2/\sqrt{\pi} \int_0^\xi e^{-\omega^2} d\omega.
\]

is the complementary error function.

**Analytical Solution for a Conduit Flow Aquifer**

For conduit flow aquifer, an analytical model developed by Quiñones-Aponte (1989) was modified. The model simulates the recovery of a conservative solute in a karstified limestone aquifer which has randomly located solution conduits and is tapped by an injection-recovery well (fig. 2). The analytical solution was derived under the following assumptions: (1) the solution conduits are of cylindrical shape and oriented perpendicular to the well (fig. 2); (2) the relatively low permeability of the limestone matrix limits its hydraulic capability to transmit water compared to that of the solution conduits; (4) the injected-water front is assumed to be at the

![Diagram](image)

*Figure 2.* Conceptual model of coupled porous media and limestone solution conduits aquifer zones.
same distance from the well in all the solution conduits; (5) because of the relatively small diameter of the solution conduits, only one-dimensional advective transport is considered through them -- thus \( \partial C/\partial z = 0 \) for \( |z| \leq r \); (6) the injected to pumping rate ratio is equal to 1, and (7) flow through the conduits is considered laminar.

Under the above assumptions the equations that govern advective transport in a single limestone solution conduit and molecular diffusion from the limestone matrix are:

\[
\frac{\partial C}{\partial t} + \frac{q}{\pi r^2} \frac{\partial C}{\partial R} = \frac{n Da}{r} \frac{\partial C'}{\partial z} \bigg|_{z=r} \quad \text{for } Rw < R \leq RI \tag{16}
\]

and

\[
\frac{\partial C'}{\partial t} - Da \frac{\partial ^2 C'}{\partial z^2} = 0, \quad \text{for } r \leq z < \infty \tag{17}
\]

where:

- \( C = \) solute concentration in the water within the solution conduits [ML\(^{-3}\)],
- \( C' = \) solute concentration of the rock matrix water [ML\(^{-3}\)],
- \( q = \) volumetric-flow rate through the solution conduit [L\(^3\)T\(^{-1}\)],
- \( r = \) radius of the solution conduit [L],
- \( R = \) radial distance from the injection-recovery well [L],
- \( Rw = \) radius of the injection-recovery well [L],
- \( RI = \) radial distance of the injected water front [L],
- \( z = \) distance in direction normal to the solution conduit [L],
- \( t = \) time [T], and
- \( n = \) matrix porosity [L\(^0\)].

The initial and boundary conditions are: \( C(Rw,t) = C_0 \), where: \( C_0 = \) tracer concentration of the injected water, \( C(R,0) = 0 \), \( C'(R,r,t) = C(R,t) \), \( C'(R,\infty,t) = 0 \), and \( C'(R,z,0) = 0 \).

The analytical solution to the coupled equations was obtained by means of the Laplace transform and inverse Laplace transform (Quiñones-Aponte, 1989). The solution for the concentration at the well during the recovery pumping, in dimensionless form, and neglecting well-bore terms is:

\[
\frac{C}{C_0} = \text{erfc} \left( \frac{\pi r n Da^{1/2} (Rp(t))}{\pi r^2 (R_p/t_p) / RI} \right)^{1/2} \tag{18}
\]
where:

\( \bar{r} \) = average radius of the solution conduits [L],

\( R_p \) = radial distance of the injected-water front during the recovery pumping period [L],

\( t_p \) = time since the recovery pumping period commenced [T],

and

\( Q_I \) = injection rate [L^3T^{-1}].

The recovery discharge was expressed in terms of the volumetric flow rate:

\[
Q_p = \frac{V_p}{t_p}, \tag{19}
\]

and the radial distance of the front during the recovery pumping (\( R_p(t_p) \)):

\[
R_p(t_p) = \frac{Q_p \cdot t_p}{\pi \bar{r}^2 \cdot n_c}, \tag{20}
\]

where:

\( n_c \) = number of solution conduits.

COUPLING POROUS MEDIA AND CONDUIT FLOW MODELS

To couple the porous media and conduit flow models some assumptions are needed. For the porous media flow zone it is assumed that: 1) flow and transport occurs only in the horizontal radial direction (Dupuit assumption), 2) an average hydraulic conductivity \( K_{pm} \) is used for the entire porous media section, and 3) the steady-state condition is reached instantaneously. For the conduit-flow zone it is assumed that: 1) a relationship between the proportionality constant of Darcy’s equation and its counterpart in Poiseuille’s equation is valid and used to characterize the conduit flow system, and 2) laminar pipe-flow is assumed to occur along the conduits.

The well bore is used to link the solutions from the two aquifer zones models by assuming mixing to be proportional to the aquifer hydraulic characteristics.

Under steady-state conditions, nonvertical-irrotational ground-water flow to a well can be represented using the cylindrical-coordinate version of Laplace’s equation:

\[
\frac{1}{r} \frac{\partial}{\partial r} \left[ r \frac{\partial h}{\partial r} \right] = 0. \tag{21}
\]

By integrating twice equation (21) and applying boundary conditions, the following solution is obtained:

\[
h = \frac{h_i - hw}{\ln(r) + hw} \cdot \ln(r) + h_i - hw \cdot \ln(r) - \ln(r_i/rw). \tag{22}
\]
To describe porous media flow to or from a well, Darcy’s equation is applied:

\[ Q = \pm 2 \pi r b K \frac{\partial h}{\partial r} \]  \hspace{1cm} (23)

From the first integral of equation (21),

\[ \frac{\partial h}{\partial r} = \frac{h_i - hw}{ln(r_1/r_w) r_1} \]  \hspace{1cm} (24)

Substituting equation (24) into (23) yields:

\[ Q_{pm} = -2 \pi r_1 b_{pm} K_{pm} \frac{h_i - hw}{ln(r_1/r_w) r_1} \]  \hspace{1cm} (25)

which represents the discharge to or from a well in a porous media aquifer, and where the subscript \( pm \) stands for porous media.

For the conduit-flow aquifer zone, an analogy between Darcy’s equation:

\[ \frac{dh}{dl} = -K, \text{ and} \]  \hspace{1cm} (26)

Poiseuille’s equation (equation (3)) is made assuming steady-laminar flow. The proportionality constant of Darcy’s equation \( K \) is equivalent to part of the left hand side of Poiseuille’s equation:

\[ K_c = \frac{\rho g r^2}{8 \mu} \]  \hspace{1cm} (27)

where \( K_c \) is the apparent hydraulic conductivity of the conduit-flow aquifer zone.

Under the previously made assumptions, the total area of conduits contributing flow to the well can be estimated by using the relationship between the apparent hydraulic conductivity of the conduit-flow zone and its counterpart term on Poiseuille’s equation. From equation (27):

\[ r^*_t = \left( \frac{K_c 8 \mu}{\rho g} \right)^{1/2} \]  \hspace{1cm} (28)

Here \( r^*_t \) can be taken as the apparent radius of the conduit sectional area, represented by a single conduit. Then the total conduit area can be approximated by:
\[ \pi r_t^2 = \frac{\pi K_c 8 \mu}{\rho g}. \]  

(29)

An expression is derived to represent the discharge rate through the conduits, on the basis of a field measured parameter \( K_c \):

\[ Q_c = \frac{\pi K_c 8 \mu R_p(t_p)}{\rho g t_p}. \]  

(30)

The radial distance of the injected water front during the recovery pumping (equation (20)) is then redefined by:

\[ R_p(t_p) = \frac{Q_c \rho g t_p}{\pi K_c 8 \mu}. \]  

(31)

The total injection or pumping rate is:

\[ Q_T = \pm (Q_c + Q_{pm}). \]  

(32)

In this first approximation, it has been assumed that the hydraulic gradient is the same within the two aquifer zones, and that their discharge components can be estimated by using the hydraulic conductivity of the two aquifer zones as weighting factors:

\[ Q_c = Q_p \left( K_c \frac{bc}{K_c bc} \right), \]  

(33)

and

\[ Q_{pm} = Q_p \left( K_{pm} \frac{bp_m}{K_c bc} \right), \]  

(34)

where \( K_0 \) represents the equivalent hydraulic conductivity of the entire aquifer section and \( bc \) the thickness.

The trace concentration in the water pumped during the recovery is determined by using the same approach applied in equation (5):

\[ C_p(t) = \frac{(C_{pm}(t) K_{pm} bp_m + C_c(t) K_c bc)}{K_0 bc}. \]  

(35)

CASE STUDY

An application of the coupled models was possible using data from a single-well injection/recovery cycles study conducted near Arecibo, Puerto Rico (fig. 3). In the case study the U.S. Geological Survey and the Puerto Rico Department of Agriculture (Quiñones-Aponte and others (1989)) explored the feasibility of underground storage of surplus water from Rio
Grande de Arecibo.

General Setting of the Study Site

The injection-recovery facilities are located in the Río Grande de Arecibo alluvial valley, north-central Puerto Rico (fig. 3). The alluvium is underlain, at about 130 feet below land surface by the most permeable limestone aquifer of Puerto Rico -- the aquifer contained within the Ayamón and Aguada limestones. The valley is a flood plain generally used for agricultural purposes. It has very little topographic relief. The Río Grande de Arecibo, which flows one half mile west of the injection-recovery facilities (fig. 3) and has a mean annual discharge of 527 ft³/s, was used as source of recharge water. The average chloride concentration of the river water was about 12 mg/L (milligrams per liter). A well that was abandoned due to salinity problems (the Monte Grande Well, fig. 3) was rehabilitated and used as an injection-recovery well.

Aquifer and Injection/Recovery-Well Characteristics.

At the injection-recovery facilities, the background hydraulic gradient of the alluvial-limestone aquifer is nearly flat (about 0.0008 ft/ft (Giusti (1976))). The aquifer consists of alluvium through the upper 100 feet and limestone (Aymanón-Aguada) through about 400 feet under the base of the alluvium. Regionally, the limestone part of the aquifer is characterized by zones of solution conduits. The aquifer transmissivity ranges from 5000 ft²/d (feet squared per day) in the alluvial part to 80,000 ft²/d in the most permeable zones of the limestone (Quiñones-Aponte (1986)). Interpretation of borehole geophysical logs of the Monte Grande Well suggested the occurrence of large solution conduits in the limestone part of the aquifer. The native aquifer water has a chloride concentration of 930 mg/L.

The Monte Grande Well varies in diameter from 12 to 16 inches and is about 150 feet deep. It is cased with slotted-steel pipe from the surface to about 100 feet through clayey and fine sand alluvium. The remaining 50 ft. are open hole to the Ayamón Limestone (fig 4).

Calibration, Verification, and Discussion

Two injection-recovery tests of different volumes (80,211 and 882,325 ft³) were used to calibrate and verify the model. In both of these tests the pumping for recovery began immediately after injection ended. Quiñones-Aponte (1989) attempted to calibrate the conduit-flow model for several parameters (matrix molecular diffusion, matrix porosity, average radius of the solution conduits, and number of solution conduits) all of them exhibiting a large degree of uncertainty. In the original model there was no representation of the actual conduit-aquifer characteristics. The advection term was
Figure 3.- Location of the study area.
Figure 4.- Lithology at the Monte Grande Well.

represented by the volumetric-flow rate and the largely uncertain dimensions and number of the solution conduits. The present conduit-flow model has been modified in order to allow representation of the conduit aquifer characteristics in the advection term. As previously described, this was done using the equivalency of the apparent hydraulic conductivity ($K_c$) of the conduit aquifer zone and its counterpart term in Poiseuille’s equation (equation (27)). As result, the matrix molecular diffusion coefficient (conduit aquifer model) and the longitudinal dispersivity (porous media model) were the only parameters adjusted during calibration.

Based on different sources of information, the following parameters were fixed for calibration purposes: $K_{pm} = 40$ ft/d (Quiñones-Aponte, 1986), $n_{pm} = 0.30$ (typical value), $n_c = 0.05$ (Jennings, 1985), and the equivalent transmissivity of the entire aquifer section $T_c = 50,000$ ft$^2$/d (Quiñones-Aponte, 1986). The coupled models were calibrated adjusting only the limestone matrix molecular diffusion coefficient and the longitudinal dispersivity of the porous media. The data set of test No. 1 (VI = 80,211 ft$^2$) was used for calibration (fig. 5a). The root mean square difference (RMSD):

$$RMSD = \sum_{i=1}^{N} \left[ \left( C_i - \hat{C}_i \right)^2 / N \right]^{1/2}$$

where:
$C_1 =$ predicted concentration [ML\(^{-3}\)],
$C_1 =$ actual concentration [ML\(^{-3}\)], and
$N =$ number of data points,

Figure 5.- Model calibration (a) using data set No. 1 (VI = 80,211 ft\(^3\)) and model verification (b) using data set No. 2. (VI = 882,325 ft\(^3\)).

was used as objective function to minimize the differences between the actual and predicted concentrations. Calibrated values of $D_w$ and $\alpha$ were: $D_w = 30,000$ ft\(^2\)/min (46.4 m\(^2\)/sec) and $\alpha = 10$ ft (3 m). Using the calibrated parameters, the coupled models were applied to data set No. 2 (VI = 882,325 ft\(^3\)) (fig. 5b). It can be observed from this simulation that the models closely approximate the data set of test No. 2 (fig. 5b). However, the matrix molecular diffusion coefficient has a functional relation with the matrix porosity. Since the matrix porosity value was fixed to 0.05, the resulting $D_w$ value is somewhat predetermined by the matrix porosity ($n_c$). This indicates that $D_w$ cannot be accepted as a definite value, and its variability should be further evaluated using additional information on $n_c$. A single value of $n_c$ (representative of the primary porosity) might not be an adequate descriptor. Perhaps, a composite value, representative of the different porosity components (fracture, vug, moldic, and others) may be more adequate for the simulation of the diffusion term. The longitudinal dispersivity ($\alpha$) was not a significant parameter for these data sets.

A model that describes the transients and turbulent flow through the conduits might improve the simulation, in terms of
representing the actual transport processes. An indication of the necessity for a model capable of simulating turbulent transport was an estimated Reynolds number greater than 2,000 for the conduit aquifer flow zone. The estimate was made based on the apparent hydraulic conductivity of the conduit flow zone and its relation to Poiseuille's equation.

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REFERENCES


ANALYSIS OF INJECTION WELL PLUGGING
DURING LONG-TERM PILOT RECHARGE TESTS
TUACON BASIN, ARIZONA

Mark M. Cross, Laura J. Strauss, and Errol L. Montgomery
Errol L. Montgomery & Associates, Inc., Tucson, Arizona

ABSTRACT

Injection of surplus treated Central Arizona Project water is being investigated by Tucson Water as part of the Tucson Recharge Feasibility Assessment Project. Injection well pilot recharge testing has been completed for six Tucson Water production wells. For three of the wells, the testing program included long-term constant-rate injection tests. Duration of the long-term injection tests ranged from 105 to 258 days; average injection rates ranged from 1,060 to 1,220 gallons per minute.

Water level rise due to plugging which occurred in the injection wells was computed using four methods. Methods 1, 2, and 3 are based on analysis of water level rise during the long-term constant-rate injection tests. Method 4 is based on analysis of water level rise during step-injection tests conducted before and after the long-term injection tests. Analyses indicate that water level rise occurred in the injection wells due to plugging of the well casing perforations and/or plugging of the aquifer adjacent to the wellbore. Magnitude of water level rise due to plugging ranged from about 15 to 40 feet, and represented about 15 to 30 percent of the total water level rise at the end of the long-term injection tests.

INTRODUCTION

Feasibility of injection of surplus treated Central Arizona Project (CAP) water using existing municipal water wells is being investigated by Tucson Water as part of the Tucson Recharge Feasibility Assessment Project. Injection well pilot recharge tests have been conducted for six Tucson Water wells in the Tucson Water Interior Wellfield. Goals of the injection tests are to project feasible long-term injection rates, to project groundwater mounding effects, and to obtain data needed to estimate operational costs for full-scale long-term injection recharge operations.

Injection rates and operational costs during full-scale recharge operations at Tucson municipal wells will depend in part
on rate of decline in hydraulic efficiency of the wells, and on frequency of redevelopment required to sustain large injection rates at the wells. Hydraulic efficiency is expected to decline during injection operations due to plugging of the wells. This paper summarizes the procedures used to select pilot injection wells for testing, and gives methods and results for assessment of plugging during long-term injection tests which were conducted at Tucson Water wells C-26A, B-44B, and C-14B.

The technical investigations described in this paper were conducted by Errol L. Montgomery & Associates, Inc., under the general supervision of R. Bruce Johnson, Chief Hydrologist, and his staff at Tucson Water. The investigations were conducted in conjunction with CH2M Hill and Dr. Gray Wilson.

SELECTION OF WELLS FOR PILOT RECHARGE TESTING

Based on detailed evaluation of well construction details and aquifer data for 212 wells in the Tucson Water Interior Wellfield, 15 wells were selected for inspection by wellbore television surveys. These surveys indicated substantial thickness of scale and encrustation on the perforated casing for most of the wells surveyed. After reviewing results of the wellbore television surveys, five of the 15 wells were redeveloped by brushing, swabbing, and bailing with a cable-tool drilling rig. Wellbore television surveys were repeated at the five wells to document the effects of redevelopment. After review and analysis of hydrogeologic conditions, well construction details, wellbore television surveys, and of results of redevelopment operations, six wells were selected for pilot recharge tests.

Tucson Water wells C-26A, B-44B, and C-14B were selected for long-term constant-rate injection tests. Well C-26A is retired from service; wells B-44B and C-14B are active Tucson Water production wells which were temporarily removed from service for recharge testing operations. The production water wells were drilled using the cable-tool method. Casing perforations are torch cuts for well C-26A, machine-cut slots for well B-44B, and horizontal louvers for well C-14B. Casing perforations extend above pre-injection groundwater level at each of the production wells. Distance from top of the perforated casing interval to pre-injection groundwater level was about 170 feet at well C-26A, about 80 feet at well B-44B, and eight feet at well C-14B. Well construction details are summarized in Table 1.
TABLE 1. SUMMARY OF WELL CONSTRUCTION DETAILS FOR PILOT INJECTION WELLS C-26A, B-44B, AND C-14B

<table>
<thead>
<tr>
<th>WELL IDENTIFIER</th>
<th>DIAMETER (inches)</th>
<th>DEPTH (feet)</th>
<th>DIAMETER (inches)</th>
<th>DEPTH (feet)</th>
<th>PERFORATED INTERVAL (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-26A</td>
<td>12</td>
<td>350</td>
<td>12</td>
<td>0-350</td>
<td>128-350</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>600</td>
<td>10</td>
<td>337-596</td>
<td>337-596</td>
</tr>
<tr>
<td>B-44B</td>
<td>16</td>
<td>500</td>
<td>16</td>
<td>500</td>
<td>141-487</td>
</tr>
<tr>
<td>C-14B</td>
<td>16</td>
<td>800</td>
<td>16</td>
<td>800</td>
<td>260-780</td>
</tr>
</tbody>
</table>

During the injection tests, water level measurements were made in the injection well and in nearby observation wells. Observation wells at the C-26 site include wells C-26B and WR-155A. Well C-26B is an active Tucson Water production well, and is located 32 feet from well C-26A. Well WR-155A was constructed as an observation well, and is located 50 feet from well C-26A. Observation wells at the B-44 site include wells B-44A and WR-156A. Well B-44A is an inactive Tucson Water production well, and is located 10 feet from well B-44B. Well WR-156A was constructed as an observation well, and is located 149 feet from well B-44B. Observation wells at the C-14 site include wells C-14A and WR-154A. Well C-14A is an inactive Tucson Water production well, and is located 67 feet from well C-14B. Well WR-154A was constructed as an observation well, and is located 60 feet from well C-14B.

PILOT RECHARGE TESTING PROCEDURES

Injection well pilot recharge tests included pumping and injection tests, borehole fluid movement investigations, in-situ testing for suspended solids, and laboratory chemical analyses of water samples. The sequence of tests included an initial step-pumping test, an initial step-injection test, a long-term constant-rate injection test followed by a water level recovery period, a second step-injection test, and a second step-pumping test. A third step-injection test was conducted after the second step-pumping test at well C-14B. In addition, a 48-hour pumping test was conducted after the initial step-pumping test at wells C-26A and C-14B.

Injection equipment installed in the pilot injection wells included two injection pipes, which provided a means of achieving a range of injection rates while maintaining positive water pressure in the injection piping. A three-inch diameter steel injection pipe was set below groundwater level with a diffuser screen in the lower five feet. A four-inch diameter steel injection pipe was set below groundwater level with a diffuser screen in the lower 10 feet. The screens were capped on the bottom. An in-line orifice plate was installed in each injection pipe above the top of the screen section. One-inch diameter PVC pipe was set below groundwater level and above the injection
screens for installation of a pressure transducer. Four-inch diameter PVC pipe was set below the injection screens to provide access for borehole geophysical logging equipment during borehole fluid movement investigations.

Step-injection and long-term injection tests were conducted using potable water from the Tucson municipal distribution system. Injection rates were measured with a McCrometer impeller flowmeter and with a Data Industrial impeller flowmeter installed in the water supply pipeline. Measurements of injection rate from the Data Industrial flowmeter were recorded automatically using an electronic datalogger. Constant injection rates were maintained for the long-term injection test and for each step during the step-injection tests by adjustment of a pressure reducing valve in the water supply pipeline. Positive gage pressure was maintained at the wellhead during injection. Water level measurements were obtained using electric water level sounders and using pressure transducers installed in the injection well and in observation wells. Water level measurements from the pressure transducers were recorded automatically by the datalogger.

Each step-injection test included three steps, conducted sequentially, one step per day. Because water level was allowed to recover after each step, the water level response for each step was independent of the water level response from the previous step. Results from the step-pumping and step-injection tests were used to compute specific injection capacity and well efficiency before and after the long-term injection test, and to evaluate magnitude of water level rise due to well plugging which occurred during the long-term injection test.

SPECIFIC INJECTION CAPACITY AND WELL EFFICIENCY BEFORE AND AFTER LONG-TERM CONSTANT-RATE INJECTION TEST

Specific injection capacities and well efficiencies were computed from step-injection test A, conducted before the long-term injection test, and from step-injection test B, conducted after the long-term injection test. Specific injection capacity was defined as average injection rate divided by feet of water level rise after four hours of injection. Well efficiency represents the ratio of formation loss to total hydraulic head loss. Formation losses are hydraulic head losses associated with flow of water through the aquifer. Well losses are hydraulic head losses associated with flow of water into or out of the well and include head losses related to plugging.

Well efficiency was computed using three methods. Results for specific injection capacity and well efficiency for step 2 of each step-injection test are summarized in Table 2. Injection rate for step 2 of each test was similar to the injection rate for the long-term test. Due to the effects of plugging during the long-term injection tests, specific capacities and well efficiencies were substantially smaller for step-injection test B conducted after
long-term injection than for step-injection test A conducted prior to long-term injection (Table 2).

<table>
<thead>
<tr>
<th>WELL IDENTIFIER</th>
<th>INJECTION RATE (gpm)</th>
<th>INJECTION CAPACITY AT FOUR HOURS (gpm/ft)</th>
<th>SPECIFIC INJECTION CAPACITY</th>
<th>WELL EFFICIENCY AT FOUR HOURS (percent)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-26A 2A</td>
<td>1,060</td>
<td>15.2</td>
<td>44</td>
<td>44</td>
</tr>
<tr>
<td>2B</td>
<td>1,070</td>
<td>10.0</td>
<td>26</td>
<td>13</td>
</tr>
<tr>
<td>B-448 2A</td>
<td>1,195</td>
<td>28.3</td>
<td>81</td>
<td>71</td>
</tr>
<tr>
<td>2B</td>
<td>1,185</td>
<td>21.4</td>
<td>60</td>
<td>54</td>
</tr>
<tr>
<td>C-148 2A</td>
<td>1,240</td>
<td>13.8</td>
<td>88</td>
<td>66</td>
</tr>
<tr>
<td>2B</td>
<td>1,225</td>
<td>11.1</td>
<td>66</td>
<td>56</td>
</tr>
<tr>
<td>2C</td>
<td>1,235</td>
<td>12.1</td>
<td>74</td>
<td>61</td>
</tr>
</tbody>
</table>

ANALYSIS OF RESULTS FROM LONG-TERM INJECTION TESTS

Injection operations for the three long-term constant-rate injection tests were conducted during the period from October 1989 through June 1990. Duration of long-term injection tests ranged from 105 to 258 days. Average injection rate ranged from 1,060 gallons per minute (gpm) at well C-26A, to 1,220 gpm at well C-14B. Duration of injection, average injection rate, and water level rise at the end of the constant-rate injection period are summarized in Table 3.

<table>
<thead>
<tr>
<th>WELL IDENTIFIER</th>
<th>TIME AT START OF TEST</th>
<th>DURATION OF LONG-TERM INJECTION PERIOD (days)</th>
<th>AVERAGE RATE (gpm)</th>
<th>WATER LEVEL RISE AT END OF LONG-TERM INJECTION PERIOD (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-26A 2A</td>
<td>09:00 hrs 10-11-89</td>
<td>258</td>
<td>1,060</td>
<td>C-26A 123.37</td>
</tr>
<tr>
<td>2B</td>
<td>1,225</td>
<td>C-26B 19.03</td>
<td></td>
<td></td>
</tr>
<tr>
<td>B-448 10:00 hrs 11-17-89</td>
<td>128</td>
<td>1,210</td>
<td>B-448 64.99</td>
<td></td>
</tr>
<tr>
<td>C-148 2A</td>
<td>105</td>
<td>C-14B 135.45</td>
<td></td>
<td></td>
</tr>
<tr>
<td>2C</td>
<td>11-28-89</td>
<td>WR-154A 43.28</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Corrected for regional water level decline

AQUIFER PARAMETERS

Water level data obtained during the 48-hour pumping tests and the long-term injection tests were analyzed for transmissivity and storage coefficient using the modified non-equilibrium equation semi-logarithmic graphical method (Cooper and Jacob, 1946), the log-log graphical method for unconfined aquifers (Neuman, 1975), and the semi-logarithmic distance versus water level rise method. The distance versus water level rise method uses the Thiem (1906) equilibrium equation for radial flow to a well. Prior to analysis
for aquifer parameters, water level rise data from the long-term injection tests were corrected for regional groundwater level decline using baseline water level data from Tucson Water monitor wells in the vicinity of the test sites.

Transmissivity for the basin-fill deposits aquifer during the long-term injection tests was 50,000 gallons per day per foot width of aquifer at 1:1 hydraulic gradient (gpd/ft) at site C-26, 65,000 gpd/ft at site B-44, and 25,000 gpd/ft at site C-14. Average horizontal hydraulic conductivity for the aquifer was 280 gallons per day per square foot of aquifer at 1:1 hydraulic gradient (gpd/ft²) at well C-26A, 230 gpd/ft² at site B-44, and 75 gpd/ft² at site C-14. Analysis of results from pumping and injection tests indicated that magnitude of the storage coefficient for the basin-fill deposits aquifer was approximately 0.1 (dimensionless; volume of water released from or taken into storage per unit surface area of aquifer per unit decrease or increase in head).

ANALYSIS OF PLUGGING DURING INJECTION TESTING

Plugging of perforations in the injection well casing and/or plugging of the aquifer adjacent to the wellbore may occur due to physical, chemical, or biological effects resulting from the introduction of injection water into the well and from movement of injection water from the well into the aquifer. Physical effects may include rearrangement of particles in the aquifer near the wellbore or introduction of suspended solids or air with the injection water. The effect of plugging is to impede movement of water from the injection well and into the aquifer; this plugging would cause excessive water level rise to occur in the injection well. Water level measurements from the long-term injection tests at sites C-26, B-44, and C-14 were analyzed to estimate magnitude of water level rise in the injection well which may have been caused by plugging of the well casing perforations and/or plugging of the aquifer adjacent to the well.

Four methods were used to compute water level rise due to plugging. Method 1 is based on comparison of observed water level rise in the injection well to observed water level rise in nearby observation well(s). Methods 2 and 3 are based on comparison of observed water level rise in the injection well to theoretical water level rise for the injection well. For method 2, theoretical water level rise for the injection well was projected using the semi-logarithmic distance versus water level rise method. For method 3, theoretical water level rise for the injection well was projected using the Theis (1935) non-equilibrium equation. Method 4 is based on comparison of water level rise after four hours of injection during step 2A for step-injection test A, before the long-term injection test, and during step 2B for step-injection test B after the long-term injection test.

Analyses of water level rise due to plugging using methods 1 through 4 are described below for well C-26A. For methods 1 and 2, water level rise data from nearby observation well C-26B were used.
for the analysis. Similar analyses using methods 1 through 4 were conducted for wells B-44B and C-14B. Results of analyses of plugging for wells C-26A, B-44B, and C-14B are summarized in Table 4.

**Table 4. Summary of results for water level rise due to plugging during long-term injection tests at pilot injection wells C-26A, B-44B, and C-14B**

<table>
<thead>
<tr>
<th>WELL IDENTIFIER</th>
<th>METHOD 1</th>
<th>METHOD 2</th>
<th>METHOD 3</th>
<th>METHOD 4</th>
<th>AVERAGE FOR METHODS 2, 3, AND 4</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-26A</td>
<td>27</td>
<td>45</td>
<td>39</td>
<td>36</td>
<td>40</td>
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<tr>
<td>B-44B</td>
<td>4</td>
<td>14</td>
<td>17</td>
<td>14</td>
<td>15</td>
</tr>
<tr>
<td>C-14B</td>
<td>6</td>
<td>19</td>
<td>20</td>
<td>22</td>
<td>20</td>
</tr>
</tbody>
</table>

*a Water level rise due to plugging during limited period of analysis
Well C-26A: Period from 7,000 minutes to end of injection test
Well B-44B: Period from 50,000 minutes to end of injection test
Well C-14B: Period from 4,000 minutes to end of injection test

*b Total water level rise due to plugging during long-term injection test

**Method 1.** Method 1 is based on comparison of observed water level rise in the injection well to observed water level rise in a nearby observation well. The Theis non-equilibrium equation predicts that, when the argument of the well function "u" is sufficiently small, water level rise would occur at the same rate for nearby observation wells and the injection well. Water level rise would increase linearly with the logarithm of time after injection started, and could be described using the Cooper-Jacob approximation to the Theis equation. The argument of the well function becomes smaller as duration of injection becomes larger. Water level rise at the observation wells was in accordance with the Cooper-Jacob approximation to the Theis equation beginning about 7,000 minutes after injection started at site C-26, beginning about 50,000 minutes after injection started at site C-14, and beginning about 4,000 minutes after injection started at site B-44. Beginning at these times, the difference between water level rise at the injection well and the observation well should be constant, if no plugging of the injection well would occur.

**Figure 1** is a graph of difference between water level rise at injection well C-26A and at observation well C-26B. The difference between water level rise at the injection well and water level rise at the observation well became larger during the injection period (Figure 1). These results indicate that rate of water level rise in the injection well was larger than rate of water level rise in the observation well. At observation well C-26B, rate of water level rise was predicted closely by the Cooper-Jacob approximation to the Theis equation beginning about 7,000 minutes after injection started. Because the theoretical rate of water level rise for the injection and the observation wells beginning 7,000 minutes after injection started would be similar if no plugging occurred, the
FIGURE 1. DIFFERENCE BETWEEN WATER LEVEL RISE AT INJECTION WELL C-26A AND AT OBSERVATION WELL C-26B DURING LONG-TERM INJECTION TEST
observed larger rate of water level rise in the injection well is attributed to plugging.

The amount of plugging, in feet of water level rise, was computed for the period beginning 7,000 minutes after injection started as the difference in water level rise at any time after 7,000 minutes, minus the difference in water level rise at 7,000 minutes. Figure 2 is a graph of difference between water level rise at injection well C-26A and water level rise at observation well C-26B, beginning 7,000 minutes after injection started. The difference between water level rise at injection well C-26A and at observation well C-26B, during the period from 7,000 minutes after injection started to the end of the injection period, was 27 feet (Table 4; Figure 2). Results for method 1 indicate that magnitude of water level rise due to plugging at injection well C-26A was about 27 feet during the period from 7,000 minutes after injection started to the end of the injection period.

Method 2. Method 2 is based on comparison of observed water level rise and theoretical water level rise for the injection well. Theoretical water level rise was computed based on observed water level rise for one or more observation wells, using the semi-logarithmic distance versus water level rise method. When only one observation well was used, the slope of the distance versus water level rise graph was computed using aquifer transmissivity obtained from prior pumping tests and from the long-term injection test. Effective well radius was assumed to be one foot. Theoretical water level rise corresponds to water level rise which would occur if well losses were negligible and well efficiency was 100 percent. Theoretical water level rise for the injection well was subsequently adjusted to include well losses based on computed injection well efficiency before commencement of the long-term injection test.

For injection well C-26A, average well efficiency was 40 percent after four hours of injection during injection step 2A, conducted before the long-term injection test (Table 2). For injection step 2A, observed water level rise was about 70 feet after four hours of injection at a rate of 1,060 gpm. Based on well efficiency of 40 percent, magnitude of water level rise due to formation losses would be about 28 feet and magnitude of water level rise due to well losses would be about 42 feet. Average injection rate for the long-term injection test was about 1,060 gpm. If no plugging occurred after injection step 2A, magnitude of water level rise due to well losses would not change and would be about 42 feet during the long-term injection test. For analysis of plugging at well C-26A using Method 2, adjusted theoretical water level rise was computed by adding 42 feet of water level rise due to initial well losses to theoretical water level rise computed using the distance versus water level rise method. Adjusted theoretical water level rise corresponds to water level rise which would be observed if no plugging occurred after injection step 2A and if well efficiency remained at 40 percent during the long-term injection test.
If no plugging of the well casing perforations and/or the aquifer adjacent to the injection well occurred, then observed water level rise at the end of the long-term injection test would be equal to adjusted theoretical water level rise. The difference between observed water level rise and adjusted theoretical water level rise at the end of the injection test can be attributed to water level rise which occurred due to plugging.

Observed water level rise, corrected for regional groundwater level decline, was 123.37 feet at injection well C-26A and 19.03 feet at observation well C-26B (Table 3). Based on the distance versus water level rise method using observation well C-26B, theoretical water level rise at the injection well was 35.83 feet at the end of the long-term injection test, and adjusted theoretical water level rise was 77.83 feet. Total water level rise due to plugging was 45 feet, based on the difference between observed water level rise and adjusted theoretical water level rise at the end of the long-term injection test (Table 4).

Method 3. Method 3 is also based on comparison of observed water level rise and theoretical water level rise for the injection well. However, for method 3, theoretical water level rise was computed using the Theis non-equilibrium equation. Effective well radius was assumed to be one foot. Theoretical water level rise corresponds to water level rise which would occur if well losses were negligible and well efficiency was 100 percent. Theoretical water level rise was subsequently adjusted to include well losses based on computed injection well efficiency before commencement of the long-term injection test. For analysis of plugging at well C-26A using method 3, adjusted theoretical water level rise was computed by adding 42 feet of water level rise due to initial well losses to theoretical water level rise computed using the Theis equation.

Figure 3 is a graph of adjusted theoretical water level rise, and observed water level rise corrected for regional groundwater level decline, for well C-26A. At the end of the injection test, observed water level rise was 123.37 feet, and theoretical water level rise, adjusted for an initial well efficiency of 40 percent, was 84.50 feet. Total water level rise due to plugging was 39 feet, based on the difference between observed water level rise and adjusted theoretical water level rise at the end of the injection test (Table 4).

Method 4. Method 4 is based on comparison of water level rise after four hours of injection during step 2A for step-injection test A, and during step 2B for step-injection test B. If no plugging occurred between the end of injection step 2A and the end of injection step 2B, and if injection rates were identical for both steps, water level rise would be similar for both steps. Because injection rates for injection step 2A and injection step 2B were not identical, observed water level rise was adjusted to remove effects of differences in injection rates.
Magnitude of water level rise due to plugging was computed as water level rise after four hours for injection step 2B minus adjusted water level rise after four hours for injection step 2A. Adjusted water level rise for injection step 2A was computed by multiplying observed water level rise for injection step 2A by the ratio of injection rate for step 2B to rate for step 2A. Observed water level rise in the injection well after four hours of injection was 69.92 feet for step 2A and 106.89 feet for step 2B. Average injection rate was 1,060 gpm for injection step 2A, and 1,070 gpm for injection step 2B. Water level rise after four hours for injection step 2A, adjusted for comparison to water level rise for injection step 2B, was 70.58 feet. Total water level rise, due to plugging which occurred from the end of injection step 2A to the end of injection step 2B, was computed to be 36 feet (Table 4).

SPECIFIC INJECTION CAPACITY AND WELL EFFICIENCY BEFORE AND AFTER REDEVELOPMENT FOR WELL C-14B

Well C-14B was redeveloped by pumping and surging after the long-term injection test and step-injection test B were completed. Step-injection test C was conducted after redevelopment to evaluate magnitude of recovery of specific injection capacity and injection well efficiency. The redevelopment procedure consisted of a pumping period of five to 15 minutes, followed by a recovery period of five to 15 minutes. This procedure was repeated until turbidity of the pumped water could not be detected by visual inspection.

Specific injection capacity increased from 11.1 gpm/ft for injection step 2B before redevelopment to 12.1 gpm/ft for injection step 2C after redevelopment (Table 2). Average of well efficiencies computed using three methods increased from about 60 percent before redevelopment to 66 percent after redevelopment. Other redevelopment methods which are being evaluated to mitigate the effects of injection well plugging include wire brushing, swabbing, and bailing; and airlift pumping and surging.

SUMMARY AND CONCLUSIONS

Results of analysis of water level rise in the injection well and observation well(s) during long-term injection tests indicate that additional water level rise occurred in the injection well due to plugging of the well casing perforations and/or plugging of the aquifer adjacent to the well. Magnitude of water level rise due to plugging during the long-term injection tests was computed using four methods.

Results indicate that at the end of the long-term injection tests, magnitude of water level rise due to plugging was about 40 feet for well C-26A, 15 feet for well B-44B, and 20 feet for well C-14B (Table 4). The fraction of total water level rise at the end of the long-term injection period which is attributed to plugging
was about 32 percent for well C-26A, 23 percent for well B-44B, and 15 percent for well C-14B.

Interim results of pilot recharge testing indicate that large rates of injection could be sustained during long-term injection recharge operations if the injection well is redeveloped at regular intervals to mitigate the effects of plugging and maintain reasonable hydraulic efficiency of the well. Results of ongoing pilot injection testing are being used to evaluate sustainable long-term injection rates, and optimum methods and frequency of required redevelopment. Similar investigations are planned using treated CAP water. Results of these investigations will be used to refine estimates of operational costs for long-term injection recharge operations.

REFERENCES


BIOGRAHMICAL PROFILES

BOUWER, Herman
Chief Engineer
U.S. Water Conservation Laboratory
4331 East Broadway Road
Phoenix, AZ 85040
(602) 379-4556

Herman Bouwer received BS and MS degrees in land drainage and irrigation from the National Agricultural University of Wageningen, The Netherlands, in 1949 and 1952, and a Ph.D. degree in hydrology and agriculture water management at Cornell University in 1955. In 1959, he became research hydraulic engineer with the U.S. Water Conservation Laboratory in Phoenix, Arizona, and served as Laboratory Director from 1972 until 1991. He is leader of a research group in subsurface water management, whose main projects deal with renovation of sewage effluent by groundwater recharge and effect of irrigated agriculture on groundwater.

BRAL, Kevin M.
Engineer
CH2M HILL, S.E., INC.
7201 N.W. 11 Place
Gainesville, FL 32605
(904) 331-2442

Kevin M. Bral received his MS degree in civil engineering from Ohio University in 1982. He received a Bachelors degree in civil engineering from the same institution in 1981. He is currently a registered Professional Engineer in the state of Florida where he specializes in the study, design, and implementation of artificial recharge systems.

BURNETT, Roger P.
Civil Engineer
US Bureau of Reclamation
Great Plains Region
Nebraska-Kansas Projects Office
203 West 2nd Street
Grand Island, NE 68801
(308) 381-5534

Roger Burnett is the head of the Projects Drainage Branch. He is a graduate Great Plains Region of South Dakota State University, Brookings, South Dakota with a BS degree in agricultural engineering, and is a registered P.E. As head of the Drainage Branch, Mr. Burnett’s duties include monitoring groundwater levels at all Bureau of Reclamation irrigation projects in Nebraska and northern Kansas. He is also involved in the investigation and design of subsurface drainage features to correct high groundwater conditions resulting from project water use, as well as other groundwater concerns associated with these projects.

BUSHNER, Greg L.
Hydrologist
Surface Water/Recharge Section
Hydrology Division
Arizona Department of Water Resources
15 S. 15th Avenue
Phoenix, Arizona 85007
(602) 542-1586

Greg Bushner received a BS degree in geology from Northern Arizona University, Flagstaff, Arizona, in 1983. He joined the Arizona Department of Water Resources in March, 1985 as a hydrologist. While at ADWR, he has participated in a number of projects including an investigation of the water resources of the Upper San Pedro Basin and a contaminant transport modeling investigation at the Phoenix-Goodyear Airport Superfund site. He is currently managing the Surface Water/Recharge Section of the Hydrology Division.

CAST, Larry D.
Project Geologist
US Bureau of Reclamation
Great Plains Region
Nebraska-Kansas Projects Office
203 West 2nd Street
Grand Island, NE 68801
(308) 381-5513

Larry D. Cast received a BS in geology from the University of Nebraska in 1964. He is presently working on the North Loup Project located in central Nebraska. The project, when completed, will consist of two dams and a reservoir, diversion works, and 150 miles of canal. His engineering, geology, and groundwater background includes working in Kansas, Colorado, and California.

CLINE, David J.
Research Assistant
Department of Hydrology and Water Resources
University of Arizona
Tucson, AZ 85721
(602) 621-8792

David Cline received a BS in geology from Northern Arizona University. He is currently a graduate student in the Department of Hydrology and Water Resources at the University of Arizona and is working with Dr. Gray Wilson on soil aquifer treatment of treated effluent. His thesis title is "Tracer Movement in the Vadose Zone During Rapid Infiltration of Wastewater."

CONROY, Aimée D.
6921 E. Thunderbird Road
Scottsdale, AZ 85254
(602) 998-1073

Aimée Conroy received her BS degree in civil engineering from Santa Clara University in Santa Clara, California and her MS degree in environmental engineering from the University of Arizona. She was a research assistant when included research took place.

COOLEY, Wayne R.
Hydrologist
Surface Water/Recharge Section
Hydrology Division
Arizona Department of Water Resources
15 S. 15th Avenue
Phoenix, Arizona 85007
(602) 542-1586
Wayne Cooley is currently a hydrologist for the Arizona Department of Water Resources in the Surface Water/Recharge Section. His prior experience with the Department was in the Modeling Section. Prior to working as a hydrologist, Mr. Cooley worked for Chevron, USA as a petroleum geologist from 1982 through 1986. Most of his work experience with Chevron consisted of reservoir geology characterization for well injection waterfloodling. He received his undergraduate degree in geology and environmental science from Hunter College and has completed some graduate level course work in geology from the University of Texas.

CROSS, Mark M.
Project Hydrogeologist
Errol L. Montgomery & Associates, Inc.
1075 East Fort Lowell Road, Suite B
Tucson, Arizona 85719
(602) 881-4912

Mark M. Cross received a BS degree in geology from Northern Arizona University in 1978 and a MS degree in hydrology from the University of Arizona in 1983. He has been a hydrogeologist with consulting firms in Arizona and Nevada since 1980, and has been with Errol L. Montgomery & Associates, Inc., since 1988. Areas of professional specialization include hydrogeologic assessment for artificial groundwater recharge, aquifer testing and analysis, and groundwater flow and chemical transport modeling.

DETONASSO, Stephen C.
President and General Manager
McCueKen Drilling, Inc.
1509 E. Ellwood
Phoenix, AZ 85040
(602) 268-0785

During the 15 years he has been with the firm, Stephen C. DeTommaso has been involved in all phases of the company's work relating to design, manufacture and installation of drainage systems. He also has participated in a number of drainage studies including installation of a test well at the University of Arizona Recharge Facility in Tucson, Arizona, the Maricopa Association of Governments Drywell Monitoring Project and preliminary feasibility studies for recharge and recovery projects. Mr. DeTommaso received a BS in construction engineering from Arizona State University in 1975.

DOS SANTOS, Placido
Water Resources Supervisor
Arizona Department of Water Resources
Tucson Active Management Area
310 South Meyer
Tucson, AZ 85701
(602) 626-5838

Placido Dos Santos is a water resources supervisor with the Arizona Department of Water Resources' Tucson Active Management Area office. Prior to joining the ADWR he earned a BS degree in geology from the University of Colorado, performed graduate studies in geosciences at the University of Arizona, and was employed as a metals exploration geologist in Colorado and Chile. Mr. Dos Santos currently manages the Tucson AMAs operations and compliance activities and is ADWR's project director for the Rillito Demonstration Recharge Project, a joint effort with Tucson Water and the Pima County Flood Control District.

DuBOIS, James F.
Environmental Program Supervisor
State Programs Unit, Hydrology Section
Arizona Department of Environmental Quality
2005 N Central Avenue
Phoenix, AZ 85004
(602) 257-2105

James DuBois is a hydrogeologist and manages the State Programs Unit of ADEQ's Hydrology Section. He holds a BA in geology from Carleton College and an MS in geology/geochemistry from the University of Kansas. At ADEQ he conducts hydrologic reviews of permit applications for Recharge and Underground Storage and Recovery projects as well as other facilities with potential to affect groundwater quality. He has helped shape agency policy for incorporating Recharge and Underground Storage and Recovery into its Groundwater and Aquifer Protection Permit Programs.

ERLEWINE, Terry L.
Senior Engineer, W.R.
California Department of Water Resources
San Joaquin District
3374 East Shields Avenue
Fresno, CA 93726
(209) 445-5100

Terry Erlewine received a BS and a MS in civil engineering from the University of California at Davis. He joined the Department of Water Resources San Joaquin District in 1978. There he has worked on groundwater studies for the San Joaquin Valley, including groundwater modeling. Since 1986, he has supervised many technical activities associated with planning for the Kern Water Bank. These have included development of a monitoring network, groundwater modeling, and related investigations.

FIELDEN, John R.
Senior Engineering Geologist
California Department of Water Resources
P.O. Box 942836
Sacramento, CA 94236-0001
(916) 322-1570

John Fielden received a BS degree in geology from the University of Oregon in 1973 and an MS degree in geology from Arizona State University in 1975. He is a Senior Engineering Geologist, specializing in hydrogeology, with the California Department of Water Resources. He is the coordinator of groundwater development activities within the Division of Planning. As such, he has been involved in all phases of numerous groundwater investigations undertaken by the Department.

GERBA, Charles P.
Professor
Department of Soil and Water Science
University of Arizona
Tucson, AZ 85721
(602) 621-6906

Charles Gerba received a BS in microbiology from Arizona State University and a Ph.D. from the University of Miami (Florida) also in microbiology. He currently has a joint appointment in the Departments of Soil and Water Science, and Microbiology and Immunology. His research interests include virus and bacterial survival and transport through the subsurface.

GOODRICH, James A.
Geologist
Orange County Water District
P.O. Box 8300
Fountain Valley, CA 92728-8300
(714) 963-5661

298
James Goodrich received his BS degree in geology from UCLA in 1973, his MS in hydrogeology from USC in 1978, and has completed post graduate studies in water resources engineering. The first fourteen years of his career were spent in the private sector working as a hydrogeologist, initially with emphasis on water supply, and later, groundwater contamination assessment. Since 1987, Mr. Goodrich has been the District Geologist and Director of Basin Management for the Orange County Water District in southern California. In this capacity, he heads five departments: Hydrogeology, Water Quality, State Certified Laboratory, Applied Bio-Technology Research, and Recharge Facility Operations.

GORDON, Grisel Z.
Research Microbiologist
Orange County Water District
P.O. Box 8300
Fountain Valley, CA 92728-8300
(714) 963-5661

Grisel Gordon received her undergraduate training at the University of Panama, and obtained a BS degree in biology in 1982. She pursued graduate studies in environmental microbiology at California State University, Long Beach (CSULB). From 1983 to 1987 she performed research on isolation and chemical characterization of marine bacterial exopolymers as a graduate assistant in the Microbiology Department of CSULB. She joined the Biotechnology Research Department at Orange County Water District in Fountain Valley, CA in 1988 to work with the recharge enhancement project concerning aquifer clogging by physical and microbial processes.

GOREY, Tim
Senior Geohydrologist
Salt River Project
1521 Project Drive
Tempe, AZ 85281
(602) 236-3575

Tim Gorey received a MS in geology from Arizona State University in 1990 and a BS in geology from Humboldt State University in 1983. He joined the Salt River Project in 1985 as a Geohydrologist I. While at SRP, he has managed projects in groundwater monitor and production well design and installation, conducted and analyzed aquifer tests, worked extensively on the proposed Granite Reef Underground Storage and Recovery Project and managed a well recharge project that used water treated at the well head.

GRAHAM, David D.
Hydrologist
U.S. Geological Survey, WRD
375 S. Euclid Ave.
Tucson, AZ 85719
(602) 670-6903

David Graham is currently a hydrologist working in Tucson for the Arizona District of the U.S. Geological Survey, Water Resources Division. He holds a BS in geology from the State University of New York at College at Cortland and did graduate work in geology at the State University of New York at Albany before joining the USGS in 1974. In Tucson he has been working on studies involving artificial recharge, movement of solutes through the unsaturated zone, and the potential for groundwater contamination.

HALPENNY, Leonard C.
President
Water Development Corporation
3938 East Santa Barbara

Tucson, AZ 85711
(602) 327-7412

Leonard Halpenny has worked in hydrology in Arizona for over 50 years, and has been a consultant since 1954. He has been working on recharge projects throughout that period, beginning with Willow Dam on Queen Creek in the 1940’s. A particularly interesting project was the Salt River Project Estrella Pumpback Project in the 1970’s. He is currently working on a recharge permit application on the Agua Fria River on behalf of the Sun City West Wastewater Treatment Plant, and on the recharge potential of the Hassayampa River west of the White Tank Mountains.

HARTLING, Earle C.
Water Reuse Coordinator
Sanitation Districts of Los Angeles County
P.O. Box 4998
Whittier, CA 90607-4998
(213) 699-7411

Mr. Hartling is a graduate of Loyola Marymount University in his native Los Angeles, receiving his Bachelor’s degree in biology in 1978, and his Master’s Degree in environmental engineering in 1981. Mr. Hartling has been a project engineer for the Sanitation Districts of Los Angeles County since 1981. He is currently serving as the Water Reuse Coordinator for that agency and has been involved in developing water reuse projects. He is working on new projects which will bring reclaimed water to the communities of Downey, Paramount, Bellflower, South Gate, Lynwood, Santa Fe Springs, Whittier and Santa Clarita.

JOHNSON, Bruce
Chief Hydrologist
Tucson Water
P.O. Box 27210
Tucson, AZ 85726
(602) 791-2685

Bruce Johnson received a BS and an MS in hydrology from the University of Arizona in 1968 and 1980 respectively. He began work at Tucson Water in 1975 as a hydrologist specializing in groundwater exploration, groundwater quality and hydrologic data evaluation. Since 1978 he has been chief hydrologist with Tucson Water and involved in data evaluation, groundwater development and recharge programs.

KUBE, Michael D.
Supervisory Civil Engineer
US Bureau of Reclamation
Great Plains Region
Nebraska-Kansas Projects Office
203 West 2nd Street
Grand Island, NE 68801
(308) 381-5525

Michael D. Kube is presently Chief of the Office Engineering Branch in the Kansas Projects Office, where the majority of his work involves the supervision of designs of civil features relating to the North Loup Irrigation Project. He has been employed with the Bureau of Reclamation for the last 16 years involved with all phases of planning, design, and construction of water resource projects. Mr. Kube graduated from the University of Nebraska in Lincoln in 1974 with a BS in agricultural engineering and is a registered civil engineer licensed in the State of Nebraska.

KWIAWTOWSKI, Peter J.
Hydrologist
CH2M HILL, S.E. Inc.
800 Fairway Drive, Suite 350
Deerfield Beach, FL 33441
(305) 426-4006

Peter J. Kwiatkowski is currently a hydrologist in the Groundwater Resources Department of CH2M HILL's Deerfield Beach, Florida office. He holds a BS in geology from Rensselaer Polytechnic Institute and an MS in hydrogeology from the University of South Florida. At CH2M HILL, Mr. Kwiatkowski has provided hydrogeologic expertise for a number of projects including groundwater remediation, water supply and permitting, aquifer evaluation, injection well testing, and aquifer storage and recovery (ASR).

LASSON, Richard O.
Civil Engineer
U.S. Bureau of Reclamation
Upper Colorado Region
125 South State Street
Salt Lake City, UT 84147
(801) 524-5520

Richard Lasson holds a B.S. degree (1972) in civil engineering and a Master of Engineering Degree (1972) from Brigham Young University. He has been employed with the Bureau of Reclamation since June 1972. He has worked extensively in hydrology, planning, and reports on many water resource projects in the Upper Colorado River Basin. He is currently the Chief of the Planning Coordination Branch in the Planning Division of the Upper Colorado Regional Office. He also serves as Regional Coordinator of the Groundwater Recharge Demonstration Program and the Superfund Technical Assistance Program.

LLURIA, Mario
Senior Principal Geohydrologist
Salt River Project
1521 Project Drive
Tempe, AZ 85281
(602) 230-5520

Mario Lluria received a science bachelor degree in geology and geophysics from Massachusetts Institute of Technology and a Doctor of Science in Chemistry from Universidad Central. He is a diplomate in Applied Hydrology of the E.S.T.I.M. of Spain and has done additional research and graduate work at the Universidad de Sevilla, Tennessee Technological University and Newark College of Engineering. Mr. Lluria is presently a Senior Principal Geohydrologist in the Water Resources Department where he works on both ground-water resources and environmental and water quality projects. He is the Project Manager for the Granite Reef Underground Storage and Recovery Project.

LUTTON, Richard J.
Geologist
U.S. Geological Survey
Geotechnical Laboratory
USAER Waterways Experiment Station
Vicksburg, MS 34380-6199
(601) 639-3393

Richard Lutton has been with the Corps of Engineers WES for 28 years working on a variety of projects in the fields of soil and rock mechanics, construction excavation, site characterization, and environmental engineering. He holds a PhD in geology from the University of Arizona and is registered in Mississippi as a professional engineer.

MACK, K. Bruce
Supervisor, Ground Water Division
Salt River Project
1521 Project Drive

Bruce Mack received a BS degree in hydrology from the University of Arizona in 1979. Mr. Mack currently supervises the Groundwater Resources Division that addresses groundwater issues and programs related to SRP's water and power operations. Responsibilities at SRP have included: technical team support for various groundwater contamination studies including three U.S. EPA Superfund sites in Arizona; designing and constructing development and monitoring wells for water supply and environmental assessments; and managing project teams involved with groundwater recharge projects of well injection and spreading basin.

MARCEAU, William
Recharge Program Specialist
Arizona Department of Environmental Quality
Central Palm Plaza Building
2005 N. Central Avenue
Phoenix, AZ 85004
(602) 257-2270

William Marceau holds an MA in biology from the New York State University College in Geneseo, New York. Since 1987 he has been with the Arizona Department of Environmental Quality, where he is currently employed as an Aquifer Protection Permit writer. Within the ADEQ Water Permits Unit he is the program specialist for recharge and underground storage and recovery (USR) projects. He is a member of the Society of Wetland Scientists, and is also involved with the permitting of wastewater treatment facilities which utilize constructed wetlands in the treatment process.

McLEOD, John
Project Manager
CH2M HILL, Inc.
8140 McPac, Bldg. 1, Suite 200
Austin, Texas 78759
(512) 346-2001

Mr. McLeod is a water supply engineer located in CH2M HILL's Austin, Texas office and has over 12 years of engineering and management experience. He is the project manager and project engineer for the ASR project. His engineering responsibilities include the development of the ASR well design and preparation of the ASR implementation plan. Other relevant experience includes all areas of project design for water systems including water treatment and water distribution. He received his MS in environmental engineering and is a registered professional engineer in Texas.

MEDINA, Miguel A., Jr.
Associate Professor
Department of Civil and
Environmental Engineering
Duke University
Durham, North Carolina 27706
(919) 660-5195

Miguel Medina holds a Ph.D. degree in water resources and environmental engineering sciences from the University of Florida, and is currently Associate Professor of Civil and Environmental Engineering, Duke University. He has conducted funded research in hydrology and water quality modeling. He is a consultant to the U.S. Environmental Protection Agency, the World Health Organization, the Research Triangle Institute of North Carolina, the Inter-American Development Bank, the Pan American Health Organization, the Ministry of Natural Resources in Venezuela and a UNESCO lecturer in Latin America.

Tempe, AZ 85281
(602) 236-2579
MERRITT, Michael L.
Research Hydrologist
US Geological Survey
Water Resources Division
9100 NW 36th Street, Suite 107
Miami, FL 33178
(305) 594-0655

Michael Merritt received a BS in physics from the University of Notre Dame in 1963, an MS in mathematics from New Mexico State University in 1970, and an MS in applied mathematics from Florida State University in 1976. His work has principally focused on applying mathematical and digital modeling techniques to hydrologic studies of surface-water and ground-water systems. Particular projects have included subsurface storage of portable water in brackish aquifers, surficial aquifer containment plume delineation, coastal saltwater intrusion simulation, and deep-well injection of municipal and industrial wastes.

MILLS, William R., Jr.
General Manager
Orange County Water District
P.O. Box 8300
Fountain Valley, CA 92728-8300
(714) 983-5881

William R. Mills Jr. was appointed General Manager of the Orange County Water District (OCWD) in fall 1987. OCWD is responsible for management of the groundwater basin in Northern Orange County. Mr. Mills is a graduate geological engineer from the Colorado School of Mines, with a MS degree in civil engineering from Loyola University of Los Angeles. Active in numerous organizations, he is also a Board Member of the California Water Resources Association, Chairman of the Groundwater Committee of the Association of California Water Agencies, Board Member of the National Water Supply Improvement Association, Chairman of the Water Management Committee, California-Nevada Section, AWWA, and Watermaster for Santa Ana River.

MOCK, Peter A.
Senior Hydrogeologist
CH2M HILL, Inc.
1620 West Fountainhead Parkway
Suite 550
Tempe, Arizona 85285
(602) 966-8188

Peter Mock is a Senior Hydrogeologist in CH2M HILL’s Tempe, Arizona office. Mr. Mock graduated from the University of Arizona in 1981 with a Bachelor’s degree in hydrology. He currently works on groundwater-related challenges of an applied nature. His experience ranges from field data collection through hydraulic and geochemical evaluation to presentation and consensus building. His primary professional interest is understanding and describing the occurrence and movement of groundwater in the subsurface.

MONTGOMERY, Errol L.
President
Errol L. Montgomery & Associates, Inc.
1075 East Fort Lowell Road, Suite B
Tucson, Arizona 85719
(602) 881-4912

Errol L. Montgomery received a BS degree in geology from Oregon State University in 1962, a MS degree in hydrogeology from the University of Arizona in 1963, and a Ph.D. degree in Hydrogeology and Geophysics from the University of Arizona in 1970. Dr. Montgomery was Assistant Professor of Geology at Northern Arizona University from 1970 to 1977 and thereafter was fully engaged in groundwater consulting. He is a Registered Professional Geologist in Arizona and California, and is President of Errol L. Montgomery & Associates, Inc.

MUNIZ, Albert
Engineer
CH2M HILL S.E., Inc.
800 Fairway Drive, Suite 350
Deerfield Beach, Florida 33441
(305) 426-4008

Albert Muniz received his engineering degree from the University of Florida in 1980. He has been a practicing Professional Engineer since 1984. Since joining CH2M HILL in 1983, Mr. Muniz has been a project manager for projects involving deep injection wells, water supply well field designs, recharge studies, aquifer storage/recovery, hazardous waste investigations and remediation, and saltwater intrusion studies. Mr. Muniz currently serves as the manager for the Water Resources and Civil Engineering Divisions in the Southeast Florida Region of CH2M HILL.

PETRUS, Rich
Senior Hydrogeologist
CH2M HILL, Inc.
5339 Alpha Road, Suite 300
Dallas, Texas 75240
(214) 980-2170

Mr. Petrus is a senior hydrogeologist in CH2M HILL’s Dallas office and has over 12 years of experience. He is the project hydrogeologist for the ASR project in Kerrville, Texas, and is responsible for all field studies as well as the development of the hydrogeologic model for the project. He has extensive experience in subsurface investigations including hazardous wastes investigations, landfill siting and leachate protection, as well as mine development and permitting for the coal industry. He received his BA and MA in geology and is a certified professional geological scientist.

PHIPPS, Donald W.
Research Microbiologist
Orange County Water District
P.O. Box 8500
Fountain Valley, CA 92728-8300
(714) 963-5661

Donald Phipps received a BS degree in biological science at the University of California, Irvine, in 1977. He obtained his MS degree from the School of Life Sciences at University of Nebraska in 1982. He joined the Biotechnology Research Department of Orange County Water District in 1985 to work with the gasoline hydrocarbon biodegradation program. Mr. Phipps possesses expertise in the application of electromechanical devices and computer-linked instrumentation to investigate biological programs, particularly relating to hydrocarbon biodegradation and groundwater infiltration processes.

POOL, Donald R.
Hydrologist
U.S. Geological Survey
Water Resources Division
375 S. Euclid Avenue
Tucson, AZ 85719
(602) 670-6729

Donald Pool received a BS degree in geology at Indiana University in 1979 and a MS degree in hydrology at the University of Arizona in 1986. He has worked for the Water Resources
Division of the U.S. Geological Survey since 1979. His primary duties have been the description and simulation of groundwater flow systems in Arizona basins. His other interests include the application of geophysical techniques in groundwater investigations. He has recently been cooperating with Dr. John Sumner of the University of Arizona Geosciences Department in the investigation of gravity techniques for estimating groundwater storage changes and aquifer storage properties.

POWELSON, David K.  
Research Associate  
Department of Soil and Water Science  
University of Arizona  
Tucson, AZ 85721  
(602) 621-6910

David Powelson received a BA in zoology from Pomona College, a MS in Watershed Management from University of Arizona, and a Ph.D. in Soil and Watershed Science from University of Arizona. His dissertation title was "Virus transport and survival in unsaturated and unsaturated flow through soil columns." He is continuing to study contaminant transport with Dr. Charles F. Gerba at the University of Arizona.

PYNE, David  
Water Resources Engineer  
CH2M HILL, Inc.  
P.O. Box 1647  
Gainesville, FL 32602  
(904) 331-2442

Mr. Pyne is the firmwide director of aquifer recharge programs at CH2M HILL. He has pioneered the development of the aquifer storage recovery, or ASR, concept in the United States for storage of water in fresh or brackish aquifers to meet seasonal, long-term or emergency demands. His experience with CH2M HILL also includes the areas of engineering design, ground and surface water hydrology. He graduated from Duke University of 1966 with a BS in civil engineering and from the University of Florida in 1967 with an MS in environmental engineering, specializing in water resources.

QUINONES-APONTE, Vincente  
Department of Civil and  
Environmental Engineering  
Duke University  
Durham, North Carolina 27706  
(919) 660-5197

Vicente Quinones-Apone holds a BS degree in land surveying and mapping from the Polytechnic University of Puerto Rico, and a major in water resources and sanitary engineering from the same university. During the last 8 years, as a Hydrologist with the U.S. Geological Survey W.R.D., he has conducted several groundwater resources studies which include; digital modeling, groundwater/surface-water interrelation, analyses of complex aquifer tests, and artificial recharge studies in karst aquifers. He is currently a graduate student in the Department of Civil and Environmental Engineering at Duke University.

RIDGWAY, Harry F.  
Chief Research Microbiologist  
Orange County Water District (OCWD)  
P.O. Box 8300  
Fountain Valley, CA 92728-8300  
(714) 963-5661

Harry Ridgway received a BS degree in microbiology in 1971 at San Diego State College. He did graduate studies in marine microbiology at Scripps Institute of Oceanography in La Jolla, CA and received his doctorate in 1976. In 1981, Dr. Ridgway joined the Orange County Water District in Fountain Valley, CA where he has performed extensive research in the areas of biofouling of reverse osmosis membranes and biodegradation of petroleum hydrocarbons by groundwater microorganisms. For the last several years, Dr. Ridgway has also managed research programs concerning the use of chemolithotrophic bacteria to remove nitrate from groundwater and aquifer clogging by microbial processes.

RIGBY, Martin F.  
Chief Research Microbiologist  
Orange County Water District (OCWD)  
P.O. Box 8300  
Fountain Valley, CA 92728-8300  
(714) 963-5661

Martin G. Rigby received his Ph.D. from the University of California, Irvine in 1981. Specializing in Water Resources Management, he has been employed since 1981 by the Orange County Water District as Chief Research Analyst, Director of Operations, and now, Director of Water Reclamation. Major interests include health and regulatory issues involving the use of reclaimed water and research into and development of new and improved reclamation technologies.

SKEHAN, Sean T.  
Hydrogeologist  
CH2M HILL S.E., INC.  
800 Fairway Drive, Suite 350  
Deerfield Beach, Florida 33441  
(305) 426-4008

Sean Skehan is presently a hydrogeologist with CH2M HILL S.E., INC. He holds a BS in geology from the University of Miami (Miami, Florida). Mr. Skehan has been a project manager for design, construction and testing of deep injection well systems, groundwater remediation, water supply wells and aquifer storage and recovery investigations. He is a registered Professional Geologist and was self employed as a certified general contractor prior to joining CH2M HILL in 1980.

STRAUSS, Laura J.  
Hydrogeologist  
Errol L. Montgomery & Associates, Inc.  
1075 East Fort Lowell Road, Suite B  
Tucson, Arizona 85719  
(602) 881-4912

Laura J. Strauss received a BA degree in geology and environmental studies from the University of California at Santa Barbara in 1983 and a MS degree in geology from the University of Arizona in 1986. She is a Registered Professional Geologist in Arizona. Since 1987, she has been a hydrogeologist with Errol L. Montgomery Associates, Inc. Areas of professional specialization include groundwater flow modeling, isotope hydrogeochemistry, hydrogeologic assessment for artificial groundwater recharge, and aquifer testing and analysis.

TINNEY, J. Craig  
Principal Hydrologist  
Flood Control Planning and Development Division  
Pima County Department of Transportation and Flood Control District  
201 N. Stone, 4th Floor  
Tucson, AZ 85701  
(602) 740-6350
J. Craig Tinney holds a Ph.D. in watershed hydrology and a MS in agricultural economics from the University of Arizona. He has held positions with the University of Arizona Water Resources Research Center, the Arizona Department of Water Resources, and the U.S. Department of Agricultural Economic Research Service. Among the topics that he has investigated and published are: groundwater recharge, the economics of impaired water quality, flood prone land use, and stormwater quality management. Most recently, he has been involved with the development of NPDES stormwater quality management response for Pima County which heavily utilize Geographical Information Systems.

TUBBS, Michael K.
Director
Tucson Water
P.O. Box 27210
Tucson, AZ 85726
(602) 791-2666

Michael K. Tubbs graduated from Southern Methodist University in 1961 with a BS in electrical engineering. In 1982 he graduated the University of Michigan Graduate School of Business Public Utility Executive Program. He has held various positions with the Dallas Water Utilities, including engineer (1961-1966), assistant director (1980-1981), and director (1985-1988). He is currently the director of Tucson Water.

WILSON, Lorne G.
Hydrologist
Department of Hydrology and Water Resources
University of Arizona
Tucson, AZ 85721
(602) 621-9108

L. G. Wilson earned a BS in agriculture at the University of British Columbia, a MS in irrigation at University of California, and a Ph.D. in soil physics at the University of California. His research interests include artificial recharge of groundwater, vadose zone monitoring for hazardous wastes, and soil aquifer treatment.

YAHYA, Moyasar T.
Research Associate
Department of Microbiology and Immunology
University of Arizona
Tucson, AZ 85721
(602) 621-6910

Moyasar T. Yahya received a BS in 1977 and a MS in 1979 in food science and technology, University of Mosul, Iraq. Besides working on his Ph.D. dissertation, Yahya conducted several research projects on the nutritional and chemical and microbial evaluation of some food material in the United States. During the last three years, he participated in several projects to study the fate of microorganisms in groundwater in the United States and Canada. Yahya has several publications on the inactivation of bacteria, viruses, and parasites by different chemical disinfectants.