THE 8TH BIENNIAL SYMPOSIUM
ON THE ARTIFICIAL RECHARGE
OF GROUNDWATER

Symposium Proceedings

RECHARGE FOR THE PEOPLE!

Embassy Suites Hotel
Tempe, Arizona
June 2-4, 1997

Sponsored by

Arizona Hydrological Society
U.S. Water Conservation Laboratory of USDA-ARS
Salt River Project and Arizona Department of Water Resources
RECHARGE FOR THE PEOPLE

The Arizona Hydrological Society,
U.S. Water Conversation Laboratory of the USDA-ARS,
Salt River Project, and Arizona Department of Water Resources
are proud to present

The 8th Biennial Symposium
on Artificial Recharge of Groundwater

to be held June 2-4, 1997 at the
Embassy Suites in Tempe, Arizona

SCHEDULE
Symposium Schedule

(Only primary authors are shown)

Monday - June 2, 1997

7:00 a.m. - 4:00 p.m.  Registration

8:00 a.m. - 8:30 a.m.  Opening Remarks - Mario Luriea
                      Keynote Speaker - David S. "Sid" Wilson, General Manager
                      Central Arizona Water Conservation District

8:30 a.m. - 10:15 a.m.  SESSION 1  Moderator, Doug Bartlett
  - H. Bouwer  Predicting and Managing Infiltration for Artificial Recharge
  - C. Neal  Avra Valley Recharge Project: A CAP Spreading Basin Recharge
             Operation in the Tucson Active Management Area Developed through
             Local and State Partnering
  - D. Stous  Equus Beds Groundwater Recharge Demonstration Project, South-
              Central Kansas
  - W. Monheiser  Results of the High Plains States Groundwater Demonstration Program
  - M. Luriea  Operation of the Granite Reef Underground Storage Project (GRUSP)

10:15 a.m. - 10:30 a.m.  Break

10:30 a.m. - 12:15 p.m.  SESSION 2  Moderator, Sheila Ehlers
  - T. Henley  The Arizona Water Banking Authority-Storing Colorado River Water for
               Arizona’s Future
  - E. Weiland  Geochemical Investigations to Evaluate Compatibility Between Recharge
                Sources, Groundwater Aquifers, and Vadose Material during Recharge
                Feasibility Investigations, Pima County, Arizona
  - S. Eden  Regional Recharge Planning in the Tucson Active Management Area
  - M. Light  Operating The Sweetwater Recharge Facilities
  - A. Conroy  A Water Reuse Partnership for Today, Tomorrow and the 21st Century:
              The Roosevelt Irrigation District Exchange Agreement
Tuesday - June 3, 1997

7:00 a.m. - 12:00 p.m. Registration

8:30 a.m. - 10:15 a.m. SESSION 5  Moderator, Aimee Conroy

D. Swieczkowski  Arizona Department of Water Resources Underground Water Storage Program: From Application to Permit

C. Close  Electronic Permitting

D. Murphy  Flow and Transport Modeling Beneath Artificial Recharge Basins

K. Lansey  Model Development for the Operation of the Granite Reef Underground Storage Project

G. Johnson  Recharge Potential on the Snake River Plain, Idaho: A Drop in the Bucket?

T. Thompson  The Wetlands of Avondale: A Water Treatment System Using Constructed Wetlands and Artificial Recharge

L. Frost  The Town of Gilbert Experience with Aquifer Storage and Recovery of Reclaimed Water

M. DeRosa  Artificial Groundwater Recharge and the Flood Control District: Work in Progress

M. Johnson  Artificial Recharge in the Las Vegas Valley: An Operational History

10:15 a.m. - 10:30 a.m. Break

10:30 a.m. - 12:15 p.m. SESSION 6  Moderator, Gary Small

B. Lauerman  Results of a Virus Seeding Experiment in an Unconfined, Cold-Water, Sand and Gravel Aquifer, Frenchtown, Montana

P. Fox  Investigation of Soil Aquifer Treatment for Sustainable Water Reuse

J. Johnson  Chemical Processes In the Vadose Zone: Implications for Groundwater Recharge of CAP Water and High Quality Effluent at the Scottsdale, Arizona "Water Campus"

L. Ester  SRP Case Study in Water Measurement - Application of the AxSys Remote Station At the Granite Reef Underground Storage Project (GRUSP)

C. Close  Vadose Zone Recharge Wells: Field Evaluation of an Innovative Design

R. Randall  Vadose Zone Injection Well Demonstration for Town of Gilbert - The HoHum Well

G. Bushner  Recharge at the Ken McDonald Golf Course, Tempe, Arizona

P. Fox  Safe Well Maintenance Technology for Direct Recharge of Tertiary Effluent

D. Toy  Aquifer Recharge and Recovery - A Case Study of the Sun Lakes Recharge/Recovery Pilot Project

12:15 p.m. - 01:30 p.m. Hosted Lunch  Moderator, Rita Pearson, Director

Rita Pearson, Director  Arizona Department of Water Resources
01:30 p.m. - 03:15 p.m.  SESSION 7  Moderator, Drew Swieczkowski

03:15 p.m. - 03:30 p.m.  Break

03:30 p.m. - 05:15 p.m.  SESSION 8  Moderator, Marie Light

- G. Bushner  Augmenting Existing Water Supplies for the City of Chandler, Arizona Through Aquifer Storage Using Four Direct Injection Wells
- M. Burke  A Comparative Study of Injection Well Rehabilitation Methods
- R. D. Pyne  Regulatory Developments Relating to Aquifer Storage Recovery (ASR) with Raw and Reclaimed Water
- S. Whitmer  Hydraulic Characterization of Wetlands to Improve Treatment Efficiency
- S. Gerke  Evaluation of Nitrogen Removal Rates in the Wetlands Treatment System of Kingman, Arizona
- K. Lansey  The Effect of Ozonation on Organic Residua...
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RECHARGE FOR THE PEOPLE!
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Clifford A. Neal, Tom Harbour, and Michael W. Block

**Chemical Processes in the Vadose Zone: Implications for Groundwater Recharge of CAP Water and High Quality Effluent at the Scottsdale, Arizona “Water Campus”**
Juliet S. Johnson, Lawrence A. Baker, and Peter Fox

**Electronic Permitting**
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**Evaluation of Nitrogen Removal Rates in the Wetlands Treatment System of Kingman, Arizona**
Sara Gerke, Lawrence Baker, and Don Manthe

**Geochemical Investigations to Evaluate Compatibility Between Recharge Sources, Groundwater Aquifers, and Vadose Material During Recharge Feasibility Investigations, Pima County, Arizona**
Eric Wellan

**Hydraulic Characterization of Wetlands to Improve Treatment Efficiency**
Shawn Whitmer and Roland Woss

**Investigation of Soil Aquifer Treatment for Sustainable Water Reuse**
Peter Fox, Margaret Nellor, Robert Arnold, Kevin Lansey, Charles Gerba, Gary Amy, William Yanko, Roger Baird, Martin Reinhard, and Sandra Houston

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FOREWORD

This year, the 8th Biennial Symposium on the Artificial Recharge of Groundwater suffered the loss of a long-time organizer and sponsor of past symposia - the University of Arizona, through the Water Resources Research Center in Tucson. Funding limitations did not allow the University to participate; an unfortunate sign of our times. This loss resulted in a late start to organization of the symposium, but we believe the content and value of the symposium has not suffered as the reader will see in the following papers presented at the symposium on June 2nd and 3rd, 1997 in Tempe, Arizona. To offset the loss of the University of Arizona, the organizing committee enlisted the assistance of the Arizona Hydrological Society to provide funding, organizational skills, and support. As in past symposia, assistance was also provided by the U.S. Water Conservation Laboratory of the USDA-ARS, the Salt River Project, and the Arizona Department of Water Resources. Our heartfelt “thanks” to these organizations for their support!

Special thanks and recognition also go to the Organizing Committee. Dr. Herman Bouwer of the U.S. Water Conservation Laboratory of the USDA-ARS and Mario Lluria of the Salt River Project have continued their individual, long-time commitment to the symposium and provided guidance and historical perspective for continuity of the symposium program. The Arizona Hydrological Society was represented by Doug Bartlett, Lynda Person, and Suzanne Kirk (Dames & Moore); Greg Bushner (HydroSystems); Gary Burchard and Troy Day (Arizona Department of Environmental Quality); Lee Wilkening (AGRA Earth and Environmental); Herb Schumann (Herbert S. Schumann & Associates); and Floyd Marsh (City of Scottsdale). The Arizona Department of Water Resources participated through the services of Greg Wallace, Drew Swieczkowski and Jim Swanson.

Our financial sponsors included the Salt River Project, Dames & Moore, Errol L. Montgomery & Associates, and Kleinfelder. Contributors included the Arizona Department of Water Resources, Brown and Caldwell, Carollo Engineers, Hargis & Associates, HydroSystems, and Southwest Ground-water Consultants. The financial generosity of these organizations allowed us to keep the registration fee for the symposium as small as possible. All of the participants in the Symposium have benefited through their financial support. Many thanks on behalf of the Organizing Committee.

A special thanks goes to Suzanne Kirk of Dames & Moore for the many hours she lent to preparing mailings, keeping track of registration, organizing papers for this proceedings volume, reminding the Organizing Committee to assemble, and generally keeping the rest of us on track.

I hope you will enjoy the Symposium and find the information contained herein of value. May we never forget that Arizona’s future lies in the successful application of groundwater recharge as presented in these papers!

Sincerely,
Doug Bartlett, Chairman. Symposium Organizing Committee
LIST OF SPONSORS AND CONTRIBUTORS

The organizers of the 8th Biennial Symposium on the Artificial Recharge of Groundwater wish to express their gratitude to the following sponsors and contributors for their financial support of this symposium:

Sponsors:

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Contributors:

Arizona Department of Water Resources
Brown and Caldwell
Carollo Engineers
Hargis + Associates, Inc.
HydroSystems, Inc.
Southwest Ground-Water Consultants
ACKNOWLEDGMENTS

Special recognition goes to the Symposium Organizing Committee members and their organizations for contributing their valuable time, effort, and resources to plan, organize, and conduct this symposium:

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# PROGRAM

**MONDAY, JUNE 2, 1997**

## INTRODUCTION

**Mario Lluria**

08:00a - 08:30a    **KEYNOTE: SPEAKER - Sid Wilson**

## SESSION 1

**Moderator - Doug Bartlett**

08:30a - 10:15a

1. Herman Bouwer  
   - Predicting and Managing Infiltration for Artificial Recharge

2. Clifford A. Neal, Tom Harbour, and Michael W. Block  
   - *Avra Valley Recharge Project: A CAP Spreading Basin Recharge Operation in the Tucson Active Management Area Developed through Local and State Partnering*

3. David H. Stous, Jerry Blain, and Michael Dealy  
   - *Equus Beds Groundwater Recharge Demonstration Project, South-Central Kansas*

4. H. Douglas Yoder and William Monheiser  
   - *Results of the High Plains States Groundwater Demonstration Program*

5. Mario Lluria  
   - *Operation of the Granite Reef Underground Storage Project (GRUSP)*

10:15a - 10:30a    **BREAK**

## SESSION 2

**Moderator - Sheila Ehlers**

10:30a - 12:15p

6. Timothy J. Henley and James G. Jayne  
   - *The Arizona Water Banking Authority - Storing Colorado River Water for Arizona's Future*

7. Eric Weiland  
   - *Geochemical Investigations to Evaluate Compatibility Between Recharge Sources, Groundwater Aquifers, and Vadose Material during Recharge Feasibility Investigations, Pima County, AZ*

8. Susanna Eden and Katharine Jacobs  
   - *Regional Recharge Planning in the Tucson Active Management Area*

9. Marie Light, Bruce Prior, and Peter Chipello  
   - *Operating the Sweetwater Recharge Facilities*

10. Aimee Conroy and Stan Ashby  

12:15p - 01:30p    **LUNCH** - Karl Kohlhoff, Assistant Utilities Director, City of Mesa  
                    Presentation of Groundwater Recharge Award
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                      Arizona Department of Water Resources Underground Water Storage Program: From Application to Permit  
                      "Electronic Permitting"  
                      - David Murphy  
                      Flow and Transport Modeling Beneath Artificial Recharge Basins  
                      - Kevin Lansey, Feyzan Misirli and Maili Wang  
                      Model Development for the Operation of the Granite Reef Underground Storage Project |

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                      Results of a Virus Seeding Experiment in an Unconfined, Cold-Water, Sand and Gravel Aquifer, Frenchtown, Montana  
                      Investigation of Soil Aquifer Treatment for Sustainable Water Reuse  
                      - Peter Fox, Margaret Nellor, Robert Arnold, Kevin Lansey, Charles Gerba, Gary Amy, Rodger Baird, Martin Reinhard and Sandra Houston  
                      Chemical Processes In the Vadose Zone: Implications for Groundwater Recharge of CAP Water and High Quality Effluent at the Scottsdale, Arizona "Water Campus"  
                      - Juliet S. Johnson Lawrence A. Baker and Peter Fox  
                      SRP Case Study in Water Measurement - Application of the AXSYS Remote Station at the Granite Reef Underground Storage Project (GRUSP)  
                      - Lee W. Ester                                             |
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<td>10:30a - 12:15p</td>
<td>Christine H. Close, Floyd Marsh, and Gary G. Small - <em>Vadose Zone Recharge Wells: Field Evaluation of an Innovative Design</em></td>
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<td>Richard A. Randall and Lonnie K. Frost - <em>Vadose Zone Injection Well Demonstration for Town of Gilbert - The HoHum Well</em></td>
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12:15p - 01:30p **LUNCH** - Rita Pearson, Director of Arizona Department of Environmental Quality
SESSION 7

01:30p - 03:15p

❖ Panel Discussion

Planned Groundwater Recharge Projects

03:15p - 03:30p

BREAK

SESSION 8

03:30p - 05:15p

❖ Greg L. Bushner, Gary G. Small and Christine H. Close

Augmenting Existing Water Supplies for the City of Chandler, Arizona Through Aquifer Storage Using Four Direct Injection Wells

❖ Michael S. Burke and Stephen P. Tanner

A Comparative Study of Injection Well Rehabilitation Methods

❖ David Pyne

Regulatory Developments Relating to Aquifer Storage Recovery (ASR) with Raw and Reclaimed Water

❖ Shawn Whitmer and Roland Wass

Hydraulic Characterization of Wetlands to Improve Treatment Efficiency

❖ Sara Gerke, Lawrence Baker and Don Manthe

Evaluation of Nitrogen Removal Rates In the Wetlands Treatment System of Kingman, Arizona

❖ Kevin Lansey, John Hillman, David Quanrud, and Robert Arnold

The Effect of Ozonation on Organic Residuals in Secondary Effluent Including Biodegradability during Soil-Aquifer Treatment

WEDNESDAY - JUNE 4, 1997

6:00a - 3:00p

FIELD TRIP

A full-day field trip is planned. Travel by bus to see existing and planned recharge projects in the Phoenix area. The cost is included in the symposium registration fee. Space is limited. Check at the registration table to confirm your reservation or to sign up.
A WATER REUSE PARTNERSHIP FOR TODAY, TOMORROW AND THE 21ST CENTURY: THE ROOSEVELT IRRIGATION DISTRICT EXCHANGE AGREEMENT

Aimée Conroy, P.E.  and Stan Ashby

ABSTRACT

Water is a very precious commodity in the desert. Consequently, many different organizations are trying to preserve this resource in various ways. One water conservation method is to use treated wastewater effluent for agricultural irrigation and other industrial uses. In most cases, the end user buys treated effluent. Sometimes effluent is given away. In the City of Phoenix’s case, an exchange agreement has been instituted between the City of Phoenix, the Roosevelt Irrigation District, Salt River Project and the Salt River Pima-Maricopa Indian Community.

In 1994, the City of Phoenix (Phoenix), Roosevelt Irrigation District (RID), Salt River Pima-Maricopa Indian Community (SRPMIC) and Salt River Project (SRP) entered into an innovative four party agreement. In the RID Exchange Agreement, Phoenix agreed to deliver 30,000 acre-feet of treated wastewater effluent annually from the 23rd Avenue Wastewater Treatment Plant to RID. RID then curtailed pumping 30,000 acre-feet of groundwater within the SRP service area. This freed up 30,000 acre-feet of surface water allocation from reservoirs along the Salt and Verde Rivers. SRP, as its part, agreed to deliver 20,000 acre-ft per year of this allocation to Phoenix water treatment facilities in exchange for Phoenix’s effort in delivering the treated effluent. The 10,000 acre-ft per year left over from this allocation is delivered to the Salt River Pima-Maricopa Indian Community as part of the SRPMIC’s water settlement with Phoenix and SRP. The exchange agreement in reality enables RID to pump less groundwater in SRP project areas, while SRP transfers raw water from surface water impoundments to both Phoenix and SRPMIC. This reduces groundwater pumping in already overdrawn areas. This paper describes the RID Exchange Agreement and the modifications made by both Phoenix and RID to make the agreement a reality.

1Paper presented at the 8th Biennial Symposium on Artificial Recharge of Groundwater, Tempe, AZ, June 2-4, 1997
2Aimée Conroy, City of Phoenix Water Services Department, 23rd Avenue WWTP, 2301 W. Durango Street, Phoenix, AZ 85009. Telephone No. (602) 534-2976.
3Stan Ashby, Superintendent, Roosevelt Irrigation District, 103 W. Baseline, Buckeye, AZ 85326. Telephone No. (602) 386-2046.
INTRODUCTION

To those of us who live in the arid southwest, especially the Sonoran Desert, water is a commodity. During those hot summer days when you can figuratively “fry an egg on the sidewalk,” a cool glass of water is a necessity. Yet, how does the water reach the tap, and where does it go after it goes down the drain? That is an age old question that people all over the world spend a lifetime answering and working on. In the Metropolitan Phoenix area, much of the water we drink is supplied by surface water sources operated by the Salt River Project (SRP) and treated to drinking water quality by municipalities, such as the City of Phoenix (Phoenix). Did you ever stop and think about how water delivery and supply is managed to ensure that it is delivered to your home or work? The purpose of this paper is to describe one small piece of the puzzle that makes up the answer to this question.

In 1994, the City of Phoenix, the Roosevelt Irrigation District (RID) and the Salt River Project entered into an innovated three part agreement to delivery water to all three entities and the Salt River Pima-Maricopa Indian Community (SRPMIC). A large part of the agreement includes the transfer of tertiary treated reclaimed wastewater to the RID in exchange for them not pumping a number of wells in the SRP service area. SRP, in turn, supplies surface water from reservoirs along the Salt and Verde Rivers to both Phoenix and the Salt River Pima-Maricopa Indian Community. This agreement, commonly referred to as the RID Exchange Agreement, did not come together without a lot of work, time and effort on the part of many individuals within all the organizations. A number of structural modifications also had to be made at facilities within the City of Phoenix and RID. This paper discusses the impacts of the RID Exchange Agreement on the RID and the City of Phoenix 23rd Avenue Wastewater Treatment Plant (23rd Avenue WWTP).

BACKGROUND

The City of Phoenix 23rd Avenue WWTP is a 57 million gallon per day (MGD) tertiary wastewater treatment plant that is solely owned and operated by the City of Phoenix. It was the first wastewater treatment plant in Phoenix and was started in 1923. The plant was been upgraded and expanded a number of times. The last expansion and upgrade was completed in 1995. The plant treats a mixture of domestic and industrial wastewater from the central Phoenix corridor, roughly located east of Interstate 17 and south of the Central Arizona Project Canal. The plant is also currently treating approximately 20 MGD of wastewater pumped from a large interceptor sewer which delivers wastewater from Mesa, Phoenix, Scottsdale and Tempe to the 91st Avenue WWTP, which is owned by the Subregional Operating Group (SROG) and operated by the City of Phoenix. The water pumped into the plant supplements the gravity flow from its service area.
The treatment process used is an advanced activated sludge system which uses anoxic selectors within the aeration basins to biologically remove ammonia and other nitrogen containing compounds from the wastewater. The biological system is followed by essentially a water treatment plant with rapid mixers, flocculation basins, the ability to add chemicals to assist the filtration process and monimedia filters. The treatment process and the modifications required to meet the requirements of the Exchange Agreement are discussed in more detail in later sections.

Wastewater effluent from the 23rd Avenue WWTP has been continuously discharged to the Salt River bed since the original primary treatment facility was built in 1923. A local area farmer has disputed grandfathered water rights to a portion of the wastewater flow, because the effluent originally flowed through his irrigation canal on its way to the river. This amounts to approximately 10 MGD, most of which is unused and discharged to the river bed. Even though effluent was continuously discharged to the Salt River, it wasn’t until approximately 1967 that the flow was significant enough to reach the 91st Ave WWTP where effluent is also discharged to the Salt River (Halpenny and Green, 1975). Halpenny and Clarke (1977) estimated that in the reach between 23rd and 91st Avenue WWTP’s, approximately 80% of the effluent recharges the groundwater, 17% joins the discharge of the 91st Avenue WWTP and 3% is lost to evapotranspiration.

The Roosevelt Irrigation District was formed in 1923. It consists of slightly more than 38,000 acres of land north of Buckeye, Arizona and west of the Agua Fria River. The District water supply, prior to the Exchange Agreement, was derived entirely from groundwater. Groundwater is pumped from 102 wells, 52 located east of the Agua Fria River (east side wells) within the SRP project area. The remaining 50 wells are located west of the Agua Fria River within the RID service area. Well water is collected and distributed by 50 miles of main canals and 136 miles of laterals, most of which are concrete lined (ADWR and UA, 1983).

The CC1 or Main Canal originates near the 23rd Avenue WWTP at 19th Avenue just north of Lower Buckeye Road. The Main Canal flows east to west and gradually increases in size and carrying capacity.

RID EXCHANGE AGREEMENT

The RID Exchange Agreement is a complex mixture of water rights related legal documents that stemmed from a lawsuit originated by the Salt River Pima-Maricopa Indian Community against SRP and the City of Phoenix. The politics of the original lawsuit are much too complex to describe in much detail, but resulted in the RID Exchange Agreement which is officially titled Exhibit “3.k” to the SRPMIC Water Rights Settlement Agreement. The Exchange Agreement took many years of negotiation to complete with the final agreement completed in the summer of 1994. (SRPMIC, 1994)
The heart of the RID Exchange Agreement is the transfer of water between the RID and Phoenix in the form of wastewater effluent from the 23rd Avenue WWTP; between SRP and RID in the form of well water transferred from SRP and RID wells into the SRP distribution system; and between SRP, Phoenix and the SRPMIC in the form of surface water transferred from the reservoirs along the Salt and Verde Rivers to Phoenix and SRPMIC. All in all, 90,000 acre-ft of water is transferred between the four entities. This may seem like a simple concept, but it is much more complicated than it seems. A number of different agreements were reached along with a number of engineering studies to ensure the water was transferred efficiently. The responsibilities of each of the three main participants are described in more detail below.

Phoenix

Phoenix is responsible for supplying 30,000 acre-ft of advanced treated or tertiary effluent, which meets the requirements of the Arizona Department of Environmental Quality’s (ADEQ) Wastewater Reuse Regulation standards for effluent used on food crops that could be consumed raw, to RID. The effluent delivery will occur “upon the request of RID and in such increments and at such times and delivery points that can be fully utilized by RID for satisfaction of RID’s then existing water delivery needs” (Phoenix and RID, 1994). For example during 1997, RID has requested that effluent be supplied to the district between the months of March and October at an average amount of 39 MGD. The actual amounts transmitted to RID varies from day to day based on time of year, RID cropping patterns and current water demands and rainfall events. It is Phoenix’s responsibility to maintain accurate records of how much effluent has been delivered to RID and to maintain an on-going record of the total amounts delivered. The cost of providing effluent is solely Phoenix’s responsibility. (Phoenix and RID, 1994)

In order to provide effluent at the quality required by the Exchange Agreement, numerous facility upgrades were developed for the 23rd Avenue WWTP. These upgrades are described in the following sections. Phoenix was responsible for planning, design and construction of the connection from the 23rd Avenue WWTP effluent discharge line to the RID Main Canal and the improvements and partial relocation of the RID Main Canal that borders the plant. The cost of the improvements and relocation were shared by RID and Phoenix (Phoenix and RID, 1995(a)). Phoenix also has a cost sharing role in the transfer of water by RID to SRP. Phoenix is responsible for 60% of the Operation, Maintenance and Replacement (OM&R) cost of pumping water from SRP Exchange wells leased by SRP to RID and are used to transfer water from RID to the SRP distribution system, along with 50% of the well pump tax assessed to RID for each acre-ft of groundwater pumped from these wells (SRPMIC, 1994).

Effluent water quality is an important issue. Phoenix is responsible for ensuring that effluent delivered to RID continuously meets the water quality criteria set out by ADEQ, as well as any subsequent changes to the regulations in the future. As a result,
Phoenix is solely responsible for obtaining any permits required for the Exchange Agreement and for ensuring that the water quality meets the standards of the Wastewater Reuse regulations. It is Phoenix’s responsibility to notify RID, if the quality of the delivered effluent is below these water quality standards (Phoenix and RID, 1995 (b)).

RID

RID’s responsibilities include transferring 30,000 acre-ft of groundwater, pumped from their east side wells and SRP Exchange wells leased by RID, to the SRP distribution system. RID is required to pay SRP a rental fee for each Exchange well based on the OM&R cost associated with pumping their own wells. RID agreed to pay SRP a rental fee equal to 110% of the average OM&R cost associated with operating their own wells, excluding the well pump taxes, for each acre-ft of water pumped from the Exchange wells to meet the Exchange Agreement. RID delivers to SRP no more that 1.0 acre-ft of water for every acre-ft of effluent it receives from Phoenix. RID is required to deliver water from its east side wells into the SRP distribution system, based on the amount water RID’s facilities can deliver, at the times and locations specified by SRP. RID is not required to deliver water to SRP from the east side wells, if at anytime the delivery disrupts RID’s ability to meet its own irrigation needs. It is also RID’s responsibility to pay SRP the lease payment and to bill Phoenix for its portion of the costs associated with the operation of the SRP Exchange wells. (RID and SRP, 1994)

Salt River Project

SRP’s role in the process is simpler, but just as important. As water is received from RID, SRP credits Phoenix’s and SRPMIC’s water delivery account for the exchange based on 1 acre-ft of water for every acre-ft of water delivered by RID. The exchange credits will accrue at either the SRP Exchange well or RID’s east side well pump outlets. SRP will allocate these credits in the two accounts based on transferring two-thirds of the credits to Phoenix and one-third of the credits to SRPMIC. Deliveries of surface water by SRP to Phoenix and SRPMIC are deducted from their accounts up to the annual amounts of 20,000 acre-ft of water to Phoenix and 10,000 acre-ft of water to SRPMIC. SRP will deliver Phoenix’s portion of the water at any present and future water treatment plant connected to one of SRP’s canals. (SRPMIC, 1994)

MODIFICATIONS TO 23RD AVENUE WASTEWATER TREATMENT PLANT

Modifications to the 23rd Avenue WWTP took place during two construction projects, which took place between 1988 and 1995. The first construction project upgraded the plant to a biological nutrient removal (BNR) activated sludge system. The BNR process uses a series of anoxic selector zones and internal recycle of treated wastewater to biologically degrade ammonia and other nitrogen containing compounds. It was determined that due to expected changes to the Arizona Surface Water Quality
Standards that the amount of total nitrogen discharged to "Waters of the United States" (which the RID Main Canal is considered) would be limited to less than 10 mg/l. This limit would become a requirement of the plant's National Pollutant Discharge Elimination System (NPDES) permit which regulates the water quality of the plant's effluent. This limit was not achievable by the existing biological treatment process, but was possible if the treatment process was converted to an BNR system. The second phase of construction expanded the plant to a nominal 57 MGD and included the construction of the tertiary treatment facilities. These facilities are required to achieve an effluent quality that meets the Wastewater Reuse water quality standards for effluent used to irrigate food crops that can be consumed raw.

The tertiary facilities or the Filter Plant (as it is referred to by plant staff) is designed to treat a peak flow of 74 MGD. It consists of a Filter Influent Pump Station, rapid mix facilities, flocculation basins and twelve monomedia 1.5 micron anthracite coal filters. There is also the capability to add alum and polymer as coagulant aids. The Filter plant has been consistently able to meet our effluent turbidity requirements of 1 NTU without chemical addition. The 1 NTU standard is a requirement of Phoenix’s agreement with RID.

MODIFICATIONS TO RID FACILITIES

A number of modifications were made to RID’s existing canal structure to ensure that treated effluent could be transferred from the 23rd Avenue WWTP to RID. These modification included increasing the capacity of the main canal adjacent to the 23rd Avenue WWTP, increasing the freeboard in the canal by adding more concrete lining to portions of the canal starting at the Agua Fria River heading east, burying a portion of the canal within the boundaries of the 23rd Avenue WWTP, and relocating the canal from the south side of Lower Buckeye Road between 23rd Avenue and 27th Avenue. The cost of these improvements was shared by both RID and Phoenix with Phoenix paying for the majority of the work within and around the plant boundaries. Modification to existing pump stations and wells along the alignment of the canal were also made to upgrade the existing electrical system. (Phoenix and RID, 1995(a))

OPERATIONAL CHALLENGES

Once the RID Exchange Agreement and facilities were in place, coordination work started to ensure that RID received effluent from the 23rd Avenue WWTP when it was required. Coordination between the 23rd Avenue Operations Supervisor and RID’s Watermaster is critical to the delivery of effluent to RID’s customers, who can be as far as 50 miles away from the plant. As a result, a communication chain was established to enable everyone involved in the process to easily communicate with one another.
The first step in the communication chain is a annual water delivery plan, which outlines what RID expects for daily effluent delivery to the irrigation system. This allows both parties to know what to expect. Any deviations from the plan by either party requires at least 24 hours prior notice. For example, if 23rd Avenue WWTP is experiencing problems with effluent water quality, the Operations Supervisor would contact the RID Watermaster to let RID know there was a problem and the plant would need to discontinue water deliveries until the problem is solved. The Plant Operations Supervisors were also given a pager which is to be used only by the RID Watermaster. If there is a problem and the Watermaster can not contact the Operations Supervisor by phone, the pager is used. This ensures that the lines of communication are continuously open.

RID has three wells located in close proximity to the 23rd Avenue WWTP. These well can be controlled by plant personnel at times of low effluent flow to maintain water surface elevation in the canal system. This is necessary to ensure that irrigators do not break suction on their irrigation siphons.

**CONCLUSION**

The RID Exchange Agreement is a complex combination of water rights agreements which holds together four distinctly separate entities for a common purpose - supplying water in the arid southwest. This innovative agreement shows that a possibly negative situation (the SRPMIC lawsuit) can be turned into a positive outcome for everyone involved. The RID Exchange Agreement is a monument to the fact that there are innovative ways to solve water rights issues. The key is that everyone involved has to work together to reach a common goal and then work to make that goal a reality.

**REFERENCES**


Phoenix, City of and Roosevelt Irrigation District, 1994. Intergovernmental Agreement Between Roosevelt Irrigation District and City of Phoenix - Cost Sharing for Conversion
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Phoenix, City of and Roosevelt Irrigation District, 1995 (a). Relocated Pipeline Maintenance Agreement.

Phoenix, City of and Roosevelt Irrigation District, 1995 (b). Sale of Effluent Agreement.


ARIZONA DEPARTMENT OF WATER RESOURCES UNDERGROUND WATER STORAGE PROGRAM: FROM APPLICATION TO PERMIT

Drew M. Swieczkowski, James E. Swanson and Cassandra L. Mocarski

Introduction

In response to an increased need to acquire and manage water supplies in Arizona, the Underground Storage and Recovery Act was passed in 1986. In 1994, the State of Arizona combined the 1986 Act with new legislation to create the Underground Water Storage, Savings, and Replenishment Program (recharge program). The Arizona Department of Water Resources (ADWR) recharge program allows for the legal storage of water underground. Under the recharge program there are four permits which can be obtained: 1) underground storage facility (USF) permits, 2) water storage permits, 3) groundwater savings facility permits, and 4) recovery well permits.

This paper will focus on USF permits. There are two types of underground storage facilities, constructed and managed. Constructed facilities utilize basins, injection wells, or other man-made structures that facilitate recharge and enhance infiltration. Managed facilities employ the use of a natural stream channel with minimal construction. For each type of facility, ADWR identifies both pilot and full-scale projects. Pilot projects (maximum of 10,000 acre-feet over a two year period) are typically precursors to full-scale projects and are designed to gather the necessary data to support a full-scale application.

ADWR and the Arizona Department of Environmental Quality (ADEQ) require preparation of a hydrologic report to support an application for an underground storage facility permit. The hydrologic report should demonstrate that the project is hydrologically feasible and will not cause unreasonable harm to land and water users. The report must show that the applicant has the technical capability to construct and operate the proposed facility. All recharge activities must comply with the established aquifer water quality standards and may require an Aquifer Protection Permit (APP) from ADEQ. In addition, the applicant must obtain any required floodplain use permit from the appropriate county or local flood control district prior to issuance of an USF permit.

The application packet, which contains guidelines for a USF hydrologic report, was recently revised by ADWR. The new packet contains a detailed description of USF hydrologic report requirements. Since each recharge project is unique in detail and design,

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the guidelines are written to be all inclusive and cover a wide variety of recharge situations. This paper will give the reader a general understanding of the USF permitting process and USF hydrologic report requirements.

**Permitting Process**

The permitting process required to obtain a USF permit from ADWR consists of a number of steps designed to aid the applicant in filing an application that can be reviewed in a timely manner. The process starts with a pre-application meeting with the Hydrology Division, Legal Division, and the local Active Management Area of ADWR. This is usually followed by the submittal of a draft proposal for the hydrologic report for review and comment, and then the actual submittal of the application and hydrologic report to ADWR. The goal for the applicant is to have a complete and correct application and hydrologic report submitted to ADWR. A wide range of criteria for the USF permit must be met to actually issue the USF permit. These criteria are highly dependent on the complexity of the recharge project itself, the hydrogeology, and the local and regional setting of the proposed recharge site.

**Pre-application Meeting**

The purpose of the pre-application meeting is to provide ADWR a project overview of the proposed USF recharge project. The overview must describe the method of recharge, the local hydrogeology, and the regional setting of the project. The overview should also summarize the project goals and objectives and describe the method(s) to be used to achieve these goals. The information collected for the pre-application meeting can also be helpful for the hydrologic report. The following information, when available, should be presented at the pre-application meeting:

- General location map of the proposed facility
- Topographic map (USGS 7.5 minute quadrangle) which shows the location of other surface water bodies near the site, including canals and other recharge projects
- Site map which shows existing and planned features of the site
- Description of the purpose and scope of the proposed project
- Description of the proposed method of recharge
- Description of the methods to be used to analyze impacts from the recharge activities
- Scope of work for proposed field investigations to include: the installation of monitor wells, infiltration testing, methodologies for obtaining aquifer
parameters, groundwater level and quality monitoring plans, surface water flow and quality measurements, subsidence measurements, groundwater and/or surface water modeling

- Other uses associated with the facility, such as recreation, water quality treatment, wetlands, etc.

- Hydrogeologic information
  - Depth to groundwater and elevation of groundwater in the general area
  - Perching groundwater conditions
  - Groundwater flow direction
  - Rock and soil types in the vicinity of the site
  - Subsurface lithology
  - Aquifer test parameters
  - Groundwater quality data
  - Wells within a one mile radius of the proposed site
  - Subsidence data

- Planned duration of facility

- Information on past and present land use

- Demonstration that the project will meet all ADWR and ADEQ legal requirements for recharge projects

- Demonstration that the project will be consistent with the active management area (AMA) water management goals, when applicable

*Project Proposal*

Following the pre-application meeting, the applicant may submit a project proposal. The purpose of the proposal is to outline the available information, to scope the additional investigations, and to detail the methods which will be used to estimate impacts. Review of a proposal will enable ADWR and ADEQ staff to make suggestions for tailoring site investigations to the individual site. The proposal should be based on issues presented at the pre-application meeting. The proposal should be reviewed by ADWR and ADEQ before the applicant initiates site investigations and other activities necessary to gather data for the hydrologic report.

*Hydrologic Report*

The hydrologic report must show that the applicant has the technical capability to plan, construct, and operate the project, that the project is hydrologically feasible, and that
the project will not cause unreasonable harm to land or other water users. The hydrologic report should be concise and well organized. The report must contain the purpose and scope of the project along with text, maps, figures, and supporting data. The hydrologic report must contain all hydrogeologic information applicable to the project including any information agreed upon during the pre-application meetings and as stated in the project proposal. The hydrologic report must be submitted to ADWR at the same time that the application and fees are submitted. The applicant is strongly advised to organize the hydrologic report as outlined in the Hydrologic Report Checklist for Underground Storage Facilities, located in the revised USF application packet.

The hydrologic report for a pilot project should follow the same format as the full-scale hydrologic report. If the necessary technical information is not available, the applicant must include a description of the proposed methods for collecting and evaluating the additional site data. The report also must demonstrate how the pilot project will gather the necessary data to support a long-term, full-scale facility application. Sufficient monitoring must be conducted to adequately characterize the hydrogeologic environment, evaluate facility operation, and examine potential unreasonable harm. Proximity to landfills, Superfund, WQARF, other contaminated areas, and/or waterlogged areas may subject a pilot proposal to a full-scale permit review process.

The information presented in the hydrologic report should be summarized in an executive summary. The executive summary should include an overview of the project which briefly describes the main points of interest in the hydrologic report.

The project's goals and objectives and the methods which will be employed to achieve the goals and objectives should be described in detail in the hydrologic report. Other uses associated with the proposed facility should also be described.

ADWR must be assured that the applicant is technically capable of operating the facility according to the conditions of the permit. Technical ability or capability is most important in the design, construction, and operation of the facility, including installation and maintenance of the facility's control measures and compliance with monitoring requirements. If an entity other than the applicant is responsible for any of these areas, the name of the entity, their responsibilities, and proof of their technical capability should be stated.

Hydrologic Feasibility

Hydrologic feasibility is used to evaluate a recharge project. ADWR's interpretation of hydrologic feasibility, in the most general sense, is that a recharge project is "possible" and "practical". The design of the facility, site layout, infiltration rates, injection rates, storage potential, and site characterization (e.g., geology, hydrology) may be used to determine if a project is feasible. The complexity of a project may dictate the necessity for reviewing additional factors in determining feasibility. The following list includes some considerations ADWR evaluates to assess hydrologic feasibility:
- Demonstration that the estimated losses will not exceed the recharge estimates of the facility
- Demonstration of adequate aquifer storage potential for the proposed recharge volume
- Ability to recharge the volume of water stated on permit application
- Evidence that recharged water will not migrate to an area where it cannot be available for use (such as physically leaving an AMA boundary)

Unreasonable Harm

A recharge project may not cause unreasonable harm to land and other water users within the area of hydrologic impact. ADWR has interpreted “unreasonable harm” to mean, in part, the quantitative effect of adding water to an aquifer which may cause a waterlogging problem, the migration of a contaminant plume or poor quality groundwater, or leaching of contaminants from the vadose zone such that an aquifer water quality standard is exceeded. Waterlogging occurs when a rise in water levels causes localized flooding of basements, septic, or other features. Groundwater intrusion into landfills may constitute unreasonable harm. Changes in the groundwater quality due to the addition of the recharged waters (degradation and/or mobilization) are also a consideration for determining unreasonable harm in the event that water quality standards are exceeded. Unreasonable harm may also occur when recharge exacerbates land subsidence problems.

Source Water

The report must include a discussion of the source water characteristics of each type(s) of water that will be recharged at the facility. Source water types may include CAP water, municipal and industrial waste water, storm water or other surface water types.

Facility Description

The description of the recharge facility must contain a detailed explanation of the physical design of the facility and the facility’s plan of operation and maintenance. The design and operational aspects of the recharge facility must be consistent. The description also must indicate whether the project will utilize man-made structures to facilitate recharge, requiring a constructed underground storage facility permit; or if a natural stream channel will be used for recharge, requiring a managed underground storage facility permit. Items that must be included in the facility description are a site description, facility design, recharge method design, facility operation and maintenance, operating parameters, and proposed maintenance of all recharge systems.
Hydrogeologic Characterization of the Recharge Site

Geologic and hydrologic conditions at the recharge site must be thoroughly described in reference to the proposed recharge project. The existing physical groundwater conditions of the site must be defined in order to identify and evaluate any possible impacts resulting from the recharge activities and to show that recharge of the proposed volume of water is hydrologically feasible. This entails describing both the surficial and subsurface geology and groundwater and surface water hydrology of the proposed recharge site. Aquifer and vadose zone characterization, groundwater conditions including a well inventory, water level and water level change mapping, and a complete analysis of the groundwater quality at and surrounding the proposed site should be included.

Impacts of Proposed Recharge

Before ADWR issues an USF permit, potential adverse impacts to the area surrounding the proposed recharge facility must be examined. A recharge project may not cause unreasonable harm to land and other water users. Problems with waterlogging, the migration of a contaminant plume, or the migration of poor quality groundwater may be determined to be “unreasonable harm” resulting from the facility. Changes in the hydrology of the area due to the operation of the facility must be quantified. All data and calculations for quantifying the impacts from the proposed recharge facility must be provided. A groundwater mounding analysis is also necessary to demonstrate that recharge will not cause unreasonable harm.

ADWR examines the maximum area of impact (AOI) of a proposed facility to determine if the facility will cause unreasonable harm to land and water users in the AOI. The maximum AOI is also used for the purpose of public noticing prior to issuance of a USF permit. When determining the maximum AOI, the applicant must assume a maximum storage scenario. This scenario assumes that the entire proposed annual volume is recharged over the life of the facility. Once this area is defined, the areal extent of the one foot rise must be drawn on a map as projected on land surface.

The applicant must use the method most applicable to the proposed facility for calculating the theoretical rise or mounding of the groundwater surface. Analytical solutions, analytical modeling, or numerical modeling may be used to complete the mounding analysis, depending on the hydrology, geology, and facility characteristics.

Impacts on water quality as a result of the recharge activities must also be addressed in the report. Although recharge with CAP and other non-effluent waters is exempt from Aquifer Protection Permit (APP) requirements, any discharge must still comply with aquifer water quality standards (AWQSS) and/or numerical water quality standards. The impact and mounding analysis must demonstrate that the recharge will not carry pollutants to the groundwater in concentrations that would exceed an AWQS, or exacerbate movement of poor quality groundwater. Groundwater modeling work may be conducted to determine the sensitivity of zones of contaminated groundwater to recharge at potential sites. Parameters
such as recharge volume, configuration of project, distance from the plume, existing hydrologic conditions, and proposed project operational parameters could be simulated to develop a general sense of what impact can be expected from the standpoint of plume management.

Monitoring Plan

An applicant must submit a monitoring plan for the facility. The plan must be designed to observe changes in water quantity and quality, water levels, and subsidence during recharge. A monitoring plan can be used to show that the facility is not causing unreasonable harm, to determine the area of impact for credit calculations, and to review the facility’s operation. Information on water level changes during project operation is also needed to define storage potential, flow direction, and rate of groundwater movement. Recharge systems are typically site specific; therefore, monitoring needs and program design vary widely. The specifics of each monitoring program should be tailored to the individual facility features, characteristics of the source water, and the purpose of the monitoring. The goals and objectives of the monitoring plan should be clearly stated. A final monitoring plan must be submitted prior to issuance of a USF permit. Any changes in the monitoring plan may require a public notice. If the applicant anticipates any reasonable changes in the final monitoring plan, contingencies should be included in the plan. The contingencies should address any possible deviations from the final monitoring plan.

Contingency Plans

A contingency plan must be developed to address potential problems that may occur over the duration of the permit. The plan must address situations or equipment failures which may result in unreasonable harm to nearby wells and landowners or have a negative impact on the hydrologic feasibility of the project. Alert levels must be established and should be based on site-specific conditions. The established alert levels should provide early enough warning to correct a problem before an exceedance of aquifer water quality standards, water level, and or other unreasonable harm occurs. For each alert level there must be a clearly defined course of action designed to verify and correct the condition that has developed. The plan must include the name and phone number of a coordinator who will be responsible for notifying ADWR and ADEQ, implementing the contingency plan, and addressing the conditions that represent unreasonable harm. ADWR and ADEQ should be notified when the permittee determines that an alert level has been exceeded. After ADWR has been notified, the permittee should in writing, identify the alert level exceeded, when it occurred, the reason it occurred, the method used to correct it, and the affect of the exceedance on the project and/or the surrounding area.

Summary

In conclusion, the application process for obtaining an USF permit is a complicated task which can be aided by proper planning and communication with ADWR and ADEQ. The newly revised USF guidelines are very detailed and can assist the applicant in the
organization of the hydrologic report. The hydrologic report checklist can be used to demonstrate to both the applicant and reviewing parties that the required hydrologic information has been provided in the hydrologic report. For a copy of the complete USF application guidelines please contact ADWR Hydrology Division at 417-2448.

**References Cited**

Arizona Revised Statues, Title 45, Chapter 3.1.

Arizona Department of Water Resources, Hydrology Recharge Files.

Aquifer Recharge and Recovery
A Case Study of the Sun Lakes Recharge/Recovery Pilot Project

Doug Toy, Stephen Noel, and Fred Goldman

Abstract
Golf course irrigation is seasonal and is usually out of phase with effluent production in many Maricopa County developments. Off season effluent storage is required if golf course irrigation is the primary mode of effluent disposal. Recharge/recovery of effluent directly into the upper alluvial aquifer (UAA) of the Salt River Valley will be used in Sun Lakes to provide seasonal storage. Aquifer water quality effluent will be recharged into the UAA during the low water demand winter months, and recovered for irrigation use (e.g. golf course) from the recharge/recovery wells and other designated recovery wells during the high water demand summer months. The recharge/recovery system will be permitted as an Underground Storage Facility by ADWR and will be issued an Aquifer Protection Permit from ADEQ. A pilot recharge/recovery well was drilled and tested with potable water. The recharge test exceeded all expectations by recharging 725 gpm for 20 days with only 17 feet of water level rise in the recharge well. This paper will report on the method employed to conduct the recharge/recovery test and the effects of recharge/recovery on the UAA in the Sun Lakes area.

Introduction

Pima Utility Company (Pima) is a water and wastewater utility operating in the Sun Lakes area of Maricopa County, in central Arizona. Pima has developed an overall effluent management plan for the beneficial use of effluent for irrigation of an 18-hole championship golf course (soon to be expanded to 27-holes), common area, and lake replenishment. Pima has recently completed a state of the art wastewater treatment plant and is continuing to improve and expand the effluent distribution system. The WWTP produces an effluent suitable for reuse and which also meets Arizona’s aquifer water quality standards.

The Pima facilities serve the retirement community of Sun Lakes and their effluent is reused by a golf course. Golf course irrigation is seasonal and is out of phase with the Pima’s effluent production. Off season effluent storage is required if golf course irrigation is to be the primary mode of effluent disposal. The recharge and recovery of effluent directly into the upper alluvial unit (UAA) of the regional aquifer will be used to


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provide seasonal storage. The effluent will be recharged into the UAU during the low irrigation demand winter months, and recovered from the recharge/recovery wells and other designated recovery wells during the high water demand summer months. The recharge/recovery system will be permitted as an Underground Storage Facility by the Arizona Department of Water Resources (ADWR) and will be issued an Aquifer Protection Permit from the Arizona Department of Environmental Quality (ADEQ). To test the viability of the aquifer storage recovery concept, a pilot recharge/recovery well was drilled and tested with potable water. A discussion of the pilot program activities are presented below.

Aquifer Storage and Recovery Operation Concept

The rate at which effluent will be recharged is dependent upon the effluent generated by the WWTP and the irrigation requirements of the receiving facilities. At project build-out, approximately 1,427 acre-feet per year of effluent will be generated. Approximately 628 acre-feet will need to be recharged during the winter for recovery during the summer. The recharge requirement will vary from day to day and season to season. Although the WWTP has flow equalization which evens out the daily flow variations, it has no on-site storage. During the winter, the irrigation system may not be able to take any water due to weather conditions and the entire daily flow will need to be recharged. As a result, the recharge capacity of the R/R wells needs to match the peak day flow of the plant. The WWTP flows are projected to ramp up to 2.4 mgd when Pima’s service area is built out. The number of R/R wells to be installed will be designed to meet the peak day WWTP flow with one backup well.

Hydrogeology of the Area

The regional geology of the area has been extensively studied by others. Alluvial deposits in the study area have been divided into three distinct units. These three units from top to bottom are called: the Upper Alluvial Unit (UAU), the Middle Alluvial Unit (MAU), and the Lower Alluvial Unit (LAU). Together, they comprise the major water bearing formation in the Salt River Valley which includes the study site. The UAU (which averages about 210 feet thick in the study area) is comprised mostly of very porous and unconsolidated gravel, sand, and silt. The MAU (which averages about 370 feet thick in the study area) is mostly weakly consolidated, but localized formations of moderately-cemented to well-cemented siltstone. The LAU (which averages about 1,020 feet thick in the study area) consists of clay, silts, mudstone, evaporite, sandstone, gravel, conglomerate, and andesitic basalt (US Bureau of Reclamation, 1977 & Laney and Hahn, 1986).

Local Conditions

The UAU, as modeled by the ADWR in their Salt River Valley Model, is about 210 feet thick and has a hydraulic conductivity values of about 50-feet/day. The UAU is not of drinking water quality because of its high nitrates and total dissolved solids (TDS). Since the area was historically farmed, it is suspected that the historical agricultural practices contributed to the observed high TDS and nitrate concentrations. The MAU water quality meets drinking water standards and is used for domestic well purposes.
Since the UAU is not used as a drinking water source, it is an ideal aquifer to store and recover effluent for non-potable uses.

The purpose of the pilot recharge/recovery project was to obtain site specific hydraulic information on the aquifer’s ability to store and recover water.

**Well Drilling**

In April 1996, one 12-inch diameter recharge/recovery well and three 2-inch diameter monitor wells were drilled. All four wells were drilled to a depth of about 200 feet to penetrate only the UAU. The three monitor wells were drilled to form a 600-foot equilateral triangle surrounding, but slightly offset from the R/R well. All the wells were drilled in the northeast corner, of the southwest corner, of Section 30, T.2S., R. 5 E., S.R.B. & M. Depth to ground water was approximately 63 feet and the pre-test ground water gradient was relatively flat.

An examination of the drill cuttings showed the UAU to be consistent with literature. However, the cuttings showed light to strong reaction to 10% hydrochloric acid indicating some cementation. Observations during drilling of the recharge-recovery well borehole indicate that flowing sands were a problem at depth. Significant lenses of clay or silty material were not identified in the drilled cuttings.

**Well Injection Hydraulics**

Potable water, with electrical conductivity of about 1200 millimhos/cm, from a nearby fire hydrant was used as the source recharge water.

A multi-injection pipe concept was used so that various injection rates could be tested. The injection piping was designed to maintain positive pressures at the surface to minimize air entrainment (Driscoll, 1986). Air entrained in the injection piping could be forced into the aquifer and cause it to air bind, thus reducing the ability of the aquifer to accept recharge water.

To prevent air entrainment, the injection pipes were sized at 1 ½ inch, 2 inch, and 3 inch diameter injection pipes. PVC pipe was used in all downhole and wellhead injection pipe. Figure 1 shows a schematic of the injection piping. Each injection pipe was equipped with a throttling valve and a combination air release/air vacuum valve located on top of the well to purge air from the system before it was injected into the aquifer and to eliminate negative pressures in the pipeline by allowing air to enter into the recharge system as the recharge stopped to prevent collapsing the injection piping. Positive pressure at the well head was maintained by creating a sufficiently large back pressure from head losses in the injection pipes.

**Aquifer Characterization**

Following installation of pilot R/R well and monitor wells MW-1, MW-2, and MW-3, the R/R well was developed and tested. Initial testing consisted of five 2-hour steps step-test and a 24-hour constant rate test. The results of the step pumping test are presented in Figure 2. The step test was designed to evaluate the production capacity of the aquifer and to establish the pumping rate for the 24-hour constant rate test. The impact of the step test on monitor well MW-1, 60 feet from the production well, is shown on Figure 2. As observed, the drawdown in MW-1 parallels the drawdown in pilot R/R well.
Based on the results of the step test, the 24-hour constant rate aquifer test was conducted at a pumping rate of 1,050 gpm. Review of the semilog drawdown versus time plot (Figure 3) indicates that the monitor wells were only slightly impacted by pumping, and that the calculated transmissivity value of the UAU is 350,900 gallons per day per foot of drawdown (gpd/ft). Based on an average saturated thickness of the UAU of 150 feet (210 feet - 60 feet), the UAU hydraulic conductivity is 2,340 gpd/ft² or 312 ft/day. This value is approximately 6 times greater than ADWR's Salt River Valley model values.

**UAU Ambient Water Quality**

The three monitor wells were sampled from 80-foot depth to 200-foot depth. Each monitor well was sampled between the 80 to 100 foot depth and also between the 140 to 200 foot depth. The TDS varied from 2800 to 4400 mg/l while the total nitrates as nitrogen varied from 16 to 31 mg/l.

The pilot R/R well was sampled at the end of the step test when the well was being pumped at 1,050 gpm. This sample was considered a composite sample of the UAU and yielded TDS of 3700 mg/l (5800 millimhos/cm conductivity) and nitrate at 15 mg/l.

**Recharge Testing**

Following aquifer testing of the UAU, a pilot recharge program was initiated to evaluate the capability of the aquifer to receive water (recharge water), and to document the capability of the recharge-recovery well design. Testing consisted of a 6-step recharge step test followed immediately by a 19-day constant rate recharge test. A total of 19 million gallons of potable water was recharged in about 20 days.

The recharge rates for the step test were determined by the injection piping configuration. The three injection pipe valves were opened individually and in combination to give the 105, 230, 340, 500, 610, and 725 gpm flow rates. The final step and long term recharge rate (725 gpm) was determined by opening all three valves. During the initial phase of the recharge test, there was positive pressure in the injection pipes at the well head and air vacuum valves were not allowing air into the injection pipes. Had a higher water pressure been available at the fire hydrant, a higher injection rate would have been used.

A semilog plot of the step test and constant rate recharge test is plotted on Figure 4. Review of the data indicated that the ground-water rise in pilot R/R well was approximately 17 feet over the 20-day recharge period. The observed rise in MW-1, 60 feet from pilot R/R well was 1.46 feet. It appears that at the end of the recharge test, the rate of rise increased which may be an indication that air entrainment was occurring in the recharge-recovery well. Field observations on the final day of the recharge test indicated that air vacuum valves were allowing some air to be entrained into the 3-inch diameter injection pipe.

Following 20 days of recharging, the downhole pumping equipment was turned on and pumped at a rate of 500 gpm. The drawdown observed during this test is consistent with the initial aquifer test. Recovery measurements were conducted following testing. Results of these data indicate that ground-water levels in the UAU returned to pre-testing conditions.
Recovery Testing

Following the completion of the recharge tests, two 48-hour recovery tests were conducted to study recovery. Both recovery tests were conducted using a 460 gpm pumping rate and about 1.3 million gallons of water were withdrawn for each test. For both recovery tests, drawdown stabilized very quickly to about 3 feet.

The first recovery test began one hour after the completion of the constant rate recharge test. Conductivity measured 1300 millimhos/cm at the start of the test and steadily increased to 2900 millimhos/cm at the test's completion. Based on conductivity, it is estimated that at the end of the first recovery test, 60% of the recovered water was the potable injected water.

The second recovery test began after the well was rested 92 days. The initial conductivity was measured at 5500 millimhos/cm and slowly dropped to 5200 millimhos/cm. Based on conductivity about 10% of the recovered water was the potable injected water.

Conclusions

The UAU in the Sun Lakes area is suitable for aquifer storage and recovery of effluent. The UAU is considered an impaired aquifer due to the high TDS (3700 mg/l) and nitrates (15 mg/l). The effluent to be recharged is a higher quality water than the ambient UAU water and will improve the UAU water quality over time.

The UAU’s hydraulic conductivity of 2,340 gpd/ft² is ideal for recharge. The mound created by the 20-day test at 725 gpm showed a rise of only 17 feet in the R/R well and a rise of 1.46 feet in a monitor well 60-feet away. The recharge rate was limited by the amount of water that could be withdrawn from the nearby fire hydrant.

Because of the UAU’s large hydraulic conductivity, recharged water appears to migrate off site rather rapidly and the direction of movement of recharge water is related to local pumping rather than the regional water table. Efficient recovery of the recharged water may not be feasible.
References Cited


Sun Lakes Recharge/Recovery Pilot Project, Sun Lakes, Arizona

STEP PUMPING TEST

FIGURE 2
24-Hour Constant Rate Pumping Test

Sun Lakes Recharge/Recovery Pilot Project, Sun Lakes, Arizona

Q = 1,050 gpm

T = 264(1050 gpm)/(13.68 ft - 12.89 ft)
T = 350,900 gpd/ft

Drawdown (ft)

Time (min)

- R/R  - MW-1  - MW-2  - MW-3
ARTIFICIAL GROUNDWATER RECHARGE
AND THE FLOOD CONTROL DISTRICT:
WORK IN PROGRESS\textsuperscript{1}

Marilyn DeRosa, R.G.\textsuperscript{2}

Abstract

The Flood Control District is uniquely positioned to advance the development of artificial groundwater recharge projects. The District owns large parcels of vacant land with good hydrogeologic characteristics conducive to rapid infiltration. In addition, the properties are generally located in areas with limited groundwater contamination potential, close to water available for recharge projects, and in the vicinity of aquifer cones of depression.

In 1988 an inventory was conducted which ranked the suitability of groundwater recharge on 34 District parcels. The three sites with the highest rating included: the impoundment area of McMicken Dam; the confluence of the Agua Fria and New Rivers; and Queen Creek west of the CAP Aqueduct. Currently, the District is working to update and expand the existing inventory of potential sites. The suitability criteria will be revised and the inventory will include both old and recently acquired properties.

At this time, the District does not engage in the development or operation of groundwater recharge facilities. The Board of Directors for the District, however, recently approved a policy allowing other agencies to lease property for recharge projects. The District is eager to coordinate with outside agencies for the development of multi-use facilities at existing flood control locations.

As the County rapidly urbanizes, the District is striving to coordinate more aesthetic projects that combine environmental, ecological and recreational benefits. The District is also very supportive of innovative techniques for managing stormwater and the conservation of water resources. In fulfilling its obligation as a flood control agency, the District is uniquely positioned to facilitate the development of artificial recharge projects.

\textsuperscript{1}Paper presented at the 8\textsuperscript{th} Biennial Symposium on the Artificial Recharge of Groundwater, Tempe, AZ, June 2-4, 1997.

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Introduction

Since its inception in 1959, the mission of the Flood Control District has been, “to provide flood and stormwater management services for the benefit of the people of Maricopa County.” These services are provided through regulatory activities, master planning, technical assistance, and public works projects such as dams, channels, and storm drains. These structural projects typically collect and convey excess runoff to one of several intermittent rivers in the Salt River Valley.

In many western states, flood control has been recognized as a clear corollary to water conservation. Los Angeles County, for example, assigns the Public Works Department the dual responsibilities of flood control and water conservation. This is a logical arrangement as the organization entrusted with the collection and conveyance of excess surface runoff is in the best position to develop methods of conserving or assuring beneficial use of the excess water. In the case of LA County, which uses groundwater to meet approximately 40 percent of its water needs, this has resulted in the development of dozens of artificial groundwater recharge projects for replenishment of groundwater supplies. Since 1919, LA County has recharged millions of acre-feet of stormwater, wastewater and surface flows, in thousands of acres of spreading basins. As a result, only 15 percent of stormwater that falls in LA County returns to the ocean (LA County FCD, 1982).

In Maricopa County, with its arid climate, local water supplies are insufficient to meet the expanding needs of municipal and industrial users. Approximately 60 percent of the region’s water needs are met with groundwater supplies (ADWR, 1988). Clearly, the need for water conservation and groundwater replenishment is acute.

With the 1989 passage in the Arizona State Legislature of Senate Bill (S.B.) 1424, the Flood Control District was empowered to begin water conservation projects in the form of artificial groundwater recharge on behalf of the State and other jurisdictions.

For a variety of reasons, the District is uniquely positioned to advance the development of artificial groundwater recharge projects:

- The District owns large parcels of vacant land, including coarse-grained river bottom properties, with good hydrogeologic characteristics conducive to rapid infiltration.
- The properties are generally located outside densely urbanized areas where the potential for groundwater contamination (related to industrial activities) is minimal.
• The properties tend to be close to water that could be utilized for recharge projects (wastewater treatment plants, Central Arizona Project [CAP] Aqueduct, Salt River Project [SRP] canals).

• The parcels are generally in regions of high groundwater overdraft and aquifer cones of depression.

In anticipation of the passage of S.B. 1424, an inventory was conducted in 1988 ranking the suitability of groundwater recharge on 34 District parcels (CH2M Hill et. al., 1988). At the conclusion of the study, the Flood Control Advisory Board (FCAB) determined that none of the proposed projects provided enough flood control benefits to justify their implementation. In addition, the FCAB passed Resolution FCD 88-24 restricting staff involvement in artificial groundwater recharge projects, except in those cases where flood control benefits would result.

Recent History

In the last ten years, Maricopa County has experienced rapid urban growth. Many smaller cities fringing the urban core are now engaged in long range water planning. They are developing assured water supplies to meet the future needs of their citizens and to satisfy continued growth. Several cities have undertaken artificial groundwater recharge projects as a component of their water resources planning. The District has holdings in several of these rapidly growing communities.

Simultaneously, the District’s constituent cities were seeing a change in their flood control needs and the District was experiencing a commensurate shift in its vision. The need for large structural flood control solutions including dams, levees and channels, was being replaced by the desire for smaller-scale, neighborhood-sized flooding solutions. Several local jurisdictions approached the District with interest in using District property for artificial groundwater recharge projects. Beyond assured future water supplies for increased residential and commercial development, a successful program can provide exciting environmental, ecological and recreational benefits. Recharge can also generate cost savings by alleviating the need for conventional surface water treatment plants, restoring continually decreasing groundwater levels, and helping to slow rates of associated land subsidence.

In response to the needs of the cities, the District worked with local jurisdictions and other agencies to develop a policy allowing the use of District property for recharge projects. Resolution FCD 95-13 allows the issuance of leases for such projects and was approved by the Board of Directors for the District in September 1996.
Previous Property Inventory

With approval of Resolution FCD 95-13, the District reviewed the 1988 inventory in preparation for inquiries regarding the suitability of District parcels for groundwater recharge projects. During the inventory, suitability criteria were developed and each of 34 potential sites was evaluated. The 34 sites were then ranked and the 15 highest ranking sites were evaluated in greater detail. The list was eventually narrowed to three sites. The initial inventory criteria included:

- **Recharge water availability.** Availability of excess runoff was a key consideration. In addition, the potential for joint projects using CAP water, wastewater effluent or SRP water was considered.

- **Flood control considerations.** Flood control benefits were of chief concern. In addition, modification of existing structures or change of operations to enhance recharge was considered.

- **Water quality impacts.** The chemical quality of groundwater and/or the proximity of known groundwater contamination was of primary concern.

- **Hydrogeologic conditions.** Hydrogeologic conditions such as depth to groundwater, depth to bedrock, thickness of upper alluvial unit, aquifer transmissivity, occurrence of perched groundwater zones, and recoverability of recharged water were evaluated.

- **Soils and infiltration rates.** The suitability of soils for recharge, estimated infiltration rates, and potential for adverse geochemical reactions were evaluated.

- **Land ownership and use.** Current land ownership, the availability of undeveloped lands, and compatibility of recharge operations with present and future land use were weighed.

The inventory was completed using only existing data; no additional site investigations or data collection was conducted. Using the limited information available, a preliminary screening was conducted and 19 of the 34 potential recharge sites were eliminated from further consideration for having "fatal technical flaws." These fatal flaws included unfavorable hydrogeologic conditions, existing groundwater contamination, and/or lack of excess stormwater runoff or supplemental sources of recharge water. The report indicated that construction of recharge facilities solely for recharging excess flood waters was not practical at several of the potential sites. Available stream flow data suggested that a significant portion of runoff from the more frequent flood events was recharged naturally and that "excess" flood waters were rare.
With the help of a District Review Committee and secondary technical criteria, the 15 remaining sites were narrowed to the three most likely candidates. These included the impoundment area of McMicken Dam, the confluence of the Agua Fria and New Rivers, and Queen Creek west of the CAP Aqueduct. Two of these proposed candidates (the McMicken Dam and Queen Creek sites) are in areas identified by the Arizona Department of Water Resources (ADWR) as, “... locations ... with current and projected groundwater declines which would benefit greatly from additional storage facilities” (ADWR, 1997).

While the inventory was successful in compiling valuable information, it was completed using only preexisting data and many uncertainties remained at its conclusion. To complete a thorough and detailed evaluation, the report sited the need for additional field investigations and data collection including site-specific hydrogeology, soils and infiltration rates, land ownership and use, floodplain impacts, and water sources.

**Current Property Inventory**

Currently, the District is working to update the inventory of potential sites. The criteria will be revised and the inventory will include both old and recently acquired properties. Many sites excluded from further evaluation during the 1988 inventory will be reevaluated. While several of these sites were eliminated due to lack of excess runoff, many are located near other potential sources of recharge water. Such sources include wastewater treatment plant effluent and untreated surface waters such as the CAP, Verde River or SRP canal water.

The District has property rights for more than 60,000 acres of land in Maricopa County; approximately 20,000 acres in fee and another 40,000 acres in easement. Of the hundreds of parcels owned by the District, more than 20 are large contiguous parcels ranging from 40 to 1,000 acres in size. A preliminary evaluation indicates that 16 of these locations could potentially and easily accommodate groundwater recharge projects for interested municipalities. These sites are primarily located along the northern and eastern edges of the urbanized area, near the CAP alignment and in close proximity to several of the 28 wastewater treatment plants in the County. In addition, these sites are also close to the SRP water delivery system and areas identified as having critical groundwater overdraft problems that could benefit from recharge activities (ADWR, 1997).

Figure 1 is a preliminary map of District lands that could potentially be utilized by other agencies to conduct groundwater recharge. In addition to parcel locations and approximate acreage, the CAP Aqueduct alignment, the SRP water delivery system, and wastewater treatment plant sites within Maricopa County are displayed.
Figure 1. Potential Groundwater Recharge Sites
Current Projects and Future Plans

Current Projects

At this time, the District does not engage in the development or operation of groundwater recharge facilities. The recently approved policy, however, allows other agencies to lease property for recharge projects. The Cities of Surprise and Avondale are currently negotiating the use of District land for groundwater recharge under this policy. Both municipalities have developed plans for a combined recreational and groundwater recharge facility.

The City of Surprise recently completed a one-acre pilot project within the impoundment area of McMicken Dam. The results of this preliminary effort were encouraging. According to their recently completed feasibility study, "... the McMicken reservoir area has the capacity to recharge large amounts of water for long times, for example, 100,000 acre-feet per year for 50 years for a total volume of 5 million acre-feet, without causing undue rises of groundwater mounds in the recharge area..." (Bouwer, 1996). The City of Avondale will be constructing a pilot-scale recharge project on District property along the east bank of the Agua Fria River later this year. The pilot project will recharge approximately 5,000 acre-feet/year and will test the design parameters and operational techniques of their proposed full-scale facility (BCI Geonetics, Inc., 1993). Geotechnical work already completed suggests site soils will exhibit relatively rapid hydraulic conductivities and good infiltration rates (RAM Associates, Inc., 1996).

The District is optimistic about the preliminary results of both projects and is eager to coordinate with other agencies for the continued development of these multi-use facilities at existing locations.

Future Plans

In addition to recharge facilities at existing flood control structures, the inclusion of groundwater recharge aspects to future flood control features holds great promise.

The District's Area Drainage Master Studies (ADMS) program was developed to analyze watersheds and identify flooding and drainage problems. From such studies, potential solutions are developed to reduce or eliminate flooding hazards. These solutions often include structural features such as onsite retention, stormwater collection systems, retention basins, drainage ways, and floodwater conveyance channels. Initial considerations during the planning phase of proposed structural solutions include the ability and value of utilizing groundwater recharge, facilities designed to enhance floodwater recharge, and the ability to combine open space amenities with recharge.
During the design phase of a project, flood control facilities can be easily designed to augment recharge. Recharge volumes depend upon the residence time of the water, infiltration rates, and the wetted area over which infiltration takes place. Guidelines for planning these facilities to promote recharge should include:

- The increased use of retention basins thereby increasing residence times.
- The decreased use of low permeability channel-bottom linings which hinder natural infiltration.
- The increased use of shallow slopes and maximum widths for drainage ways and channels thereby increasing the wetted area over which infiltration can take place.

Furthermore, recharge projects can be naturally combined with open space requirements. In open spaces that are set aside as environmental impact mitigation measures, the open water contained in the spreading basins can enhanced the growth of riparian vegetation. These sites can provide ideal habitat for water fowl and other wildlife providing recreational and ecological enhancement as well as flood control benefits.

Maricopa County will continue to rapidly urbanize as the population is expected to double by the year 2020. In response to these increasing pressures on the environment and water resources, the District will strive to coordinate more aesthetically acceptable projects offering a combination of environmental, ecological and recreational benefits. The District is very supportive of innovative techniques for managing stormwater and the conservation of water resources. In fulfilling its obligation as a flood control agency, the District, more so than any other public entity, is uniquely positioned to facilitate the development of artificial recharge projects.

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ARTIFICIAL RECHARGE IN THE LAS VEGAS VALLEY:
AN OPERATIONAL HISTORY

Abstract

Artificially recharging the Las Vegas Valley (Valley) ground-water system with treated Colorado River water is one water resource management option employed by the Las Vegas Valley Water District (District) to help meet future long-term and short-term peak water demands. The District began operation of an artificial ground-water recharge program in 1988 in order to bank water for future use and to slow declining water levels. Artificial recharge occurs in the winter months, typically from October to May, when there is excess capacity in the Southern Nevada Water System (SNWS), currently a 400 Million Gallon per Day (MGD) treatment and transmission system.

Treated Colorado River water is recharged into the principal aquifer through the District’s existing distribution system, to a network of production wells or dual-use wells for both recharge and production. The water is then stored until recovered from the wells, during the high demand summer months. The water recovered is injected Colorado River water, however, this water is accounted against the District’s ground-water rights. Credits in the artificial recharge account accrue until needed to cover pumpage in excess of permitted ground-water rights.

Wells used in the program were drilled and constructed in a variety of ways, and have responded differently to artificial recharge operations. The majority of the wells now used for artificial recharge and production were completed prior to 1980, using the cable-tool drilling method, perforated in place and naturally developed. Other wells were installed using the reverse-circulation drilling method with filter packs. The District commenced drilling dedicated injection wells in 1993, to address operational concerns observed in some of the production wells. Several types of installation and drilling methods have been used to optimize injection. The types of drilling methods used for the injection wells include, reverse circulation, air-foam, cable tool and dual rotary.

In 1988, two dual-use wells where used for production and artificial recharge, injecting an annual total of 1,153 acre feet of water. Since 1988 the artificial recharge program has expanded, using up to 40 wells with eight dedicated injection wells. Total water banked for future use, as of January 1, 1997 is 114,126 acre feet of water. Static water levels in the principal aquifer have risen from 10 to 40 feet in the main area of artificial recharge. Water levels in other areas of the Valley also show increases, indicating that the rise is not isolated, but is occurring throughout the principal aquifer. The artificial recharge program is currently expanding to utilize 31 dual use and 19
injection wells, for a potential capacity of 62,000 gallons per minute of injection or 45,000 acre feet per year of recharge by the fall of 1999.

Introduction

Most of the potable water used in the Valley comes from the Southern Nevada Water System, a 400 MGD treatment and transmission facility located on the shore of Lake Mead. The remaining supply is obtained from the principal alluvial aquifer in the Valley. Prior to importation of Colorado River water through SNWS in 1971, the principal aquifer was the sole source of potable water in the Valley. As a result, the ground-water system was over drafted and water levels in the aquifer have declined up to 280 feet by 1990 in some areas of the Valley (Morgan and Dettinger, 1994). The ground-water gradient of the aquifer is from the northwest part of the valley towards the southeast. Currently, (1997) 85 percent of the water supply for the Valley is obtained from the Colorado River. Ground water contributes 15 percent of the annual supply, but wells are pumped only during the summer months, to meet peak water demands. During the months of May through September, ground water accounts for nearly 33 percent of the Valley’s water supply. In order to optimize available water supplies, and to help meet future anticipated demands, the District, the largest member of the Southern Nevada Water Authority (SNWA), has incorporated artificial recharge of ground water as part of the overall water management program. Up to 50 MGD has been injected during the cooler, winter months, when demand for water declines below the capacity of SNWS.

Description of Study Area

The Las Vegas Valley Hydrographic Basin is located in Southern Nevada, about five miles west of Lake Mead and the Colorado River (Figure 1). The basin is a structurally formed, alluvial-filled depression rimmed by bedrock uplifts during Cenozoic extension of the Basin-and-Range Physiographic Province. The alluvial sediments are made up of primary clastic rocks eroded from surrounding mountains. The principal alluvial aquifers contain clastic sediments ranging from fine-grained silts and clays in the central part of the basin, to coarse-grained sands, gravels, pebbles, and boulders (Donovan, 1996). The thickness of the alluvial sediments in most of the areas of the Valley is at least 1,000 feet. In the central and southeastern areas of the Valley, thicknesses reach 3,000 to 5,000 feet (Plume, 1989). The Valley floor elevation ranges from 3,000 feet above mean sea level (asl) in the west to 1,500 feet asl in the east. This low elevations, combined with the Valley’s location in the Mojave Desert results in a mean annual precipitation to the Valley floor of four inches per year. Surrounding mountain peaks reach of maximum elevation of nearly 12,000 feet asl and receive over 20 inches of precipitation annually.
Pilot and Demonstration Project

To assess the feasibility of Artificial Recharge the District first conducted a pilot project in 1987 using existing unused production Well 21. During the pilot project 5000 gallons of treated Colorado River water were injected. The results of the pilot project are described by katzer and Brothers (1989). After successfully testing the feasibility in the pilot program, two production wells where selected for the Demonstration project injection. Production wells 16 and 17 were selected because of the close proximity to the main field to prove recovery of the injection water for compliance with Nevada water law. The chemical compatibility and aquifer hydraulics of the demonstration project are summarized by Brothers and Katzer (1990). The demonstration project proved that artificial recharge was feasible in the Valley.

Expansion of Artificial Recharge

Figure 2 displays the relative location of the District wells and documents the expansion of the program through time. Retrofitting of existing production wells for recharge/recovery operation occurred from 1989 to 1991 in proximity to the District’s main well field. The wells used for recharge where predominately drilled prior to 1980 by the cable tool method. The casing was perforated in place and the wells were naturally developed. The existing wells were retrofitted with minimal modifications. Pump impellers were prevented from rotating, check valves opened up to allow water
Figure 2. -- Location of District Wells Used for Artificial Recharge
down the pump column and flow meters manually reversed to monitor injection rates. Stationary pump impellers provided sufficient friction loss to fill the pump column, preventing air entrainment. During this time period, 15 wells where used for artificial recharge.

During the 1992 to 1993 artificial recharge season, 10 more wells were added to the program. The majority (seven) of these wells were like previous wells used in the program -- constructed more than 20 years ago by cable-tool methods. Three of the wells were recently drilled by reverse circulation drilling methods and gravel packed. During the following season (1993-94), eight more of the recently, reverse circulation drilled wells were brought on-line as dual use wells.

As each well was added to the program, its performance, both as an injecting and pumping well, was closely monitored and it became apparent that the two types of wells responded quite differently to artificial recharge operations. Figure 3 displays the performance of a subset of the dual-use wells located in the main well field. Of seven total wells, three of the wells are the older cable-tool (CT) drilled type and the other four are the newer reverse-circulation (RC) wells. Figure 3 compares the recharge water level recorded in the artificial recharge wells over the 1993-94 injection with the rising static water level. In three of the older CT wells the recharge water level rise approximates that observed in the static well - indicating the recharge water level rise is a result of reservoir filling. All four of the newer RC wells, however, show recharge water level rises much steeper than the static water level rise, suggesting near well bore clogging.

![Figure 3](image-url)  
**Figure 3.** -- Comparison of recharge water level rise to static water level rise in the main well field during the 1993-94 season. (CT) wells are older cable-tool drilled wells; (RC) are newer reverse circulation drilled wells.
The clogging observed in the newer wells when they were operated for artificial recharge also became apparent when the newer wells were pumped during the following summer production season. Flow rate declines and production specific capacity reductions occurred in all 11 wells, and the reductions were significant (from 30 to 60 percent). Because the District was so dependent on the water supply in the summer from these wells, they were removed from the artificial recharge program and the challenge focused on how to expand the artificial recharge program without incurring significantly increased well maintenance and operational costs on the rapidly clogging wells.

Instituting a program of re-development at each well has been recommended and is common practice at other aquifer storage and recovery programs in the United States (Pyne, 1994; Bay and Bowser, 1993). However, instituting such a program would significantly increase operational costs of these newer wells and there was no guarantee that the benefits of re-development would be cost-effective. Two other options were also examined: 1) constructing gravel-packed single use injection wells with reverse circulation or air foam drilling methods (addressing the "particle rearrangement" hypothesis of clogging (Pyne, 1994)) and 2) constructing injection-only or dual-use wells with cable tool methods and natural development.

To investigate the "particle rearrangement" hypothesis, five gravel packed wells were constructed and equipped for use as injection only wells (wells AR-6, AR-7, AR-8, AR-9, and AR-10). These wells were placed into operation during the 1994-95 injection season. Specific injection capacity declines recorded in these wells was also much greater than that anticipated from rising static water level.

Use of the cable-tooled drilling method to construct injection-only and dual-use wells began late in 1996. Currently three wells are under construction using the cable-tool drilling method and natural development (AR-94, AR-95 and AR-100). Another well (AR-93) will employ the dual rotary method of well construction. These methods were selected to minimize the "damage-zone" of the aquifer that occurs during drilling, and also to eliminate the need for a gravel pack in the annulus of the well bore. It is anticipated that these types of wells will have a smaller clogging potential then the reverse-circulation drilled, gravel-packed wells numbered 68 through 85 and the dedicated artificial recharge wells AR-6 through AR-10.

System Operations

Distribution system operations at the District changed significantly due to the continued expansion of the artificial recharge program. Maximizing artificial recharge requires certain regional pumping stations and parts of the distribution system to be operating near full capacity year round. Traditionally during the winter months maintenance of both production wells and pumping facilities occurred. Maximizing artificial recharge meant that routine maintenance was rescheduled. The continual
maintenance of both production wells and pumping facilities occurred. Maximizing artificial recharge meant that routine maintenance was rescheduled. The continual pumping of distribution facilities resulted in stresses not anticipated when they were designed.

During the early years of the artificial recharge program all valving changes at the wells and related system facilities were accomplished manually. The lack of automatic control at the recharge sites meant that system demand had to drop and stay down before a field crew was sent out to perform the site modifications to bring the well online for artificial recharge. To make artificial recharge operation more efficient, valving at recharge sites were automated to minimize the turn time between injection and production. Automated bypass valve were installed to route water around the check valve. Magnetic flow meters were installed on all wells so that flow measurements were possible without having to manually reverse flow meters. With the automated system in place, artificial recharge operations can be initiated from the central computer any time there is excess capacity available from SNWS.

**Results of Artificial Recharge**

The volume of water injected by the artificial recharge program on a monthly basis is summarized in Figure 4. The volume of recharge increased steadily from 1989 to 1994. The maximum volume of artificial recharge occurred from October 1993 to May of 1994. During this time period, a monthly maximum of 5,200 acre feet of water was injected in January 1994. Since 1994 artificial recharge volumes have declined as a result of increased demands on the SNWS system. Total volume of water banked as of January 1, 1997 is 114,126 acre-feet. With the expansion of the SNWS system schedule for completion in June 1997, artificial recharge volumes are anticipated to exceed the 5000 acre feet of water per month. System modeling predicts that for the 1998 recharge season up to 38,000 acre feet may be available for injection.
Figure 5 shows the change in the potentiometric surface in the principal aquifer from October 1990 to October 1996. Evaluation of the October water levels annually are critical to observing impacts to the Valley ground-water system. Generally, District production occurs from May through September. October represents a static condition following up to five months of maximum pumping stress. Changes observed in the October potentiometric surface are conservative measurement of the positive effect the recharge program is having on the water levels in the valley. Static water levels have risen from ten to forty feet in areas surrounding the District’s artificial recharge program. Water levels in other areas of the Valley also show increases, indicating that the rise is not isolated, but is occurring throughout the principal aquifer.

Figure 5.— Difference in Potentiometric Surface of the Principal Aquifer: October 1990 to October 1996.
Summary

Since the start of artificial recharge in the Valley with the pilot and demonstration projects, many modifications have occurred to the well operations and the distribution system. Different types of well completions have been used by the District throughout the project to optimize recharge. The primary method of injecting water has been through dual use wells drilled by the cable tool method. In 1994 single purpose injection wells were added to the program to address impacts of plugging in the gravel envelope wells. Major distribution systems changes have occurred as the project has evolved. The system has been automated to more efficiently recharge when ever sufficient capacity is available from SNWS. With the expansion of system capacities in 1997, the artificial recharge program should expand accordingly. Water levels throughout much of the principal aquifer are rising as a result of the program.

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AUGMENTING EXISTING WATER SUPPLIES FOR THE
CITY OF CHANDLER, ARIZONA
THROUGH AQUIFER STORAGE USING FOUR DIRECT INJECTION WELLS

Greg L. Bushner, Gary G. Small, and Christine H. Close

ABSTRACT

The City of Chandler, Arizona, has undertaken a project to conserve and store 2.3 million gallons per day (mgd) of highly purified reclaimed water from the Intel Fab 12 site. Four direct injection wells will be used to recharge the water using back pressure from the existing reclaimed water line. The target injection zone is an interbedded layer of sands and gravels with minor amounts of clays, within the Middle Alluvial Unit (MAU), as defined in the Basin and Range Geomorphic Province. This injection zone is approximately 400 to 600 feet below land surface at the site. In addition to the injection wells, six monitor wells were constructed at the site. Three monitor wells were constructed in the Upper Alluvial Unit (UAU), two were constructed in the target recharge zone, and one was constructed in the very upper portion of the Lower Alluvial Unit (LAU), beneath the target injection zone.

Each direct injection well was constructed with 110 feet of stainless steel louvered casing along with 50 feet of blank stainless steel casing. Only well IW-1 was completed with stainless steel casing for the entire well. The remainder of the casing in the other three wells was high alloy carbon steel. High quality Colorado Silica sand was used for the gravel pack around the louvered casing. The annular space in each well above the gravel pack was sealed using a bentonite grout that congeals into a cohesive mass filling the space between the well and the casing.

Water quality data were evaluated to predict the reactive capacity of the highly purified reclaimed water with the ambient groundwater. These data were used to determine the need for stainless steel well casing. Using the water quality reaction analyses, only the louvered section

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of casing needed to be constructed with stainless steel. Additional water quality analyses is being undertaken to evaluate the mixing of the purified reclaimed water and the groundwater.

An injection well test was conducted using well IW-1 to determine the injection capacity of the aquifer and make a determination of the total number of injection wells needed. IW-1 was tested for a period of 37 days during which time the water level within the well rose to a maximum of 136 feet. The pumping rate averaged 394 gallons per minute (gpm). Based on the test results, it was confirmed that three additional injection wells were needed. This will allow the City of Chandler to inject an average of 410 gpm in each of the four wells to achieve the capacity of 2.3 mgd. A backup well may be required for long term project operation.

INTRODUCTION

The City of Chandler, Arizona, has undertaken a project to conserve and store 2.3 mgd of high quality reclaimed water from the Intel Fab 12 Plant. The Intel Fab 12 Plant is located approximately 6 miles west of the Industrial Process Water Treatment Facility (IPWTF) where the water is stored (Figure 1). Four direct injection wells are being used to recharge the water using back pressure from the existing reclaimed water line.

The target injection zone is an interbedded layer of sands and gravels with minor amounts of clays, within the MAU, as defined in the Basin and Range Geomorphic Province. This injection zone is approximately 400 to 600 feet below land surface. In addition to the injection wells, six monitor wells were constructed at the site. Three monitor wells were constructed in the UAU, two were constructed in the target recharge zone, and one was constructed in the very upper portion of the LAU, beneath the target injection zone.

GENERAL GEOLOGY

The City of Chandler is located in the East Salt River Valley (SRV) which is part of the Basin and Range Geomorphic province. The East SRV is a basin filled with alluvial sediments several thousand feet thick. Within the study area, it can be divided into three primary water bearing units that include: (1) the UAU, (2) the MAU, and (3) the LAU. Wells within a 25 square mile area encompassing the IPWTF site were evaluated to assist in determining the geology surrounding the project site. The site-specific cross-section shown in Figure 2 was completed by Kenneth D. Schmidt & Associates. The cross-section illustrates the relationship between the three units at the site.

The UAU is the primary yielding aquifer unit because of its unconsolidated coarse grained materials, consisting of gravel, sand, and silt (Laney and Hahn, 1986). It ranges in thickness from 100 to 300 feet, thinning toward the mountain fronts. Well yields from the UAU can exceed 4,000 gpm. However, the water quality is usually poor as compared to the other two aquifer units.
Figure 1
City of Chandler
Industrial Process Water Treatment Facility Location
Map and Site Plan

Explanation
- Monitor Well (CI)
- Direct Injection Well (IW)
The MAU is generally a finer grain material with sequences of coarse grain sediments. The MAU ranges from approximately 100 to 1,000 feet thick within the East SRV (Laney and Hahn, 1986). It is generally thicker near the valley center and thins or is missing toward the basin margins. The MAU is the poorest yielding aquifer unit of the three however, water quality is generally of better quality than the UAU. Well yields generally range up to 1,000 gpm.

The LAU is made up of several types of sediments ranging from silts, clays, sands and gravels, to conglomerates, and evaporite deposits, with interbedded volcanics. In general it is coarser grained as compared to the MAU. The LAU ranges in thickness from 600 to 10,000 feet (Laney and Hahn, 1986). Again, it is generally thicker near the valley center and thinner at the basin margins. These sediments are usually more consolidated than the UAU with yields ranging up to 2,000 gpm. The LAU has varying water quality, due to evaporite deposits.

GENERAL HYDROLOGY

Wells within the study area range from small domestic wells that have a pumping capacity of less than 35 gpm to large irrigation production wells that have an average pumping capacity of 2,100 gpm. The majority of the water supply wells are generally completed within the MAU or LAU to depths that range from 400 to 800 feet below land surface. These wells tend to be smaller domestic wells and perhaps some smaller capacity irrigation wells. There are several large diameter deep irrigation and municipal production wells within the study area that tap the LAU, and range in depth from 1,000 to 1,500 feet below land surface. The deepest well within the study area is reported to be 2,000 feet.

There are two expressions of the groundwater table that have been observed within the study area. The uppermost groundwater table is in the UAU as documented by data collected from the on-site shallow monitor well (CI-4). Water levels in this unit did not respond to the injection test conducted with well IW-1.

Water level elevations in the regional groundwater table range from approximately 1130 feet above m.s.l. to 1030 feet above m.s.l. The groundwater flow direction is generally to the northwest and may be influenced locally by pumping wells. A hydraulic gradient of 0.007 feet per mile was calculated for the IPWTF site with a flow direction to the north, northwest.

MONITOR WELLS

There were six monitor wells installed by Kenneth D. Schmidt and Associates at the IPWTF site (Figure 1). Monitor wells CI-1 and CI-2 are shallow monitor wells that penetrate the UAU only. Monitor wells CI-3, CI-4, and CI-5 were installed in a cluster and are located just west of the first injection well IW-1. Monitor well CI-6 is a deep monitor well that taps the target injection zone of the MAU.
The monitor wells CI-3, CI-4, and CI-5 show that the target injection zone is isolated from the UAU by several hundred feet of clays (Figure 2). Wells perforated beneath these clay units often have water levels that rise above the top of the water producing unit. Well CI-3 is the deep monitor well and is perforated in the LAU beneath the injection zone. Well CI-4 is a very shallow well that is perforated in the UAU. Well CI-5 is perforated in the same zone (MAU) as the injection wells.

The water level in well CI-4 changes in response to atmospheric pressure and to recharge from the land surface, whereas wells CI-3 and CI-5 show a potentiometric surface that changes in response to pumping of water supply wells in the area. Response to the injection and withdrawal of water within the confined aquifer is shown on Figure 3.

WATER QUALITY ANALYSIS

Water quality data were evaluated to predict the reactive capacity of the highly purified reclaimed water with the ambient groundwater. These data were used to determine the need for stainless steel well casing. Using the water quality reaction analyses, only the louvered section of casing needed to be constructed with stainless steel. Additional water quality analysis is being undertaken to evaluate the mixing of the purified reclaimed water and the groundwater.

AQUIFER TESTING

The aquifer test results for each of the direct injection wells is provided in Table 1. Transmissivity values ranged from 36,200 gpd/ft for wells IW-1 and IW-2, respectively. These values are based on calculated recovery data collected at the end of the aquifer test when water levels in the wells were recovering or attempting to achieve a state of equilibrium.

Storage coefficients were determined from the measurement of the closest observation (monitor) well perforated in the same intervals as the pumping wells. Storage coefficients ranged from 1.14E-03 to 4.72E-04 for wells IW-1 and IW-4, respectively.

<table>
<thead>
<tr>
<th>Table 1. Results of Aquifer Testing</th>
</tr>
</thead>
<tbody>
<tr>
<td>Well</td>
</tr>
<tr>
<td>IW-1</td>
</tr>
<tr>
<td>IW-2</td>
</tr>
<tr>
<td>IW-3</td>
</tr>
<tr>
<td>IW-4</td>
</tr>
</tbody>
</table>
Figure 3

Water Level Changes in Monitor Well CI-5
During Injection Test of Well IW-1
DATA COLLECTION

Each of the direct injection wells is equipped with an ACT-PCK controller, flow meter, and pressure transducer. The controller is attached to the inside of the vault structure and is housed in a water-proof sealed container in case the vault structure is flooded (either by overflow of the direct injection wells, or by land surface flooding). Using the controller, the flow rate and pressure in each well can be monitored. The water level in each well is measured through the use of a water level pressure transducer directly wired to a Programmable Logic Controller (PCL) or data logger. From there, the signal is relayed to the control room within the City of Chandler’s Operations Facility.

For the pilot injection testing, the monitor well cluster to the west of well IW-1 (Figure 1), were equipped with pressure transducers in order to collect real-time water level data as the test progressed. Using a data logger, real-time data were collected from the monitor well cluster, and the direct injection well IW-1. The data logger queried the sensors, and recorded and coded the data. Coding consists of identifying the well, and adding a date and time stamp for each query. The data recorded included water level height, flow rate, and total flow for the direct injection well, and water level height in each of the adjacent monitor wells (CI-3, CI-4, and CI-5).

These data were downloaded on a weekly basis via lap-top computer that was taken into the field and connected to the data logger. These data were received by the data logger at five-minute intervals and stored in a data file on the lap-top computer. These data were then downloaded from the lap-top computer to a computer at the HydroSystems’ office and stored in separate files for permanent storage and analysis. These data were then imported into the Excel software to format for analysis and data evaluation.

PILOT INJECTION TESTING

A pilot injection test was conducted with well IW-1 from May 8, 1996 through June 13, 1996. Groundwater water pumped from an irrigation well approximately ½ mile from the injection well provided the source water for testing. The average flow rate during the test was approximately 390 gpm which converts to about 63 acre-ft (Figure 4).

It is evident that the impacts from the injection did not affect either the shallow groundwater present at the site (UAU aquifer) or the LAU aquifer, as determined from monitor well CI-3. Water level changes that occurred during the test in monitor well CI-3, can be attributable to regional groundwater pumping by wells perforated in the deeper aquifer in the vicinity of the IPWTF site. Well CI-3 is perforated in the upper portion of the LAU aquifer, below the level which water was injected into the MAU aquifer.
Water levels in well CI-4 did not change much and remained at about 52 feet below land surface throughout the test, indicating that the injection test had no affect on the shallow groundwater. This well is completed in the upper perched aquifer of the UAU at the site.

Groundwater levels in the area fluctuated during the testing period due to local pumping. This is evident by the changes in water levels that occurred in monitor well CI-5, which is perforated in the target injection zone of the MAU aquifer (Figure 3).

SUMMARY

The IPWTF is able to receive high quality reclaimed water from the Intel Fab 12 Plant and store it in the MAU aquifer through the use of four direct injection wells. At the present time well field is effectively recharging the product water flow from the plant. The current water production from the plant is approximately 800 gpm.

Significant and unique features within Arizona of the IPWTF project are as follows:

- The use of gravity flow direct injection wells for recharge is unique. These wells operate at very low system back pressure.

- Each of the direct injection wells occupies relatively little space onsite. The area required for the wells is less than 10 X 10 feet. This can be compared to the requirements for a spreading basin to recharge a similar volume of water.

- Through the use of an effective well design, water was recharged in the target injection zone. The target injection zone is separated by units of lower permeability both above and below.

This project was completed in conjunction with the following engineering and groundwater quality consulting firms: HydroSysts, Inc., was responsible for the direct injection well design, construction and testing. Kenneth D. Schmidt and Associates was responsible for the recharge and water quality permitting, monitor well design and installation, and water quality data collection. Black & Veatch is the principal engineering firm that was responsible for the piping design and construction and the evaporation basins.

REFERENCES

AVRA VALLEY RECHARGE PROJECT: A CAP SPREADING BASIN RECHARGE OPERATION IN THE TUCSON ACTIVE MANAGEMENT AREA DEVELOPED THROUGH LOCAL AND STATE PARTNERING

Clifford A. Neal, Tom Harbour, Michael W. Block

INTRODUCTION AND BACKGROUND

The Avra Valley Recharge Project (AVRP) is located in the northwest portion of Arizona's Tucson Active Management Area (AMA) and was the first spreading basin project to recharge untreated Central Arizona Project (CAP) water in Pima County. Development and operation of the project has occurred through coordination with and cooperation of many state and local agencies. The project concept was originated by Metropolitan Domestic Water Improvement District (Metro), a local area municipal water provider, and BKW Farms, Inc. (BKW), a local farming company. The facility was designed, permitted, and constructed by the Central Arizona Water Conservation District (CAWCD) as a State Demonstration Project for the underground storage of excess CAP water. Funds used in the project's development were accrued through a county-wide ad valorem property tax. CAWCD is responsible for facility operations which began on July 18, 1996, shortly after the Arizona Department of Water Resources (ADWR) issued an Underground Storage Facility Permit to CAWCD. This permit authorizes recharge of up to 8,300 acre-feet (AF) of CAP water during a two year pilot phase. Metro agreed to purchase all of the water to be stored under the project's pilot phase, for which it will accrue underground storage credits. Key goals of the project are to assess the feasibility of operating a long-term spreading basin recharge project in Pima County, to assist in meeting the assured water supply needs of water providers in the Tucson AMA, and to provide space to store Colorado River water during times of abundant supply. Figure 1 is a map showing the general location of the AVRP.

Hydrogeologic Features

The AVRP site is located in the northern part of the Avra Valley, a tectonically depressed trough that contains several thousand feet of basin-fill deposits eroded from


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surrounding mountain blocks. The principle geologic units of interest at the site include, in
descending order: recent alluvium, Fort Lowell Formation of Quaternary age, and Tinaja
beds of Tertiary age. These units are underlain by a faulted and tilted sequence of older
sedimentary rocks and a basement complex of igneous and metamorphic rocks
(Montgomery & Associates, 1995).

The AVRP site is located about one-half mile south of the Santa Cruz River, which
flows perennially as a result of treated effluent releases into the river from Tucson’s Ina
Road and Roger Road treatment plants located about 17 miles and 25 miles upstream,
respectively. The site is surrounded by agricultural land which has been farmed using
groundwater since the 1930’s, resulting in area groundwater table declines in excess of 120
feet. Depth to groundwater immediately below the site was about 290 feet prior to
initiation of recharge.

DESCRIPTION OF FACILITIES

Recharge Facilities

AVRP consists of eleven acres of spreading basins situated on state land outside of
the 100-year floodplain of the Santa Cruz River. The project was sited in an abandoned
materials borrow pit in an effort to reduce excavation costs associated with construction of
spreading basins. The facility consists of four basins, ranging in size from two to three
and one-half acres. Untreated CAP water is transported by gravity flow approximately one
mile from the CAP aqueduct to the recharge facility via a concrete-lined conveyance canal
which was constructed by BKW with funding assistance through a Tucson AMA
augmentation grant and the City of Tucson. The BKW canal continues past the AVRP
facilities, carrying CAP water to agricultural fields located to the northwest.

CAP water is diverted from the BKW canal into the recharge facility through a
turnout structure located upstream of an adjustable check gate. The check gate raises the
water level in the conveyance canal sufficiently to guarantee minimum stage for inflow to
the facility. Water enters the recharge facility distribution system after passing through a
trash rack to remove any large debris. A main slide gate controls the volume of water
passing through the turnout and into a buried 24" pipeline leading to the approach pool of
the facility’s primary weir. The primary weir provides a continuous measurement of
facility inflow. Water stage is measured in a stilling well adjacent to the approach pool as it
passes over the 4.5 foot sharp-crested contracted rectangular weir. Water flowing over the
weir crest free-falls into the downstream plunge pool where a series of slide gates control
flows to the four individual basins. Inflow to each recharge basin is measured by smaller,
90 degree V-notch weirs. V-notch weirs were selected to measure basin inflows because
of superior measurement accuracy at low flows. Water passing over the crests of the basin
weirs enters distribution pipes leading to the basins. Water levels in the basins are
maintained at depths no greater than 2.5 feet.

All weirs are equipped to provide a continuous measurement of stage for calculation
of flow rate and volume. Each basin is equipped to provide a continuous record of stage.
Water levels are measured every 15 minutes in weir and basin stilling wells using water
level transducers. Data from each transducer is transmitted via buried cable back to a
measurement and control unit (MCU) located at the recharge facility. Data are transmitted
by radio from the MCU to the CAP’s Twin Peaks Pumping Plant about five miles from the
AVRP. The information is integrated into CAWCD’s communications system and
transmitted back to CAP headquarters in Phoenix where staff can monitor facility

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operations and retrieve stored data for further analysis.

Groundwater Monitoring Facilities
Wells identified in Table 1 comprise the monitor well network being used to monitor groundwater levels and quality in the regional aquifer during pilot recharge operations. Locations of these wells are also shown on Figure 1.

**TABLE 1: AVRP GROUNDWATER MONITORING WELLS**

<table>
<thead>
<tr>
<th>CADASTRAL</th>
<th>IDENTIFIER</th>
<th>WELL USE</th>
<th>WATER LEVELS</th>
<th>WATER QUALITY</th>
<th>WELL OWNER</th>
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<tr>
<td>D(12-11)02abb</td>
<td>TANG-1</td>
<td>Monitor</td>
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<td>Yes</td>
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<tr>
<td>D(12-11)02abc</td>
<td>TANG-2</td>
<td>Monitor</td>
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<td>D(12-11)02ada</td>
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<td>Monitor</td>
<td>Yes</td>
<td>Yes</td>
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<td>D(12-11)02ccd</td>
<td>TA-47</td>
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<td>No</td>
<td>USBR</td>
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<td>Yes</td>
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<td>Monitor</td>
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<td>SC-09</td>
<td>Monitor</td>
<td>Yes</td>
<td>Yes</td>
<td>Pima County</td>
</tr>
</tbody>
</table>

Of the eight monitor wells included in the monitoring network, only AVMW-01 was constructed as part of the AVRP. The remaining wells were already in place prior to development of the project. Use of Pima County’s monitor wells is made possible through a data sharing agreement between Pima County and CAWCD. This agreement provides for an exchange of water quality and water level information. Pima County provides results of water quality analyses from its existing monitor wells to CAWCD in return for results from the AVRP on-site monitor well and pilot recharge operations. This cooperative agreement reduces project costs by minimizing the need for redundant sampling and completion of additional monitoring wells.

Groundwater levels are measured weekly in the on-site monitor well (AVMW-01) and monthly in the seven off-site monitor wells. Groundwater samples are obtained for chemical analysis quarterly at AVMW-01, SC-09, and SC-10. TANG-1, TANG-2, and TANG-4 are sampled twice per year.

Vadose Zone Monitoring Facilities
Soil borings conducted early in project development identified a ten-foot thick layer of sandy clay about 30–35 feet below land surface which appeared to be laterally continuous. It was feared that this layer would inhibit recharge by creating a perched groundwater mound below the basins. To monitor for this potential occurrence, seven piezometers, designated AVPZ-01 through AVPZ-07, were constructed in and around the facility. AVPZ-01 and AVPZ-02 are nested piezometers located about 300 feet downstream of the basins and were installed to provide a measurement of the lateral extent of perched groundwater migration. AVPZ-01 was completed to the top of a fine-grained unit about 73.5 feet below land surface. AVPZ-02 was completed to the top of the sandy-clay
unit about 30 feet below land surface. AVPZ-03 through AVPZ-07 are located directly adjacent to the recharge basins and are all completed to the top of the first substantial fine-grained unit approximately 30-40 feet below land surface. AVPZ-03 is on the west side of the basins, AVPZ-04 is south, AVPZ-05 is east, AVPZ-06 is north, and AVPZ-07 is on a berm in the middle of the basins. Depth to perched groundwater below land surface is measured weekly at each of the piezometers.

OPERATION AND MONITORING RESULTS

Recharge Volumes

During the eight-month period from July 1996 through February 1997, a total of 3,963 AF of CAP water was delivered to AVRP. Figure 2 provides a graph showing monthly inflow to the facility during this period. Deliveries prior to mid-October were restricted due to capacity limitations of BKW’s temporary pump station diverting water from the CAP aqueduct into the BKW conveyance canal. In October, BKW completed installation of a larger capacity, permanent pump station and deliveries to AVRP from then until mid-February were not restricted by canal or pump station limitations. February deliveries are significantly lower than those in the previous three months because CAP water was not available after February 13 due to an outage on the main CAP aqueduct system.

Figure 2 also shows the monthly volumes of water delivered to each of the four AVRP basins. As this graph indicates, deliveries were made to all four basins simultaneously from October through mid-February, with no drying cycles at any of the basins. Infiltration rates were lowest in Basin 4, averaging just over 0.5 foot per day. The infiltration rate in Basin 3 was initially estimated to be about 0.75 feet per day, but increased to over 4 feet per day after several weeks of continuous inflow and basin wetting. Infiltration rates in Basins 1 and 2 were difficult to estimate initially because inflow capacity through the weirs serving these basins was not sufficient to fill the basins in the first five months of operation. However, by the end of January, the bottom of both basins were covered with water and infiltration rates stabilized at about 2.5 feet per day in Basin 2, and 4.5 feet per day in Basin 1.

One of the conditions of the Underground Storage Facility permit associated with the project is to measure or calculate all water losses at the facility. Because none of the delivered water left the project boundaries except through infiltration or evaporation, the only losses were those caused by evaporation. In calculating evaporation losses, total wetted area of the basins was estimated by visual inspection and real-time monitoring of basin water level elevations. The daily volume of water lost to evaporation was calculated based on the Cooley method using the “Maximum” curve (Cooley, 1970). The total volume lost to evaporation during the initial eight-month period was calculated to be 20.0 AF, or about 0.5% of the total project inflow. Therefore, net recharge at the facility from July 1996 through February 1997 totalled 3,943 AF.

Impacts on Regional Aquifer Groundwater Levels

 Depths to groundwater were measured in the network of eight monitoring wells prior to and during recharge operations. Figure 3 provides graphs showing water level data for each of the monitoring wells. On-site monitoring well AVMW-01 is located adjacent to the recharge basins and provides the most accurate measurement of the height of the groundwater mound resulting from recharge at AVRP. At this well, a total water level elevation increase of 21.0 feet was measured between July 8 1996, and February 11, 1997,
FIGURE 3: GROUNDWATER LEVELS IN AVRP MONITOR WELLS

SC-09 Water Level Elevations
Ground Surface Elevation: 1991.54'

SC-10 Water Level Elevations
Ground Surface Elevation: 1978.07'

TA-47 Water Level Elevations
Ground Surface Elevation: 2032.90'

BKW-6 Water Level Elevations
Ground Surface Elevation: 2015.50'

TANG-1 Water Level Elevations
Ground Surface Elevation: 2018.44'

TANG-2 Water Level Elevations
Ground Surface Elevation: 2020.72'

TANG-4 Water Level Elevations
Ground Surface Elevation: 2029.82'

AVMW-01 Water Level Elevations
Ground Surface Elevation: 2015.31'
however most of the increase occurred after the end of September. Prior to September, water levels remained near static or declined slightly in response to local groundwater pumping. The water level measurement taken one week after flows into the project were discontinued in mid-February show a drop in the groundwater table of more than four feet, indicating a direct response to project operations. Off-site monitor wells show no apparent impact on water levels caused by recharge at AVRP, likely due to their distance from the project. Variability in water levels measured at the off-site monitor wells appears to be a result of other outside influences.

**Occurrence of Perched Groundwater**

The network of seven piezometers is monitored for the occurrence of perched groundwater development above fine-grained units in the vadose zone. **Table 2** provides a summary of the mound height measured at each piezometer through February 1997. Water is normally detected in AVPZ-01, however, because water was detected prior to recharge and is consistently less than 0.5 feet, it is believed to be residual drilling fluids that drained from the formation and is not derived from recharge. AVPZ-02 remained dry until late November and then developed 0.3 feet of perched groundwater by the end of January 1997. No perched groundwater has been observed in AVPZ-03, AVPZ-04, and AVPZ-05. After about two months of operation, water was observed in AVPZ-06. The perched groundwater mound measured in AVPZ-06 reached a maximum height of 4.55 feet on September 13, but steadily diminished to zero by January 8. Perched groundwater was observed in AVPZ-07 shortly after recharge operations began. The maximum mound height measured in AVPZ-07 was 2.3 feet, occurring on August 22. The mound at this piezometer then fluctuated between 1 and 2 feet until flows to the project were stopped in mid-February.

It is important to note that the height of the perched groundwater mound decreased as recharge volumes increased. The maximum mound height of 4.55 feet remained well below the invert of the basins and did not affect recharge efficiencies. At this point in the pilot operations, perched groundwater mounding does not appear to pose a threat to recharge capacity.

**Water Quality Monitoring**

Periodic groundwater quality samples are obtained and analyzed from the network of six monitoring wells to characterize ambient groundwater quality and monitor water quality transformations during recharge operations. In addition, CAP source water is sampled at the facility turnout and analyzed for the same chemical constituents as the groundwater samples. Samples are analyzed for general chemistry, total metals, chlorinated pesticides, chlorinated herbicides, volatile organic compounds, total petroleum hydrocarbons, and total coliform.

Data indicate that numeric aquifer water quality standards were not exceeded in groundwater with the exception of total coliform which was present at a concentration of 300 MPN/100 mL in a sample obtained from AVMW-01 on 8/8/96. The presence of coliform after less than one month of recharge operations suggests that contamination likely occurred during well construction. However, since the allowable aquifer water quality standard for microbiological contaminants is zero per 100 mL aqueous sample, the Arizona Department of Environmental Quality (ADEQ) was immediately consulted. Based on ADEQ recommendations, the well was disinfected by superchlorination using calcium hypochlorite granules and then purged of residual chlorine. A repeat sample obtained on 10/4/96 failed to detect coliform and an additional sample on 11/13/96 confirmed that the well was successfully disinfected.
### TABLE 2: Avra Valley Recharge Project Perched Groundwater Observations

<table>
<thead>
<tr>
<th>Date</th>
<th>AVPZ-01</th>
<th>AVPZ-02</th>
<th>AVPZ-03</th>
<th>AVPZ-04</th>
<th>AVPZ-05</th>
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<td>0</td>
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<tr>
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The volatile organic compound Toluene was detected in AVMW-01 on 6/21/96 at a trace concentration of 1.1 ug/L, well below the aquifer water quality standard of 1,000 ug/L. This constituent has not been observed in subsequent samples and is likely the result of sample handling or laboratory contamination. A chlorinated herbicide, 2,4-Dichlorophenoxyacetic Acid, (2,4-D), was detected in the CAP source water sample collected on 11/13/96. 2,4-D is a weed-control agent that is commonly used in agricultural areas. The detected concentration of 2,4-D was 1.8 ug/L, which is below the aquifer water quality standard of 70 ug/L.

CONCLUSIONS

Although preliminary operational results may not fully characterize long-term effects, analysis of data collected during the initial eight months of pilot recharge operations at AVRP support the following general conclusions:

- Recharge volumes in excess of 800 acre-feet per month and infiltration rates over 2.5 feet per day appear to be achievable. Infiltration rates remained high indicating basin clogging due to settling of suspended solids and algal growth is not occurring.

- Weekly water level measurements at on-site monitoring well AVMW-01 confirmed the development of a localized groundwater mound in the regional aquifer underlying the AVRP facility. Water level elevations measured in off-site monitoring wells showed little change during the year which may indicate that mounding is not laterally extensive.

- Development of perched groundwater above fine-grained units in the unsaturated zone does not appear to pose a threat to recharge capacity. Perched groundwater was detected in only 3 of the 7 piezometers and although mound height peaked at 4.55 feet, this is well below basin invert elevations. It is important to note that mound height has steadily decreased as recharge volumes increased and during the peak recharge months of November-December, perched groundwater mounding was less than 2 feet.

- Ambient water quality data obtained prior to recharge operations compared with data collected during the first eight months of recharge operations exhibit no apparent negative water quality impacts.

The pilot phase of the Avra Valley Recharge Project has helped demonstrate to Pima County water resource managers, public officials, and citizens that recharge of CAP water is physically possible. In addition, the cooperative atmosphere under which the project was developed and the project’s operational success has assisted in removing some of the negative image of CAP water in the region and has begun to promote the increased utilization of this renewable water resource.

References Cited


CHEMICAL PROCESSES IN THE VADOSE ZONE: IMPLICATIONS FOR GROUNDWATER RECHARGE OF CAP WATER AND HIGH QUALITY EFFLUENT AT THE SCOTTSDALE, ARIZONA "WATER CAMPUS"\(^1\)

Juliet S. Johnson, Lawrence A. Baker, and Peter Fox\(^2\)
Department of Civil and Environmental Engineering

INTRODUCTION
Water scarcity is likely to become more problematic in the near future due to rapid population growth, increasing per capita consumption of water, and geographical disparities between centers of population growth and availability of water. In many arid lands deliberate recharge of groundwater with effluent or surface water is being considered as a way to replenish overdrafted aquifers and provide sustainable water supplies (Bouwer et al., 1990). Recent have demonstrated reductions in concentrations of biological oxygen demand (BOD), suspended solids, bacteria and viruses, and nitrogen during groundwater recharge of effluent (Bouwer et al., 1990; Bouwer, 1991; Wilson et al., 1995; Kopchynski et al., 1996). In this paper, we focus on inorganic geochemical processes that occur during passage of recharge water through the vadose zone. Of particular concern is the leaching of naturally occurring contaminants that are often found in soils in arid climates (fluoride, boron, arsenic, chromium, lead, barium; Hem, 1965; Faust and Aly, 1981; Baker and Bolitho, 1995), salinization of soils by ion exchange reactions, and dissolution/precipitation reactions that may cause alter soil physical properties (Pyne, 1995; Bouwer et al., 1991; Galperin et al., 1993; Krauskopf and Bird, 1995).

We would like to thank Cristine Close (HydroSciences, Inc.), Marty Craig (City of Scottsdale), Taqueer Qureshi (ASU), and Peggy O'Day (ASU) for their contributions to this project.

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STUDY SITE

Our study site is the City of Scottsdale’s “Water Campus Project”, a state-of-the-art groundwater recharge and storage facility. Recharge water will be injected via wells drilled to a depth of 55 m. This water will then infiltrate through 107 m of vadose zone of the groundwater table located at a depth of 162 m (Figure 1). Soils at this location are comprised of Upper, Middle and Lower alluvial units. The Upper Alluvial Unit (UAU) consists of loose, unconsolidated sands and gravels interbedded with silts and clays. The Middle Alluvial Unit (MAU) consists of weakly cemented, fine grained materials. The fine-grained materials includes clastic sediments and evaporite deposits. The Lower Alluvial Unit (LAU) consists of primarily coarse-grained, unconsolidated, clastic sediments. (HydroSystems, Inc., 1995).

We used six soils taken from two pilot recharge wells in our experiments (Table 1). These soils were chosen because they had a very high percentage of fines and were located near strata of high permeability. In all experiments, special attention was paid to the fines portion of each soil. Batch experiments utilized fines that passed through a standard #200 mesh. Because of the need to infiltrate water through the flow-through columns, we used a courser fraction (passing #10 mesh; 2 mm). Previous studies have shown that pyrites, iron hydroxides and other minerals that may release metals and cause contamination are primarily associated with the fines portion of soil, (Allard, 1995; Masscheleyen, 1991; Forstner, 1991, Harmsen 1977; Beek et al., 1979).

Table 1. Description of soils used in batch and column experiments.

<table>
<thead>
<tr>
<th>Soil #</th>
<th>Recharge Well</th>
<th>Depth (m)</th>
<th>Soil Type:</th>
<th>% of fines</th>
</tr>
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<td>26-29</td>
<td>clay</td>
<td>7.1</td>
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<tr>
<td>2</td>
<td>1-3</td>
<td>30-33</td>
<td>clay + silt</td>
<td>23</td>
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<tr>
<td>3</td>
<td>1-3</td>
<td>40-41</td>
<td>silt + silty sand</td>
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</tr>
<tr>
<td>4</td>
<td>1-4</td>
<td>33-36</td>
<td>clayey gravel</td>
<td>7.9</td>
</tr>
<tr>
<td>5</td>
<td>1-4</td>
<td>40-43</td>
<td>clay + clayey sand</td>
<td>8.4</td>
</tr>
<tr>
<td>6</td>
<td>1-4</td>
<td>49-52</td>
<td>clayey gravel</td>
<td>13.1</td>
</tr>
</tbody>
</table>

Three types of water were used in these experiments (Table 2). Water from the Central Arizona Project (CAP) canal has a strong high levels of sodium, chloride, and sulfate and a TDS of 649 mg/L. Membrane-filtered effluent (MF), produced at the Water Campus, has the highest TDS (919 mg/L) and also high concentrations of sulfate, chloride, and sulfate. Both of these waters are well-buffered and circumneutral. The third water that we examined was reverse-osmosis treated effluent (RO) also produced at the Water Campus. The RO water had very low TDS (39 mg/l), almost no alkalinity, and a pH of 6.5.
Table 2. Basic chemistry of the base waters used in all experiments.

<table>
<thead>
<tr>
<th>Parameter</th>
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<th>MF</th>
<th>CAP</th>
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<td>pH</td>
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<td>7.8</td>
<td>7.6</td>
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<tr>
<td>Alkalinity (mg/L CaCO₃)</td>
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<td>108</td>
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<tr>
<td>Calcium (mg/L)</td>
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<td>39</td>
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<tr>
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<td>31</td>
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<td>5</td>
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<tr>
<td>Chloride (mg/L)</td>
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<td>216</td>
<td>91</td>
</tr>
<tr>
<td>Sulfate (mg/L)</td>
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<td>184</td>
<td>258</td>
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<td>Nitrate (mg-N/L)</td>
<td>1.1</td>
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<td>0.03</td>
</tr>
<tr>
<td>Fluoride (mg-N/L)</td>
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<tr>
<td>Bromide (mg/L)</td>
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<td>0.07</td>
</tr>
<tr>
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<td>0.018</td>
<td>0.014</td>
</tr>
<tr>
<td>TDS (mg/L)</td>
<td>39</td>
<td>919</td>
<td>649</td>
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</tbody>
</table>

METHODS

Batch Experiments

We conducted batch experiments with combinations of the three source waters plus two blended waters (CAP + MF; CAP + RO) and all six soils. In each experiment 10 g of soil fines was added to 60-ml high density polyethylene (HDPE) bottles, which were then filled with the source water. Two bottles were prepared for each treatment. One bottle was to be used for analysis of anions, pH and dissolved inorganic carbon. These bottles were soaked in distilled deionized water (DDW) for 24 hours prior to use. The other bottle was to be used for analysis of the major cations and trace metals. These were soaked in a 10% nitric acid solution for 24 hours and then rinsed three times with DDW before use. Bottles containing the soil water slurries were placed in a rotating mixer to assure thorough mixing between the liquid and solid phases. Samples were continuously mixed for 30 days at approximately 40 rpm. Dissolved inorganic carbon (DIC) and pH were measured immediately upon opening the bottles. Samples were then filtered through 0.2μm membrane filters. Filters use for anion samples were washed with 200 ml of DDW prior to filtration; filters used for metals samples were flushed with 200 ml of 10% nitric acid solution prior to filtration. After filtration, the anion samples were refrigerated until analysis. The cation/trace metals samples were acidified with concentrated Ultra Pure™ nitric acid to a pH<2 and then was also refrigerated until analysis.

Column Experiments

Small laboratory column experiments were conducted using CAP, RO and MF waters. The columns (25 cm high x 5 cm ID) were filled with each soil type and packed
to approximately in-situ soil density (**Figure 2**). Columns were operated in the upflow mode at a rate of 0.17 L/day (RO and MF water) for 100 days or 0.67 L/day for 30 days (CAP water). A summary of the column experimental matrix is presented in Table 2. Carboys containing reverse osmosis (RO) and microfiltered (MF) wastewater were kept refrigerated and water was pumped from a cold room (5 °C) directly to the columns in the lab (~ 25 °C) using a 20-channel peristaltic pump.

Samples of the inflow water and the column effluents were collected in beakers at days 1, 2, 3, 6, 15 and 30. The RO and MF water columns were also sampled on days 45 and 100. Because the beakers were open to the air, the samples were approximately at equilibrium with the atmosphere. Samples were filtered using the same procedures described for the batch studies.

Samples were analyzed for all major ions, pH, DIC, fluoride, bromide, nitrate, phosphate, and a suite of trace elements (arsenic, barium, boron, chromium, iron, lead, and selenium) using standard analytical methods (details in Johnson et al., 1997).

**Data Analysis**

Inferences regarding geochemical mechanisms that caused changes in the composition of the water during these experiments were made by examining net changes in the ionic composition of the water and by computerized equilibrium modeling.

Net changes in ion balances are a widely used tool to infer geochemical mechanisms responsible for water quality changes (e.g., Baker et al., 1991). From electroneutrality considerations it follows that

\[ \Delta Ca^{2+} + \Delta Mg^{2+} + \Delta K^+ + \Delta Na^+ = \Delta SO_{42-} + \Delta Cl^- + \Delta HCO_3^- + \Delta CO_3^{2-} + \Delta F^- \quad \text{equation 1} \]

Thus, an increase in a cation must be balance either by a decrease in another cation (implying cation exchange) or an increase in an anion (e.g., by solubilization of a mineral). Because the composition of water exiting the soil columns varied over time and the sampling intervals varied, we expressed the change in ionic composition as the flow-weighted average change (FWAC). This number represents the average change in concentration, between influent and effluent, over the course of the experiment. The flow weighted average change was calculated using the equation:

\[
FWAC = \frac{\sum_{n=1}^{N} \frac{(Cl_i Q + (Cl_i - 1) Q)}{2} \Delta t_i - (Cl_i Q \sum_{n=1}^{N} \Delta t_i)}{Q \sum_{n=1}^{N} \Delta t_i} \quad \text{equation 2}
\]

Where \( N = \) days of the experiment (= 100 for RO and MF, = 30 for CAP)  
\( Cl_i = \) concentration in the column outflow on day \( i \)  
\( Q = \) flow through columns
\[ \Delta t = \text{time interval between sampling days} \]

A chemical equilibrium modeling program, MINEQA2 (Allison et al., 1983), was used to determine when soil solutions were at equilibrium with various minerals. Inputs to the model were measured concentrations of major ions and trace elements, pH, and DIC. Oversaturated species were not allowed to precipitate. For each mineral, the proximity to equilibrium was expressed as the saturation index (SI), where

\[ SI = \log \frac{Q}{K} \]

- equation 3

Where: \( Q = \) ion activity product for a certain mineral  
- \( K = \) equilibrium constant for the same mineral

A SI of zero indicates the solution is in equilibrium with a particular mineral. A SI < 1 SI indicates undersaturated and a SI > 1 indicates oversaturation.

RESULTS

"Washout" during column startup. During the column experiments, the concentrations of many elements in the outflows increased to a peak at around day 2 and then decreased and eventually reached a steady concentration. This phenomenon is illustrated by the behavior of fluoride (Figure 3). This washout phenomenon was observed for most elements. The most likely explanation of this phenomenon is the rapid dissolution of evaporate minerals (e.g., barite, fluorite, gypsum). Oversaturated with respect to known minerals was sometimes observed, suggesting that amorphous or poorly crystallized evaporite minerals may be the source of these ions during washout. The importance of washout was expressed as a "washout ratio", the ratio of the peak concentration (generally on day 1 or 2) to the concentration at the end of the experiment. Among potential drinking water contaminants, the washout ratios were highest for fluoride (91, 140, and 7.5 for RO, MF, and CAP waters), intermediate for barium (3.2, 6.9, and 1.8, respectively), and lowest for arsenic (all waters ~ 1.4-1.5). Peak concentrations for these elements sometimes approached their MCLs. Elevated levels of several contaminants (fluoride, arsenic) immediately after the initiation of recharge were also observed in the pilot recharge wells at the Water Campus. We anticipate that because the flow paths are longer in full-scale system than in the laboratory, the magnitude and duration of the washout phenomenon will be more pronounced in the field than in the lab.

Long-term behavior of regulated contaminants. Concentrations of regulated contaminants that had increased during the washout period declined in all of the laboratory columns. By the end of the experiment, all of these contaminants were well below the MCLs. Several other contaminants (boron, lead, selenium, boron and chromium) had concentrations at or below detection limits throughout the entire experiment.

Changes in major ions and alkalinity. Over the course of the study, net changes in TDS for waters passing through the soils columns were small for both the CAP and MF waters.
There was also little change in pH or alkalinity for these waters as they passed through the columns (Figure 4). One possibly significant change for the MF water was an increase in magnesium concentrations, which were offset by small decreases in calcium and sodium and a small increase in bicarbonate. Equilibrium modeling showed that the MF source water was undersaturated with respect to magnesite (MgCO₃), whereas water leaving the MF columns were very close to equilibrium with magnesite. The implication of this is that the release of magnesium from soils in the upper part of the vadose zone (by cation exchange) may result in oversaturation of magnesite and perhaps other Mg-bearing minerals in deeper soils. Both the source water and effluent from the MF columns were oversaturated with respect to several calcium minerals (dolomite; calcite; aragonite), suggesting that precipitation of one or more of these minerals may occur deeper in the vadose zone. In contrast, RO water consistently leached minerals from the soils, as would be expected. Water exiting the RO columns remained undersaturated with respect to several calcium minerals and had lower pH (average = 6.5) and alkalinity (average ~ 50 mg/L as CaCO₃) than water exiting any of the other columns.

Comparison of batch and column experiments. Results from the batch experiments supported trends seen in the columns. Ion exchange reactions were not seen in the batch tests as would be expected. The longer term reactions, such as the dissolution of arsenic were also not seen in the batch experiments. Batch experiments did show increases in fluoride and barium. These are the same constituents that were “washed out” of the columns. Thus, batch test may be good indicators of the types of readily dissolvable minerals present in soils.

IMPLICATIONS FOR MANAGEMENT OF RECHARGE WELLS

Regulated contaminants. Soils in arid lands often contain elevated levels of metals and other contaminants that could potentially be solubilized during groundwater recharge. We observed evidence for significant leaching of fluoride, arsenic, and barium. For all three of these contaminants and for many major ions, we observed a “washout” phenomenon immediately after initiation of recharge. During this period several contaminants approached, but never quite reached, drinking water MCLs. Because the potential for leaching would be greater for in situ recharge systems, we anticipate that concentrations of these contaminants could exceed MCLs in the field. Once the washout process ended, concentrations of all regulated chemicals were well below their MCLs. Thus, at this site, we conclude that the natural soil contaminants will not pose a problem over the long term.

Physical characteristics of soils. Recharge of water may alter the physical nature of soils in ways that may be undesirable. Because the structure of soils in arid lands sometimes depends upon evapotranspiration “cements”, the dissolution of the cementing material could lead to subsidence, with concomitant loss of infiltration capacity. The dissolution process would certainly be most pronounced for the RO water, which was very aggressive in dissolving carbonates. Conversely, precipitation of minerals could also lead to clogging and reduced infiltration capacity. MF water, for example, was oversaturated with respect
to several calcium minerals that could precipitate in the vadose zone. Ion exchange in the MF columns also lead to oversaturation of magnesite, which could precipitate deeper in the vadose zone. Finally, salinization of soils (exchange of bound calcium and magnesium with sodium from recharge water) could occur during recharge, but we did not see any evidence of this in our experiments.

Conclusions
1. A “washout” phenomenon resulted in peak concentrations very early in the column experiments. This is probably due to rapid dissolution of amorphous and/or poorly crystallized evaporite minerals. During this period, which lasted a few days, the concentrations of several regulated contaminants approached their MCLs.
2. By the end of the column experiments, concentrations of all regulated chemicals were well below their MCLs.

We postulate that geochemical processes may alter soil structure and thereby change infiltration rates. We postulate that dissolution of soil cements may lead to subsidence and loss of infiltration capacity, particularly for aggressive waters, such as the RO water used in our experiments. Conversely, precipitation of soil minerals could lead to clogging. Longer-term, in situ monitoring and analysis would be required to test these hypotheses.

References


Figure 1. Cross section of the Water Campus Project. Location of the groundwater surface, pediment surface, and McDowell Mountains are shown.

Figure 2. Schematic of soil column experiments. Experimental treatments included soil type (six vadose zone soils) and inflow water (three types).
Figure 3. Fluoride concentrations in the outflow of columns operated with CAP water. Fluoride in the inflow CAP water was 0.45 mg/l. A "washout" behavior similar to this was noted in all columns.

Figure 4. Alkalinity of all three waters after passing through soil columns. Alkalinity of all six columns were averaged for each water type.
ELECTRONIC PERMITTING

Greg Bushner, Christine Close, and Drew Swieczkowski

ABSTRACT

With the emphasis today being placed on E-mail addresses and Internet access, the move to electronic permitting seems a natural progression. Electronic permitting transforms the normal paper report into an electronic document which reduces preparation costs and post-review storage requirements. The electronic permit can be placed on the reviewing agency’s local network for easy access during the review process. Technical information, such as, groundwater flow model executable files, databases, and input/output files can be made available for review in their entirety which usually is not possible through standard report documentation. The electronic document can be printed and annotated, but cannot be edited in its distributed format. Electronic permitting can increase the communication link between the reviewing agency and the consultant, reduce costs and labor, and enhance the traditional review process.

INTRODUCTION

HydroSystems, Inc. (HSI), along with the Arizona Department of Water Resources (ADWR) and the Arizona Department of Environmental Quality (ADEQ), initiated a pilot program in 1996 to investigate the feasibility of using electronic documentation to file and review permit applications for underground water storage projects. The joint effort enabled the development of specific criteria needed by the reviewers and the applicant. The pilot program consists of providing electronic copy of all permit application documentation and the necessary software to review the documentation electronically in addition to submittal of the hard copy permit application documentation. Attached to the electronic copy is annotation software which allows the reviewer to view the document in its entirety, print, and to electronically comment on areas in question. It

is anticipated that electronic permitting will increase communications, reduce costs, and facilitate the technical review process.

DEVELOPMENT OF THE ELECTRONIC PERMIT APPLICATION

The electronic permit application is developed in the same manner as a hard copy document including all reporting, figures, and tables associated with the filing of an underground water storage permit. Attached to the electronic permit application is a viewer which allows the reviewer to see the document regardless of the software they have on their computer system. The electronic document can be further enhanced by the applicant with the installation of hyperlinks which are a feature of the viewer. The hyperlinks allow the reviewer to jump from one area in the document to another with the click of an icon. For example, each section in the table of contents can have a hyperlink added which allows the reviewer to jump from the table of contents to the beginning of that section.

The electronic permit application is then saved onto a writeable CD which is able to store as much as 660 megabytes of data. This allows the applicant to submit additional reference material to the ADWR such as databases, groundwater flow models, and input/output files which normally is not possible with standard reporting techniques. The additional reference material enables the reviewer to actively analyze modeling data including executable files without changing the original files, or have an electronic copy of the original input files available to change without inputting all of the hard copy data for use with a groundwater flow model.

The writeable CD is generally referred to as write-once, read-many (WORM) media. This means that the original document can be read many times, but only written to the CD once. This provides the applicant with protection against the original permit document from being modified. However, the report or data stored on the original CD can be downloaded, imported into the appropriate program, and then changed. Meaning this is not fool proof, but the original file stored on the CD cannot be over-written. For identification and storage purposes, a serial number can be etched onto the original CD where the data are stored.

The Envoy™ 1.0 viewer was used for the pilot program. Envoy is included in the WordPerfect® Suite which is now owned by Corel®. The Envoy viewer can be distributed without restriction. The Envoy viewer includes several annotation options that can be used by the reviewer including highlighting, bookmarks, and inserting sticky notes. The electronic document can be viewed, searched, navigated, annotated, and printed.
One electronic copy (CD) and one hard copy of the electronic permit application is submitted to the ADWR. Traditional permit application submittal consists of five hard copies of each application document. The electronic permit application is then downloaded and stored onto their local network for access during the review process.

Each reviewer can customize their comments using the features available in Envoy. The bookmark option allows direction to specific items that require another’s attention. The annotations made to the electronic permit can be imported into a word processing program for development into a permit response letter. When all aspects of the review are completed, the annotated file is written to the CD under a new file name and returned to the applicant.

**BENEFITS TO ADWR**

There are many benefits to the ADWR from the electronic permit application process. Some of these benefits are measured by the increased amount of interaction and communication that occurs between the ADWR, the consultant, and the applicant as a result of submitting an application electronically. The relationship becomes more partnership or team oriented in the review process. Some benefits can be measured by the reduction in review time and storage space required. Other benefits may not be apparent at this time. The following are just a few of the benefits to the ADWR:

1. The use of CD technology to submit permitting documentation enhances the review process by providing the ADWR with additional technical data for their use. For instance, the additional data should provide insight into how the modeling of impacts was approached. If desired, the agency could change modeling scenarios to check on the impacts associated with different model parameters. This is a time saving addition for the reviewer, allowing them direct access without having to wait for changes in modeling output from an outside source.

2. For some projects, the amount of data presented in hard copy can be substantial. Using a CD to store the permitting documentation will decrease the amount of storage space required when the review is complete. Large drawing files or blueprints can be included on the CD thereby decreasing the need for storage space.

3. The communication and cooperation between the ADWR and the applicant should increase as electronic permitting progresses. Other users of electronic permitting have noted reductions in manpower needs for the review while their workload has increased.

4. The ADWR can develop a response to the permit application by importing the comments generated in Envoy into WordPerfect.
BENEFITS TO THE APPLICANT

There are several benefits to the applicant including increased communications, time saving, and cost reduction.

1. The communication and cooperation between the applicant and the reviewing agency increase as the applicant develops provisions to make this process work. It is in the applicants interest to have a process which is user friendly, cost effective, and time efficient. By jointly developing the electronic permitting process, the applicant can be assured that the reviewing agency has all of the tools needed to complete the electronic review.

2. The applicant can quickly and easily provide additional data to the reviewing agency which should expedite the review. This saves time and money by allowing the reviewer access to model parameters and databases used in the permit development. It also increases communications and cooperation between the applicant and reviewing agency. The reviewing agency now has access to everything the applicant used to develop the permit documents. If any questions arise, both parties will have a clear picture of how to address the issue(s).

3. The applicant benefits from using electronic permitting by reducing costs and labor associated with the development of hardcopy documents. It is estimated that a single hardcopy permit application can cost a minimum of $100.00 to copy, format and bind, and generally five to ten copies are made. This is a labor intensive process. The electronic permit application costs approximately $30.00 to prepare and only one copy is needed for distribution. The applicant saves approximately 70% of the cost of one hard copy by using an electronic format.

CHALLENGES TO MEET

Several challenges remain for electronic permitting including the development of electronic applications, ability to include other forms of data electronically, and manipulation of models which incorporate AutoCAD or other computer aided drawing programs. The applicant will continue to address the issue of security while the agency will have to show a “leap of faith” when accepting the electronic document’s validity in comparison to the hard copy. All of these issues can be resolved with the reviewing agency and applicant working jointly to develop solutions that benefit both groups.
OTHER ELECTRONIC PERMITTING APPLICATIONS

During the discussions concerning the use of an electronic format for permit review, several other uses for electronic applications were developed. The ideas ranged from other forms of electronic submittals to use of the Internet for access and review. Several of the ideas follow:

1. Developing underground storage facility and water storage permit application forms in an electronic format. These forms could be downloaded by the applicant from the agency’s web site.

2. Develop Intent to Drill and other permit applications in an electronic format for downloading by the applicant from the agency’s web site.

3. Applying electronic permitting to the development and submittal of an Aquifer Protection Permit application by working with the ADEQ to develop the process necessary to meet their requirements.

4. Submitting annual and quarterly reports to the ADWR and ADEQ in an electronic form for review and storage.

5. Using the CD technology to submit and store blue prints and other large scale drawings.

6. The reviewing agency and applicants should take advantage of the Internet and the World Wide Web. The reviewing agency could use their web site to post the electronic permit application for public review. This would be beneficial to outlying areas that are impacted by an application but do not have the resources to physically view the permit application. These individuals would be able to review the application by accessing the agency’s web site using a community services computer system.

CONCLUSIONS

The use of all types of electronic applications to complete daily tasks has taken hold in the United States and around the world. As these processes increase in speed and efficiency, they will become a more prevalent part of every day life. Communications between offices will use e-mail and Internet access. Electronic permitting has a place in this advanced technological network.
The reviewing agency and applicant will be able to save time and reduce costs by progressing to this electronic format. The CD technology allows for a large transfer of information in a small amount of space providing efficient storage after the completion of the review process. The viewer software attached to the electronic permit application allows the reviewer to comment within the text and develop the response letter by incorporating the comments into a separate document. Although the pilot program is not yet complete, the electronic permit has vast possibilities and advantages over the non-electronic permit format.
EQUUS BEDS GROUNDWATER RECHARGE DEMONSTRATION PROJECT, SOUTH-CENTRAL KANSAS

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Introduction
The Equus Beds Groundwater Recharge Demonstration Project is a phased, small-scale $7 million trial project being used to test the feasibility of a full-scale $106 million groundwater recharge, storage and recovery project. The full-scale project is a key part of an Integrated Water Supply Plan adopted by the City of Wichita that will provide additional water supply to the City and surrounding communities to meet growing water needs through the year 2050. The general project location is shown in Figure 1.

[Diagram showing the location of the project]

Figure 1 Site Location

The recharge project will capture above-base flow water in the Little Arkansas River, transfer and store captured water in the aquifer at a location between the Arkansas and the Little Arkansas Rivers. The stored water can then be recovered to meet City demands in times of drought. Above-base flow water is defined as water which is generated from rainfall runoff above the base flow as established by the State of Kansas. The project will benefit all users of the Equus Beds as follows:

- By adding up to 104 billion gallons (or 319,000 acre-feet) of water to aquifer storage for use to meet City demands.
- By reducing power costs for pumping because of higher groundwater levels
- By protecting the aquifer from water quality deterioration from intrusion of natural and man-made sources of salt water.

Water Supply Plan
The City now uses two principal raw water sources, the Equus Beds well field and Cheney Reservoir. A schematic of the City’s current raw water supply system is shown in Figure 2. An Integrated Water Supply Plan was adopted in August 1993 by the City
Figure 2
RAW WATER SUPPLY SYSTEM SCHEMATIC
which optimizes the use of local water resources and includes the following additional components:

- Water conservation
- Redevelopment of Bentley-Reserve well field
- Greater use of Cheney Reservoir flood storage and spillage
- Use of the Little Arkansas River
- Recharge of Equus Beds well field
- Purchase of groundwater irrigation rights in over appropriated areas

Equus Beds Aquifer

The Equus Beds Aquifer forms the eastern most portion of the High Plains Aquifer System and underlies portions of Sedgwick, Harvey, McPherson and Reno Counties, Kansas. The aquifer, named for the equine fossils found in its unconsolidated deposits, is about 900,000 acres in size and has an average annual withdrawal of 157,000 acre-feet of water. Depth to groundwater ranges from 10 to 110 feet and the saturated thickness ranges from less than 50 to over 300 feet.

Water rights and pumpage currently exceeds the aquifer’s natural recharge rate of about 3.2 inches per year. Since the 1950s, water levels in the aquifer have dropped 20 to 40 feet due to heavy pumpage throughout the area. The altered gradient is resulting in migration of high chloride water from the Arkansas River to the southeast and from leaking oil field brine disposal areas from the northwest toward the City’s well field.

Water from the City’s Equus Beds well field is of generally good quality for municipal water supply and irrigation needs. Groundwater modeling of the brine migration by the Bureau of Reclamation indicates that the average chloride concentration in the well field, an indicator of salinity, will increase from the existing 60 milligrams/liter (mg/L) to about 95 mg/L in year 2010 and 145 mg/L in year 2050. Maximum chloride levels would exceed 300 mg/L in some areas, well above the sensitivity level of 200 mg/L for agricultural uses and 250 mg/L for municipal uses.

In 1975 the Equus Beds Groundwater Management District No. 2 (GMD2) was formed to manage groundwater supplies within its boundaries. The GMD2 established two fundamental management principles: 1) Aquifer Safe-yield Principle which limits groundwater withdrawals to annual groundwater recharge; and 2) Groundwater Quality Principle which seeks to maintain by protection and remediation the naturally occurring water quality of the aquifer. The management program consists of three major components: 1) Information and Education; 2) Data Collection and Research; and 3) Administration and Regulation.
The groundwater recharge project is consistent with the GMD2’s water conservation and recycling policies, and has the support of the GMD2 Board of Directors. The project will not impair the Little Arkansas River or Equus Beds Aquifer and will be constructed and operated within state and federal water quality standards.

**Groundwater Recharge Demonstration Project**

The proposed groundwater recharge demonstration project includes four basic phases or stages of investigations and pilot testing to determine the feasibility of the plan and to obtain state approvals. The four basic parts are:

- **A Feasibility Study** to review existing geologic, hydrogeologic, water quality, water rights, and environmental data. Completed in 1994, this study confirmed the feasibility of the project based on available information.

- **A Hydrogeological and Engineering Study** which included collection of additional geology, hydrogeology, water quality, environmental, and cultural resource data. Work completed included construction and testing of one 1,000 gallon per minute (GPM) well near the Little Arkansas River to recover river water by induced infiltration, large scale infiltration tests at five sites, installation of monitoring wells and piezometers, and river stage gages; and additional computer modeling. This study was completed in late 1996.

- **Design and construction of pilot demonstration facilities.** This work includes one 1,000 gpm surface water intake, two presedimentation basins, powered activated carbon and chlorination facilities, two pipelines, infiltration basins, recharge well, recharge trench, and associated telemetry and controls. Construction of these facilities began in January, 1997.

- **Operation of the pilot demonstration facilities for a 2 to 3 year test period.** The pilot operation is designed to confirm full-scale project feasibility, collect operating data to establish groundwater recharge, storage and recovery facility design data, and collect data for GMD2, state and federal agency approvals and permits.

The following activities have occurred during first two phases of the Equus Beds Groundwater Recharge Demonstration Project.

**Facility Site Selection:** Location of facility sites were selected using original data collected and additional geologic, stream flow, and water quality data collected during the hydrogeological investigation. The availability of land and the distance from the Little Arkansas River to potential recharge sites were also important selection criteria.
After a detailed evaluation, a site located northwest of Halstead, Kansas was selected as the location for a full-size test well and piezometers. This test well was used to evaluate aquifer-river flow and water quality interactions. The test well will be used to induce above-base flows from the Little Arkansas River during above-base flow events to provide recharge water for the operation of the demonstration project.

A surface water intake will also be installed about 20 miles downstream near the USGS gage at Sedgwick, Kansas. This area was selected because of higher stream flows and more frequent above-base flow events, a higher mean concentration of atrazine to more accurately evaluate potential water treatment schemes, potential aquifer contamination, and a short distance to several City wells that exhibit potentially high recharge rates.

Environmental Assessment: Numerous activities were conducted during 1995 and 1996 to complete environmental studies for the Project. These activities included preparation of an environmental assessment (EA), environmental clearances for soil borings, reconnaissance for various testing, monitoring, and construction activities, fishery surveys and instream habitat analyses, and public hearings.

The EA was prepared to fulfill the requirements of the National Environmental Policy Act of 1969 and satisfy the requirements of the Bureau of Reclamation (Reclamation) for their participation in the demonstration project. A “Finding of No Significant Impact” was ultimately determined to be appropriate and was approved by Reclamation on September 11, 1995.

Plans for fishery surveys and stream habitat analyses were coordinated with the Kansas Department of Wildlife and Parks (KDWP). Sample locations were selected and landowner permission was obtained with the assistance of the GMD2 and the KDWP. Twenty-one fish species were collected and identified during the surveys, with the most common fish being the red shiner and the most common game fish being the channel catfish.

Geology and Hydrogeology: Extensive subsurface drilling and testing were conducted during the hydrogeological investigation phase of the demonstration project. This effort included installation of 64 test holes, monitoring wells, pilot holes, and piezometers. This testing was conducted to site additional project facilities (monitoring well strings, infiltration test pits, and the 1,000-gpm test well and piezometers). The drilling also provided additional geologic data needed to refine the regional geologic setting and illustrated the complex nonhomogeneous nature of the alluvium or unconsolidated materials, even on a local scale.

Infiltration Testing: Drilling conducted at the City wells for test infiltration basin siting showed that soils have a variable fine-grained topsoil thickness that will affect design and
construction of the recharge basins. The fine-grained topsoil layer range from 6 to 12 feet thick and is typically thicker at the northern part of the well field and thinner in the middle and southern well locations. Infiltrometers were constructed of corrugated aluminum plates to form a 11.5 foot-diameter cylinder that extended through the upper fine grained material to underlying fine to medium sand. Groundwater from nearby City wells was used to supply water for the infiltration tests. Tests were run for periods of 56 to 100 days with drying periods. Three of the four sites showed extremely high infiltration rates, with initial rates ranging from 60 to 80 feet per day. Long-term rates ranged from 6 to 12 feet per day. Based on this testing locations near City Well Nos. 4 and 36 will be used for recharge sites during the demonstration project.

Aquifer Testing: Aquifer testing was conducted to demonstrate the connection between the Little Arkansas River and the aquifer, and that water quality was acceptable for recharge through surface infiltration basins and recharge wells. During the process for the water rights application several local concerns were expressed. The primary concerns included:

- Lack of hydraulic connection between upper and lower aquifer zones, and the prevention of deep water groundwater use by the project.
- Impairment of existing water rights and contamination of the aquifer.

To address these concerns the scope of the aquifer test was increased. The study was expanded to include installation of additional shallow and deep piezometers, and increased water quality monitoring with additional data evaluations and groundwater modeling.

Three pumping tests were performed on the test well. A 24-hour well acceptance test was conducted to determine well efficiency and to determine the optimum pumping rate for a 30-day test. The 30-day test was conducted to evaluate aquifer performance and river-aquifer interaction, including changes in groundwater quality during long-term pumping periods. The 75-day aquifer test was conducted to evaluate the impacts of atrazine and other water quality concerns for an extended pumping scenario. Water level data developed in these tests was used to calibrate the subregional groundwater model.

The 24-hour test showed that the test well had a well efficiency of about 100 percent, and that the aquifer quickly recovered from the pumping stress. Hydrographs developed during the 30-day pump test revealed an immediate response to pumping in both the shallow and deep piezometers. This indicated good hydraulic connection between the aquifer zones, thus meeting the provisions in the term permit. Results from the 30-day pump test were also used to calibrate the subregional groundwater model. Model results confirmed that water pumped from the deep aquifer zone was replaced by water from the upper zone and the river within a short distance from the pumping well.
Hydrographs obtained from the 75-day pump test revealed that water levels in shallow and deep piezometers were directly affected by increases in river stage, further proving a good connection between the river and shallow and deep portions of the aquifer. Water quality data further proved the river-aquifer interaction, as chloride and specific conductance concentrations in the test well tracked the concentrations in the surface water. Based on water quality samples taken during this time, atrazine in the test well was detected 2 out of 105 days at level slightly above the detection limit. As a result, atrazine is not considered to be a major threat, but will continue to be closely monitored.

**Water Quality:** A total of 1,258 water quality samples were collected by the U.S. Geological Survey (USGS), and analyzed by the USGS and the City’s laboratory. Results indicated that 31 parameters were detected in either the Little Arkansas River surface water or the groundwater. Review of the data revealed these parameters to have concentrations slightly above the detection limit and less than 20 percent of U.S. Environmental Protection Agency’s Maximum Contaminant Level (MCL). The only exceptions were the higher readings for atrazine and cyanazine.

**Demonstration Project Facility Concepts**
Using the results of the study, concept layouts for the demonstration project recharge facilities were developed and plans and specifications prepared. Each existing City well is located on a 5-acre plot and the demonstration facilities will be constructed on land surrounding two City wells. A schematic of the groundwater recharge demonstration system is shown in Figure 3.

The northern site, known as the Halstead Recharge System, is located at City Well No. 4 and will use water obtained from induced infiltration from the test well near the Little Arkansas River during periods of above-base flow. The diversion and delivery facilities include the 1,000 gpm test well, a three-mile, 12-inch pipeline. Recharge facilities for the Halstead system includes an 18-inch recharge well, a 100-foot long by 3-foot wide recharge trench, and two approximately 1/2-acre recharge basins. The basins are excavated through the surface clay with the basin floor in fine sand. The basins have ramps to facilitate periodic scrapping and cleaning. The facilities will also include a control building with valving and metering, and a SCADA/control system with communication to the water plant control room. The system will operate automatically, when flows exceed 42 cfs as measured from a near by USGS gaging station. The USGS data collection platform (DCP) will be programed to send a signal by telephone line to start and stop the pumps at the appropriate river flow levels.

The Sedgwick Recharge System intake and delivery facilities includes a 1,000 gpm surface water intake, a package preselementation unit with chemical feed to remove turbidity, chlorine and powered activated carbon facilities and a three-mile, 12-inch
Figure 3
GROUNDWATER RECHARGE DEMONSTRATION SYSTEM SCHEMATIC

LEGEND

- USGS GAGE
- MONITORING WELL
- PIEZOMETERS
- PUMPING WELL
- EARTHEEN RECHARGE BASIN
- RECHARGE WELL
- RECHARGE TRENCH
- SURFACE WATER INTAKE
- 12" PIPELINE

Burns & McDonnell
pipeline. The recharge site around City Well No. 36 will include an earthen pre-sedimentation basin and three approximately 1/2 acre infiltration basins. This system will also have a control building, valving, metering and a SCADA/control system with communication to the water plant control room. Operation will also be automatic with pumps starting on a signal for a USGS gaging station located near Sedgwick, Kansas when river flow is greater than 40 cfs.

In addition to flow rates and time, basin levels, the trench and recharge well levels, adjacent groundwater levels, and basic water quality parameters will be continuously recorded by the SCADA system. The recharge well is equipped with a 1,500 gpm pump for redevelopment and both systems have chlorine units to control biofouling as needed.

Conclusion:
Three years of preliminary data evaluations and detailed hydrogeological studies has indicated the proposed groundwater recharge project is a feasible method of providing for the long term water supply need for the City of Wichita, Kansas. Additionally, the project will help protect the aquifer water quality. To confirm the feasibility of the full-scale concept, a small demonstration project is currently under construction with the first system coming on line in May, 1997. A two to three year pilot operation program will develop data for final design, and to provide the basis for state and federal permits required for the full-scale project.

The primary demonstration project sponsors are the City of Wichita, and the Bureau of Reclamation. Additional participants are the U.S. Geological Survey and the Equus Beds Groundwater Management District. Work is closely coordinated with the Kansas Department of Health and Environment, the Board of Agriculture, Division of Water Resources, and the U.S. EPA. Burns & McDonnell Engineering Company serves as the project manager for the groundwater recharge demonstration project.

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EVALUATION OF NITROGEN REMOVAL RATES IN THE WETLANDS TREATMENT SYSTEM OF KINGMAN, ARIZONA

Sara Gerke, Lawrence Baker, and Don Manthe

Constructed wetlands have been widely used to treat wastewater, and particularly to "polish" low quality effluent. Wetlands are also being used as a component of effluent reuse projects that recharge municipal effluent to aquifers. Thompson et al. (1994) outlined a plan to recharge nitrate-contaminated canal water. Ingersoll and Baker (1997) developed nitrate removal coefficients for wetland microcosms and proposed that wetlands could be used to treat nitrate-contaminated groundwater. In all of these applications, the capacity of constructed wetlands to remove nitrogen from effluent inexpensively is a key consideration (Horne, 1995). For 84 constructed wetlands designed to treat municipal effluent, average nitrogen removal was 64% (Knight et al., 1992), although removal rates > 90% can be accomplished under some conditions (Ingersoll and Baker, 1997). Both nitrification and denitrification occur in wetlands. Nitrification is supported by natural reaeration at the air-water interface and by diffusion of oxygen into the root zones of plants. Denitrification occurs in the anaerobic plant mat and sediments and is supported by abundant supply of carbon from decaying plants, with an optimal ratio of plant carbon: influent N of 5:1 (Ingersoll and Baker, 1997). These factors make wetlands treatment systems particularly well suited for the reduction of nitrogen contamination.

Despite the widespread application of constructed wetlands to remove nitrogen from various effluents, there is limited information in the literature on the specifics of nitrification and denitrification reactions in wetlands. Furthermore, our ability to model nitrogen removal is limited. In summarizing nitrogen removal rates, Kadlec and Knight (1996) computed an average nitrogen removal coefficient of 15. 3 m/yr, but \( k_N \) varied from < 1 m/yr to > 60 m/yr for individual systems. This uncertainty in N removal rates often dictates the development of pilot wetlands prior to construction of full-scale systems. This is expensive and adds several years to the development of the full-scale systems.

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In this paper we examine the sequence of nitrogen transformations in a treatment wetland throughout the year and evaluate the use of a simple sequential model of nitrogen transformations to predict overall nitrogen removal.

Background

Our study site is a wetland treatment system in Kingman, Arizona. The wetland was designed to treat effluent from an aerated lagoon system to a quality that could be met aquifer protection requirements for disposed by recharge. The aerated lagoon produces effluent with 15-20 mg N/L, present primarily as ammonia and particulate organic nitrogen (PON). The availability of inexpensive vacant land, the suitability of wetlands for N removal, and warm climate favored the use of constructed wetlands treatment over a conventional waste water treatment system.

The wetland treatment system consists of three separate treatment cells connected in series. Each cell is approximately 2150 feet long by 150 feet wide and less than 1 foot deep and densely planted with vegetation (predominately bulrush). Each cell contains three deep zones (3 feet depth) which were left unplanted. (Manthe et al., 1995) These provide for mixing of the water and improve the hydraulic characteristics of the wetland system. In Figure 1, the shaded areas represents densely packed vegetation while the unshaded area represent the deep, unplanted zones. An elevation drop of 5 feet between each cell allows for reaeration of water. The wetland was constructed with an impermeable lower liner to prevent the infiltration of partially treated wastewater into the aquifer beneath the wetland. The hydraulics of the wetland are essential to effective operation, sampling, and modeling of the system.

The wetland has been sampled monthly since October 1996 at 13 locations along its longitudinal axis (Figure 1). The sampling plan concentrates the collection effort on the first third of the system where the most rapid changes are likely to be occurring. Samples collected from the edge at each site are stored in a cooler for approximately six hours prior to filtration and preservation in the Environmental Engineering Lab at Arizona State University. Samples are analyzed for all major nitrogen species; ammonia, particulate organic nitrogen (PON), soluble organic nitrogen (SON), nitrite, and nitrate. To further characterize wetland processes, chloride, bromide, phosphate, biochemical oxygen demand (BOD), and dissolved organic carbon (DOC) are being analyzed. Temperature, dissolved oxygen, and pH have been measured in the field since December, 1996.

Results

The sequence of nitrogen transformations occurring in the wetland is illustrated by data collected in October 1996 (Figures 2). At this time, plants were approximately eight feet tall and had not started to senesce. Nitrogen in the effluent from the aerated lagoons was primarily ammonia and PON; the PON was presumably algae. PON declines rapidly in the first cell as algae cells are retained by sedimentation and filtration. Death and lysis of algae cells released ammonia to the system, as reflected by a small increase in ammonia concentrations in the first cell (Figure 2). Ammonia levels slowly decline in cells 2 and 3.
as the result of nitrification and plant uptake. Nitrate peaks in cell 3, but at a level of only approximately 3 mg/L, presumably because denitrification quickly removes nitrate formed by denitrification. The effluent leaving the wetland contains almost no PON or ammonia and < 2 mg/L NO₃-N/L.

The pattern of nitrogen transformations changes from the late summer to winter. Throughout this period, PON removal is still nearly complete in cell 1 (Figure 3). Ammonia concentrations decline throughout the wetland throughout October, November, and December 1996 (Figure 4). The ammonia concentrations in the lagoon effluent were about twice as high in November and December compared with October. Ammonia concentrations in the effluent were (all as N) 1 mg/L, 3 mg/L, and 7 mg/L, respectively, over the three sampling periods, corresponding to net ammonia removal rates of 90, 80 and 60 percent. The lower ammonia removal in December probably reflects two factors: (a) lower microbial reaction rates, reflecting a drop in temperature, and (b) release of ammonia by wetland plants, which were in a stage of advanced decomposition by December. Longitudinal profiles of nitrate (Figure 5) show that nitrate was generally low during October and November, with effluent levels < 2 mg NO₃-N/L. In December nitrate increased significantly by the end of cell 2 and reached a maximum of 24 mg NO₃-N/L in cell 3. Effluent from the wetland contained 18 mg NO₃-N/L. The elevated nitrate concentrations could have occurred as the result of nitrification of ammonia released during plant decomposition or as a result of slower denitrification due to lower temperature.

Overall removal efficiencies for nitrogen species analyzed to date (not including SON) were 87% in October, 75% in November, and -12 % in December. The negative retention of nitrogen in December indicates that the wetlands were a net source of nitrogen to the effluent.

**Preliminary Model**

As a first step in modeling nitrogen removal in wetlands we evaluated a classical sequential model of nitrogen transformations:

\[
\frac{d([\text{PON}])}{dt} = -k_{\text{PON}} [\text{PON}] \tag{1}
\]

\[
\frac{d([\text{NH}_4^+])}{dt} = k_{\text{PON}} [\text{PON}] - k_{\text{NO}_3} [\text{NH}_4^+] \tag{2}
\]

\[
\frac{d([\text{NO}_3^-])}{dt} = k_{\text{NO}_3} [\text{NH}_4^+] - k_{\text{N}_2} [\text{NO}_3^-] \tag{3}
\]

The terms \( k_{\text{PON}} \), \( k_{\text{NO}_3} \), and \( k_{\text{N}_2} \) are first order rate coefficients for ammonification, nitrification, and denitrification reactions.

Our attempt to fit system-wide values of \( k_{\text{PON}} \), \( k_{\text{NO}_3} \), and \( k_{\text{N}_2} \) to measured nitrogen species along the longitudinal axis of the wetland met with little success, as illustrated in Figure 6. The poor fit between the measured and predicted values indicates that a simple
sequential model is not adequate to predict nitrogen concentrations. Attempts to find system-wide coefficients for the November and December data sets also failed.

To determine why this approach failed we calibrated the model for each of the three cells separately. This exercise revealed that $k_{\text{PON}}$ was reasonably constant among cells for a given date, but $k_{\text{NO}_3}$ and $k_{\text{N}_2}$ varied among cells. $k_{\text{N}_3}$ tended to increase from cell 1 to cell 3 during all three months analyzed to date, suggesting that conditions for nitrification improve along the axis of the wetland. We postulate that this may be due to oxygen limitation in the first part of the wetland and are now looking for relationships between oxygen concentrations and nitrification rates throughout the wetland. The denitrification rate constant was consistent among cells for a given date but decreased and actually became negative during December. Nitrogen dynamics probably also depend upon the physiological condition of wetland plants. We postulate decomposition of plant material early in the winter produces ammonia, and subsequently, nitrate. The production of nitrate would yield an apparent negative denitrification rate constant. This hypothesis is supported by the fact that total nitrogen concentrations in cell 3 were higher than input total nitrogen during December. To correct for this problem, we are attempting to incorporate plant growth and senescence into the model.

Table 1. "Best fit" calibration for nitrogen transformation coefficients in the Kingman wetland, October through December, 1996.

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<td>0.3</td>
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<td>0.3</td>
<td>0.3</td>
</tr>
<tr>
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<tr>
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Conclusions

The sequence of nitrogen transformations occurs in the Kingman wetland resulted in high nitrogen removal efficiency and low effluent nitrate concentrations in October and November. In December, nitrogen removal efficiencies were negative, probably because decomposing plants in the wetland contributed nitrogen to the water. In our first attempt to calibrate a sequential nitrogen transformation model we were unable to find system-wide rate constants that yielded reasonable predictions of observed nitrogen species along the longitudinal axis of the wetland. We hypothesize that the nitrification rate constant may vary with oxygen concentrations or some surrogate of oxygen demand. The dynamics of ammonia and nitrate probably also depend upon the physiological state of wetland plants. Further modeling efforts will focus on this issues.
References Cited


Figures:

Figure 1: Kingman Wetland Cells and Sample Locations

Figure 2: October Nitrogen Concentration Profile
Figure 3: Particulate Organic Nitrogen Concentration

Figure 4: Ammonia Nitrogen Concentration Profile
Figure 5: Nitrate - Nitrogen Concentration Profile

Figure 6: Model of Nitrogen Transformation Profile October 1996 using a single rate coefficient
Figure 7: Model of Nitrogen Transformation Profile October 1996 using multiple rate coefficients

- Data PON
- Data Ammonia
- Data Nitrate
- Model PON
- Model Ammonia
- Model Nitrate
GEOCHEMICAL INVESTIGATIONS TO EVALUATE COMPATIBILITY BETWEEN RECHARGE SOURCES, GROUNDWATER AQUIFERS, and VADOSE MATERIAL DURING RECHARGE FEASIBILITY INVESTIGATIONS PIMA COUNTY, ARIZONA

Erick Weiland

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ABSTRACT

This paper presents the findings of an investigation conducted to evaluate the geochemical compatibility between recharge water, groundwater aquifers and vadose materials for the proposed surface basin recharge site in lower Santa Cruz River basin near Marana within the Northwest Replenishment Program (NRP) in Pima County. Long-term compatibility of potential recharge water sources (Central Arizona Project and Treated Effluent) and the quality of recharged water were evaluated in detail. Geochemical processes and reactions that can potentially reduce infiltration rates and degrade the quality of the groundwater due to surface spreading recharge which were evaluated include: recharge water and groundwater reactions, ion exchange, adsorption, and mineral dissolution and/or precipitation. This paper presents the results for the Santa Cruz site. Due to publishing page limitations, results of column tests for the Canada del Oro site are not included here but can be found in AGRA (1996).

Twelve column tests were conducted over a 12 week period to analyze the physical and chemical changes in vadose zone materials (recent alluvium and Fort Lowell Formation) and recharge water. Water samples were collected for chemical analysis upon breakthrough of seepage and every two weeks thereafter. Field parameters (pH, T°C, EC, DO, and ORP) were measured along with the rate of inflow of feed water and rate of outflow of seepage water on a daily basis.

Seepage breakthrough occurred by day 2 in most columns, with maximum seepage occurring by day 15 to 20, declining to much lower rates by day 40 to 50. Recent alluvium had lower infiltration rates (0.05 to 0.25 fpd) than the Fort Lowell Formation (0.2 to 1.8 fpd). It is likely that infiltration rate decreases due to swelling clays would play a significant factor in long-term recharge.

Applying CAP water to surface recharge basins would most likely increase the TDS, hardness, calcium, silica, chloride, fluoride and sulfate concentrations in the aquifer. Applying treated effluent would most likely result in additional increases in sodium and silica concentrations within the aquifer and nitrate concentrations may rise above the AWQS.

Seepage water samples are at or above saturation with calcite, montmorillonite, illite and kaolinite and their precipitation may decrease infiltration rates. Anhydrite, chloride, fluorite, gypsum, siderite, and silica cement are generally below saturation concentrations, and their dissolution may increase infiltration rates and increase TDS concentrations. A breakdown of existing clays and formation of the new clays may contribute to a decrease in infiltration rates.
Column tests provided substantial information on which to base preliminary decisions and to define critical issues for field evaluations. Laboratory data and geochemical modeling serve to identify possible outcomes. Overall site stratigraphy and operational parameters have a significant influence on long term infiltration potential.

COLUMN TESTS

Vadose zone samples for the column tests were collected by Montgomery & Associates (M&A) personnel during vadose zone investigations and were delivered to METCON Research, Inc. For a description of the sample acquisition additional information on borehole locations and lithologic descriptions for the sampled intervals the reader is referred to the Task 3 report (M&A, 1996b). Samples were composited at METCON to obtain single, representative samples for each of two principal geologic units, recent alluvium and the Fort Lowell Formation, from the project site.

Potential recharge water sources used in the testing were delivered by Metropolitan Domestic Water Improvement District and the City of Marana. METRO Water supplied CAP water pumped directly from the aqueduct near Avra Valley Road. Marana supplied treated effluent from the Santa Cruz River downstream of the Ina Road Treatment Facility.

Twelve column tests were prepared by METCON (1996). Column length was set at 10 feet of material plus 1 foot of open space at the top. This allowed for optimizing contact time between recharge water and material, while keeping the columns indoors minimizing potential contamination. Columns were designed to maintain a 6-inch head of feed water during the duration of the tests. Column diameters were 3 inches for the recent alluvium material and 4 inches for the Fort Lowell Formation material. Material was prepared and packed to approximate field conditions. Every attempt was made to blend the entire sample (all buckets of the same geological unit) so that the sample used in the column was representative of the size fraction distribution and mineralogy of the natural material. The columns were capped to reduce light availability and minimize biological activity.

Six of the columns used Recent Alluvium and six used Fort Lowell Formation material. The tests were further divided into two sets, one for Central Arizona Project water and one using treated effluent. Four of the column tests were performed under a closed cycle regime. All remaining columns were run under an open cycle, with fresh feed applied and the seepage discarded after measurement and analysis.
The column tests were run for 12 weeks with feed water applied to all columns at 9:00 am on January 2, 1996. The initial breakthrough of seepage occurred within a day for all Fort Lowell Formation test columns. Recent alluvium tests took the longest to achieve breakthrough, up to 6 days. During the column tests, field parameters (pH, electrical conductivity, dissolved oxygen, and oxidation/reduction potential) were measured along with the rate of inflow of feed water and rate of outflow of seepage water on a daily basis. Upon breakthrough of seepage from the columns, samples were collected for chemical analysis at a certified laboratory (American Environmental Network of Arizona, formerly Analytical Technologies Inc.) for inorganic constituents. Additional samples for chemical analysis by the laboratory were collected every two weeks, except for the 10th week when sampling was skipped due to budget constraints.

ANALYTICAL CHEMISTRY

Feed waters (CAP and effluent) were sampled three times during the course of the column testing. The first sample was taken on January 4, 1996 at the beginning of the tests. The three feed waters were again sampled half-way through the tests on February 13th, and then at the end of the test period on March 26th. The seepage water from each column was sampled upon initial breakthrough (once enough water had been collected, approximately 5 liters) and then approximately every two weeks for the duration of the twelve-week test period. Final samples from all columns were collected on March 26th, after which the feed pumps were turned off and the columns were allowed to drain.

Chemical analyses for inorganic constituents and total organic carbon were performed by AEN. All metal analyses were for total metals (not dissolved). Only 2 of the 2684 analyses performed had duplicate values out of range due to matrix interference and no percent recoveries were out of QA/QC range.

WATER CHEMISTRY of NATIVE GROUND WATER

Water chemistry obtained from M&A (Communication, 1996c) for two wells near the Santa Cruz site were used for this evaluation. Well SC9, (D-11-11)34aba, was the average of 14 samples and well SC10, (D-11-11)33bcb, was the average of 13 samples. Both waters are dominated by calcium (75 mg/l) and bicarbonate (200 mg/l), followed by sodium (50 mg/l) and sulfate (80 mg/l) and finally magnesium (10 mg/l) and chloride (60 mg/l), typical of much of the groundwater within the Tucson basin. The nitrate AWQS is exceeded in well SC10. TDS averages 100 mg/l and pH averages 7.2.

WATER CHEMISTRY of CENTRAL ARIZONA PROJECT WATER
CAP water is dominated by sodium (120 mg/l) and sulfate (230 mg/l). Calcium (60 mg/l) and bicarbonate (115 mg/l) follow with magnesium (33 mg/l) and chloride (90 mg/l) being the least dominate. Silica concentrations (5 mg/l) are much lower compared to groundwater. Magnesium, sodium, and chloride concentrations are significantly higher than those found in the groundwater. TDS ranges from 560 mg/l to 760 mg/l and EC ranges from 870 μS to 1060 μS, higher than that found within the Tucson Basin (CH2M Hill and others, 1988). The pH is similar to that found within the groundwater, at 8.2. Trace metals are generally less than detection limits except for barium having concentrations near 0.12 mg/l similar to that found within the groundwater. Total organic carbon ranged from 0.8 mg/l to 5.0 mg/l.

WATER CHEMISTRY of TREATED EFFLUENT

The treated effluent is dominated by sodium (110 mg/l) and bicarbonate (165 mg/l), followed by calcium (44 mg/l). Chloride and sulfate have similar concentrations near 75 mg/l. It is of note that potassium concentrations (11 mg/l) are higher than magnesium concentrations (6 mg/l). Silica concentrations (24 mg/l) are near the levels seen in the groundwater from the two sites. Sodium, potassium and chloride concentrations are significantly higher than those found within the groundwater. The pH, 7.5 to 8.0, is within the range found within the Tucson Basin (CH2M Hill and others, 1988). The TDS ranged from 440 to 490 mg/l, higher than generally found within the groundwater or the Tucson Basin. Trace metals are all below AWQS. Copper (0.01 mg/l), zinc (0.1 mg/l), and barium (0.02 mg/l) are found in the treated effluent. Total organic carbon ranged from 6.0 to 9.0 mg/l.

COLUMN TEST RESULTS - Water Chemistry

The METCON report (1996) contains raw data tabulations and graphs of infiltration rates, pH, electrical conductivity, dissolved oxygen, and oxidation/reduction potential. The following section generalizes the results of the tests. Specific results can be found in the AGRA report (1996).

Generally all of the columns followed a similar pattern for the infiltration rates. Infiltration rates increased to a maximum then a gradual decrease and a final leveling off. The first part of the pattern involves no seepage out of the column, only infiltration of feed water, and is caused by feed water filling the air filled pore space within the column. This second portion of the pattern involving steadily increasing rates of infiltration, is most likely caused by water displacing trapped air within the smaller pore spaces, thereby increasing the effective porosity of the material. Flow through the column at this time is characterized by unsaturated flow. At some point each of the columns reached a maximum infiltration/seepage rate. During this time
any air that can be displaced has been, and clogging has not become a predominant mechanism. Saturated flow conditions are now present. From this maximum, infiltration rates decrease. These rates usually decrease more quickly at first and then begin to level off at a reduced rate after some period of time. This pattern indicates the physical and chemical clogging of the material is occurring. Much of the physical clogging appears to take place at the surface of the material/feed interface. A fine-grained (usually clay or bio-material) clogs the top few millimeters of the soil column (see figure 2). If the top portion of the material is removed, infiltration rates will again increase. This was demonstrated by columns SCRACAPB and SCRAEFFB where the top segment of the material column, approximately one inch, was removed on 3/5/96. However, the increase only partially regains the maximum infiltration rate seen earlier in the test cycle.

Seepage pH in the recent alluvium columns starts out lower than the feed pH then increases until it is above the feed pH by the end of the tests. Seepage pH from the Fort Lowell Fm columns starts out higher than the feed pH then drops to levels similar to the feed pH by the end of the test.

Seepage EC starts out significantly higher than feed EC, then drops quickly (within 10 days) to levels similar to the feed EC in all tests.

Calcium is leached, higher concentrations in seepage than feed, for all columns. This leaching is greater in the initial phase of the testing. Magnesium leaching occurs initially in the effluent columns. However, magnesium is attenuated, lower concentrations in seepage than feed, in the CAP columns. Sodium concentrations in the seepage are similar to the feed for the recent alluvium columns. Initial sodium concentrations are lower in the seepage than feed in the Fort Lowell Fm columns. The sodium concentration in seepage then increases to higher than that in the feed by the end of the tests. For CAP tests, potassium concentrations are similar in feed and seepage. In the effluent test, potassium concentrations are lower in seepage than in the feed. Silica concentrations show major increase in the seepage for all tests.

Alkalinity, sulfate, and chloride are all higher in seepage than feed, especially during the initial portion of the testing. This is true except in the case of the Fort Lowell Fm - CAP test, where alkalinity, sulfate, and chloride in the seepage and feed are similar throughout the test.

Fluoride, arsenic, and barium tend to be leached from the soil into the seepage. Manganese, iron, chromium, and copper are generally similar in the feed and seepage. Aluminum concentrations in seepage are erratic with no apparent pattern.
GEOCHEMICAL MODELING

A primary objective of this investigation was to evaluate the mineral-water interactions and determine if any long term impacts to infiltration rates and/or water quality are likely. To accomplish this objective, the water chemistry and vadose zone mineralogy were assessed using a geochemical modeling approach. The latest in these modeling systems is the PHREEQC geochemical modeling application, a computer model for speciation, reaction-path, advective-transport, and inverse geochemical calculations available from the U.S. Geological Survey (Parkhurst, 1995).

The recharge of water, natural or induced, is a process involving a transformation of the native materials the water contacts and subsequent evolution of the water chemistry. First, surface water chemistry responds to its contact with air, airborne particles and minerals in the streams and/or surface soils. Next, as water infiltrates through the vadose zone, the water continuously attempts to come into equilibrium with the minerals with which it is in contact. Finally, the water reaches the primary aquifer, where it now equilibrates with the minerals of the geological unit forming the aquifer. Depending on the length of travel time, the time it would take to reach equilibrium, and the competing reactions, the water may or may not reach equilibrium at any point along its pathway to the aquifer. Each step along the way however, determines the final water chemistry and therefore quality within the aquifer.

Equilibrium Conditions - Geochemical Modeling

CAP water is over-saturated with respect to the minerals: chlorite, iron hydroxides, gibbsite, kaolinite, and sodium montmorillonite. CAP is under-saturated with respect to aluminum hydroxides, anhydrite, fluorite, gypsum, siderite, and silica cement. Minerals near saturation for CAP are: calcite, calcium montmorillonite, dolomite, illite, and quartz. Treated effluent is over-saturated with respect to the minerals: calcium montmorillonite, iron hydroxides, gibbsite, illite, kaolinite, and sodium montmorillonite. Effluent is under-saturated with respect to aluminum hydroxides, anhydrite, fluorite, gypsum, and siderite. Minerals near saturation for effluent include: calcite, chlorite, dolomite, and quartz. Groundwater is near equilibrium with calcite and under-saturated with respect to anhydrite, dolomite, and gypsum.

Geochemical Equilibrium Modeling - Column Test Changes

CAP Water - Recent Alluvium.

Seepage is near equilibrium with calcite, iron hydroxides, quartz, and silica
cement. Seepage is over-saturated with respect to: montmorillonite, chlorite, dolomite, gibbsite, illite, and kaolinite. The seepage is under-saturated with respect to aluminum hydroxides, anhydrite, fluorite, gypsum, and siderite. Minerals that were under or near saturation in the feed water and became increasingly saturated in the seepage include: calcium montmorillonite, illite, and silica cement.

**Effluent Water - Recent Alluvium.**

Seepage is near equilibrium with calcite, iron hydroxides, gibbsite, quartz, and silica cement. The seepage is over-saturated with respect to: montmorillonite, chlorite, illite, and kaolinite. The seepage is under-saturated with respect to: aluminum hydroxides, anhydrite, fluorite, gypsum, and siderite. Minerals that were under or near saturation in the feed water and became increasingly saturated in the seepage include: calcite, chlorite, and dolomite. Minerals having decreased saturation indices include: iron hydroxides and gibbsite. This would suggest that the formation of calcite, chlorite, and dolomite are possible while iron hydroxides and gibbsite may be consumed if present.

**CAP Water - Fort Lowell Formation.**

Seepage is near equilibrium with calcite, iron hydroxides, quartz, and silica cement. The seepage is over-saturated with respect to: montmorillonite, chlorite, dolomite, gibbsite, illite, and kaolinite. The seepage is under-saturated with respect to: aluminum hydroxides, anhydrite, fluorite, gypsum, and siderite. Increased saturation indices were calculated for calcium montmorillonite, illite, and silica cement. A decreased saturation index was determined for iron hydroxides. This would suggest that formation of calcium montmorillonite, illite, and silica cement is possible.

**Effluent Water - Fort Lowell Formation.**

Seepage is near equilibrium saturation for the following minerals: calcite, calcium montmorillonite, dolomite, iron hydroxides, gibbsite, illite, kaolinite, quartz, and silica cement. The seepage is over-saturated with respect to only sodium montmorillonite. All remaining minerals evaluated are under-saturated. Several of the minerals went from being over-saturated in the feed effluent to being under-saturated in the seepage water. These minerals include: calcium montmorillonite, iron hydroxides, gibbsite, and illite. These results suggest that these minerals may have precipitated within the material column along with other mineral-water interactions significantly shifting the water chemistry.

**GEOCHEMICAL MIXING MODELS**

To determine potential mineral equilibrium changes in current groundwater aquifers due to infiltration of recharge water, PHREEQC was used to calculate
mineral equilibrium under several mixing models for each of the sites based on the seepage water chemistry of the column tests and groundwater of the site. The results from these models are then compared to the mineral equilibrium computed for the aquifer groundwater to determine if significant differences exist after the addition of recharge waters. Each of the mixing models evaluate mixing 10%, 50%, and 90% infiltration seepage with 90%, 50% and 10% groundwater, respectively. The mixing models are based on the average chemistry found in each water. For the Santa Cruz site groundwater no aluminum or silica data was provided by Pima County. This lack of data may impact specific numerical values for some of the aluminosilicate minerals, however, the trends seen within the mixing models may still apply.

CAP Water.
Simulated mixing of CAP recharge water with the groundwater, indicate an increase in the saturation index for several minerals as the percentage of recharge water is increased. Saturation indices go from being under or near saturation using 10% CAP to over-saturated at 50% mixing for calcium montmorillonite, illite, and sodium montmorillonite. Dolomite and chlorite become saturated to over-saturated by the time 90% CAP is mixed with groundwater. Several minerals approach saturation as the recharge water percentage increases. These minerals include: iron hydroxides and silica cement. It is also of note, that gibbsite is over-saturated for all three mixing models but under-saturated in the mineral equilibrium model. Minerals at or near saturation over the entire mixing range include: calcite and silica cement. These results indicate that calcite, silica cement, montmorillonite, chlorite, illite, and possibly gibbsite would be stable within the mixing zone.

Treated Effluent.
Mixing of treated effluent infiltration and groundwater indicate increases in the saturation index for several minerals. Saturation indices go from being under-saturated to over-saturated for kaolinite and sodium montmorillonite. Minerals approaching saturation include: calcium montmorillonite, dolomite, iron hydroxides, illite, and silica cement. Minerals at or near saturation over the entire mixing range include: calcite, gibbsite, and quartz. These results indicate that calcite, gibbsite, quartz, kaolinite, montmorillonite, dolomite, iron hydroxides, illite, and silica cement would be stable within the mixing zone.

CONCLUSIONS

The objective of this investigation was to determine potential short-term and long-term impacts of surface basin recharge, using CAP water or treated effluent, on infiltration rates and aquifer water quality in the lower Santa Cruz.
Based on the results from these column tests, infiltration rates and possible water quality impacts were determined. Mineral precipitation and/or dissolution and ion exchange are key processes in the control of infiltration rates, quality of the recharge water, and the recharged water's ultimate impact on the groundwater. Ion exchange is a major factor in the water quality of the seepage during the beginning periods of the tests. Additionally, possible adsorption reactions and clay mineral interaction were considered to describe the changes seen within the seepage water chemistry.

The recent alluvium was determined to have the lower infiltration rates (0.05 to 0.25 fpd) than the Fort Lowell Formation (0.2 to 1.8 fpd) at the Santa Cruz site. Infiltration rates in the recent alluvium generally reached their maximum levels within the initial 15 to 20 day period and declined to much lower rates by day 40. Infiltration rates in the Fort Lowell Formation generally reached their maximum levels within a few days and then declined to lower rates by day 30. These results suggest that the recent alluvium will control the overall infiltration rate at this site.

Based on mineral equilibrium modeling, calcite, montmorillonite, illite and kaolinite are at or above saturation concentrations in the seepage and most mixing scenarios. Anhydrite, chlorite, fluorite, gypsum, siderite, and silica cement are generally below saturation concentrations.

REFERENCES CITED


The Hayfield Riparian Site is located to the southeast of the 91st Ave. WWTP on a terrace within the floodplain on the north bank of the Salt River. It is characterized by two kidney shaped surface flow treatment wetlands, H1 and H2 (Figure 1). These wetlands were constructed and planted in July and August, 1995 using local soils and two species of bulrush, Scirpus validus and Scirpus olneyi. Basin H1 is 1.33 ha (3.3 ac) where as Basin H2 is 1.29 ha (3.2 ac). Of the total surface area, each basin has approximately 20% as open-water deep zone. Each basin has an inlet and outlet open-water deep zone with Basin H1 containing five internal deep-zones while Basin H2 has only two internal deep zones. For this tracer test, depths in the shallow emergent areas were maintained at 30 cm (1 ft), while deep-zone depths ranged between 1.0 and 1.3 m (3.3 - 4.3 ft).

**Figure 1:** The Tres Rios Hayfield Riparian Site basins H1 and H2. Both basins have 20% open-water deep zones, but in H1 they are configured into 5 narrow internal zones versus Basin H2 which has its 20% configured into two large open-water deep zones with water-fowl islands.

As Figure 1 shows, characteristics of these two basins are almost identical. To compare the effects of the two open-water configurations, operating conditions for the two Hayfield basins were kept as similar as possible during the tracer tests (Table 1).

**Method**

Tracer studies were conducted on the two Hayfield wetland cells in late 1996. Figure 2 displays data obtained from one of these studies. These data were analyzed using the method of moments. The moments provide information needed to determine
the “actual” or tracer detention time, variance of the curve, and the number of CSTR’s that describe the wetlands’ hydraulic nature, and the dead volume. The dead volume is the volume of the reactor that does not readily mix with the inflow. It includes the volume taken up by the plants. In addition, we obtain a better understanding of how water is flowing through the cell from the study of the tracer retention time.

Table 1: Hayfield Physical & Operational Parameters for Tracer tests H1A and H2A.

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</tr>
<tr>
<td>Avg. Qin (m³/d)</td>
<td>2,021</td>
<td>Avg. Qin (m³/d)</td>
<td>1,992</td>
</tr>
<tr>
<td>Avg. Qout (m³/d)</td>
<td>1,719.8</td>
<td>Avg. Qout (m³/d)</td>
<td>1,736</td>
</tr>
</tbody>
</table>

**Materials & Methods**

**Tracer Material.** The tracer chosen for these tests was Br⁻, which was obtained in the form of regent grade NaBr. Br⁻ was chosen because it is present at low background levels (0.2 - 0.3 mg/L), and it is more conservative than an organic tracer such as rhodamine dye.

**Tracer Amount.** Enough tracer was used for each test to achieve a peak concentration of 10 mg/L, assuming the basins behaved as a CSTR. For Tracer Test H1A (Basin H1, first test), approximately 86.6 kg of NaBr was used which resulted in 67.3 kg of Br⁻ being added. Tracer Test H2A utilized 83.4 kg of NaBr which resulted in an addition of Br⁻ equal to 64.8 kg of Br⁻.

**Tracer Addition.** Tracer was added to the basins by means of a slug-input. To accomplish this, the pre-measured NaBr was dissolved in two large plastic containers using approximately 265 L (70 gallons) of plant reuse water (Secondary-Advanced treated wastewater used as process water within the WWTP). After complete dissolution, achieved by mixing each container for approximately 30 minutes, the Br⁻ solution was dumped immediately downstream of the Basin’s inlet weir. This condition allowed for vigorous mixing of the tracer with incoming wastewater prior to entering the inlet deep-zone of the basin undergoing testing.

**Tracer Collection.** Prior to the slug addition of tracer, an automatic sampler was set at the outlet of the Basin being tested. The sampler used for this study was an ISCO Model
HYDRAULIC CHARACTERIZATION OF WETLANDS TO IMPROVE TREATMENT EFFICIENCY

Shawn Whitmer and Roland Wass

Introduction:

The goal of this project is to relate wetland hydraulic characteristics with treatment efficiency for wetlands of similar design. This information is needed to help improve understanding of wetland performance in contaminant removal.

Wetlands provide a low tech alternative for treating municipal wastewater in most subtropical and temperate climates. They are also used to polish conventionally treated wastewater prior to groundwater recharge (ADEQ, 1995). Wetlands remove or attenuate contaminants via bacterial transformation or uptake and through physiochemical processes like adsorption, precipitation and sedimentation (Hiley, 1995). Bacterial activity occurs primarily at the soil-water interface and to a lesser extent upon surfaces of submerged aquatic vegetation.

Although they provide surface area for bacterial growth, the plants also occupy small areas which obstruct the movement of water as it proceeds through the wetland. The presence of plant material reduces the “as-built” volume of the wetland, which in turn can lead to erroneous estimates of detention time, since this parameter is dependent on flow and design volume determination. The detention time is an important parameter used in the design of wetland treatment systems. Therefore, it is prudent to understand how the water is flowing through the system (e.g. the hydraulic and hydrologic characteristics must be understood). This type of analysis has been conducted on only a few occasions (Kadlec, 1994). In this paper we examine the hydraulic characteristics of a wetland and demonstrate how this information can be utilized to improve the modeling of contaminant removal.

Background: Modeling of Treatment Wetlands

Wetlands in the past have been designed to remove specific contaminants through the use of areal rate constants, (k), through the “first-order model”, (Reed, Middlebrooks, & Crites, 1988). This type of analysis, however, for the most part ignores the “actual”

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3 City of Phoenix, Tres Rios Demonstration Project, 91st Ave. WWTP, Tolleson, AZ 85353.
hydraulic characteristics of the system (Kadlec, 1994). These rate constants have been determined by measuring the concentration of a contaminant at the inlet and outlet of the treatment cells for many different wetlands. This type of analysis assumes that the cells act as plug flow reactors (PFR’s), while in reality, many wetlands do not. Wetlands in general act more like one or a series of continuously stirred tank reactors (CSTR’s). Generalized areal rate constants are not representative for specific areas within the wetland. These areal constants do not directly take into account the individual “tanks” within the wetland cell. These tanks need to be considered individually and then as a system to obtain more accurate estimates of the rate constants. With the rate constants for each tank, future constructed wetland projects could be based on more accurate design parameters. However, this type and quantity of information will require a more intensive and in depth study in the future.

The texts by Levenspiel (1972) and Fogler (1992) set forth the basic chemical reaction engineering principles and have developed the mathematical explanations for the design of chemical reactors. Kadlec (1994) first utilized these principles in the analysis of wetland design.

To determine areal rate constants with more accuracy, a wetland’s hydraulic characteristics must be known. These characteristics can be obtained by conducting tracer tests. The method of using conservative tracers to determine the hydraulics of wetlands has been utilized in the past by many researchers. Tracer tests show how the water actually flows through the wetland cell. In addition, the removal efficiency for a non-conservative constituent by a wetland can also be estimated.

Wetlands have been studied to derive equations that link geometric parameters to desired removal efficiencies of various independent contaminates. These empirical relationships typically take into account the volume, depth, area, flow and porosity of the original design. This approach neglects the actual “active” volume and pore space in established wetland “cells” and the change in these cells as they mature and develop new hydraulic profiles. During the maturation process, vegetation fills in previously open areas and changes the hydraulic characteristics of the wetland. The proposed work accounts for the actual detention time, “active” volume, and pore space to link the desired removal efficiencies with the actual hydraulic conditions of mature wetlands.

Site Description: Hayfield Riparian

The Tres Rios Demonstration Constructed Wetlands are located at the City of Phoenix/SROG 91st Avenue Wastewater Treatment Plant in Tolleson, Arizona. The Tres Rios Demonstration Project is a 2-year coordinated research effort among the City of Phoenix/SROG, Bureau of Reclamation, Arizona Department of Water Resources and Arizona Game and Fish. The project was constructed during the spring and summer of 1995. The resulting wetland cells have been polishing approximately 2 million gallons per day of conventionally treated wastewater since that time. Three wetland sites were constructed: Cobble Site, Research Cell Site, and Hayfield Riparian Site.
3700 configured to take 24 discrete hourly samples and place them into plastic containers. The duration of test H1A was 9 days while test H2A was conducted over 11 days. These test periods correspond to 2.77 and 3.25 times the nominal detention time (V/Q) for each basin, respectively. Each afternoon samples from the past 24 hours were removed from the sampler and placed into 250 mL plastic containers and stored at 4 °C until shipment to the City of Phoenix Compliance Laboratory.

**Figure 2:** Tracer Curve for Hayfield Basin H1

Sample Analysis. Samples for both tests were submitted to the City of Phoenix Compliance Laboratory for analysis. The method employed was Ion Chromatography, EPA 300.0.

Flow Measurements. Inlet flows for each basin were recorded at the Hayfield Site inlet splitter box. This structure houses 60° V-Notch weirs which serve as the primary measurement device. Flows exiting the basins were measured with similar V-Notch weirs. Both inlet and outlet measurements were obtained each morning for the duration of each test.

Method of Moments for a Tracer Response Curve

The method for determining the moments of a curve is presented below. This is a brief summary of the work developed for chemical reactors by O. Levenspiel (1972) and has been expanded upon by Kadlec (1994) for the case of wetlands. First the mass balance of the tracer in the system is checked. This is known as the zeroeth moment of the curve and is determined by:

\[ M_0 = \int_{0}^{\infty} C(t) dt \]  

*eqn (1)*
\[ M_0 = \text{the zeroth moment of the curve } [g], \]
\[ Q = \text{flow } [L/d], \]
\[ C(t) = \text{concentration of tracer at time } (t) [mg/L], \]
\[ t = \text{time } [d]. \]

The first moment is determined which provides information necessary to calculate the actual cell detention time or tracer detention time, given by:

\[ M_1 = Q \int_0^\infty tC(t)dt \]  
\[ \text{eqn (2).} \]

\( M_1 \) has units of [g-d]. To determine the mean detention time, the time weighting around the centroid of the distribution, is needed. This is calculated from the second or central moment [g-d^2], by the following:

\[ M_2 = Q \int_0^\infty t^2C(t)dt \]  
\[ \text{eqn (3).} \]

From the moments, important hydraulic information can be calculated. This information is used to analyze and improve wetland treatment designs. From equation 1 and equation 2, we can determine the tracer detention time (\( \tau \), [d]).

\[ \tau = \frac{M_1}{M_0} \]  
\[ \text{eqn (4).} \]

The variance of the curve (\( \sigma^2 \), [d^2]), is determined from equation 1 and equation 3.

\[ \sigma^2 = \frac{M_2}{M_0} \]  
\[ \text{eqn (5).} \]

The variance of the curve correlates to the degree of mixing in the wetland cell.

From equations 4 and 5 we can determine the number of CSTR’s (n), that the wetland cell hydraulic characteristics resemble.

\[ n = \frac{\tau^2}{\sigma^2} \]  
\[ \text{eqn (6).} \]

From this information, the spreading or dispersion occurring within the wetland cell can be quantified, from the, wetland dispersion number, \( \mathcal{D} \), (Kadlec, 1994). But first we need the dimensionless variance, (\( \sigma^2_\theta \)), which is calculated from:

\[ \sigma^2_\theta = \frac{1}{n} \frac{\sigma^2}{\tau^2} \]  
\[ \text{eqn (7).} \]

the dispersion number (\( \mathcal{D} \)) can now be found by iteration:

\[ \sigma^2_\theta = 2\mathcal{D} - 2\mathcal{D}^2 \left( 1 - e^{-1/(\mathcal{D})} \right) \]  
\[ \text{eqn (8).} \]

this equation was originally derived in (Levenspiel, 1972) with the following substitution

\[ \mathcal{D} = \frac{D}{uL} \]  
\[ \text{eqn (9).} \]

where
D = dispersion coefficient (m²/d),

u = average velocity (m/d),

L = length (m), from inlet to outlet.

The values that are represented by (D) are not to be used independently since a true velocity and dispersion coefficient are difficult to measure for a wetland. From these values, we can calculate a corrected areal rate constant that would better explain how the system is treating the water. Values calculated from the tracer studies conducted at the Hayfield site for this type of analysis are presented in Table 2. Figure 2 shows the tracer curve data obtained for Basin H1.

Analyzing Moment Information

The information obtained from the tracer curves and moment calculations can be used to compare the efficiency of ideal and non-ideal chemical reactors (Kadlec, et al., 1993). Wetlands are chemical reactors and as such are often modeled as plug flow reactors (PFR). They can also be modeled as continuous stirred tank reactors (CSTR’s) or a PFR with dispersion. To represent a first-order irreversible reaction within a PFR of a non-conservative constituent we use the following:

\[
C = C_i e^{(-k\tau)} \quad \text{or} \quad \frac{C}{C_i} = e^{(-k\tau)} \quad \text{eqn (10)}
\]

where

- \(C\) = concentration (mg/L),
- \(k\) = first-order rate constant (d⁻¹),
- \(\tau\) = retention time (d).

The related equation for a complete mix system of CSTR’s is:

\[
\frac{C}{C_i} = \frac{1}{(1+k\tau/n)^n} \quad \text{or} \quad C = C_i \frac{1}{(1+k\tau/n)^n} \quad \text{eqn (11)}
\]

where

- \(n\) = number of reactors.

If dispersion is considered in the PFR model the following relationship developed by Wehner and Wilhelm (1956) must be used:

\[
C = C_i e^{\left[-\left(-k\tau\right)+\frac{D}{uL} (k\tau)^2\right]} \quad \text{or} \quad \frac{C}{C_i} = e^{\left[-\left(-k\tau\right)+\frac{D}{uL} (k\tau)^2\right]} \quad \text{eqn (12)}
\]

Remember \(D/uL = D\) = wetland dispersion number. This equation was developed for small values of \(D \leq 0.05\) for the simple Gaussian approximation. It should not be used for values of \(D \geq 0.10\) (Levenspiel, 1972). It has been shown that small amounts of
dispersion occurs when $\phi = 0.002$ while $\phi \geq 0.20$ represents a large amount. For mixed flow $\phi = \infty$. These values can be checked with the derivation in Levenspiel (1972), for dispersed plug flow.

**Data Analysis:**

Results obtained from the method of moments are presented in Table 2. These values are used in the effort to compare the type of reactors that best describe wetlands' behavior. Table 3 displays the actual volume that is actively polishing the wastewater.

**Table 2: Information from Moment Calculations**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Hayfield 1A</th>
<th>Hayfield 2A</th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tracer detention time ($\tau_d$)</td>
<td>2.70</td>
<td>2.34</td>
<td>M1/M0</td>
</tr>
<tr>
<td>Tracer curve variance ($\sigma^2$)</td>
<td>1.44</td>
<td>1.33</td>
<td>M2/M0</td>
</tr>
<tr>
<td>Number of tanks (n)</td>
<td>3.53</td>
<td>3.09</td>
<td>$\tau_d^2 / (\sigma^2)^2$</td>
</tr>
<tr>
<td>Dimensionless variance ($\sigma_0^2$)</td>
<td>0.28</td>
<td>0.32</td>
<td>$1/n$</td>
</tr>
<tr>
<td>Wetland dispersion number ($\phi$)</td>
<td>0.17</td>
<td>0.20</td>
<td>&quot;Solver&quot; $^a$</td>
</tr>
</tbody>
</table>

$^a$ Solver routine in Microsoft EXCEL.

**Table 3: Design Parameters vs. "Actual" Parameters**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Hayfield 1A</th>
<th>Hayfield 2A</th>
</tr>
</thead>
<tbody>
<tr>
<td>Design detention time ($\tau_d$) [d]</td>
<td>3.35</td>
<td>3.25</td>
</tr>
<tr>
<td>Flow (Q) [m$^3$/d]</td>
<td>2021</td>
<td>1992</td>
</tr>
<tr>
<td>Design volume (Ve) [m$^3$]</td>
<td>6761</td>
<td>6475</td>
</tr>
<tr>
<td>Active volume (Va) [m$^3$]</td>
<td>5456</td>
<td>4662</td>
</tr>
<tr>
<td>Dead volume/ Pore space (Vd) [m$^3$]</td>
<td>1306</td>
<td>1812</td>
</tr>
<tr>
<td>Percent reduction from (Ve) [%]</td>
<td>19</td>
<td>28</td>
</tr>
<tr>
<td>Flow porosity [%]</td>
<td>81</td>
<td>72</td>
</tr>
</tbody>
</table>

Using equations 10-12 we can compare the effect of the tracer detention time, ($\tau_d$), and dispersion number, $\phi$, on the first-order rate constants, (k). To illustrate the effect of hydraulic considerations on computed rate constants, consider a hypothetical situation where 75 percent removal of a non-conservative constituent is desired, from wetland cells similar in design to the Hayfield wetlands. The resulting areal rate constants, (k_a), that would be needed to attain the desired removal efficiency are presented in Table 4 for various reactor configurations. These are based on a 1 ft emergent depth in the cells.
Table 4: Hypothetical Case 1: Hayfield Basin H1 and Hayfield Basin H2
For 75% removal of contaminant and 1 ft. deep emergent water level

<table>
<thead>
<tr>
<th>Type of Reactor</th>
<th>Detention Time [d]</th>
<th>First-order Areal Rate Constant (k_a) [ft/d]</th>
<th>Detention Time [d]</th>
<th>First-order Areal Rate Constant (k_a) [ft/d]</th>
</tr>
</thead>
<tbody>
<tr>
<td>PFR</td>
<td>3.35</td>
<td>0.41</td>
<td>3.25</td>
<td>0.43</td>
</tr>
<tr>
<td>PFR w/ τ_t</td>
<td>2.70</td>
<td>0.51</td>
<td>2.34</td>
<td>0.59</td>
</tr>
<tr>
<td>One CSTR</td>
<td>3.35</td>
<td>0.90</td>
<td>3.25</td>
<td>0.92</td>
</tr>
<tr>
<td>Series of CSTR’s</td>
<td>3.35</td>
<td>0.51</td>
<td>3.25</td>
<td>0.54</td>
</tr>
<tr>
<td>PFR(^b) w/ D</td>
<td>3.35</td>
<td>0.67</td>
<td>3.25</td>
<td>0.76</td>
</tr>
<tr>
<td>PFR(^b) w/ D, τ_t</td>
<td>2.70</td>
<td>0.83</td>
<td>2.34</td>
<td>1.056</td>
</tr>
</tbody>
</table>

\(^b\)Note: The respective values for D and τ_t are located in Table 1.0.

Discussion

Results show that the actual detention time, (τ_t), makes a significant change in the rate constant. If this is not considered in the original design then the wetland could be under designed. The original value for the areal rate constant, (k_a) would have been lower which would not have accounted for the reduced amount of time in the wetland. This would mean that the actual treatment obtained was below the desired. This demonstration shows that the hydraulics do in fact, need to be considered in designing wetlands. In addition, by correcting for dispersion, (k_a) increased but, with the combination of both D, and τ_t, the rate constants increase quite significantly. The effect of the dispersion number has not as of yet been quantified since the calculated values were much higher than those recommended when using equation (12). A more complex equation is needed to describe the effects of large amounts of dispersion. When considering CSTR’s in series, it is important to remember that once 3 to 4 tanks are used in series, they are basically considered a PFR. This accounts for the values being close the PFR case with the design detention time.

For wetlands to discharge to ground water recharge projects they must meet water quality regulations but if they are not designed to account for the hydraulics they could exceed those requirements.

Additional Considerations:

Wetland configurations and designs are also expected to effect the flow of water which affects the dispersion number. This would include such design features as “deep water zones”, islands, bends or curves in the surface area layout. Other considerations could include the wetland plant type, operating temperatures, pH, flow, stratification and even the type of fauna that inhabits the wetland. The specific effects of each will be
harder to quantify than the hydraulic characteristics but they are nonetheless important considerations.

Future Work

It is desirable to consider the effects of the water budget (e.g. precipitation, evapo-transpiration, and or infiltration) during analysis of future tracer studies. These occurrences could either dilute the constituent or further concentrate it depending on which process dominates within the treatment cell. Further development of the technique to describe and model the effects of dispersion is planned. This will include the complex equation that describes dispersion when $D \geq 0.2$. Also, a sensitivity analysis on the method of moments will be more fully explored in the future, to test the effects of small parameter changes, (e.g. background concentration).

In the near future we hope to conduct dye studies at several treatment wetlands in Arizona and to compare the hydraulic characteristics with operation and design features (e.g. emergent depths, length to width ratios, ratio of pools, shallow areas, etc...)

References Cited


INVESTIGATION OF SOIL AQUIFER TREATMENT FOR SUSTAINABLE WATER REUSE

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\textsuperscript{1}Paper presented at the 8th Biennial Symposium on the Artificial Recharge of Groundwater, Tempe, AZ, June 2-4, 1997
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ABSTRACT

A three-year program of study is proposed that will establish the efficacy and sustainability of wetlands and soil-aquifer treatments leading to indirect potable water reuse. Wetlands and infiltration/subsurface treatment will be considered as a single treatment system for purposes of aquifer protection -- primarily protection from nitrate and dissolved organics of wastewater origin. Soil-aquifer treatment is envisioned as consisting of (i) infiltration through a biologically active infiltration interface (<1m in depth) at the soil/water boundary of the infiltration basin; (ii) percolation through an extensive vadose zone, 10-100 feet in depth; and (iii) storage/transport in the underlying aquifer (6-24 months, >500 horizontal feet) pending withdrawal at proximate production wells. Water quality benefits in terms of organic carbon, nitrogen and pathogen attenuations will be assigned to each treatment zone based on the proposed field program and supporting laboratory experiments. At least seven field sites have been selected for use in the study based on specific strengths such as depth to groundwater, quality of groundwater data, instrumentation and geographical considerations. Results will be analyzed within a systems framework that can be driven by local data and constraints from other geographical locations.

A multidisciplinary team consisting of environmental researchers from three western states; water and wastewater practitioners from Arizona and California -- states that will eventually depend heavily on wastewater reclamation/reuse to meet water demands; and experts in (aquatic) organic chemistry, virology, hydrology and systems analysis has been identified to overcome these shortcomings. The proposed project will generate practical tools with which to assess water quality and/or gage compliance with regulatory (reuse) criteria. These include identification of organic tracers for waters of wastewater origin (for calculation of volume contributions of wastewaters at withdrawal points), modification of flow algorithms for generation of distance/travel time relationships in groundwaters influenced by reclaimed water, correlations between numbers of infective viruses and PCR-detectable viral nucleic acid, and interrelationships between wetlands polishing and soil-aquifer treatment for nitrogen control.

NATURE, SCOPE AND OBJECTIVES

Wastewater Reclamation Using Soil-Aquifer Treatment

Groundwater recharge with reclaimed wastewater is an increasingly valued practice for replenishing aquifers used for domestic supply, especially in the arid Southwest. By far the most widely used recharge method is by infiltration from spreading basins, although direct injection of reclaimed water is also possible. Percolation through the unsaturated zone and subsequent groundwater transport and storage provide final polishing of the reclaimed wastewater such that the extracted water can be used for non-
potable purposes without further treatment and for potable purposes after disinfection. Collectively, the water quality improvements that arise from percolation and groundwater transport/storage pending reuse are termed soil-aquifer treatment (SAT).

Groundwater recharge enjoys the following advantages over recycle based on discharge into surface waters:

Groundwater recharge provides additional water quality benefits due to SAT.

Seasonal or longer-term storage can be achieved without evaporative losses.

Groundwater recharge protects water against recontamination by birds and mammals (with coliforms and parasites) and possibly even by humans.

Groundwater recharge keeps sunlight away from the water. This prevents growth of algae and associated water-quality problems such as algae-derived taste and odor and DBP formation (upon chlorination of waters containing dissolved organics of algal origin). To avoid algal growth in the surface reservoir, the wastewater must be treated to remove essentially all nitrogen and phosphorus. This is expensive and avoidable if effluent is used to recharge local groundwaters.

Infiltration removes treated wastewater from sight. When it is recovered from wells, it is conceived of as groundwater, an aesthetically superior and more publicly acceptable water source.

Soil percolation encompasses several processes that occur during downward transport in the unsaturated zone. At the basin/soil interface, the combined effects of sedimentation, filtration and microbial growth lead to the formation of relatively impermeable deposits, also termed schmutzdecke. Basin hydraulic performance is dominated by head losses during bulk transport through this zone. The schmutzdecke and the top few centimeters of soil, collectively termed the infiltration interface, is a zone of high biological activity. The residence time in the infiltration interface is only on the order of minutes. Below the infiltration interface, the unsaturated or soil percolation zone is typically 10 to 100 ft deep and provides a contact time with the soil of hours to days. After reaching the underlying aquifer, groundwater moves slowly to extraction wells, which are generally located at least 500 ft away. Groundwater transport provides 6 to more than 12 months of contact time with the unconsolidated aquifer material. Because both the time and length scales (and the biogeochemical conditions) of the infiltration interface, soil percolation and groundwater transport zones differ significantly, these three zones will be considered separately within the context of soil-aquifer treatment.

Our collective understanding of SAT at the both the system level and the process level does not meet our immediate needs. Existing information does not support rational
design and operation of SAT systems. Neither can we point at a specific level of water quality improvement during SAT that will eliminate potential public health concerns. Lack of information leads to overdesign and overregulation. At the system level, it is difficult to predict optimal operational procedures for production of acceptable effluent water quality at the lowest cost. At the process level, it is difficult to predict the fate and transport of contaminants of health concern during SAT or to compare the quality of reclaimed water with that of alternative sources. These uncertainties lead to questions regarding the long-term sustainability of water quality benefits attributable to SAT systems.

Project Overview and Goals

The central project objective is to examine the sustainability of SAT, in combination with above-ground treatments, leading to indirect potable reuse of reclaimed water. Primary questions involve the fates of nitrogen species, organic carbon, and viruses.

Specific goals of the project are:

To characterize processes that contribute to organics removal and transformation during transport through the infiltration interface, soil percolation zone and underlying groundwater aquifer;

To investigate and model relationships among above-ground treatment, wetlands polishing and SAT at the systems level;

To identify monitoring criteria that will provide proper assurances regarding the elimination of viruses and other pathogens;

To produce a framework or model within which SAT systems can be designed to meet regulatory criteria and regulatory criteria can be analyzed in terms of their impact of SAT systems design, costs, etc.

The proposal represents a regional effort by several governmental organizations and university laboratories. Proposed investigations and data gathering are primarily at the field scale, using recharge sites in California and Arizona, although laboratory work is required to support the field studies.

Water quality data will be collected from at least seven full- or pilot-scale wetlands and recharge facilities that offer a range of effluent qualities and very different physical conditions in terms of depth to groundwater, soil and sediment type, etc. Research activities will be tailored to take advantage of unique features or strengths at respective field sites.
METHODS, PROCEDURES AND FACILITIES

Soil Characterization

Soil samples will be obtained from borings taken during the installation of the monitoring wells. The soil characteristics to be determined include particle size distribution, clay content, fraction organic carbon, and ion exchange capacity. Representative soils from the borings will be used in microcosm studies and to generate adsorption isotherms.

Fate and transport of nitrogen species

Typically, water samples will be obtained for analyses of ammonia and nitrate/nitrite from infiltration basins, at several depths in the vadose zone, and at several groundwater monitoring wells that differ in terms of distance and time of travel away from points of recharge. At wetlands, samples will be obtained at the facility inlet and outlet as well as intermediate points. The Sweetwater wetlands/recharge facility provides wetlands treatment followed by disposal via infiltration. Nitrogen speciation will be monitored as a function of time or position in the wetlands. Measurements will include all major nitrogen species in order to support development of a nitrogen balance and estimate the degree of denitrification across the facility.

Changes in nitrogen speciation that accompany soil-aquifer treatment of wetlands effluent will be established by comparing depth-dependent water quality measurements in the vadose zone beneath the SAT infiltration basin and distance-dependent measurements in aquifers that receive reclaimed water. Dilution effects will be accounted for using procedures developed to monitor changes in organic water quality parameters -- by measuring changes in an as yet unselected tracer compound that is of wastewater origin, water conductivity, or both. Measurements of dissolved oxygen and/or redox potential will be used to determine if and where local conditions are likely to encourage denitrification reactions. Depth- or distance-dependent measurements of DOC concentrations will be used to determine the biodegradability of dissolved organics that arise from wetlands treatment.

Potentials for nitrification and denitrification reactions will be established in microcosms designed to provide conditions that are appropriate for those conversions. These will be set up using native or acclimated sediments and effluent or groundwater samples. Microcosms will be amended as dictated by experimental objectives with molecular oxygen (nitrification studies) or a suitable organic substrate (denitrification).

Virus transport.
The objective will be approached by measuring both numbers of infective viruses and naked viral nucleic acid in water samples derived from the vadose zone and underlying aquifer at pilot-scale recharge facilities. It is anticipated that enteroviruses will be used as the basis for development of such relationships owing to their prevalence in wastewater, the relative convenience of enterovirus measurements and their persistence in the environment, all of which have been established in previous SAT studies. Appropriate nucleic acid sequences are available for the construction of primers for enterovirus nucleic acids, a necessity for PCR-enhanced detection of nucleic acid.

Work is needed to determine if coliphages detected in groundwater near SAT sites actually originate from wastewater. Bacteriophage, especially FRNA coliphage (male-specific, RNA viruses), have been proposed as surrogate indicators for evaluating the adsorption and transport of human viruses through soil. Recent research, however, revealed that the predominant phages detected in the reclaimed water are not FRNA phage, and the validity of the native phage as tracers of human virus is unknown. In the present study, coliphage in groundwater and disinfected reclaimed water effluents will be characterized to determine relationships among the species present. Results will be used to determine whether or not the frequency of false positives undermines the utility of coliphage as indicators of pathogen transport.

The effects of chlorine dose and contact time on numbers of infective viruses and PCR-detected viral nucleic acids will be measured in soil slurries seeded with live viruses and lysed virus particles. Numbers/rates of attenuation will be used to establish expectations relative to the utility of PCR measurements to indicate the presence of viruses under field conditions. Viruses and nucleic acids will be selected for study based on environmental relevance and the availability of adequate monitoring procedures.

Relative rates of transport or attenuation of viruses and nucleic acids during flow through porous media (soil and sediments) will be established in 1- or 2-m columns packed with representative soil samples. Mechanisms for virus inactivation in wetlands (sunlight inactivation, physical removal, predation, etc.) will be established in microcosms. Inactivation rates will be compared in light and darkness, filtered and unfiltered samples.

Characterization of Organic Residuals and SAT-dependent Improvements in Organic Water-Quality Parameters.

A variety of emerging analytical procedures will be applied to generate a detailed chemical picture based on fractionation, classification by predominant functional groups, reactivity and direct chemical analysis. The resultant data base will be examined to yield correlations (expectations) relative to the fate and transport of the same chemical groups or fractions during SAT. Parallel work will be undertaken to determine which groups of organics are active in DBP formation, contribute to the gene-tox properties of the aggregate, etc. The data developed here will help us devise treatment/reuse strategies for
the management or elimination of identifiable risk associated with indirect potable reuse of reclaimed water. Organic or organic-related measurements of interest in disinfected secondary effluent and groundwater samples derived from pilot-scale facilities follow:

Total & dissolved organic carbon (TOC/DOC)

UV absorbance at 254 nm and UV absorbance spectra (200-400 nm)

Molecular weight distribution among effluent organics, derived via size-exclusion chromatography with UV detection.

Biodegradability of organic residuals and DBPs, measured via the biodegradable organic carbon (BDOC) procedure.

Disinfection by-products -- trihalomethanes (THMs) and haloacetic acids (HAAs).

Disinfection by-product formation potentials -- Bromide ion will be measured to help interpret data relative to the brominated THMs and HAAs.

Hydrophobicity of residual organics, or the fraction of such residuals that is retained on an XAD-8 resin.

$^{13}$C-NMR and elemental analysis (C, O, H, N, S, P) will be carried out on organic material that can be concentrated via a combination of XAD-8 resin separation and freeze drying.

Semi-quantitative measurement of individual compounds and/or classes of compounds via GC/MS, including those that may be effective tracer compounds such as EDTA, NTA and APECs.

Three general types of water samples will be obtained for characterization in terms of these parameters: (i) pre-SAT; (ii) post-SAT, as a function of depth in the vadose zone; and (iii) aquifer samples, as a function of distance or transit time from points of infiltration. For pre-SAT samples, we will characterize effluent organic matter (EfOM) from various chlorinated secondary effluents, including at least biotower effluent, an activated sludge effluent, and a nitrified/denitrified effluent. There are at least two (pre-SAT) sampling points of potential interest: (i) effluent entering filtration ponds and (ii) composite samples of pond-water itself, where EfOM is potentially augmented by algal organic matter (AOM). Where possible, upgradient samples of native groundwater will also be analyzed. A more complete description of analytical methods is provided as Appendix A.
Resultant data will be used to test hypotheses relative to fate and transport of specific classes of compounds that contribute to EfOM. Contemporary measurements obtained as a function of depth in the vadose or distance/travel time from infiltration points will be used to develop straightforward relationships between water quality and SAT operational parameters (depth to groundwater, distance to extraction wells, infiltration rates, pond area, etc.).

Based on previous work, potentially useful organic tracers (e.g., EDTA) are present in most if not all domestic wastewater effluents. Parameters contributing to the detailed organic analyses include a number of such candidates. Their attenuation during vadose-zone transport and aquifer storage will be evaluated using these data and supportive bench-scale experiments.

Although organic residuals in waters that have percolated through the infiltration interface tend to be refractory in time scales of hours to days (see above), their fate during percolation through an extensive vadose zone and extended aquifer storage is not known. To generate information of this sort, microcosms will be developed to simulate long-term (6-18 months) storage pending withdrawal for reuse. As a minimum, microcosms will consist of sediments with a lengthy history of contact with reclaimed water and wastewater effluent that has already percolated through the infiltration interface and vadose zone. Microcosms will be sacrificed at intervals over a two-year period. Water and sediment will be extracted separately and subjected to a range of organic analyses that is similar in scope to those developed for detailed field studies. Controls will include a zero-time sample, sediment-free, wastewater-free, and heat-sterilized reactors.

**Modeling and systems analysis.**

Mathematical models representing contaminant treatment and attenuation during above-ground treatment and vadose-zone transport will be developed or identified in the literature. Relationships for wetlands and the unsaturated soil zones will be fit to experimental and field data from all appropriate study sites. They will likely be exponential decay functions with time and/or distance traveled. Time will account for decay processes while distance will be important for sorption or filtration impacts. Decay constants will likely vary by soil type. Since flow velocity also varies with soil type, single variable (time) relationships can potentially be used. Literature equations will be used to describe above-ground treatment efficiencies. With available aquifer parameters and estimates of dispersion coefficients, transport and dilution can be modeled using existing groundwater models such as MODFLOW. Decay and sorption effects will be superimposed to yield estimates of contaminant concentration as a function of travel time/distance. Exponential decay/sorption relationships are again expected.
The mathematical models describing contaminant removal and cost equations will be used in the optimization scheme. The optimization problem is to determine the least cost system that will produce water of specified quality as it enters the aquifer. A dynamic programming approach will be employed since the processes act in series and may be discrete in nature. In the decoupled approach, a defined input contaminant concentration will be applied to the aquifer. This must be reduced to a desired or regulated concentration at points of withdrawal. First, steady-state conditions will be analyzed to represent long-term system operations. In this case, a general nonlinear programming model will be written and solved. Results from the two models will be combined to identify the overall optimal system design or operation by summing the costs of the pretreatment processes and withdrawal system.

ANTICIPATED RESULTS

(i) Quantified relationships between contaminant attenuation and process variables such as infiltration depths, travel distance/time to withdrawal wells, infiltration rate, etc.;
(ii) Comparisons between the qualities of SAT-treated reclaimed wastewaters and waters from alternative sources;
(iii) Methods for determining the volume contribution of water of wastewater origin in groundwater samples;
(iv) A systems framework within which to compare treatment alternatives, including SAT, in terms of their effect on overall process performance. The same models can be used to investigate the effects of regulations on process requirements.

Organics

(i) State-of-the-art characterization of organics that comprise the persistent fraction of DOC -- that which typically survives transport through the infiltration interface;
(ii) Separation and quantitative treatment of processes that contribute to the removal and transformation of organic contaminants during SAT;
(iii) Measurement of the SAT-dependent changes in the character of residual organics -- i.e., comparison of residual organics in terms of identifiable chemical characteristics at various points in the overall SAT process;
(iv) Quantification of the degree to which persistent organics of wastewater origin contribute to THM and HAA formation at points of groundwater withdrawal.

Nitrogen

(i) Verification of nitrogen removal during SAT;
(ii) Methods for managing nitrate levels in waters that reach groundwater extraction points through a combination of above-ground treatment, wetlands polishing and soil-aquifer treatment;

Viruses

(i) Relationships between survival/numbers of infective viruses and viral nucleic acids in secondary effluent and groundwaters influenced by the recharge of reclaimed water;
(ii) Evaluate the utility of native (wastewater) coliphages and virus-like particles (VLPs) as indicators of pathogen transport and process performance during SAT;
(iii) Application of emerging monitoring methods to determine the presence/attenuation of pathogens such as adenoviruses, DNA viruses whose fate in the environment has not been studied in groundwaters replenished with reclaimed water;
(iv) Constitutive representation of virus attenuation during transport through the vadose zone and aquifer storage.

System representation and analysis

(i) Fitted mathematical relationships between the concentrations of specific water quality indicators (dependent variables -- e.g., organic contaminants, nitrogen species and pathogens) and independent variables such as transport distance/time, soil type, and level of above-ground treatment.
(ii) Methods for finding least-cost means for satisfying groundwater quality criteria through a combination of above-ground and soil-aquifer treatments, well location and withdrawal policy.
(iii) Application of water quality relationships and optimization routines for design/operation of several infiltration sites under evaluation in this project.

□
OPERATING THE SWEETWATER RECHARGE FACILITIES

Marie Light, Bruce Prior, and Peter Chipello

Abstract

The Sweetwater Recharge Facilities have been recharging reclaimed water or secondary effluent in shallow basins for the last ten years. The stored water is recovered on an annual basis to supply non-agricultural irrigators in the Tucson metropolitan area. Changes in the recharge operating plan have impacted the facility's ability to meet reclaimed water demand and the aquifer's water quality. While hydraulic loading rates have declined reducing the annual recharge volume, soil aquifer treatment has increased significantly with respect to nitrogen removal. In response to increasing demand for reclaimed water, the City of Tucson is in the process of expanding this project. The expanded project combines constructed wetlands, recharge basins, and public use/educational amenities.

Introduction

The City of Tucson has been using reclaimed water instead of groundwater for non-agricultural irrigation since 1983. Reclaimed water is filtered and disinfected secondary effluent. The Sweetwater Recharge Facilities supplies water through the Reclaimed Water System to golf courses, parks, schools, cemeteries, and street medians. These facilities provides both water quality improvement and temporary storage of secondary effluent. Four elements comprise these facilities: pipelines, wetlands, recharge basins, and wells (Figure 1). Reclaimed water was delivered to the recharge basins until January 1994 when secondary effluent was delivered. The water is recovered throughout the year with four extraction wells. Two more wells will be added to the recovery well field along with the expansion project.

2 Marie Light, Bruce Prior, and Peter Chipello, City of Tucson, Water Department, P.O. Box 27210, Tucson, Arizona 85726-7210. Phone: (520)791-2689; FAX: (520)791-3293.
Figure 1. Site Map for Sweetwater Recharge Facilities

The source water for the facilities is secondary effluent produced by the Roger Road Wastewater Treatment Plant (RRWTP). Booster pumps transport the water from RRWTP to both the Reclamation Plant and to the recharge basins. The Reclamation Plant filters the secondary effluent with sand and anthracite media and then disinfects the water with chlorine. The periodic backwashing of the filters produces a very high turbidity water which requires additional treatment. Four recharge basins totaling 13.8 acres were built on the west side of the Santa Cruz River in 1989. An additional four recharge basins totaling 14 acres are being completed on the east side of the river. A constructed wetland, 18.2 acres in size, is also being built to treat the backwash water and to provide public use and educational amenities.

The water table is about 120 feet below land surface and the regional gradient is 0.003 to the north northwest. The water takes about one to two weeks to reach the water table once it is discharged into a recharge basin. The sediments in the basins are sand to silty sand with some gravelly areas. The vadose zone is comprised of unconsolidated sediments ranging in grain size from sandy gravels to small clay lenses.
Operational Plan

The recharge basins are operated in wet/dry cycles to maintain the infiltration rates. During a wet cycle, the depth of water in a basin is maintained at about 1.5 feet. After each basin is filled to a prescribed depth, water is added several times a day to maintain the water depth. In the early years of operation, wet/dry cycles ranged from 5/7 days to 2/4 days (Light, 1993). The longer wet cycles were used during the winter months when algal growth is typically inhibited by cooler temperatures.

In August 1994 the wet/dry cycles were changed to 2/3 days. This wet/dry cycle was applied throughout the cool and warm months of the year to allow a uniform operation of the Reclamation Plant. The pressure in the common pipeline between RRWTP and the Reclamation Plant fluctuated dramatically when flow to a basin was altered. The flow rate can be maintained at a constant rate by filling a basin for a two day period and then filling the next basin. By the time the last of the basins is filled to capacity, the water in the first basin has infiltrated and the soils have dried sufficiently to begin another wet cycle. As time proceeded, however, the wet cycle increased because the water took longer to infiltrate. At first the basins infiltrated all the water within one day. By the middle of 1996, the water took three to five days to infiltrate after being discharged to the basin. The number of wet/dry days was lengthened to 4:5 in 1995 and 6:7 in 1996. The wet/dry ratio is approaching 1 (Figure 2).

Basin maintenance includes annual plant removal, biennial disking, and ripping approximately every six years (Figure 2). Plant removal reduces the organic material in the basins and exposes the soil to direct sunlight. Disking turns the soil to a depth of about 12 inches. Ripping fractures soil structures that have developed to a depth of at least 30 inches. All maintenance is scheduled after a two month period of drying which typically occurs in July and August. The soil must be sufficiently dry to allow the heavy equipment to enter the basins and to pull the disking and ripping equipment without losing traction or getting stuck.

Reclaimed water was recharged in the basins on the west side of the Santa Cruz River until January 1994. The Reclamation Plant removes approximately 50% of the suspended material contributing to turbidity. Therefore, when secondary effluent was discharged into the basins, the suspended material doubled.

Impacts of Changes in the Operating Plan

During the pilot phase of this project the annual recharge volumes were less than 500 acre-feet per year (Figure 3). After the western basins became fully operational in 1991, they recharged between 2500 and 3200 acre-feet per year. The annual recharge volume has declined since the shorter wet/dry cycles were implemented. The infiltration
Figure 2. Annual average of wet to dry days and basin maintenance

Figure 3. Annual recharge volumes and average infiltration rates
rate did not appear to drop between January and August of 1994 when secondary effluent was first delivered. Hydraulic loading rates declined from 234 feet per year in 1994 to 186 feet per year in 1996.

While the infiltration rates have been declining, the water quality in the aquifer has been improving. Before recharge activities started in 1986 the Nitrate as Nitrogen concentration in the aquifer averaged 14.8 mg/L (Figure 4). The spatial distribution of the nitrogen in the aquifer was also very uneven (Figure 5). Soil aquifer treatment studies performed in 1992 and 1993 showed the nitrogen removal rate ranged between 30% and 36% (Santerio, 1992; Ward, 1993; University of Arizona and University of Colorado, 1992). The average Nitrate as Nitrogen concentration in the aquifer remained above 10 mg/L until the infiltration rates declined and secondary was discharged into the recharge basins. The average Nitrate as Nitrogen concentration in the aquifer in 1996 was 5.3 mg/L (Figure 4 and Figure 6). The nitrogen removal rate in 1996 was 74%. A comparison of the areal distribution of the nitrogen concentrations in the aquifer between 1986 and 1996 shows a significant reduction of the nitrogen concentrations. The nitrogen removal rate has been observed to be related to the hydraulic loading rate (Bouwer, 1991).

Figure 4. Comparison of Nitrogen Concentrations in Groundwater and Recharge Water
FIGURE 5. 1986 Distribution of Nitrate Concentration (mg/L) In Monitor Wells

FIGURE 6. 1996 Distribution of Nitrate Concentration (mg/L) In Monitor Wells
Expansion to include wetlands

The City of Tucson is completing construction of a wetland and recharge basins on forty-five acres south of the Reclamation Plant. At the Reclamation Plant, where secondary effluent is received from RRWTP, variations in the secondary effluent turbidity levels up to 30 NTU's decrease the efficiency of the tertiary Anthracite/Silica filters. This leads to increased backwashing of the filters. The Reclamation Plant now pumps the backwash water back to the RRWTP for treatment. The Sweetwater Wetlands is designed to treat both secondary effluent and backwash filter water from the Reclamation Plant. This expansion will include 1.2 acres of settling basins, 17 acres of polishing basins and 14 acres of recharge basins. The settling basins and polishing basins comprise the wetland.

The influent to the wetland will flow from two splitter structures to the settling basins at a maximum rate of 1.6 million gallons per day (MGD). From there the water will flow by gravity through the polishing basins and into two recharge basins (RB-005A and RB-006A). There is a 16 foot elevation change from the bottom of the settling basins to the bottom of the recharge basins. The settling basins and recharge basins are approximately 500 feet apart at their nearest points. These two recharge basins will take either polished water or secondary effluent. The eastern two recharge basins (RB-007A and RB-008A) will receive pumped secondary effluent. This will allow recharge operations using secondary effluent to progress regardless of the flowthrough at the wetland.

The design layout for the wetland and recharge basins took advantage of the Santa Cruz River's surficial flood plain deposits and the underlying course channel deposits. Surface sediments in the area of the settling basins and polishing cells are comprised of 20 feet of fine silty, clay overbank floodplain deposits. The settling basins and polishing cells are constructed within this fine grained surficial deposit. Dual-ring infiltrometer tests across the polishing basins produced an average infiltration rate of 0.02 ft/day which should reduce to even slower rates once the wetlands are fully developed. Beneath the silty clay lies a channel deposit of coarse cobbly sand. Excavation on the south side of the property exposed this coarser unit on the bottom of the recharge basins. Based on operation of recharge basins on the west side of the river, infiltration rates are projected to be approximately 1.6 ft/day. The annual recharge volume for the new basins is projected to be 4,000 acre-feet per year. These facilities are expected to have a recharge and recovery capacity of 6,500 acre-feet per year.

The utility anticipates a significant decrease in nitrate levels from microbial consumption and the uptake of nutrients by the emergent growth plants. Total Nitrogen is projected to be reduced within the settling basins from 25.5 mg/L to 22.5 mg/L. The polishing basins are expected to reduce the Total Nitrogen concentration to 3.1 mg/L. The settling basins are also projected to reduce the Total Suspended Solids (TSS) from 141 mg/L to 46 mg/L and the Biological Oxygen Demand (BOD) from 74.1 mg/L to 46 mg/L. Projections of TSS, BOD and Total Nitrogen are based on equations representing a first order, area-based model with
rate constants developed from data collected at existing wetlands and are conservative with respect to conditions in Tucson.

The wetland is designed to be a park-like feature which will encourage passive recreation and public awareness of the importance of creating new wildlife habitat in urban environments. The site includes public parking, restrooms, a kiosk, a ramada, and park benches along a walking path around the eastern polishing basin. Educational signage will be posted at intervals along the path describing the wetland treatment process, hydrology, and expected wildlife. There has already been a tremendous positive public response to the project for its environmental education potential. Additionally, the public will gain an appreciation of how wastewater can be used as a valuable water resource.

Conclusion

While the hydraulic loading rate has decreased from 234 to 186 feet per year, the nitrogen removal rate has increased from about 30% to 74%. The hydraulic loading rate decreased when the wet/dry cycle was shortened from 5/7 days to 2/3 days. Although the annual recharge volume has decreased, the recharge capacity will increased through the addition of four more recharge basins to 6,500 acre-feet per year. The increases in demand for reclaimed water can be met while reducing the nitrogen concentrations in the aquifer to below drinking water standards.

Acknowledgments

Special acknowledgment is made to Bruce Johnson for his support of research efforts to characterize changes in water quality at the recharge projects and for his efforts in obtaining the support of the City’s Mayor and Council to fund the research. Appreciation to the people preparing the graphics: Michael McCasland, Terry Miley, Nathan Miller, and Mare H. Yates.

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PREDICTING AND MANAGING INFILTRATION FOR ARTIFICIAL RECHARGE

Herman Bouwer

Introduction

The finer the soils that are available for groundwater recharge, the more important it is to get a good idea of the infiltration capacity of the soil so that land requirements and evaporation losses for planned recharge systems can be estimated with some confidence and the feasibility of recharge via surface infiltration can be assessed. Also, finer soils are more vulnerable to surface clogging than coarse soils, so that recharge systems in finer soils must be carefully designed and managed to minimize infiltration reductions. After infiltration, the water must be able to move freely down through the vadose zone to the aquifer, without being impeded by high groundwater levels that may be caused by rising perched water above less permeable layers in the vadose zone or by mounding of groundwater on the aquifer itself. Thus prediction of groundwater rises in response to recharge is important to determine the maximum hydraulic capacity of the system, the optimum layout and geometry of the infiltration area, and where and at what rate groundwater must be pumped to maintain adequate lateral flow in the aquifer away from the infiltration area.

Predicting infiltration

Sands and gravels are the preferred soils for groundwater recharge via surface infiltration systems. With clean water, such soils typically give infiltration rates 5 to 15 ft/day, and sometimes even more! Where such soils are not available, finer soils like fine sands, loamy sands, sandy loams, and light loams may have to be used. Infiltration rates for such soils may range from only about 0.3 to 2 ft/day. Often, such finer soils are available in large areas of alluvial valleys and plains. Even if the annual infiltration or hydraulic loading at continuous operation for such soils is only on the order of 100 ft/yr, the system would still be more than 90% efficient since evaporation losses range from about 6 ft/yr in warm dry areas like Phoenix, Arizona, to about 1 ft/yr in cooler, more humid climates like NW Europe.

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Because low-infiltration recharge systems require more land, they offer excellent opportunities for environmental and public benefits. The infiltration systems could then be designed as attractive aquatic parks with nature, scenic, and recreational amenities. If the infiltration area is relatively flat and only mildly sloping, the supply channel could be constructed as a meandering stream (possibly with some wetland sections) down the slope. Shallow infiltration basins would then be obtained with low berms on the contours (Figure 1). The berms would be constructed from soil on the uphill side, leaving a deeper channel that is kept filled with water during drying for survival of aquatic life, including mosquito fish. Rock piles in the supply channel would back the water up for flowing into the basins. Systems as in Figure 1 require minimum earth moving and give minimum intrusion into the land area. They could be attractively landscaped and equipped with walking and biking trails, picnic areas, and other facilities for maximum public enjoyment.

After promising soils have been identified by studying soil maps and doing on-site inspections, “wet” infiltration tests should be performed. These are typically done with metal, cylinder infiltrometers about one foot in diameter. However, use of such small infiltrometers can seriously overestimate the large-area infiltration rates because of lateral flow (divergence) around the cylinder due to capillary suction in the soil (Bouwer, 1986; Bouwer et al, 1997). Double ring or “buffered” infiltrometers are not the solution because the divergence also causes overestimation of infiltration in the center portion of the cylinder. The obvious approach then is to use larger infiltration test areas like, for example, 10 x 10 ft bermmed areas, where divergence effects are less significant. These tests are laborious and they can also take large volumes of water because it may take more than a day to reach or approach “final” infiltration rates. Another approach would be to use conventional single cylinders with significant water depth to speed up the infiltration process so that tests can be completed in a relatively short time (5 hours, for example). The resulting data are then corrected for water depth, limited depth of wetting, and divergence effects to get an estimate of the long-term infiltration rate for a large inundated area. This rate should be about equal to the hydraulic conductivity K of the wetted zone. The following procedure can be used:

The infiltrometers are single steel cylinders 24" in diameter and 12" high with beveled edge (Figure 2). A piece of 2x4 lumber is placed on top of the cylinder and the cylinder is driven straight down with a sledge hammer to a depth of about 1" to 2" into the ground. The soil is packed against the inside and outside of the cylinder with a piece of 1" x 2" furring strip that is held at an angle on the soil against the cylinder and tapped with a light hammer to get good soil-cylinder contact. If the soil contains clay, the water used for the test should be of the same chemical composition as the water used in the recharge project to avoid complications due to effects of water on status of clay. A dish or pie tin is placed on the soil for erosion prevention. The cylinder is filled to the top, and clock time is recorded. Water is allowed to drop about 2" to 5", the drop is measured with a yardstick, clock time is recorded, and the cylinder is filled back to the top. This is repeated for about 6 hours or when the accumulated infiltration has reached about 20", whichever comes
first. The last drop \( y_n \) is measured and clock time is recorded to get the time increment \( \Delta t_n \)
for \( y_n \). A shovel is used to dig outside the cylinder to determine the distance \( x \) of lateral wetting (divergence, Figure 2)). The infiltration rate \( i_n \) inside the cylinder during the last water level drop is calculated as \( y_n / \Delta t_n \). The corresponding downward flow rate or flux \( i_w \)
in the wetted area is then calculated as

\[
i_w = \frac{i_n \pi r^2}{\pi (r + x)^2}
\]

(1)

The depth \( L \) of the wet front at the end of the test is calculated from the accumulated drop \( y_i \) of the water level in the cylinder as

\[
L = \frac{y_i \pi r^2}{n \pi (r + x)^2}
\]

(2)

where \( n \) is the fillable porosity of the soil. The value of \( n \) is estimated from soil texture and initial water content. For example, \( n \) may be about 0.3 for dry uniform soils, 0.2 for moderately moist soils, and 0.1 for relatively wet soils. Well-graded soils would have lower values of \( n \) than uniform soils. The value of \( L \) can also be determined by digging down with a shovel right after the test and seeing how deep the soil has been wetted. This works best if the soil initially is fairly dry, there is good contrast between wet and dry soil, and there are no rocks. Applying Darcy’s equation to the downward flow in the wetted zone then yields

\[
i_w = \frac{K z + L - h_{we}}{L}
\]

(3)

where \( z \) is the average depth of water in the cylinder during the last water level drop. The term \( h_{we} \) is the water-entry value of the soil and it is used to estimate the suction at the wet front as it moves downward. The water-entry value is about half the air-entry value (due to hysteresis) and may be estimated as follows (in inches of water):

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>coarse sands</td>
<td>-2</td>
</tr>
<tr>
<td>medium sands</td>
<td>-4</td>
</tr>
<tr>
<td>fine sands</td>
<td>-6</td>
</tr>
<tr>
<td>loamy sands - sandy loams</td>
<td>-10</td>
</tr>
<tr>
<td>loams</td>
<td>-14</td>
</tr>
<tr>
<td>structured clays (aggregated)</td>
<td>-14</td>
</tr>
<tr>
<td>dispersed clays</td>
<td>-40</td>
</tr>
</tbody>
</table>
Since \( K \) is now the only unknown in eq. 3, it can be solved for \( K \) as

\[
K = \frac{i_w \cdot L}{(z + L - h_{we})}
\]

(4)

The calculated value of \( K \) can be used as an estimate of long-term infiltration rates in large and shallow inundated areas, without clogging of the surface and without restricting layers deeper down. Because of entrapped air, \( K \) of the wetted zone is less than \( K \) at saturation, for example, about 0.5 of \( K \) at saturation for sandy soils, and about 0.25 of \( K \) at saturation for finer soils. If the \( K \)-values calculated with eq. 4 look promising for an infiltration system, the next step is to put in some test basins of about one acre for long-term flooding to evaluate clogging effects and potential for infiltration reduction by restricting layers deeper down. Good agreement has been obtained between predicted infiltration rates (\( K \) in eq. 4) and those of larger basins (Bouwer et al., 1997). If the infiltrometer tests give infiltration rates that are too low for surface infiltration systems, alternate systems such as excavated basins, recharge trenches, recharge shafts or vadose zone wells, or aquifer wells can be considered (Bouwer, 1996).

The above procedure is by no means exact. However, in view of spatial variability (vertical as well as horizontal) of soil properties, exact procedures and measuring water level drops with vernier equipped hook gages are not necessary. The main idea is to account somehow for divergence and limited depth of wetting, rather than applying a flat reduction percentage to go from cylinder infiltration rates to long-term large area infiltration rates. Because of spatial variability, cylinder infiltration tests should be carried out at various locations within a given site. Finally, the resulting infiltration rates should never be expressed in more than two significant figures!

**Hypothetical example.** A cylinder 24" in diameter and 12" high was driven 1" into a relatively dry sandy loam soil. The following time and drop-of-water level data were recorded:

- 0800 filled with water to top
- 0830 water dropped 4", cylinder refilled to top
- 0900 water dropped 3", refilled to top
- 1000 water dropped 5", refilled to top
- 1100 water dropped 4", end of test

Digging with a shovel showed a lateral wetting of 12" outside cylinder, \( n \) was taken as 0.2 and \( h_{we} \) as -10". The value of \( i_w \) is 4/1 = 4" / hr which when substituted into eq. 1 yields \( i_w = 4 \pi 12^2 / \pi (12 + 12)^2 = 1" / hr \). The value of \( y_i \) is 16", which when substituted into eq. 2 yields \( L = 16 \pi 12^2 / 0.2 \pi (12 + 12)^2 = 20" \). Since the average water depth in the cylinder during the last measured water level drop was 11 - 4/2 = 9", \( K \) is calculated with eq. 4 as \( 1 \times 20 / (9 + 20 + 10) = 0.5" / hr \). Thus, the last measured cylinder infiltration rate of 4" / hr is reduced to 0.5" / hr to eliminate effects of divergence, limited
depth of wetting, and water depth in cylinder to produce a value that can be used to predict potential infiltration rates for extended inundation of large areas.

**Controlling clogging**

Finer soils in recharge basins are more vulnerable to clogging than coarser soils because pore sizes are smaller and easier to block by suspended solids. Thus, water going into these basins must be as free as possible from suspended solids. Even then, biofilm growth on the soil particles could significantly reduce effective pore diameters and, hence, infiltration rates. Also, clay and other small soil particles could migrate downward due to the “seepage force” of the infiltrated water and accumulate a small distance (often only a few mm or less) below the surface where it can form a “micro” clogging or restricting layer. This process, called fine particle movement or wash-out wash-in, is an important factor in soil crusting due to rainfall and has been well documented in the soils literature (Sumner and Stewart, 1992). Because of this fine particle movement, drying of basins for infiltration recovery and periodic removal of the cracked and curled up clogging layer may have to be followed by shallow disking or harrowing to break up any wash-in layers that may have formed some small distance below the surface. This leaves a rough surface and loose soils so that when the basins are filled again the surface soil may cave and slough, causing soil to be stirred up and become suspended. This soil would then move with the water, and settle out to start a new clogging layer on the soil surface. Also, as water rapidly infiltrates when it first enters the loose soil, fine-particle migration may be enhanced and the fine particles could migrate further down and accumulate on the underlying undisturbed soil, where they could form a mini-restricting layer. This layer may then be at a depth of 5 to 15 cm below soil surface, depending on how deep the soil was disked or harrowed. This would require deeper disking, harrowing, or even ripping the next time the basin is dried and cleaned, etc. To prevent this situation from developing, the basin after disking or harrowing may have to be smoothed and somewhat compacted, as achieved by rolling, so that surface soil is not stirred up when the basin is filled and fine particles cannot as readily move downward through the soil.

If infiltration basins are constructed on sand or gravel, erosion of the banks (wave action, sloughing) or bottom (splashing at inlet and overland flow when filling) is not all that critical for infiltration because the suspended material quickly settles out and forms a sand layer on top of the original sand. This layer probably still is quite permeable and may have a minor if any effect on infiltration. However, if the same erosion happens to a basin in fine soil, the suspended material settles out more slowly and there will be segregation of particles. The fine sand particles will settle out first, followed by silt particles which then will be topped with a blanket of clay particles that can significantly reduce infiltration rates. Because of this, infiltration basins in fine soils must be designed and managed to completely avoid soil erosion. Berms or dikes must be compacted to maximum density and must have mild slopes that are vegetated or otherwise protected against erosion by rain or wave action. Also, when the basins are filled, water should be admitted very
slowly to avoid erosion by splashing and overland flow. In the system of Figure 1, this can be achieved by filling the basin from the survival channel, which is on the contour and thus can fill the basin over its entire length with minimal flow per unit length of basin. Thus, survival channels serve a dual purpose: protection of aquatic life during drying and even distribution of inflow during filling. Shallow basins with water depths less than about 10" generally are preferable to deep basins because the clogging layer then tends to be looser and, hence, more permeable than under larger water depths where it may be compressed by increased intergranular pressures (Bouwer and Rice, 1989). Shallow basins also dry sooner by infiltration after turning off the inflow when decreasing infiltration rates indicate the need for infiltration restoration by drying and possibly cleaning of the bottom.

The role of vegetation in basins still is not clear. While it may aggravate insect problems, it also provides shade which could reduce algae growth in the water and on the bottom and, hence, clogging. Root activity may help keep fine soils "open" and, hence, more permeable. Dead vegetation that accumulates on the soil (thatch, detritus) may protect underlying soil against blockage of soil pores by suspended solids. However, vegetation also retards drying and recovery of infiltration rates. This may cause problems in the winter which already is a critical period for infiltration because lower temperatures increase the viscosity of the water which gives lower infiltration rates.

Some recharge systems must be able to accept a certain inflow, for example, sewage effluent that cannot be discharged somewhere else. Where the total basin area is barely sufficient to handle the inflow, operators of the system then tend to frequently disk or harrow the basins to break up clogging layers and to keep infiltration rates as high as possible. However, this can give soil compaction problems deeper down on sensitive (finer-textured) soils, which leads to reduced infiltration rates. To maintain system capacity, basins then must be flooded longer and dried shorter, which reduces infiltration recovery until eventually all basins must be filled to accept the inflow with no opportunity for drying. Infiltration rates then continue to decline due to increased clogging and eventually the whole system fails and drastic measures must be taken.

If the soil contains a lot of large stones, diskling or harrowing is not possible and the soil must be ripped to restore infiltration rates. Ripping, however, causes stones to move upward through the soil profile so that frequent ripping eventually produces a surface that is almost completely covered by stones, like an armored streambed. Infiltration rates then will be low, because infiltration cannot take place where the soil is covered by stones. Infiltration then can occur only between the stones, where suspended solids and other clogging materials accumulate on the soil between the stones from where they can not be removed by drying and scraping. The only solution then is to remove the stones and expose as much of the soil surface as possible.
Controlling groundwater effects

Infiltration rates also are reduced when groundwater levels below the recharge area rise too high. In most systems, infiltration is controlled by a clogging layer on the bottom and the soil in the vadose zone below the clogging layer is unsaturated. In that case, there is no direct hydraulic connection between basin and aquifer and infiltration rates are unencumbered by groundwater level as long as the top of the capillary fringe above the groundwater table is below the bottom of the basin. Static capillary fringes in permeable soils are less than about 2 ft thick. Allowing for some additional thickness due to downward flow, groundwater levels thus could rise to as high as about 3 to 5 ft below the bottom before they start to reduce infiltration rates.

If there is no clogging layer on the bottom, as can be expected for ultra clean water and basins in coarse sands or gravels, there is direct hydraulic continuity between the water in the basin and the groundwater. The groundwater table will then rise up to the basin and connect with the water surface in the basin (Bouwer, 1978, and references therein). If the groundwater level then is relatively high, the flow below the water table will be mostly lateral and controlled by the slope of the water table (hydraulic gradients much less than one). In this case, the flow in the aquifer and, hence, the infiltration and outflow from the basin will vary linearly with the slope of the water table (or with depth to groundwater at some distance from the basin, as measured vertically down from the water level in the basin). In that case, rising groundwater levels will reduce infiltration rates. If, on the other hand, the groundwater level is relatively deep, the outflow from the infiltration basin will be mostly downward and controlled by gravity (hydraulic gradient about one). In that case, rising groundwater levels will not reduce infiltration rates. The depth to groundwater (as measured from the water surface in the basins) where the flow system changes from gravity controlled to slope-of-the-watertable controlled is equal to about twice the width of the entire infiltration area (Bouwer, 1990). This applies to uniform, isotropic underground formations. Anisotropic or stratified situations need to be considered on a case-by-case basis.

Groundwater rises below recharge areas can be controlled by the hydraulic capacity and size and geometry of the recharge system, and by groundwater abstractions from the aquifer. Also, there can be perched groundwater mounds that develop on layers of lower permeability in the vadose zone. Where infiltration is controlled by a clogging layer on the basin bottom, the basin infiltration rate i will be much less than the saturated hydraulic conductivity \( K_s \) of the vadose zone and, hence, of the soil in the perched mound. If the hydraulic conductivity \( K_r \) of the restricting layer then is much less than \( K_s \), the height \( L_p \) of the perched mound above a restricting layer of thickness \( L_r \) can then be estimated as \( i L_r / K_r \). This is a simplified version of the general equation for \( L_p \) based on applying Darcy's equation to the downward flow in the perched layer and through the restricting layer (Bouwer et al, 1997). If the calculated \( L_p \) is large enough to reduce infiltration rates, the recharge basins can be spread out over a larger area to reduce the regional value of i, or
they can be arranged in a long, narrow strip to create two-dimensional flow in the perched mound. The lateral flow then reduces the height of the mound (Bouwer, 1978).

Several analytical procedures have been developed to calculate the rate of rise of the groundwater mound on the aquifer itself (Warner et al., 1989). The equations, such as the Hantush or Glover equation (Bouwer, 1978, and references therein) can be used to determine short or long-term rises of the groundwater mound in the absence of any other recharges to, or discharges from, the aquifer. For shallow water tables, this is useful to predict how long the basins can be flooded before they need to be dried again to allow recession of the mound. For deep water tables, the equations can predict how long recharge can continue and how much water can be stored or "banked" underground before the groundwater mound rises too high and abstraction of groundwater farther away is needed to stabilize groundwater mounds below the recharge area at acceptable levels.

Mound rises on the aquifer can be reduced by reducing i (recharging less water or spreading the basins out over a larger area to reduce regional i), or by arranging the basins in a long, narrow strip rather than in a more square or round area. Equations have been developed to predict at what distance groundwater pumping must occur, and how deep the groundwater must be maintained at that distance, to create a steady-state flow system where recharge and abstraction are in equilibrium and the groundwater mound in the recharge area is sufficiently low (Bouwer et al. 1997). Sometimes, maximum permissible groundwater mound rises are determined by factors other than maximizing infiltration rates in the recharge system, such as presence of solid waste disposal areas (landfills), sewers or other underground pipelines, deep basements, building foundations, cemeteries, etc. The calculations of mound heights and ultimate abstraction requirements will be useful in initial feasibility studies and planning. More refined estimates can be obtained by computer simulation of the entire recharge and recovery system, using MODFLOW or other model. Then, when the project is built, it should be constructed in phases so that the experience obtained with the first phase can be used to refine the design and management criteria for the second phase, etc. The performance of groundwater recharge systems is so site specific that a complete project with guaranteed hydraulic capacity can never be designed at once. Since it almost takes a recharge system to design a recharge system, the preferred approach is to start small and simple, learn as you go, and expand as needed!
REFERENCES CITED


Figure 1. Schematic of meandering supply stream (top) with rock dams (dots) and outlets (small bars) to basins that are formed by berms on the contours, and of cross-section AA (bottom) with contour berms, infiltration basin upslope from berm, and survival channels.

Figure 2. Geometry and symbols for single-ring infiltrometer.
RECHARGE AT THE KEN MCDONALD GOLF COURSE, TEMPE, ARIZONA

Thomas D. Ankeny, Eric Kamienski, and Greg L. Bushner

ABSTRACT

The City of Tempe (COT) is in the process of developing a Constructed Underground Storage Facility (USF) at the Ken McDonald Golf Course located in South Tempe. Once completed the Ken McDonald Golf Course Recharge Facility will store high quality reclaimed water from the Kyrene Water Reclamation Facility. This water will be conveyed from the treatment plant to the recharge facility, where it will be injected into the Upper Alluvial Unit (UAU) aquifer at the site.

Initially, the COT will store up to 1.5 million gallons of water per day (mgd) using the vadose zone recharge well method. This method incorporates wells that are designed specifically to inject water into the vadose zone above the water table. Each recharge vadose zone well at the Kyrene Recharge Facility will be designed to recharge between 565 to 645 acre-feet/year (ac-ft/yr), at rates ranging from 350 to 400 gallons per minute (gpm). These rates are based on the pilot recharge testing conducted at the site. Ultimately, this site may be used to store an average of 3 mgd of high quality reclaimed water from the Kyrene Water Reclamation Facility.

A pilot test was recently conducted on vadose zone recharge well RW-1. This well was oversized and equipped to operate at a potential recharge flow rate of approximately 700 gpm. During the pilot test, RW-1 achieved a flow rate that ranged between 540 to 430 gpm, and averaged 435 gpm. Water levels within the well casing stabilized between 25 to 30 feet for the majority of the test period. Groundwater levels at the monitor wells rose approximately 1 to 1 ¼ feet. The monitor wells were strategically placed to characterize the mounding that occurred as a result of the recharge and any changes in groundwater quality due to the recharged water.

The pilot test confirmed that the vadose zone recharge well method will work for the Ken McDonald Golf Course Groundwater Recharge Facility (KMGCRF), and that the unsaturated materials

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at this location will readily accept the recharge water. Based on the average flow rate of 435 gpm, to achieve a recharge facility capacity of 4 mgd, six additional vadose zone recharge wells will be required (without backup capacity).

**INTRODUCTION**

The KMGCGRF is located in south Tempe on the Ken McDonald Golf Course near Rural Road, between Elliot and Guadalupe Roads. The project site encompasses an area that includes the Ken McDonald Golf Course and the adjacent Salt River Project (SRP) Kyrene Generating Station. *Figure 1* shows the locations of the monitor wells (COT-1, COT-2, and COT-3), the Ken McDonald Golf Course irrigation well (production well), and the vadose zone recharge well (RW-1). Monitor well COT-1 is located between fairways for the 3rd & 9th holes, whereas monitor wells COT-2 and COT-3 are located on SRP property. *Figure 1* also shows two proposed vadose zone recharge wells and potential locations for additional monitor wells.

**SITE GEOLOGY**

The site geology has been described based on the drillers and geophysical logs of the monitor wells, recharge well, and the production well. *Figure 2* is a schematic cross section showing the local lithology of the UAU based on the cuttings returns and the geophysical logging of the vadose zone recharge well and monitor wells.

The upper most portion of the project site from land surface to approximately 30 feet is predominantly composed of silty sands, followed by a thin layer of clayey sands and sandy clays to about 60 feet below land surface. The target zone for recharging is found below this and is composed of poorly graded gravels with a gravel sand mixture and interbedded with some finer clayey gravels. This zone ranges from 60 to approximately 130 feet below land surface. The recharge target zone, consisting of poorly graded gravels, and gravel-sand mixtures is approximately 70 feet thick in the vicinity of the recharge well RW-1. This grades into a zone that consists of finer materials consisting of more clay, silt, and sand to a depth of 250 feet. The water table was at approximately 95 feet below land surface after these wells were drilled during the fall of 1994; groundwater flow direction is to the southwest.

**RECHARGE TESTING**

The COT received a permit from the Arizona Department of Water Resources (ADWR) to operate the KMGCGRF from April 22, 1996 through April 22, 1998. The Underground Storage Facility (USF) permit allows the COT to store up to 1,000 acre-feet of water from April 22, 1996 to April 22, 1998, and the Water Storage permit allows the COT to store water (CAP) from their potable water supply at this facility during the pilot testing phase of the project. The operation of this facility was dependent on several conditions as stated in both the facility and water storage permits.
Ken McDonald Golf Course
Groundwater Recharge Project
Site Map Showing Existing
Well Locations and
Proposed New Well Locations

**EXPLANATION**
- ☐ Production Well
- ☑ Monitor Well
- ☑ Vadose Zone Recharge Well
- ☑ Proposed New Vadose Zone Recharge Well Sites
- ☐ Pond
- ☐ Turfed Area

HydroSystems, Inc.  Figure 1
Lithologic Cross Section B - B'
COT-3 - RW-1 - COT-1

EXPLANATION
- CL: Gravely Clay, Sandy Clay, Silty Clay
- GC: Clayey Gravels, Gravel-Sand-Clay Mixtures
- GC-CL: Clayey Gravels and Gravely Clay Mixture
- CP: Poorly Graded Gravels, Gravel-Sand Mixtures
- GP-GC: Gravel with Clay and Sand
- SC: Clayey Sands, Sand-Clay Mixtures
- SC-CL: Sandy Clay to Clayey Sand
- SM: Silty Sands, Sand-Silt Mixtures
- Approximate Static Water Level
- Perforated Interval

Figure 2
The groundwater recharge system for the KMGCGRF was designed with instrumentation and equipment to facilitate the collection of needed data during the 60 day recharge test to: (1) meet all of the permit conditions, and (2) support the full-scale recharge and APP permit applications. The instrumentation and equipment used during the recharge test is discussed below along with the testing procedure and data management.

EQUIPMENT AND INSTRUMENTATION

There are several components to the KMGCGRF that include: (1) the vadose zone recharge well, associated equipment and instrumentation that is housed in the well vault, (2) the three monitor wells, and (3) the data collection system associated with the well vault structure.

The vadose zone recharge well RW-1 is equipped with a McCrometer flow meter and flow sensor, pressure cell, and water level transducer. These components were all connected to the Remote Operations Controller (ROC) or data logger to record and store the data electronically. The flow meter used both manual and electronic measurement methods that could monitor real time rate data along with a manual back-up data system if needed. The pressure cell was used to monitor the real time pressure in the eductor line at the well head during the test. The well was also equipped with a manual pressure gage that was installed upstream of the flow meter. This essentially measured system pressure. The pressure transducer was used to measure water level changes in the vadose zone recharge well during the test. An access port in the top of the well head plate was used to sound water levels during the test in order to calibrate the pressure transducer.

The three monitor wells were each equipped with a pressure transducer to monitor changes in groundwater levels during the test. This allowed for continuous groundwater level data collection at the project site and provided baseline information before the test began. Each of the pressure transducers in the monitor wells were set at 100 feet below land surface. The pressure transducers are wired to a transmitter which transmits the signal to a converter at the monitor well that outputs an infra-red signal to a receiver located at the ROC adjacent to RW-1.

The COT chose to install dedicated bladder-type sampling pumps in each of the monitor wells to meet the groundwater sampling conditions set forth in the ADWR recharge permits. A submersible pump is utilized to purge three well casing volumes from the wells before sampling with the bladder-type pumps. These dedicated pumps were installed in time for the second round of samples taken during the pilot test.

There were several challenges the COT faced when developing the data collection system for the KMGCGRF. The site constraints included limited access due to the golf course (burying a cable was not allowed in the fairway), high tension power lines overhead, and monitor wells COT-2 and COT-3 were constructed on land owned by Salt River Project (SRP). The COT came up with an innovative solution to these constraints by using infra-red radiation to transmit the data collected at the monitor wells to the receiver mounted at the ROC.
The ROC stores the data collected from the recharge well (flow, pressure, and water level) and groundwater levels from the three monitor wells. All of these data were collected at five minute intervals. The data retention period was approximately three days. Data could be transmitted via radio to the COT’s SCADA system directly, or using a cellular modem that was installed along with the ROC and associated software, could be remotely obtained.

TESTING PROCEDURE

Prior to the start of the recharge test all equipment was pre-tested to ensure proper operation. Data recording devices were also tested to ensure proper operation. In addition to testing and calibrating the equipment before starting the test, water was transmitted through the Rain-for-Rent pipe in order to flush the pipe of any sediment that may have been present. Water was discharged to the Ken McDonald Golf Course lake/irrigation system.

The test began by opening a valve at a fire hydrant located at the Kyrene Water Reclamation Facility. A hose was attached from the fire hydrant to the main reclaimed water line that is normally used to transmit reclaimed water from the Kyrene Water Reclamation Facility to the Ken McDonald Golf Course. A temporary Rain-for-Rent pipe was used to convey the water from the end of the reclaimed line on the golf course to the recharge well RW-1. The isolation valve located on the Rain-for-Rent pipe and the butterfly valve located at the well head were both in the closed position. Once the Rain-for-Rent pipe was completely full of water, the isolation valve was opened slowly. Completely filling the Rain-for-Rent pipe with water allowed air that might be present to escape through the air-relief valves located in the piping train. Opening the isolation valve allowed water to flow into the vault structure and through the flow meter where it was stopped by the butterfly valve. Once water reached this point the water pressure could be measured from the pressure gage upstream of the flow meter. The butterfly valve was slowly opened to allow water to flow into the eductor system and into the recharge well RW-1. To end the test or turn off the well, this process was reversed.

RECHARGE TESTING RESULTS

The recharge test began on June 6, 1996, at approximately 1:50 pm and continued until August 6, 1996 at approximately 8:45 am when the fire hydrant was shut off, a period of 61 days. Using a water level transducer and flow rate meter, the resultant water level rise and recharge rate were monitored continuously in RW-1. Groundwater levels in the monitor wells were also monitored continuously during the recharge testing using water level transducers. Both field and office notes were taken during the test to monitor and record additional information not recorded by the data logger.

The test began at a flow rate of approximately 540 gpm and ended at 430 gpm. The average flow rate during the test was 435 gpm. By the end of the test 38,906,368 gallons of water were recharged into the UAU aquifer. This converts to 119.4 acre-feet of water that was stored based on the totalizing flow meter reading. This equates to 1.96 acre-feet per day or 0.64 mgd. The
projected future demand to store water at the KMGCGRF is estimated to range from 1.5 to 3.0 mgd with peak demands of 4.0 mgd. Using the higher demand number of 4.0 mgd and the average recharge rate of 435 gpm, six additional vadose zone recharge wells will be required to meet this peaking demand (with no backup).

During the test water levels rose a maximum of 51 feet in well RW-1, or 24 feet below land surface. Figure 3 is a graph showing depth to water and flow rate for well RW-1 versus time. At no time during the test did water levels rise above the 20 foot depth to water cut-off level. The well did operate in the zone between 20 and 30 feet below land surface, however.

During the first two weeks of operation, the water levels in recharge well RW-1 rose and continued an upward trend. It was necessary to adjust the flow rate to reduce the upward trend in water levels. The flow rate was reduced by approximately 75 gpm from 518 gpm to 443 on June 14, 1996 (Figure 3).

The overall performance of recharge well RW-1 for this initial pilot test resulted in a recharge capacity of 8.5 gpm/ft. This parameter was determined by dividing the average flow rate by the total head rise in the well, or rather 435 gpm divided by 51 feet. Recharge capacities for other similar vadose zone recharge wells within the East Salt River Valley ranged from a low of 2.9 gpm/ft to a high of 12.2 gpm/ft, and averaged 6.7 gpm/ft. Well RW-1 performed well within the range of values for other similar vadose zone recharge wells. This comparison was provided to give the COT a sense of performance capability of well RW-1. Although these wells and tests are similar, there are significant differences between site conditions, well construction, and operational parameters. The COT will have better site-specific data for comparative purposes after the installation and testing of additional recharge vadose zone wells at the KMGCGRF.

MONITOR WELL RESPONSE

Each of the monitor wells responded during the recharge test of well RW-1. This is shown in Figure 4, a graph of adjusted depth to water for RW-1 and depth to groundwater for each monitor well over time. This graph includes data from before the test began and extends until after the test ended.

Almost immediately from the start of the recharge test, groundwater levels in the monitor wells began to rise. Each monitor well showed the same trends and tracked almost identically. A groundwater sampling round was conducted at the monitor wells just prior to the start of pilot recharge testing. Figure 4 shows that COT-1 was the first well to be impacted from the recharge test. COT-1 is also the closest well to the RW-1, a distance of 359 feet. COT-1 showed a rise of approximately ½ foot over a four day period. COT-2 and COT-3 did not show a distinct upward trend in water levels from the start of the recharge test, rather these monitor wells responded with a gradual continuous rise in water levels.
Graph Showing Adjusted Depth to Water and Flow Rate for Well RW-1 During the Recharge Test
June 6, 1996 (1:02 pm) to August 7, 1996 (8:11 am)

Flow rate adjustment - decrease
Flow rate adjustment - increase
Data logger problems
End of test, 8/6/96
Start of Test, 6/6/96
(Data dashed where inferred)

Time (hourly intervals after test began)

Figure 3
Graph Showing Adjusted Depth to Water for Well RW-1 and Depth to Water for Monitor Wells COT-1, COT-2, and COT-3 Before, During, and After Pilot Testing May 31, 1996 to August 14, 1996

- Start of Test, 6/6/96
- Data logger problems
- End of Test, 8/6/96

Figure 4
It also appeared that the three monitor wells were reaching a state of equilibrium a little more than halfway through the test on July 8-9, 1996 (~975 hours). It is unlikely that there was any pumping interference from nearby production wells because none were pumping at the time with the exception of SRP well 21.5E-1.0S. This well supplies coolant water to the adjacent SRP Generating Station and runs continuously. Although it is adjacent to the recharge project, it withdraws groundwater from the MAU and LAU and has little affect on UAU groundwater levels.

Groundwater levels in well COT-1 (southeast of recharge well RW-1) rose a total of 1.75 feet from 87.25 to 85.50 feet below land surface. Groundwater levels in well COT-2 (southwest of the recharge well) rose a total of 1.18 feet from 88.09 to 86.91 feet below land surface. Groundwater levels in well COT-3 (northwest of the recharge well) rose a total of 1.41 feet from 87.25 to 85.84 feet below land surface (Figure 4).

At the end of the test the resultant mound began to dissipate rapidly as shown in Figure 4. Groundwater levels in all three of the monitor wells decreased by approximately ½ to ⅓ feet within a week after the test ended. Monitor well COT-1 again responded the most rapidly of the three monitor wells and showed the most response to groundwater level dissipation. This downward trend continued until groundwater levels in the monitor wells reached the static water levels they were at before the test began.

PILOT TEST RESULTS

The pilot test of vadose zone recharge well RW-1 can be deemed a success, although this well was oversized and equipped to operate at a potential recharge flow rate of approximately 700 gpm. During the pilot test, RW-1 achieved a flow rate that ranged between 540 to 430 gpm, and averaged 435 gpm. Water levels within the well casing stabilized between 25 to 30 feet for the majority of the test period. Groundwater levels at the monitor wells rose approximately 1 to 1 ¾ feet. The monitor wells were strategically placed to characterize the mounding that occurred as a result of the recharge and any changes in groundwater quality due to the recharged water.

The pilot test confirmed that the vadose zone recharge well method will work for the KMGCGRF, and that the unsaturated materials at this location will readily accept the recharge water. Based on the average flow rate of 435 gpm, to achieve a recharge facility capacity of 4 mgd, six additional vadose zone recharge wells will be required (without backup capacity).
REGIONAL RECHARGE PLANNING
IN THE TUCSON ACTIVE MANAGEMENT AREA

Susanna Eden and Katharine Jacobs

Abstract: Artificial recharge of ground water has been identified as a key tool in addressing water management problems in the Tucson area. The Tucson Active Management Area (AMA) office of the Arizona Department of Water Resources has been coordinating an effort to overcome the various institutional and political limitations associated with artificial groundwater recharge. The regional recharge planning effort was conceived as a voluntary, consensus-based process in which interested parties work together to achieve common goals.

The first phase of plan development involved working with 22 hydrologists and engineers to produce a report representing a consensus view regarding a number of controversial technical issues associated with artificial groundwater recharge. In the second phase, a needs assessment was performed to determine the goals, objectives, and concerns with respect to recharge of the various potential partners in regional recharge projects. Plan development is proceeding on the basis of these two fact finding activities.

This paper will describe the Tucson AMA's regional recharge planning process, results, and implications for the future of artificial groundwater recharge within a regional water management context.

Background: The Tucson Active Management Area (AMA) covers 3,866 square miles in southeastern Arizona. It includes most of Pima County, small parts of Santa Cruz and Pinal counties, and five incorporated cities and towns, of which Tucson is the largest. The AMA also contains Tohono O'odham (San Xavier and Schuk Toak Districts) and Pasqua Yaqui reservation lands. There are two major groundwater sub-basins: the Avra-Altar Valley sub-basin, extending along the western side of the AMA from the Mexican border to Picacho Peak; and the Upper Santa Cruz Valley sub-basin, which crosses the AMA from south to north and includes the Rillito (Pantano and Tanque Verde) and Canada del Oro drainages.


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Total water use in the AMA was approximately 318,000 AF in 1995. Currently, annual natural recharge is estimated at 59,000 AF and incidental recharge is estimated at 77,000 AF. Therefore, more than half of the total water supply is mined groundwater.

Although substantial quantities of ground water exist in the aquifers, sustained groundwater mining in the AMA has major negative consequences. Falling groundwater levels have eliminated the free-flowing rivers, streams and associated riparian habitat in much of southern Arizona. The most productive portion of the aquifer is becoming depleted, with the likely result of decreased well productivity and increased pumping costs. The salinity of the groundwater tends to increase with depth, so continuing to mine ground water also will result in declines in the quality of water available for potable purposes. In addition, the risk of land subsidence increases with continued depletion of groundwater from Tucson’s Central, Southside, and Santa Cruz well fields, and is quite severe in the Central Wellfield. Aquifer compaction and associated land subsidence has already occurred in several areas of the AMA; fissuring has occurred locally near the CAP canal in the north Avra Valley, and sink holes have developed near the Santa Cruz River within the San Xavier District.

The Tucson AMA is one of five AMAs in Arizona established pursuant to the 1980 Groundwater Management Code. The Tucson AMA has a statutory goal of achieving safe-yield by 2025, which means that the amount of groundwater used in the AMA on an average annual basis must not exceed the amount that is naturally and artificially recharged. The goal of reaching safe-yield by 2025 cannot be achieved without augmenting the water supply.

The primary source for augmenting the Tucson AMA water supply is the Central Arizona Project (CAP). The City of Tucson’s CAP allocation is the largest allocation in the state: 148,200 AF. This allocation was intended to cover the demand of several other water providers in the region and allow for increasing demand associated with population growth. Allocations to other municipal providers, Indian Tribes, and the State Land Department within the AMA bring the total to 215,333 AF. By contrast, total use of CAP in the Tucson AMA was approximately 10,000 AF in 1995. Most of this use was agricultural.

The City of Tucson delivered CAP water to its customers from the fall of 1992 to the fall of 1994, but multiple problems with delivered water quality, including corrosion in older parts of the distribution system, caused the treatment plant and delivery system to be shut down until problems could be solved. On November 7, 1995, the citizens of Tucson approved Proposition 200, the Water Consumer Protection Act, which limits the ways in which the City’s CAP allocation can be used. The proposition prohibits delivery of CAP to potable water customers, unless the CAP water is treated to the same quality as Avra Valley ground water for hardness, salinity and dissolved organic material. This can only
be accomplished through advanced treatment, such as reverse osmosis. Because such techniques have never been applied at this scale, extensive engineering studies and pilot plant operation would be required prior to operation of an advanced treatment plant, should such a plant prove to be the community’s choice.

The political agreement that gave Arizona congressional authorization of the CAP in 1968 also gave the state the lowest priority allocation of Colorado River water among the Lower Basin States (California, Nevada and Arizona). Thus, Arizona’s allocation is the most likely to be affected by shortages on the Colorado River system, and the Tucson AMA is at the end of the CAP pipeline. Reliability planning for CAP was based on projections that the canal would be out of service to the Tucson AMA for 60 days per year. System capacity limits during peak demand months, however, may significantly increase outage risks. Direct delivery of CAP requires a reliability feature, originally envisioned as a reservoir to be built by the Bureau of Reclamation, with backup groundwater facilities provided by Tucson. The use of CAP water envisioned by Proposition 200 sponsors was recharge, agricultural use and industrial use, rather than direct delivery for potable use. Such a strategy reduces the need for reliability and may result in deletion of the terminal storage facility as a component of the CAP project. Vulnerability to CAP shortages has increased the importance of recharge as a buffer against future drought.

Although these physical factors are driving the need for recharge, other incentives dominate the decisions of most potential participants in recharge projects:

- The Central Arizona Groundwater Replenishment District (CAGRD) was created to allow new residential development to proceed in accordance with Assured Water Supply (AWS) rules. The CAGRD has a legal obligation to replenish ground water mined by its customers: water providers and subdivisions that expect growth within their boundaries.
- These same providers and subdivision developers also may choose to meet AWS requirements by recharging for themselves, outside the CAGRD.
- The Central Arizona Water Conservation District (CAWCD), the agency that operates the CAP canal and is responsible for Colorado River water deliveries to CAP subcontractors, must develop long-term reliability storage to “firm up” future deliveries to municipal subcontractors.
- Water providers with CAP allocations who currently are unable to take direct delivery of their allocation or deliver it directly to customers can recharge their allocations for annual and long-term storage credits.
- The Arizona Water Banking Authority (AWBA) will use recharge to accomplish its four objectives: 1) protect municipal CAP deliveries from supply shortages; 2) help accomplish local water management objectives; 3) facilitate Indian water rights settlements; and 4) provide for interstate water banking.
- The Tohono O’odham are studying recharge without recovery for riparian
enhancement, subsidence avoidance, groundwater level stabilization and containment of poor quality ground water.

Other entities may choose recharge to accomplish a variety of objectives, including offsetting municipal water conservation target violations or replenishing ground water mined to support turf (e.g., golf courses).

**Regional Recharge Planning:** As Tucson was struggling with problems associated with direct delivery of CAP and momentum was gathering for the Water Consumer Protection Act, long-standing disagreements, distrust and litigation divided various segments of the community from each other and made cooperation difficult. Late in 1993, the Santa Cruz Valley Water Replenishment District lost the political support of Tucson’s City Council. As a result, the only institution with regional authority for recharge dissolved, and progress on recharge lagged. Although some sub-regional cooperative recharge planning developed to fill the void, a region-wide approach was lacking.

At the state level, however, recharge related activities were accelerating. Plans took shape to increase the use of CAP water within Arizona. These plans included pricing incentives and the creation of the Arizona Water Banking Authority (AWBA). Without regional cooperation on recharge, the Tucson AMA was at a disadvantage in its ability to make use of such State programs.

With support from the Ground-water Users Advisory Council, the Tucson AMA stepped forward to initiate a regional cooperative process for recharge planning. A process was envisioned that would depend on the voluntary participation of two committees of representatives from a broad spectrum of interests. One committee would be composed of policy-oriented representatives, who would establish the principles and goals, shape and direct the process, and be instrumental in communicating the results to their respective publics. The other committee would be made up entirely of technical experts in fields related to recharge. They also would represent a wide variety of interests to ensure the objectivity of their technical conclusions.

The planning process was designed on these broad outlines: The technical committee was to resolve controversial technical issues to the extent possible with current information and identify where more information was needed. They were to identify potential recharge project sites, including groundwater savings facilities, develop criteria for evaluating technical and economic feasibility, and evaluate the sites on the basis of these criteria. The policy committee was to take the information provided by the technical committee and combine it with information about the goals, objectives, and constraints of the entities potentially involved in recharge in order to further evaluate, rank, and select projects to meet regional recharge needs. A public review process was to follow. The product of these activities would be a Regional Recharge Plan. The process was designed in this way so that the consensus on technical issues could inform the discussions of
political and institutional participants, and appropriate and proportional use of input from the technical and political groups would lead to a more meaningful public process.

The regional recharge planning process was initiated by the Tucson AMA’s Groundwater Users Advisory Council (GUAC) when they recommended hiring a recharge coordinator. After consultation with several knowledgeable individuals, a proposal was drafted to develop a regional recharge plan for the AMA. On November 3, 1995, the draft was circulated to about 20 representatives of institutions identified as instrumental in creating water policy. These individuals later formed the Institutional/Policy Advisory Group (IPAG). The document proposed the organization of the technical Regional Recharge Committee (RRC).

The process began with formation of this purely technical committee to ensure that the regional recharge planning effort would be based on sound information. To maintain productivity and minimize the effect of non-technical issues, RRC was limited to individuals with the appropriate technical credentials. The Tucson AMA initially invited 18 hydrologists, engineers and hydrogeologists from government, water providers, the University of Arizona and consulting firms to sit on the RRC. The original list was expanded because of interest and enthusiasm in the community. All participants donated their time to the process.

The RRC met regularly from January through July of 1996, in full committee and in subcommittees formed for specific tasks. Their objectives were to 1) achieve an understanding of the physical and institutional setting for recharge in the Tucson AMA, 2) develop siting criteria, 3) apply the siting criteria to potential recharge sites around the Tucson AMA, and 4) prepare a report on their results including identification of needs for further research/information.

Their first order of business was to define the physical and institutional issues in need of clarification within the committee and in the community at large. The RRC identified eight such issues. They included questions about opportunities and constraints on replenishing Tucson’s Central Wellfield, the normal range of recharge projects costs, effects of in-channel recharge on flood hazards and natural recharge, landfill risks, plume management, subsidence, and habitat development. Individual committee members or sub-committees produced study papers on specific questions and brought them back to the full committee for discussion and to develop consensus on these issues. When consensus was reached on each issue, conclusions were included in the committee’s report. All the findings published in the RRC report were reviewed and approved by the entire RRC. This fact is significant because some of their conclusions potentially could have a profound impact on how recharge is perceived in the community and consequently what recharge projects are pursued.
The next order of business for the RRC was development of criteria for siting recharge facilities. The criteria were intended to allow evaluation of projects for their ability to meet the dual goals of maximizing short-term (five years) volume of recharge and long-term benefits. The original site evaluation criteria were long-term storage rate, storage capacity, need for additional conveyance systems, impact on groundwater quality, annual cost for recharge, annual cost for recovery, annual cost for treatment, impact on groundwater level decline, extent of regional benefit, potential for recreational use, regulatory considerations, and time required to implement.

In a brainstorming session, the committee created a list of possible recharge sites that included existing, planned, investigated and completely conceptual projects. A subcommittee identified the sites that merited further investigation. Subcommittee members conducted an initial screening to eliminate from further consideration those sites with factors that rendered them unfavorable for implementation within the next 5 years. They then reviewed general and specific criteria to be used for evaluation of the remaining possible recharge sites, and agreed on assignments whereby each subcommittee member evaluated the sites based on one or more evaluation criteria in accordance with the member’s particular expertise.

A total of 34 potential recharge sites were initially evaluated. CAP was the source water considered in each case. Seven existing or possible groundwater savings facilities (GSF) were evaluated along with direct recharge projects. Of the 34 projects on the initial list, 16 were chosen to be evaluated in greater detail and included in the Committee’s report: 11 direct recharge projects and 4 GSFs. All of the projects suggested by the sponsors of the Proposition 200 received initial evaluation and three received fuller evaluations.

A joint meeting of the RRC and IPAG was held August 22, 1996, to create a smooth transition from the technical phase to the policy phase of the process. By this time, a final draft of the RRC report had been completed and sent to all members of both committees. It was agreed at the joint meeting that two indispensable components of a successful planning process would be institutional cooperation and public outreach. The RRC was tasked with completing and publishing their report and preparing an executive summary that could be used to inform institutional participants and elected officials. The RRC’s report was published and distributed in September 1996. More than 150 copies have been distributed, and response has been enthusiastic to both the report and the process by which it was developed.

The first task the IPAG undertook was to define the objectives and principles of the regional recharge plan. These were captured in a principles document designed to be distributed to elected officials.
The objectives of the Plan were:

- Provide a forum for regional cooperation regarding recharge activities
- Maximize the use of renewable water supplies in the Tucson AMA
- Optimize sharing of recharge, pumping and transmission facilities
- Expedite selection, testing and construction of groundwater recharge facilities
- Facilitate equitable access to recharge capacity
- Provide a background document for the facilities plan that will be required by the Arizona Water Banking Authority

The principles were:

- All entities are welcome to participate in the regional plan.
- It is essential to develop a united front in soliciting resources and funding from outside the Tucson AMA.
- All entities will be encouraged to support the Regional Recharge Plan and cooperate in meeting individual and regional water management needs.
- Participants are expected to pay project costs in proportion to benefit derived, to the extent this is feasible.
- Projects will be added to the Regional Recharge Plan as they meet established criteria.

In accordance with these objectives and principles, the IPAG wanted to base their planning activities on an inclusive assessment of the recharge-related needs of the people who are going to use the water. The Group identified the need to prepare a questionnaire and personally interview all parties interested in developing recharge projects to obtain information for a needs assessment. IPAG members suggested a list of entities to be interviewed for the needs assessment and emphasized that the focus of questions should be to reveal common goals and highlight points of contention so that they could be resolved.

An effort to inform elected officials about the process and to solicit their moral support occurred simultaneously with needs assessment interviews. In most cases, the responses of elected officials and their staff were positive. It was acknowledged that cooperation on recharge among the various entities in the Tucson AMA was desirable.

The attitude of interview respondents also was positive. Information for the needs assessment came primarily from these survey interviews, which were conducted from November 1996 through January 1997. The needs assessment survey was designed to elicit information about goals, concerns, operating constraints, recharge project involvement and interest, and assessments of the relevant issues associated with recharge. The survey was not intended to produce data for statistical analysis nor to establish a factually accurate picture of physical or legal/institutional conditions. An attempt was made to interview representatives from all entities likely to participate in recharge in the Tucson TAMA, and all but three of the 29 entities initially identified as likely participants
provided some information in response to the survey.

The entities that provided information for the Needs Assessment were state, federal, and regional agencies (4), local jurisdictions (4), municipal water providers: public (6) and private (4), agricultural water users (4), industrial water users (2), and Indian Tribes (2). Elected (and appointed) officials who received briefings included the Tohono O’odham Nation Water Resources Committee, City of Tucson Mayor and Council and City Manager, and Pima County Board of Supervisors and County Manager.

Just as there were technical issues that required clarification, there were a number of policy-related issues that required airing. These issues were raised in survey interviews, so the first analysis effort was to summarize the opinions of respondents on these issues. Issue descriptions were developed from responses. Unlike the technical issues, policy issues were not resolved in this process. Participants were asked only to agree on a fair description of them.

All interview survey responses were reviewed, along with audio tapes whenever such tapes had been made. Categories of responses were identified and responses were tabulated by category. Respondents’ goals associated with recharge were defined, and level of interest (primary, secondary, tertiary) in each goal by each responding entity was tabulated. The process was repeated for concerns about hazards associated with recharge. The entire survey of each respondent was then reviewed to ensure no information pertinent to goals and concerns had been omitted.

Issue descriptions, definitions of goals and risk concerns were developed entirely from interviews to faithfully represent the expressed opinions of respondents. Respondents were given the opportunity to review a draft of the needs assessment to correct any misinterpretation of their responses and add information or clarification.

Major objectives for regional recharge were defined by combining the respondents’ goals and concerns into logically consistent groups. A simple ranking and weighting method was used to order objectives. Responses were summed for each objective with the following weighting: primary 3 points, secondary 2 points, and tertiary 1 point. Ties were broken on the basis of number of entities naming that objective.

On the theory that there might be geographically identifiable areas of the AMA where recharge needs were not being addressed, objectives were clustered that indicated a desire to put wet water in a geographically identifiable location. Responding entities were divided into groups on the basis of their interest in those objectives. Northwest, City of Tucson, and southern (Green Valley) groups emerged. Because entities in all three of these areas already were progressing toward development of projects to meet their sub-regional needs, focus shifted to regional needs. The same method was used to group region-wide objectives.
After scoring and ranking regionally focussed objectives, ten top ranked objectives were identified as key. These are listed in their order of priority:

1. Reliable supply of wet water for future use, at the desired location, in the needed quantity, at the needed time
2. Most economical way to meet objectives, short-term = least cost, long-term = costs consistent with benefits
3. Storage credits to meet entity-specific needs, with certainty, at low cost, with equal access to all
4. Groundwater and drinking water quality protection from mobilization of contaminants, from degradation due to long-term recharge of “lower-quality” water, through soil-aquifer treatment, by phase-in of water quality changes
5. Riparian enhancement and/or nature-based recreation along natural stream channels and/or as part of planned systems of parks and/or trails
7. Local influence on decisions that affect land use and development and/or recovery of storage credits
8. Increased use of CAP water within the TAMA
9. Reduced groundwater mining
10. Increased use of effluent

Regional recharge project evaluation criteria were derived from these objectives. Each objective was redefined in terms of sub-objectives with measurable results, preserving all the detail of the needs assessment. Each sub-objective was then translated into one or more criteria. Sub-objectives that could not be met by project siting, design, or operations were dropped. Criteria were divided into two groups: those that apply to individual projects and those that apply to the Regional Recharge Plan.

The next step was to match physical evaluation criteria (from the RRC) with criteria developed from the needs assessment survey of entity goals. This permitted project evaluation based on its suitability for achieving project objectives. The sixteen projects evaluated last year by the RRC were chosen to undergo the first round of evaluation and ranking based on the regional objectives. Information from the RRC’s project evaluations was used to answer the questions how well or to what extent did the project meet each criterion. The evaluations were updated to the extent that new information had become available and all evaluations were reviewed by the IPAG members and their technical advisors. Methods for scoring and ranking projects on the basis of these evaluations were discussed at a meeting in May 1997.

Once projects are ranked, a draft plan, composed of several projects, will be evaluated on how well it meets Regional Recharge Plan criteria; unmet needs can be identified; and additional projects can be conceived to meet those needs.
The public review process as envisioned includes public workshops and other mechanisms for public review of the findings. If public review uncovers unmet needs or different goal priorities, the planning process is designed for flexible response to such new information, and can allow revisions and improvements as long as the voluntary commitment of participants persists.

The intention throughout this process has been to arrive at a procedure for ranking projects that was perceived as objective and fair by all participants. The participants represent extremely divergent interests. It was hoped that the results of a procedure perceived as fair and objective by these participants would be acceptable to the divergent interests. It will be the job of these participants in the next phase of the plan to build acceptance among their publics.

**Results:** The success of this endeavor ultimately rests with the community itself, and the willingness of individual water providers and political entities to cooperate in ensuring future adequate water supplies in the region. We are cautiously optimistic about our chances for success. For example, the AWBA, established by the State Legislature in 1996, purchases water for drought storage with funding from a tax on property in counties served by the CAP, which must be spent for the benefit of the county. When originally developing its 1997 operating plan, the AWBA was unable to identify sufficient recharge capacity to store water paid for by Pima County taxes in the Tucson AMA. The Regional Recharge Planning (RRP) process, then a year old, had not progressed far enough, fast enough to keep pace with outside events, but ad hoc cooperation by the same entities involved in the RRP led to revision of the operating plan to include recharge within the Tucson AMA. The AWBA now is relying on Tucson’s RRP to provide the basis for its Recharge Facilities Plan for the Tucson AMA.

Interest and involvement in the process has remained consistently high, and may be growing. Consensus among the participants is holding as we approach the point at which actual projects are ranked. Project rankings may be perceived to have real-world implications for political and financial support, so this is where the strength of any consensus will be tested. Among the challenges that lie ahead are describing the process and communicating the results to the community activists and groups who did not participate in the process.

**Implications:** Recharge occurs in a complex interrelated social/political/cultural setting. Productive debate requires a sound technical basis; but in highly charged debates, even the technical facts must be arrived at by an open consensus-based process among experts to ensure the results are credible. Looking at recharge from only the technical and legal perspectives ignores important factors that ultimately may be more relevant to achieving community goals. Voluntary consensus-based processes that specifically address the expressed preferences of potential participants may offer the best hope for success.
RESULTS OF THE HIGH PLAINS STATES GROUNDWATER DEMONSTRATION PROGRAM

H. Douglas Yoder and William Monheiser

Abstract: This paper summarizes activities that have take place under Phase II of the High Plains States Groundwater Demonstration Program, authorized by Public Law (P.L.) 98-434, as amended by P.L. 102-575. Phase I was completed in December 1987 by a report to Congress from the Secretary of the Interior recommending 21 projects for construction. Phase II, the construction, monitoring, and evaluation of the demonstration projects, was initiated in fiscal year 1989. The program is being carried out through cooperative agreements being executed with other Federal and State cooperators and local sponsors who are responsible for a minimum of 20 percent of project costs. This paper discusses the results of the projects completed to date.

Background

The High Plains States Groundwater Demonstration Program's purpose was to study the potential for artificial groundwater recharge in the 17 Western States and demonstrate artificial recharge technologies under a variety of hydrogeologic conditions. Demonstration sites were located in areas having a high probability of physical, chemical, and economic feasibility for recharge.

The High Plains States Groundwater Demonstration Program Act of 1983 (Public Law [P.L.] 98-434) was enacted on September 28, 1984. Funds were first appropriated in fiscal year (FY) 1985. As specified in the act, the program was carried out in two phases. Phase I was a 2-year site selection process during which the Bureau of Reclamation (Reclamation) worked cooperatively with the 17 Western States, the U.S. Geological Survey (USGS), and the Environmental Protection Agency (EPA) to select 21 demonstration projects for construction in Phase II. EPA has the additional responsibility of reporting to Congress at the conclusion of the program on the impacts of the demonstration projects on surface water and groundwater quality.

Although not specified in the authorizing legislation, coordination was also carried out with the U.S. Fish and Wildlife Service (Service) and State fish and game agencies to assure that any adverse impacts to fish and wildlife resources were mitigated and that opportunities to enhance wetlands and wildlife resources were developed, where practical, as part of the groundwater recharge demonstration projects.

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Phase I was completed on December 1, 1987, when the Secretary of the Interior forwarded the “Phase I Report to Congress” containing the recommendations of 21 projects for Phase II construction.

The Reclamation Projects Authorization and Adjustment Act of 1992 (P.L. 102-575) was enacted on October 30, 1992. Title XXVI, High Plains States Groundwater Demonstration Program, increased the appropriations ceiling for the High Plains States Groundwater Demonstration Program from the existing $20 million to $31 million. This title also clarified the reporting requirements of the act, the duration of Phase II of the program and permitted adjustments in the appropriation ceiling for fluctuations in construction costs. The additional ceiling funded modifications or extensions of existing projects to obtain better data, assure aquifer protection, and/or provide additional recharge cycles due to drought conditions. Figure 1 shows the location of the projects.

In 1993, auditors from the Office of Inspector General (OIG) reviewed the program to determine whether the legislative requirements and objectives were being met. Overall, the OIG found excellent progress in meeting the program’s legislative mandate of studying and demonstrating the potential for artificial recharge of aquifers. Accordingly, the OIG concluded that further involvement in the construction and operation of eight deferred demonstration projects was not needed to meet the intent of the program’s legislation. Therefore, the program was limited to funding of the 13 sites that were underway.

Environmental Compliance

The original 21 projects recommended in the Phase I report were subject to compliance with the National Environmental Policy Act (NEPA). A programmatic Environmental Assessment was prepared and a finding of no significant impact was determined.

Regulations

The major Federal statutory provisions relevant to the Program include the Safe Drinking Water Act (SDWA), the Clean Water Act (CWA), and NEPA. Other relevant statutes and regulations included:

- Federal agency adherence to State wellhead protection programs
- Emergency powers to prevent imminent and substantial endangerment to human health from any contaminant that is likely to enter a designated potable groundwater or surface water supply
- Discharges to surface waters (and to groundwaters in some very limited circumstances) under the National Pollutant Discharge Elimination System
- Dredge and fill operations within defined waters of the United States (e.g., for project construction or maintenance)
- Review authority by EPA over certain Federal projects (NEPA section 309)
Figure 1. - Location of recharge demonstration sites.
Federal statutes provide guidelines for State program development and minimal standards for compliance. Most of the States have been granted primacy for administering regulations. Primacy States are required to apply standards that are equivalent to, or more stringent than, the Federal "baseline." In addition, many States have enacted laws to cover specific activities not controlled by Federal laws. The current status of State regulatory development has important implications for the various types of groundwater recharge technologies.

Regulating projects that use surface infiltration is largely left to the States, although broad provisions exist in the CWA and the SDWA concerning the protection of all groundwater resources. Increased attention has recently been focused on retaining storm runoff and storage of reclaimed wastewater for groundwater recharge.

Under the authority of the SDWA, EPA has promulgated minimum requirements for underground injection control (UIC) programs. These regulations are designed to prevent endangerment of underground sources of drinking water (USDW's) from contamination. The regulations define a USDW as any aquifer or portion thereof which currently supplies a public water system or which contains a sufficient quantity of water to supply a public water system and either currently supplies drinking water for human consumption or contains fewer than 10,000 milligrams per liter (mg/L) of total dissolved solids (TDS). The regulations also provide for some narrow exemption criteria, whereby aquifers or parts of an aquifer that are not currently used and have no potential as drinking water sources can be exempted from protection as USDW's.

Recharge wells are considered Class V wells, which is a broad category encompassing a diverse group of wells for which EPA has not promulgated specific technical requirements. Under Federal regulations, these wells are authorized by rule as long as they do not endanger USDW's. However, the UIC program director may require owners and operators of Class V wells to obtain a permit under certain conditions to prevent endangerment of USDW's. In some instances, States with primacy to administer these programs may have promulgated more extensive regulations governing these wells.

On January 26, 1989, EPA clarified its groundwater monitoring policy for the High Plains States Groundwater Demonstration Program in a memorandum from the Director of EPA's Office of Drinking Water, and the Director of EPA's Office of Groundwater Protection. This memorandum provided guidance to project sponsors on EPA's groundwater protection goals, determining the hydrogeologic framework, selecting constituents for monitoring, collecting baseline data, and monitoring frequencies.

Reclamation and the Administrator of EPA entered into a memorandum of understanding to evaluate the impacts to surface water and groundwater quality resulting from the groundwater recharge demonstration projects. EPA consulted with USGS and was directed to make maximum use of data, studies, assistance, and other technical resources available from State and local entities. The evaluation of water quality impacts will be included in the Secretary of the Interior's final report to Congress. Project-specific coordination was formalized by execution of interagency agreements between EPA and
Reclamation regional offices.

Demonstration projects were designed, completed, and operated to protect human health and the environment and to comply with the memorandum of understanding on evaluation of water quality impacts:

- First, no endangerment will 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 [MCL's]) promulgated in 40 Code of Federal Regulations (CFR) Part 141, or EPA-recommended, health-based limits which have been peer-reviewed by EPA, such as health advisories.
- Second, where such standards are exceeded due to activities not related to this demonstration program, no endangerment will occur if constituents in the injectate do not exceed ambient concentrations in the groundwater or otherwise cause the quality of a USDW to deteriorate.

Under the Program, all projects have been held to a high level of monitoring scrutiny relative to typical projects undertaken outside of this program. The intent has been to go beyond minimal regulatory requirements to better assess the water quality implications of groundwater recharge operations under different project site conditions, water sources, and hydrogeologic characteristics.

The program anticipated limited instances where a project may experience injectate water that does not meet all health-based standards. These projects were continued on a case-by-case basis if the sponsor could assure that no drinking water well would be affected for the duration of the project and for as long as significant impacts from the recharged water persist on the receiving aquifer. For such projects, EPA required a permit that specified an allowable mixing zone within the receiving aquifer and expected that delegated States would also require permits. Compliance with the protection standard had to be demonstrated at the limit of the mixing zone. The sponsors also had to demonstrate institutional controls not only over the mixing zone but also over a buffer area. The permit would also require a very carefully designed monitoring program which would:

- Measure the impact of the project on the receiving aquifer
- Demonstrate compliance with the standard at the chosen point or points
- Give advance warning of the possibility that an MCL or other health-based standard may be exceeded outside of the allowed mixing zone
- Be active both during and after completion of the project until the applicant could demonstrate that the project no longer poses an adverse impact in the receiving aquifer

It was essential that data generated on all projects in this program be of a known and generally high quality so that conclusions regarding the impacts of recharge activities on surface water and groundwater quality could be made with confidence.

Each region of EPA approved the quality assurance and quality control programs of each project sponsor within their respective EPA regions. They also conducted audits of
the laboratories performing the analytical work to assure that all analysis and testing was performed to the standards that will met the specific needs of the High Plains States Groundwater Demonstration Program.

Program Status

Six projects, Southeast Salt Lake, Hueco Bolson, Highline Well Field, Rillito, Washoe County, and Denver Basin are completed, with final project and summary reports printed.

Title XXVI, High Plains States Groundwater Demonstration Program, of the Reclamation Projects Authorization and Adjustment Act of 1992 (P.L. 102-575) allowed for 5 years of operation after the date on which construction of the project was completed. Some project sponsors have requested this extension to gather a more complete set of monitoring data.

Project Descriptions

This section briefly describes each completed project.

- The **Southeast Salt Lake County Project** which is located in Salt Lake City, Utah, was designed to inject water from Deer Creek Reservoir into the unconfined portion basin-fill aquifer during the winter to recover for municipal use during the summer.

  This project involved two different water treatment processes, conventional and rapid chemical mix with pressure filtration. The sponsor also tested aquifer storage and recovery technology against dedicated injection and recovery wells.

  The sponsor determined that this project is an economically sound way to meet summer peak demand. Salt Lake County Water Conservancy District current average base unit range from $90/160 per acre feet and recharged water calculated cost of $246/346 per acre foot. These cost are high compared with base unit cost but compare very favorably with other peak options.

  The project operated through three injection/recovery seasons during the High Plains Program and is continuing and being expanded as part of the Central Utah Project completion.

- The **Hueco Bolson Project** is located near El Paso, Texas, and was designed to document the results of an ongoing recharge project. A series of technical research memoranda were prepared to address various aspects of design, operation, and aquifer responses of the Hueco Bolson recharge project. Powdered activated carbon treatment was demonstrated to be highly effective at removing priority pollutants, pathogens, as well as general wastewater cleanup.

  A USGS proposal to study the formation and transport of trihalomethane (THM) in chlorinated water injected into the subsurface was incorporated into the study and completed. Chemical analysis show that the predominate form of THMs changed
from bromoform in the injection wells to chloroform in the production wells. Results also demonstrated significant removal of THMs.

- The **Highline Well Field Project** in Seattle, Washington, was designed to recharge aquifer storage during off-peak demand periods so that yields from production well fields would increase without long-term groundwater declines. The testing program and full recharge at two sites have been successfully completed. The project successfully demonstrated Aquifer Storage and Recovery technology using wells to recharge into the intermediate aquifer. Results show that artificial recharge is economically viable when compared with other methods of meeting peak demands. Project difficulties include biofouling due to algae, particulate plugging, and concerns with possible THMs formation. THMs were not found to be a problem. The facilities are in full operation.

- The **Rillito Creek Project** was located on the north side of Tucson, Arizona, and examined the feasibility of utilizing an inflatable dam to capture stormwater flows and passive recharge into the stream bed. The projects was the first of its kind in Arizona. A wide variety of in- and off- channel recharge techniques were evaluated for the project. The sponsor decided not to construct the recharge project due to technical, economic, and institutional considerations, including limited aquifer storage capacity, lack of economic benefits, staffing needs, and development costs.

- The **Washoe Project** is locate near Reno, Nevada, and was designed to inject treated potable water from the Truckee River into the East Lemmon and Golden Valley Basins during years with above-average precipitation. The project was successful in stabilizing or raising groundwater levels in the Golden Valley by injecting 50 acre feet of water. In Lemmon Valley the project allowed the sponsor to use diverted water for municipal use allowing the aquifer to recover from over pumping. Experience to date has not shown artificial recharge to be economically viable at this site.

  Based on the successes in the Golden Valley, the County is proposing recharge at other sites (e.g., Spanish Springs Valley) with artificial recharge the sponsor projects costs of $380/590 per acre foot for recovered water.

- The **Denver Basin Project** was operated by the Willows Water District on the south end of metro Denver, Colorado, area and was designed to inject potable surface water from Denver's transmountain supply 1,500 feet into the Arapahoe Aquifer. The project was dedicated in April 1992 and completed and placed on standby status in the spring of 1997. A total of 34 recharge periods injected 1,283 acre-feet. Over the life of the project it was found that the temperature differential between the receiving aquifer and the injection water caused problems with the specific capacity of the well. The water being injected was colder than the ambient temperature in the aquifer. During the summer 1995, two injection studies were made to determine the effects of injecting water that was warmer than that injected previously, better injection rates were found with the warmer water. Minimal problems with aquifer
clogging have been observed. Overall injection efficiency was in excess of 98%. Water levels in the Arapaho Aquifer were increased as a result of the project.

Final Report

Individual, technical project final reports will be written at the conclusion of each project and are referenced in the annual interim reports to Congress. A report to Congress, required by P.L. 102-575, will summarize these final reports for individual projects and address the items listed below:

- A detailed evaluation of the demonstration projects with an evaluation by EPA of the impacts to surface water and groundwater quality resulting from the demonstration projects
- Results of a study to identify and evaluate alternative means by which the costs of groundwater recharge projects could be allocated among the beneficiaries of the projects and to identify and evaluate the economic feasibility and legal authority of using groundwater recharge in water resources development projects
- Specific recommendations regarding the location, scope, and feasibility of operational groundwater recharge projects to be constructed and maintained by Reclamation
- An evaluation of the feasibility of integrating these groundwater recharge projects into existing Reclamation projects

Overall the High Plains Groundwater Demonstration Program has been a success for those projects that were constructed. Many states and local entities have had to develop special legislation or recognize the legal and institutional ties between surface and groundwater posed by these projects. Because of the drought in the early 1990's many of the projects were not able to recharge or demonstrate to the extent that was expected.

In general, no sponsor was able to recharge at the rate that was anticipated. Wells either experienced biofouling, viscosity, or other clogging problems that restricted the volume of water recharged, or maintenance problems were greater than anticipated. Total THM formation remains a concern for chlorinated source waters. Most sponsors found that recharge is a viable tool for water management, and for urban areas, can be a cost effective method for meeting peak demand requirements.

Table 1 describes all of the projects.
<table>
<thead>
<tr>
<th>Project</th>
<th>Local sponsor</th>
<th>State</th>
<th>Water source</th>
<th>Treatment</th>
<th>Technology</th>
<th>Purpose</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rillito Creek</td>
<td>Pima County Flood Control District</td>
<td>AZ</td>
<td>Rillito Creek storm runoff</td>
<td>NT¹</td>
<td>River impoundment</td>
<td></td>
</tr>
<tr>
<td>Denver Basin</td>
<td>Willows Water District</td>
<td>CO</td>
<td>Denver Water Department</td>
<td>T²</td>
<td>Deep well injection</td>
<td>M&amp;I³</td>
</tr>
<tr>
<td>Southwest Irrigation District</td>
<td>Southwest Irrigation District</td>
<td>ID</td>
<td>Snake River</td>
<td>NT</td>
<td>Passive injection</td>
<td>Irr⁴</td>
</tr>
<tr>
<td>Equus Beds</td>
<td>City of Wichita</td>
<td>KS</td>
<td>Little Arkansas River, Equus Beds Aquifer</td>
<td>NT</td>
<td>Passive recharge</td>
<td>WQ⁵</td>
</tr>
<tr>
<td>Turner-Hogeland</td>
<td>Montana Bureau of Mines and Geology</td>
<td>MT</td>
<td>Snowmelt</td>
<td>NT</td>
<td>Land percolation</td>
<td>Irr</td>
</tr>
<tr>
<td>Wood River</td>
<td>Central Platte Natural Resources District</td>
<td>NE</td>
<td>Platte River</td>
<td>SD⁶</td>
<td>Spreading basins</td>
<td>Irr/M&amp;I</td>
</tr>
<tr>
<td>York</td>
<td>Upper Big Blue Natural Resources District</td>
<td>NE</td>
<td>Runoff and industrial cooling water</td>
<td>PT⁷</td>
<td>Reservoirs, basins, injection</td>
<td>M&amp;I</td>
</tr>
<tr>
<td>Washoe</td>
<td>Washoe County Department of Public Works, Utility Division</td>
<td>NV</td>
<td>Truckee River</td>
<td>T</td>
<td>Injection</td>
<td>M&amp;I</td>
</tr>
<tr>
<td>Blaine Gypsum</td>
<td>Oklahoma Water Resources Board</td>
<td>OK</td>
<td>Runoff and agriculture return flows</td>
<td>NT</td>
<td>Passive injection</td>
<td>M&amp;I</td>
</tr>
<tr>
<td>Hermiston</td>
<td>City of Hermiston</td>
<td>OR</td>
<td>Shallow aquifer</td>
<td>NT</td>
<td>Injection</td>
<td>M&amp;I</td>
</tr>
<tr>
<td>Huron</td>
<td>South Dakota State University</td>
<td>SD</td>
<td>James River</td>
<td>T</td>
<td>Injection</td>
<td>M&amp;I</td>
</tr>
<tr>
<td>Hueco Bolson</td>
<td>El Paso Water Utilities Public Service Board</td>
<td>TX</td>
<td>Reclaimed wastewater</td>
<td>WWT⁸</td>
<td>Injection</td>
<td>M&amp;I</td>
</tr>
<tr>
<td>Southeast Salt Lake County</td>
<td>Salt Lake County Water Conservancy District</td>
<td>UT</td>
<td>Deer Creek Reservoir</td>
<td>SD</td>
<td>Injection</td>
<td>M&amp;I</td>
</tr>
<tr>
<td>Highline Well Field</td>
<td>City of Seattle</td>
<td>WA</td>
<td>Cedar River</td>
<td>NT</td>
<td>Injection</td>
<td>M&amp;I</td>
</tr>
</tbody>
</table>

¹ No treatment.
² Treatment to varying standards.
³ Municipal and Industrial.
⁴ Irrigation.
⁵ Water quality.
⁶ Sedimentation treatment.
⁷ Partial treatment.
⁸ Wastewater treatment.
SAFE WELL MAINTENANCE TECHNOLOGY FOR DIRECT RECHARGE OF TERTIARY EFFLUENT

Peter Fox, Paul Johnson, Sandra Houston, William N. Houston, Ling-Xiao Wang, Michael Johnson and Phil Brown

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Executive Summary

This project focuses on direct groundwater well recharge of tertiary effluent. The goal is to design and conduct laboratory- and pilot-scale model aquifer studies to determine:

- if well injection systems can be optimized by controlling biofilm growth and minimizing aquifer clogging over time,
- if there is a way to restore the aquifer transmissivity in cases where wells have become clogged,
- if disinfection by-products are produced and eliminated by natural processes when chlorination is used to reduce aquifer clogging, and
- if injected waters should meet primary drinking water standards, or do natural aquifer filtration processes sufficiently improve the quality of injected waters so that water quality requirements for injected waters can be relaxed.

Introduction

In Arizona, as well as other areas of the world, populations are increasing and water quality standards are becoming more stringent over time. These conditions are forcing water suppliers to better manage the available water supplies as well as to seek alternate means of water supply augmentation. One approach currently being practiced and studied in Arizona is the soil-aquifer treatment (SAT) system that involves the infiltration of treated wastewater. Geologic conditions limit SAT systems to areas where the vadose zone soils are sufficiently permeable to allow recharge of the underlying groundwater. This same geologic constraint also limits the applicability of other surface recharge options, including constructed wetlands and enhanced stream bed recharge.

Another groundwater recharge option that is often discussed involves the direct injection of treated waters (waste water, groundwater from remedial systems) into water supply aquifers. Unlike SAT and other surface infiltration approaches, direct injection of water can be employed for a wider range of soil conditions because it requires only the presence of a high yield aquifer, which can normally be found at some depth. Direct injection is also attractive for arid regions from the standpoint that significant evaporative losses associated with infiltration recharge ponds are eliminated.

While direct aquifer recharge of treated waters has a number of obvious advantages relative to surface infiltration technologies, technical and regulatory impediments are currently restricting its use. All recharge systems are known to be susceptible to clogging over time, and this reduction in permeability is generally attributed to a combination of biomass growth and physical filtration of suspended solids. Unlike surficial SAT systems, however, clogging layers in aquifers are not easily scraped away.
Considerable pretreatment of the water and periodic well redevelopment are necessary, and even then, new injection wells often must be installed as existing ones become plugged. Second, many regulatory agencies are hesitant to allow direct recharge of any waters not satisfying primary drinking water requirements. In effect, no allowances are currently made for the natural capacity of the aquifer to improve water quality within a very short distance of the injection well. Thus, water quality requirements are significantly more stringent for waste water treatment systems connected to direct aquifer recharge systems than for those feeding surface infiltration systems. The net result is that the current regulatory climate indirectly discourages the use of direct aquifer recharge.

Summary of Research Goals

The discussion above indicates that use of direct aquifer recharge systems could be increased the interaction between aquifers and injected waters was better understood. Specifically, the goal of this study is to provide answers for the following questions:

- Can well injection systems be optimized to control biofilm growth and minimize aquifer clogging over time? If aquifer clogging occurs, is there a way to restore the aquifer transmissivity?

- If chlorination is used to reduce aquifer clogging, will disinfection by-products be eliminated in the controlled biofilm region surrounding the injection well?

- Is it necessary for injected waters to meet primary drinking water standards, or do natural aquifer filtration processes sufficiently improve the quality of injected waters so that water quality requirements for injected waters can be relaxed?

Summary of Research Approach

To answer the questions introduced above, both bench-scale and large-scale aquifer model studies are being done. The specific goals of each study are given below:

- **bench-scale one-dimensional aquifer simulations**: these studies are being conducted in order to determine which disinfection schemes have the potential to prevent biological clogging around a reinjection well, while maintaining a biologically active treatment zone in the aquifer. The bench-scale aquifer simulators are to be designed, characterized, and then used to assess the performance of the various disinfection schemes.
- **large-scale two-dimensional aquifer physical model**: A large-scale two-dimensional aquifer physical model will be designed, constructed, and tested. This apparatus is extremely valuable to the long-range research plan as it provides a unique opportunity to cost-effectively simulate recharge systems on a scale that is only slightly smaller than that of real systems. It provides a "bridge" between one-dimensional bench-scale studies and real recharge systems.

Results from the bench-scale aquifer model studied have been completed and a brief summary of these results is presented in this paper.

**BENCH-SCALE AQUIFER MODEL STUDIES**

Figures 1 and 2 present schematics of the bench-scale aquifer simulation tanks that have been constructed for this project. Each is constructed from 1/2 in clear acrylic.

![Schematic of bench-scale aquifer model construction](image)

**Figure 1.** Schematic of bench-scale aquifer model construction.
Figure 2. Schematic diagram of bench-scale aquifer model experiments.

The tanks are filled with a medium grade sand taken from the Aqua Fria River bed. It has a saturated hydraulic conductivity, $K_{sat}$, of 240 ft/d. One inch thick gravel packs at the inlet and outlet ends of the tanks are filled with the same sand but with all particles passing a no. 20 sieve size also removed. The gravel packs are held in place with no. 20 stainless steel wire mesh (Figure 2). Laboratory-scale aquifer injection simulation tests using tertiary effluent included monitoring levels of, and significant changes in: biomass growth, flow rates, head pressure, total organic carbon (TOC), turbidity, biochemical oxygen demand 5-day test (BOD$_5$), disinfection by-products (DBP’s), and types of disinfectant used. The utility of BOD$_5$ testing has been determined to not be relevant since BOD$_5$ levels of the effluent have generally been $<2-4$ mg/L and BOD$_5$ testing is not accurate in this range. Measurements of electron acceptors including oxygen and nitrogen species has been done to evaluate oxygen demanding reactions in the aquifers.

During operation, water is pumped horizontally through 45 inches of soil in the tanks which are 6 inches tall and 3 inches wide. Hydraulic conductivity measurements and tracer studies were used to demonstrate similar hydraulic properties in each tank. F-curves produced by tracer studies on each tanks were almost identical and significant reduction in short circuiting resulted from the use of 1/2" baffles.
Following about one week of hydraulic testing, pumping of an advanced treated wastewater from the City of Tempe Kyrene Water Reclamation Plant through the tanks was begun. Some typical ranges of water quality parameters of the wastewater are:

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>TSS</td>
<td>0 - 0.5 mg/L</td>
</tr>
<tr>
<td>COD</td>
<td>9 - 29 mg/L</td>
</tr>
<tr>
<td>BOD</td>
<td>&lt;2 - 4 mg/L</td>
</tr>
<tr>
<td>Turbidity</td>
<td>0.2 - 0.6 NTU</td>
</tr>
<tr>
<td>NO₃⁻</td>
<td>2.0 - 5.3 mg-N/L</td>
</tr>
<tr>
<td>TKN</td>
<td>0.8 - 5.0 mg-N/L</td>
</tr>
</tbody>
</table>

Constant rate, peristaltic pumps operating at 6 RPM pump the water from the reservoir to the tanks (Figure 2). A second pump adds concentrated sodium hypochlorite solution directly to the tank influent line. A 500 ml dark amber jar provides approximately 1/2 hr. contact time before the chlorinated wastewater enters the tank.

Initial target residual chlorine concentrations entering the tanks were chosen to be: 0 mg/L (tank 4), 2 mg/L chloramine tank 5, 2 mg/L free chlorine (tank 2), and 10 mg/L free chlorine (tank 3). After several months of operation, the target residual chlorine concentrations were changed to 0 mg/L (tank 4), 2 mg/L chloramine (tank 5), 2 mg/L free chlorine (tank 2), and 5 mg/L chlorine (tank 3). Tank 3 was switched to a lower chlorine dose since there was 2 mg/L free chlorine residual in tank 3 effluent at the 10 mg/L free chlorine dose and there was very little loss in hydraulic conductivity. Tank 1 had an initial target chloramine concentration of 5 mg/L, however, this was impossible to achieve with the low ammonia concentrations in the source wastewater, therefore it was considered impractical to operate at this chloramine concentration and operation of Tank 1 was terminated soon after start-up.

After several months of operation, one more major operational change was made. The flowrates to tank 2 and tank 5 were doubled to approximately 45 L/d while the target residual concentrations were maintained at 2 mg/L free chlorine and 2 mg/L chloramine, respectively. Operation of tanks 1, 3 and 4 was stopped. Tank 1 had never shown any change in hydraulic conductivity and was serving as a control fed tap water. Increasing the flowrate was considered necessary to observe greater variations in the hydraulic conductivity of the tanks. Practical limitations on transporting effluent to the laboratory prevented operation of all tanks at the higher flowrate.

Variations in wastewater quality and pumping rates result in some variability in the actual chlorine residual concentrations entering the tanks. Very low ammonia concentration in the wastewater initially made it difficult to achieve chloramine concentrations greater than 0.5 mg/L without the addition of ammonia to the water. After October 20th, a nitrification failure at the Tempe Kyrene plant resulted in ammonia concentrations exceeding 10 mg-N/L which required very high doses of chlorine to
achieve a residual free chlorine concentration, however, obtaining a chlormine residual become simpler. The plant began to recover in December, however, a period of several months is anticipated before efficient nitrification will achieve ammonia concentrations less than 1 mg-N/L. Table 2 includes a summary of typical operating conditions for each of the tanks:

Table 2 Summary of Typical Operating Conditions for Bench-Scale Aquifer Models

<table>
<thead>
<tr>
<th>tank #</th>
<th>2</th>
<th>3*</th>
<th>4</th>
<th>5</th>
</tr>
</thead>
<tbody>
<tr>
<td>chlorine dose (mg/L) (before 9/10/96)</td>
<td>6</td>
<td>17</td>
<td>0</td>
<td>3</td>
</tr>
<tr>
<td>chlorine residual at tank inlet (mg/L) (before 9/10/96)</td>
<td>2</td>
<td>10</td>
<td>0</td>
<td>0.5</td>
</tr>
<tr>
<td>chlorine residual in tank effluent (mg/L) (before 9/10/96)</td>
<td>0.2</td>
<td>2</td>
<td>0</td>
<td>0.1</td>
</tr>
<tr>
<td>flow rate at start of experiment (L/day)</td>
<td>22</td>
<td>24</td>
<td>25</td>
<td>27</td>
</tr>
<tr>
<td>flow rate after 9/30/96 (L/day)</td>
<td>45</td>
<td>0</td>
<td>0</td>
<td>45</td>
</tr>
</tbody>
</table>

* On September 10, tank 3 was changed to a target residual of 5 mg/L chloramine.

**Hydraulic Conductivity**

Throughout the first month of tank operation, the hydraulic conductivity of the soil media in all five tanks was observed to increase from an initial value of 70 ft/day to approximately 250 ft/day by the end of the month. This final value was in agreement with the measured hydraulic conductivity of the soil (240 ft/day).

The overall hydraulic conductivity of all the tanks was similar with the exception of tank 5. Tank 5 had the highest dose of free chlorine and this dose of free chlorine appeared to be maintaining hydraulic conductivity near 240 ft/d. Decreases in hydraulic conductivity were observed in tanks 2, 3, and 4 after day 30 the overall hydraulic conductivity of these tanks had similar trends. After doubling of the flowrate, a significant and steady decrease in hydraulic conductivity was observed in tanks 2 and 5. The higher flowrate essentially doubled the loading of solids and biodegradable material that can influence clogging in the tanks.

Clogging effects were observed in tank 4, receiving no chlorination, within the first week of operation with wastewater as the hydraulic conductivity in the first 12 inches of the tank decreased to approximately 150 ft/day. No similar affect was observed in the conductivity of the same zone for the three tanks receiving chlorinated wastewater. Superchlorination of tank 4 was done on day 60 in an attempt to recover hydraulic conductivity. Superchlorination was done by spiking the effluent with 2 ml of 5.25% by weight chlorine solution which is equivalent to dosing 75 mg/L chlorine for 1 hour.
Superchlorination resulted in a rapid increase in hydraulic conductivity within the following seven days, however, the hydraulic conductivity returned to values similar to those observed prior to superchlorination within 20 days. The effect of superchlorination was apparently temporary. A small spike in turbidity in effluent from the tank might have indicated that superchlorination did result in the passage of dead microorganisms through the aquifer material.

In tank 3, decreasing the target free chlorine concentration from 10 mg/L to 5 mg/L did not have a major effect on the hydraulic conductivity (Figure 3). One possible effect of the decrease in chlorine concentration on day 66 was a decrease in the hydraulic conductivity of the first 12 inches. The results appear to indicate that a 5 or 10 mg/L free chlorine residual will prevent biological clogging through the distance of the tank at the flowrate tested.

![Figure 3. Hydraulic Conductivity Results for Tank#3 (5-10 mg/L of free chlorine)](image)

Hydraulic conductivity results for tank 2 are presented in Figure 4. The most pronounced effect on hydraulic conductivity in tank 2 is in the last 9 inches. This indicates that the 2 mg/L free chlorine residual could be effective at preventing biological clogging for the first 36 inches of the tank and dissipation of the residual allows for biological growth near the end of the tank. The effect of doubling the flowrate on day 83 did not cause a significant decrease in the hydraulic conductivity at the beginning of the tank. The chlorine loading on the system was also doubled as the flowrate was doubled and these
results might indicate that dissipation of chlorine is the most important factor in controlling growth as opposed to loading of biodegradable material.

Figure 4. Hydraulic Conductivity Results for Tank#2 (2 mg/L free chlorine)

In tank 5 after day 60, the greatest decrease in hydraulic conductivity is evenly distributed across the first 36 inches of the tank while the highest hydraulic conductivity is greatest in the last 9 inches of the tank. This is exactly the opposite of tank 2 and might demonstrate some critical differences between the behavior of chloramine and free chlorine. Tank 5 receives chloramine which apparently does not prevent biological clogging but is capable of inhibiting biological growth resulting in evenly distributed clogging. The lack of clogging in the last 9 inches might be from lack of available biodegradable material. Chloramines release ammonia when they decay and this could also stimulate growth evenly distributed growth.

The overall results for tanks 2 and 5 indicate a 50% decrease in hydraulic conductivity. However, the spatial distribution of clogging is very different and this will have important implications for field reductions in hydraulic conductivity.

Dissolved Oxygen
Changes in dissolved oxygen concentrations followed intuitive trends throughout each of the tanks. Dissolved oxygen in the tank receiving no chlorine (tank 4) dropped to zero in about 10 days. Tank 3 had dissolved oxygen concentrations near the influent concentrations indicating that the high free chlorine dose was maintaining a high redox potential and probably inhibiting biological growth. Tank 2 had a decrease in dissolved oxygen of 2 to 5 mg/L indicating that some biological activity was occurring and that aerobic conditions were present through the tank. Tank 5 had negligible effluent DO concentrations after 30 day of operation indicating that biological activity developed initially and has maintained anoxic conditions in the tank. These differences between tank 2 and tank 5 might also represent differences in biological clogging.

Chlorine demand within the tank reflects the same pattern of reduction as observed with dissolved oxygen.

Results from tanks 2 and 5 indicate distinct trends that are consistent with the dissolved oxygen levels in the tanks. Effluent nitrate concentrations from tank 2 are 0.1 to 2.6 mg-N/L higher than the influent indicating that some nitrification might be occurring. Effluent nitrate concentrations from tank 5 are approximately 1 mg/L less than the influent which could indicate denitrification.

Total Organic Carbon

Following about three weeks of operation with wastewater, a pattern of slight reduction in total organic carbon (TOC) was observed in all but the tank with the highest chlorine dose. It seems likely that that during the first few weeks, populations of microbes responsible for the oxidation of TOC in the wastewater were getting established in the tanks. Subsequently, there has been very little difference between influent and effluent TOC indicating that the majority of organic carbon is not easily biodegradable in the time scale (6-12 hours) of the experiment. This result is consistent with the low BOD values of the source wastewater.

Turbidity

When wastewater was pumped through the tanks, turbidity levels increased in all of the tanks to varying degrees and durations. The increases in turbidity are likely biological in origin, corresponding with periods of bio-growth and not simply the result of the mechanical migration of soil fines. The lowest increase in turbidity was in tank 3 where high free chlorine concentrations suppressed biological activity. Turbidity levels increased rapidly in tanks 2, 4, and 5. Tank 2 did have an apparent delay of approximately one week before turbidity levels began to increase indicating that the lower dose of free chlorine could have inhibited development of biological activity. Tanks 3 and 4 both showed consistent removal of turbidity until their operation was stopped.
SRP CASE STUDY IN WATER MEASUREMENT - APPLICATION OF THE AXSYS REMOTE STATION AT THE GRANITE REEF UNDERGROUND STORAGE PROJECT (GRUSP)\(^1\)

Lee W. Ester\(^2\)

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Introduction

The cornerstone of any controlled water resource management project is the installation, collection and processing of accurately gathered flow (water) measurement data. Within the world of water measurement instrumentation and automation, there is no shortage of equipment to choose from. The end-user must consider with ample forethought what they wish to accomplish with the available funds they have to spend.

Instrumentation costs are quite variable among manufacturers within the industry. The end-user must be pragmatic and decisive in selecting a water measurement instrument and system that meets the needs for their application. Such is the case for SRP, who chose the AxSys Remote Station (ARS) for water depth monitoring of the four recharge basins at its Granite Reef Underground Storage Project (GRUSP).

SRP

Salt River Project (SRP), with headquarters in Tempe, Arizona, is the nation’s third-largest public power utility and Arizona’s oldest, largest single manager of one of the state’s most precious resources—water.

SRP was born of necessity and since its inception, has played a unique role in the development of the Salt River Valley by maintaining a constant vigil over its water supplies. SRP delivers approximately 1 million acre-feet of water annually to about 1.2 million users in the Phoenix metropolitan area. Inherent in SRP’s stewardship of water is the ability it has developed to measure and operationally deliver precise amounts of gravity-fed water. SRP is a leader in the design and application of unique water measurement instrumentation.

GRUSP - Project Location

SRP manages GRUSP, Arizona’s largest recharge facility. The premise of GRUSP is to recharge the underground aquifer with water for use at a later date.

GRUSP is located on the Salt River channel, within the lands of the Salt River Pima Maricopa Indian Community, in Maricopa County, Arizona. The elevation ranges from 380 meters to 396 meters above mean sea level. It is within the Basin and Range physiographic province of Arizona. The climate is semi-arid with annual precipitation of under .20 meters and evaporation rates of up to 2.2 meters per year.

Problem Statement

During the early design stages of GRUSP, it became apparent that water measurement would be a critical element in 1), satisfying the reporting requirements necessitated by the state’s regulators and 2), providing for accurate collection of scientific data while 3), addressing what was then an unknown—the physical issues of
actual operation and maintenance (O&M) and subsequent efficiency evaluations of operating the recharge facility economically.

Specifics

Selecting the water measurement hardware for GRUSP was a challenge for SRP’s instrumentation specialists to be innovative—yet cost effective in meeting the requirements of data reporting (regulators) and real-time data collection for operations and maintenance activities. Also, of high importance was the accuracy of the water measurement stations—obviously, maximizing recharge credits received would be based upon the accuracy of the measuring of the water that passed through these measurement stations.

In addition, SRP’s staff had to focus on obvious limitations of the local GRUSP environment—which would include the physical challenge of adapting water measurement hardware in a gravel-rich, unstable, hot/cold environment. In addition, the staff were charged with controlling and minimizing up-front and future operation and maintenance costs, environmental concerns, coping with no available electrical power, fending off instrumentation vandalism and finally, they were to plan for emergency removal of the water measurement instrumentation in the event the Salt River channel should become flooded.

State regulations

GRUSP was not exempt to Agreement Issues, land use permitting, IGA’s and State Regulations and Permitting. Pertinent to the discussion within this paper, the State’s storage permit was for 200,000 acre feet of water per year. This permit was used as basis for the storage entitlements of all the GRUSP participants. For example, the least percentage entitlement is just over 2%, the highest just under 27%. SRP recognized that these types of “book-keeping” requirements mandated accurate water measurement.

Operational requirements

Operationally, real-time data collection of inflow water measurements and basin depth measurements were requested. Inflow measurements would be required to determine the basin specific infiltration rates and to assist in the control of the basin specific water depths. Early on it was recognized that through the collection of basin specific data, optimum operating criteria (potential) could be established for each of the four recharge basins. In addition, preventive maintenance had to be minimal due to cost and the remote location of GRUSP. Correlating the collected water measurement data with actual performed site and basin maintenance would be instrumental in the identification of efficient operations of water recharging (cost/benefit) and in preparing future operating budgets.
Controlling costs

Once the reporting formats and operational issues were identified and understood, SRP’s instrumentation staff narrowed the field of possible instrumentation solutions to the four scenarios identified in the following figure.

As can be seen in figure 1, the up-front costs for the instrumentation necessary for the four basins at GRUSP varied greatly depending on the chosen package. But, necessary for the analysis were those costs anticipated for the future operation and maintenance of the chosen package--generally, that of labor.

As shown, the farthest left scenario if selected would prove to be the most flexible and achievable for SRP at the lowest cost. Note the necessary labor dollars required for five years of operation and maintenance requirements ($17,171) and the first year hardware and installation costs ($20,160) is lowest utilizing the AxSys Remote Station with ORBCOMM satellite capabilities.

The highest future cost is with the scenario that requires manual field observations daily--far right in figure 1. No data logging instrumentation is provided in that scenario. The five year scenario totals approximately $290,675 of operation and maintenance charges while only requiring $1000 of first year hardware and installation costs--for
clarity, this scenario would simply have fixed staff gages, where all measurements would be facilitated manually.

In conclusion, as shown in figure 2 below, some form of automated remote water measurement at GRUSP would prove to be cost effective for SRP. In short, SRP’s specialists chose the AxSys Remote Station anticipating the future upgrading of the instrument stations with the ORBCOMM satellite telemetry system.

Figure 2
Instrumentation selection—environmental concerns

For any water measurement instrument to be considered for application, it would have to be able to withstand high summer temperatures—consisting of diurnal fluctuations of nearly 60 degrees Fahrenheit, blowing dust, lightning strikes and driving rain (those conditions typical to Arizona’s monsoon season storm events). In addition, the instrumentation would have to be packaged in such a manner as not to become home to insects and rodents—creatures who typically chew on electrical wiring or can become a safety hazard (due their possible bites). In addition, the overall basin-specific installation and “look” of the instrument had to be as unobtrusive as possible—no sense attracting attention in such a remote location which might have encouraged vandalism.

Further, due to the very nature of the “looseness” of the surface soils at the GRUSP facility, SRP’s standard use of water measurement “stilling wells” would be challenged. Stilling wells, as currently employed throughout the SRP system, have been used as safe havens (vandalism, temperature and storm protective environments) successfully on nearly all SRP water measurement projects.

At GRUSP, stilling wells were ruled out very early in the specifications—the luxury was deemed not practical due to the maintenance required (plugging and subsequent pumping of the stilling well needed to clear the inlet plumbing) and thus would not be economically feasible.

Availability of electric power

GRUSP has no conventional (115v AC) electric power for easy access. Consequently, it was recognized that the water measurement instrumentation would have to be low-powered and capable of being battery powered.

The power issue became one of the first “narrowing” elements of the instrument selection processes. Many instrumentation packages simply used too much power and were eliminated from the selection process.

Ultimately, SRP would design a specialized solar charging system uniquely configured to power the AxSys Remote Station with the ORBCOMM satellite capabilities. The solar panels are extremely small—1.1 watts each—two per station—measuring a meager 5” x 5” each. The solar panels are mounted horizontally on each roof-top of the instrumentation stations. The panel configuration is benign in its appearance and is vandal proof. To date, there has been no successful tampering with the solar power arrays at the four basin measurement stations.

Vandalism

Vandalism was another issue to be resolved. The GRUSP facility is located in a remote, but urbanized location. Any damage to the water measurement stations would take away from the integrity of the data collected and raise the maintenance costs of the
facility. The final instrument station would be configured with a 1/4 inch thick protective enclosure surrounding the AxSys Remote Station. From the outside, a quality padlock would be used. In essence, the steel enclosure acts as armor for the sensitive instrumentation system housed inside a NEMA type 3R (rainproof) enclosure.

**Ease of recovery--emergency removal**

The water measurement instrumentation was designed for timely and easy removal due to the chance of flooding within the Salt River channel. For obvious reasons, SRP felt it was important to safeguard the instrumentation investment. By design, the instrument configuration had to be installed in such a manner as to be removable in short notice--quite possibly with only hours of prior notice.

Interestingly, this feature was called into action at the flooding of GRUSP that occurred in 1994. All water measurement instrumentation was removed before any damage was sustained. It took SRP staff a little under four hours to facilitate the necessary removal of instrumentation from all four of the GRUSP basins.

**Hardware Selected**

SRP developed the instrumentation configuration that was totally unique for the specialized requirements of water measurement necessitated at the GRUSP basins.

At the heart of the water measurement station is an adequate supply of electrical power via solar energy. Surrounding the solar system is the Stevens AxSys Monitoring/Processing Unit--the on-board water measurement “computer” and subsequent data logger. Making all this functional is the Milltronics ultra-sonic detection device equipped with independent temperature compensating software. The sonic device eliminated the need for SRP’s standard style stilling wells.

Finally, the telemetry capabilities of the configuration is the ORBCOMM Satellite system because of its very low-power requirements and the relative low costs and economies of scale that were involved.

**AxSys remote station (ARS)**

In December of 1996, SRP, in a collaborative effort with Stevens Water Monitoring Systems, installed the first Arizona beta-site utilizing the ORBCOMM satellite system. ORBCOMM is the world’s first wireless, two way data communications system. The water measurement data collected by the AxSys Remote Station is sent via e-mail to operators at SRP every four hours. Data is collected at the basins every thirty minutes (see examples figures 3 and 4). In the future, as more satellites are positioned in orbit above the earth, collection of the GRUSP data will become as real-time as needed for the operation of the basins. (As a sidenote, there is a monthly fee for users to access the ORBCOMM satellite system).
Figure 3 - GRUSP Basin #1 - no water being recharged.

"ID ORB 01 DATE 04/29/97 TIME 20:00:00 INTERVAL 00:30:00"
"Power: 13.4v"
00000.00 00000.00 00000.00 00000.00 00000.00 00000.00
00000.00 00000.00 00000.00 00000.00 00000.00 00000.00
00000.00 00000.00 00000.00 00000.00 00000.00 00000.00

Figure 4 - GRUSP Basin #1 - water being recharged (5 days later from figure 3).

"ID ORB 01 DATE 05/04/97 TIME 21:30:00 INTERVAL 00:30:00"
"Power: 13.2v"
00002.22 00002.26 00002.26 00002.28 00002.29 00002.28
00002.30 00002.31 00002.30 00002.31 00002.32 00002.32
00002.34 00002.33 00002.34 00002.35 00002.35 00002.34
00002.36 00002.37 00002.37 00002.38 00002.37 00002.36
00002.35 00002.39 00002.39 00002.36 00002.38 00002.38
00002.35 00002.39 00002.98 00002.97 00002.93 00002.97
00003.01 00003.05 00003.05 00003.16 00003.12 00003.16

Lessons Learned--Recommendation

SRP configured the AxSys Remote Station for a variety of water measurement requirements. Whether measuring water flow, basin depth, groundwater observation well depths, water quality or weather station components, the installation and low-cost of the AxSys Remote Station is formidable for most competitors. The unit is highly cost effective when compared to other methods of water measurement. When compared to SCADA or manual data collection methods, it has proven to offer SRP the highest return on its investment.

The AxSys Remote Station is sold as a stand-alone water measurement station. By utilizing sonic level detection devices, SRP eliminated the physical requirements of installing water monitoring stilling wells at each of the four basins--this construction savings alone was more than required to purchase the ARS units themselves.

The AxSys Remote Station has been instrumental in reducing the cost of the data collection requirements at GRUSP. In addition, the ARS has very minimal maintenance requirements. Utilizing solar power and the ORBCOMM Satellite System, the ARS has been a perfect match for SRP enabling it to effectively manage the operation of the Granite Reef Underground Storage Project.
References

For a free VHS video tape of the SRP/GRUSP water measurement application of the AxSys Remote Station, please contact:

SRP
Water Management Services
P.O. Box 52025
Mail Station PAB-112
Phoenix, AZ 85072-2025
Tel. 602-236-5592  FAX 602-236-2159
Website: http://www.srp.gov

For additional technical instrument information, please contact:

Milltronics
709 Stadium Dr. East
Arlington, TX 76011
Tel. 817-277-3543

ORBCOMM
21700 Atlantic Blvd.
Dulles, VA 20166
Tel. 703-4-6-5000  800-ORBCOMM  FAX 703-406-3508
Website: http://www.orbcomm.net

Stevens Water Monitoring Systems
P.O. Box 688
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THE WETLANDS OF AVONDALE: A WATER TREATMENT SYSTEM USING CONSTRUCTED WETLANDS AND ARTIFICIAL RECHARGE

Timothy J. Thompson, Steven B. Bachman & David Parkinson

ABSTRACT

In an effort to utilize a large supply of untreated Central Arizona Project water that contains significant portions of agricultural return flows, a combination of constructed wetlands and artificial recharge infiltration basins has been designed to treat and deliver potable water to existing municipal wells in Avondale, Arizona. Although wetlands treatment is becoming more common in many wastewater applications, special considerations are associated with the treatment of surface water containing periodically high concentrations of nitrates and low amounts of organic carbon. In this project, both the constructed wetlands system and the recharge area’s vadose zone are managed as treatment mechanisms to reduce levels of nitrates and other potentially problematic water quality constituents.

The Avondale wetlands consists of a series of open water treatment cells containing central, shallowly submerged islands where emergent vegetation will satisfy both treatment capacity requirements and aesthetic considerations. To ensure the viability of the wetlands treatment process, a 5,000 AFY pilot program will determine the effectiveness of the denitrification reaction using either internally-produced or externally-added organic carbon.

The wetlands treatment system consists of three separate groups consisting of 5 cells each. Within each group, the water flows sequentially through each of the 5 cells to optimize mixing and contact time with the nitrate-reducing bacteria that live at the water-plant and water-substrate interfaces. Three species of bulrush (Scirpus sp.) are planted to ensure optimal treatment opportunities as well as an aesthetic appearance. Following wetlands treatment, the water is transferred by a 1.5-mile long pipeline to infiltration ponds located along the eastern bank of the Agua Fria River.

The approximately 20 acres of infiltration ponds will be located in a pre-existing channel of the Agua Fria River. Hydrogeologic investigations determined lithologic information,

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vertical percolation rates, review of existing well logs and areas to install dedicated monitoring wells. Mounding analyses conducted as part of the application for an Underground Storage Facility permit from Arizona Department of Water Resources (the permit has since been awarded), indicate that, after 20 years of continuous recharge at 5,000 AFY, a groundwater mound would be created that is approximately 9 ft above static water level (SWL) at 1,000 ft from the center of the recharge area, and less than 1 ft above SWL at 18,000 ft away. At the full-scale rate of 40,000 AFY, mounding analysis indicates SWL will rise to approximately 65 feet above the original SWL at a distance of 5,000 ft, and 10 ft at 50,000 ft away. In actual operation, the mound will not reach these heights because of on-going local groundwater extraction.

This project also represents the value of public-private partnerships for both the development and the funding of new concepts in the water supply industry. The land surrounding the treatment area is incorporated into an overall residential development by a local developer. The recharge basins and surrounding land were purchased by the City using public funding.

**INTRODUCTION**

The City of Avondale, Arizona, located west of Phoenix, currently relies on groundwater resources to meet its municipal and agricultural water demands. The City has substantial allocations for surface water supplies from the Central Arizona Project (CAP) and Salt River Project (SRP) that require treatment prior to potable use. In lieu of a traditional, relatively expensive water treatment plant, the City has created a project, the Wetlands of Avondale, to design a treatment system that moves the untreated CAP and SRP water through constructed wetlands, transfers the water to spreading basins for aquifer recharge, and finally extracts the potable water using existing municipal production wells (see Figure 1). This technique uses the natural biochemical processes of plant-hosted bacteria and vadose zone soils as a substitute for the mechanical and chemical cleaning of typical water treatment facilities.

The project is designed in two phases beginning with a 2-year pilot project. An Underground Storage Facility permit granted by Arizona Department of Water Resources (DWR) allows for 5,000 acre-feet per year (AFY) of recharge during the pilot project. A full-scale project may treat 40,000 AFY or more. The project is interdisciplinary, involving contributions from hydrogeologists, water treatment engineers, biologists, planners, assessment district specialists, and developer liaisons.

**SOURCE WATER TREATMENT REQUIREMENTS**

The City of Avondale has a 4,100 AFY allocation of CAP water which, like many other Phoenix-area communities, is currently unusable because of a lack of treatment capability. For most of these communities, the historic use of inexpensive groundwater requiring minimal
treatment has allowed them to operate without water treatment plants. With the recent addition of abundant CAP water to the area’s resources and State mandates to minimize overdraft, an increasing demand now exists for this water supply. Because the construction of treatment plants is expensive, many communities are searching for more cost-effective alternatives, often involving artificial groundwater recharge and/or wetlands as alternative treatment mechanisms.

The City of Avondale is located near the western terminus of SRP’s Grand Canal which transports water from a variety of sources, including: CAP canal, SRP’s Salt and Verde river system, groundwater, agricultural runoff and storm drain returns. The latter three sources cause the distal reaches of the Grand Canal to occasionally have relatively poor water quality, with high nitrate concentrations being of primary concern (see Figure 2). SRP pumps groundwater into the canals as needed to balance the fluctuating water demand at the distal reaches of the canals. The agricultural runoff and storm drain returns are an artifact of the original design of the canals as an agricultural water supply and reuse system. Typically, nitrate concentrations are well within Maximum
Contaminant Levels (MCL’s), but during limited periods the concentration can increase dramatically (see Figure 2). The wetlands are designed to treat the water to ensure that these high nitrate concentrations are removed prior to aquifer recharge and subsequent potable use.

WETLANDS DESIGN

DESIGN CONSIDERATIONS

Prior to design of the wetlands component of the system, a series of fundamental considerations were identified. These were:

1) the source water has highly variable amounts of nitrate (as N), with peaks as high as 18 mg/l;
2) the source water contains relatively low concentration of organic carbon which is necessary for the viability of the nitrate-reducing bacteria;
3) the annual month-long cleaning of the SRP canal system will require that an alternative water source be used, such as one of several nearby wells;
4) the wetlands needed to be designed as an aesthetic amenity for both a residential development and wildlife; and,
5) the wetlands capital construction and operating/maintenance costs needed to be significantly lower than alternative forms of surface-water treatment.

Natural wetlands environments are well documented as being highly effective in reducing nitrate concentrations because of the nitrogen demand of various microbial organisms that grow on the surfaces of roots, rhizomes, stems and leaves of specialized aquatic plants (e.g., Crumpton et al., 1993). Although constructed wetlands treatment processes are becoming more commonly employed as either an alternative or an enhancement to the traditional municipal wastewater treatment, the source water is typically comprised of a much higher percentage of biological material than is present in the Avondale case. This wastewater-derived biological material contains significant concentrations of organic carbon which is a critical component in the biochemical process of nitrate-reducing biota. For low-carbon source water, as in the case of Avondale’s CAP-SRP water supply, evaluation of the potential for the natural systems to effectively reduce the nitrate concentration will be required. Although it is expected that the growth and subsequent decay of the bulrush plant material will provide a sufficient in-situ source of organic carbon, regular monitoring of the combined system’s organic carbon content and nitrate-reducing capability will be conducted. Once the cycle of growth and decay is fully developed, we expect that this type of system may be more efficient than a wastewater-sourced wetlands because in a wastewater system, the excess organic material from the source water may actually be in competition with the biological processing capacity of the system (see Baker, 1993 and Wetzel, 1993). If, however, operation of the pilot project indicates that an outside source of carbon is required, there are several sources possible, such as the addition of processed agricultural by-products, or other carbon-based solutions.
DESIGN CRITERIA

The anticipated performance of the treatment process includes nitrate concentration reduction of approximately 50%, to produce treated water with a nitrate concentration (as N) of less than 10 milligrams per liter. The efficiency of the treatment will be measured by regular sampling efforts at the SRP outfalls, key locations within the wetlands, at the wetlands outflow and in key monitoring wells.

Sedimentation Basins: Two sedimentation basins (8 acres total surface area) are included at the beginning of the system which slow velocities of the canal water and to allow settling of suspended solids (see Figure 1). These basins were excavated sufficiently deep to allow for many decades of sediment accumulation before the need for clean-out.

Treatment Cells: A series of 12 treatment cells provide a continuous, gravity flow system for the CAP-SRP water. The treatment cells have a combined surface area of 64 acres, of which approximately 32 acres are shallowly-submerged “islands”, populated with bulrush (see Figure 1). The system is designed for a flow capacity from 5,000 AFY (4.5 mgd) to 15,000 AFY (13.5 mgd). At the lower flow rate, the longer residence time in the treatment system is calculated to provide sufficient treatment to reduce a nitrate concentration of 20 mg/l to 10 mg/l. The maximum flow rate will provide less total treatment, but can be used to increase treatment volumes during periods when source water is of better quality.

The treatment cells are separated into three groups consisting of 5 cells each. Within each group, the water flows sequentially through each of the 5 cells to optimize mixing and contact time with the nitrate-reducing bacteria that live at the water-plant and water-substrate interfaces. The water flow is gravity-driven by a 2 foot head differential between each successive cell, and controlled by a series of fixed weirs. To ensure adequate residence time in the planted areas, and to minimize the potential of short-circuiting, the water enters and exits each cell through a riser pipe located near the center of the distal end of each central island (see Figures 3 and 4). Gravel berms surrounding island areas and central risers are also included to ensure sufficient contact time with the wetland plants.

On each wetland island, three species of bulrush (Scirpus californicus, S. Acutus, and S. Americanus) are planted to provide optimal treatment opportunities as well as an aesthetic appearance. The City has established a bulrush nursery to propagate a sufficient number of plants.

Water Transfer Pipeline: Following wetlands treatment, the water is transferred 1.5-miles to the west in a gravity-driven pipeline to a series of infiltration basins located along the eastern bank of the Agua Fria River.
RECHARGE DESIGN

Following the wetlands treatment, the treated water is transferred to 28 acres of spreading basins where the water infiltrates into the unconfined aquifer. For the pilot project, the City of Avondale’s existing municipal production wells are used for recovery. An expanding recharge and recovery program could be designed to improve water quality in poor water quality areas of the aquifer. Design of recharge facility also included consideration of future expansion as additional water supplies are added to the system.

RECHARGE CONSIDERATIONS

Studies were conducted of the expected infiltration rates of the spreading basins, design and maintenance criteria, effect of spreading on adjacent portions of the aquifer, and effects on regional water quality.

Infiltration Rates: The recharge site is located in the stream gravels adjacent to the Aqua Fria River, which is a typical desert wash that varies from high flow rates during storms
to no surface flow during much of the year. Shallow sediments are primarily sands and
gravels with local sandy clay or silty clay layers. Discontinuous one- to two-foot zones of
cemented sands and gravels are also present. Hydrogeologic investigations conducted to
determine site characteristics included nine borings for lithologic information and assessment
of vertical percolation rates, review of existing well log and boring data from surrounding
areas and siting of three dedicated monitoring wells. Considering the necessity of drying out
the basins on a regular basis, which could be up to 50% of time, conservative estimates of
long-term infiltration rates range from 1.0 to 2.0 ft/day.

**Impacts on Local Groundwater Regime:** As part of the permitting process required by
DWR, an analytical model was prepared to determine the amount of groundwater build-up
generated by a given quantity of recharge, assuming that there was no additional pumping in
the basin to recover this recharged groundwater. The recharge mound created by the 5,000
AFY pilot project is approximately 9 ft at 1,000 ft from the center of the recharge area, and
diminishes to less than 1 ft high at 18,000 ft away. In the full-scale project model, assuming a
maximum of 40,000 AFY is being recharged for 20 years, the mound would build up to
approximately 65 ft high at 5,000 ft away, and 10 ft high at 50,000 ft away. In actual
operation, the mound will never reach these heights because the recharged water will not be
stored in the aquifer over these time periods. The actual effects of the mounding will be
closely monitoring to ensure that groundwater flow patterns at a Superfund site at Goodyear
Airport, 3.7 miles southwest of recharge project, are not changed.

**Groundwater Quality:** The water quality of the combined SRP and CAP waters in the
Grand Canal is highly variable depending on the relative proportions of the various water types
present and the time of year. The chemistry of the native groundwater is also variable de-
pending on both the location and depth of sampling. In general, however, most chemical
parameters in the surface water and groundwater are similar, such as alkalinity, pH, calcium,
magnesium, and hardness. Total dissolved solids (TDS) content in the groundwater ranges
from 310 ppm to 1,000 ppm, whereas TDS of the combined CAP-SRP water at Avondale
ranges from approximately 200 to 600 ppm, with rare periods as high as 850 ppm. Significant
changes are not expected in TDS concentrations during the wetlands treatment process.

**Permitting:** Both an "Underground Storage Facility Permit" and a "Managed Facility
Permit" have been issued by DWR for the pilot scale project. A hydrologic study was prepared
as part of that permitting process, and several meetings and discussions were held with DWR
and Department of Environmental Quality (DEQ) staff concerning the design issues of the
project. An area of interest to most parties was the calculated effects of the maximum recharge
volume on groundwater flow directions and flow rates at the Phoenix-Goodyear Airport to the
southwest of the project area, where a full scale groundwater remediation program is being
conducted. The groundwater modeling for the pilot project showed little effect of recharge at
the contaminated site. The actual water level build-up is concentrated between the recharge
basins and Avondale's production well locations. DEQ recommended that future Avondale
production well sites be located between the recharge site and clean-up site to ensure mitigation of the potential effects of future larger scale recharge operations.

**DESIGN CRITERIA**

Based on these considerations, the following design criteria were applied:

1. the recharge site was selected based on appropriate location and optimal sediment characteristics;
2. the size of the basins was based on expected infiltration rates of approximately 1 ft/day;
3. basin layout was conducted to utilize existing topography as much as possible;
4. inflow to the basins was designed to allow independent operation of each basin which is necessary to provide operational flexibility based on the infiltration rate decay characteristics of each;
5. Mass grading of the recharge site was coordinated with the site grading of an adjacent residential development to provide a two-way benefit of removing excess soil from the site and providing it to the development to regrade the site for sewer line minimum gradient requirements.

**BENEFITS OF A COMBINED CONSTRUCTED WETLANDS AND ARTIFICIAL RECHARGE PROGRAM**

**COST EFFECTIVENESS**

One of the driving forces behind the evaluation of a constructed wetlands and recharge basin system *in lieu* of a traditional water treatment plant was the potential for a significant cost savings. Based on design and construction costs, the project cost is approximately 60% of a comparable water treatment plant. Similarly, the operations & maintenance costs are expected to be approximately 50% of a comparable treatment plant.

**MITIGATION OF GROUNDWATER OVER-PUMPING AND WATER QUALITY DEGRADATION**

The long history of groundwater dependence in this part of Arizona has resulted in greatly overdrafted aquifers. This usage combined with nitrate-generating agricultural practices has led to both lowered water levels and water quality degradation in many parts of the West Salt River basin. By recharging the aquifer with better quality water, further overdraft can be mitigated and water quality improvements may be attained. By implementing this program utilizing renewable water resources, Avondale will be in compliance with the 1980 Groundwater Management Act, which requires groundwater usage be reduced to within the safe yield of the aquifer system. A further advantage of this program is the control of migration of poor quality groundwater into the remaining "sweet" zones currently used by Avondale municipal wells.
RECREATIONAL AND WILDLIFE AMENITY

Incorporated into the overall project design is planning for both recreational aspects and wildlife habitat creation. The wetlands treatment areas will be aesthetically designed and will be bordered by recreational areas which in turn will blend into an adjacent residential development area. An important additional benefit is the added habitat created by the wetlands itself, as well as the park area for birds and other wildlife. These combined uses essentially spread the costs of the facility across a much wider range of benefits.

MUNICIPAL-PRIVATE SECTOR COOPERATIVE EFFORT

This project represents the value of public-private partnerships for both the development and funding of new concepts in the water supply industry. Private funding allowed the project to move from concept to feasibility to permitting. Public financing, which included a combination of City grants, development fees, and property taxes, was then used for design and construction funding. The land surrounding the treatment cells is incorporated into an residential development plan by a local developer. The recharge basins and surrounding land will be purchased by the City using public funding and issuance of public bonds.

DISCUSSION

By combining the recent advances in both constructed wetlands treatment and artificial recharge technology, this project is an important, cost-effective addition to the capabilities of water purveyors to provide potable water supplies from untreated sources. Wetlands projects also provide aesthetic enhancements to community environments and add riparian habitats. For many Phoenix area communities, the high-cost of constructing a water treatment plant to treat their allocation of CAP water is prohibitive, especially considering the current low cost of pumping groundwater. Spreading basins alone could be used if the imported water meets quality standards, but when it is substandard, the combined methods of wetlands treatment and artificial recharge can provide a promising alternative.

REFERENCES


THE TOWN OF GILBERT EXPERIENCE WITH AQUIFER
STORAGE AND RECOVERY OF RECLAIMED WATER

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1Paper presented at the 8th Biennial Symposium on the Artificial Recharge of
Groundwater, Tempe, AZ June 2-4, 1997
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ABSTRACT: The Town of Gilbert is experiencing rapid growth, which has challenged local officials and consultants to plan for the most efficient use of ground water and renewable water resources. One important element of the planning is the 100% reuse of the reclaimed water produced in the Town. This has been accomplished by implementing direct, nonpotable reuse and an aquifer storage and recovery program. The Town delivers reclaimed water to three (3) regional parks, two (2) housing subdivisions, two (2) farms, its municipal complex lake, and three (3) golf courses for irrigation and lakes maintenance. Water that cannot be reused directly is recharged through percolation basins and a vadose zone injection well. The stored water is recovered through shallow wells for use in two (2) housing subdivision lakes on which water-skiing is approved. The Town has constructed a deep well within the hydrologic impact area of the recharge project and can recover water for potable use. Recent expansion of the recharge facility incorporated wildlife viewing facilities and enhanced riparian habitat. The Town of Gilbert recharge and recovery project is the focus of this paper, and includes discussion of management, operations, monitoring, habitat improvements, and volunteer efforts.

INTRODUCTION

The Town of Gilbert is located in Central Arizona approximately 15 miles southeast of Phoenix. Geographically, this area is at the northern end of the Sonoran Desert and receives an average of 7.5 inches of precipitation a year. Summer temperatures reach an average high of 108 degrees (F) and winter temperatures can drop into the upper 20’s. With the taming of the Salt and Verde Rivers and the construction of the Salt River Project (SRP) Irrigation System, the area has been actively farmed since the early 1900’s.

The Town of Gilbert is experiencing rapid growth with an incorporated area population swelling from 5,700 in 1980, to 72,000 in 1997. Based on the Town’s General Plan, the ultimate population is estimated to be 323,100 (Camp Dresser & McKee, 1996).

In 1980 the Arizona Legislature passed the Ground Water Management Act. The Act’s municipal conservation program is based on continued, “reasonable reductions in per capita use and such other conservation measures as may be appropriate for individual users.” (ARS 45-564.A.2) As an incentive to use reclaimed water, its use is not counted in the per capita use calculations for determination of compliance with the Act. In addition, recharged and stored reclaimed water retains its identity as reclaimed water when withdrawn from the area of hydrologic impact (geographic area where the effects of the recharge can be detected and measured).
A significant portion of the Town's water demand can be satisfied with reclaimed water. Direct uses (reclamation plant to the customer) of the reclaimed water can include landscape and crop irrigation, lake makeup water, and industrial applications. Indirect use (recharge and recovery) can include any water demand the Town chooses to satisfy with this resource. The recharge basins also provide a riparian habitat for wildlife nesting and foraging, as well as birding opportunities for the public.

WATER RECLAMATION FACILITY

The Gilbert Water Reclamation Facility (WRF) was constructed and was originally owned and operated by the Parsons Corporation. Privatization of this facility was necessary to meet the needs of the growing community. The Town is currently purchasing the facility from Parsons, and will continue to contract the operation and maintenance of the facility. The facility began operation on October 1, 1986 and has a design capacity of 5.5 MGD, with provisions for an ultimate capacity of 11.0 MGD. It uses the oxidation ditch method for treating raw sewage. The facility was designed with 100% reuse in mind, and effluent must meet state requirements for open access landscape irrigation. These requirements include: 10 milligrams per liter (mg/L) biological oxygen demand (BOD), 10 mg/L suspended solids (SS), fecal coliform of 25 CFU/ml geometric mean or 75 CFU/100 ml single sample maximum, and maximum turbidity of 5 NTU. In actual operation, the facility effluent averages less than 5 mg/L BOD, less than 1 mg/L SS and about 0.5 NTU of turbidity. Effluent that is sent to the recharge basins is not filtered or chlorinated to prevent the formation of trihalomethane compounds (THMs). A denitrification system was placed in operation on May 26, 1996 and has enabled the plant to discharge reclaimed water with a total nitrogen concentration below 10 mg/L, with the average being 7.5 mg/L.

DIRECT REUSE

The Town currently has sixteen miles of reclaimed water mains that deliver water to three (3) regional parks, three (3) golf courses, farm land, the municipal complex, and two (2) housing subdivisions.
RECHARGE

Reclaimed water that is not consumed by customers is stored in an underground storage and recovery project. The recharge site is 73 acres in size and located immediately south of the WRF (figure 1). The facility's eleven ponds were constructed by excavation and berming. The six easterly ponds were constructed in 1984 (Carollo Engineers) and water is distributed by way of a concrete ditch running through the middle of the site. The five westerly ponds were constructed in 1993 (Engineering Science) and water is delivered by means of an underground pipe system. The eleven basins are cycled through wet and dry periods. Each basin is plowed as needed to control vegetation and break up the soil surface. At least once each year, each basin is cross-ripped (deep plowed in perpendicular directions) to 30 inches in depth to further break up the surface to restore infiltration capacity.

The Town received a $55,000.00 grant from the Arizona Department of Water Resources to construct and pilot test a modified dry-well for reclaimed water recharge. The modified dry-well is a four (4) foot diameter, ninety foot deep drilled shaft with a twelve inch PVC casing centered in it. The annulus is backfilled with 3/8 inch gravel and a four (4) inch injector pipe and two 1-1/2' sounder and bailer pipes are housed in the casing. A concrete pad is placed at ground surface to protect the well head and provide additional support for metering and valving (CH2M Hill, 1995). The modified dry-well, which is now referred to as a vadose zone injection well (VZIW), was constructed near the recharge basins south of the WRF for seasonal storage of effluent. The VZIW began injecting tertiary treated (filtered and disinfected) effluent on January 7, 1997. The VZIW has been operated continuously with frequent monitoring of flow, water levels, water quality, and system pressure. The initial target rate for continuous injection through February 24 was 350 GPM, or about 0.5 MGD. After step tests were performed on February 25, the target flow rate was increased to 700 GPM, or 1.0 MGD. As of April 30, 1997, the VZIW had not exhibited any signs of clogging, and water levels have fluctuated less than 0.3 foot in a monitoring well that is approximately 50 feet north of the VZIW.

The geology of the area is characterized by three (3) alluvial units to a depth of approximately 2,000 feet where crystalline rocks are encountered. The Upper Alluvial Unit is 250 -- 300 feet in thickness and is composed of boulders, cobbles, sand and gravel deposited by the Salt River. The upper 50 -- 100 feet of this unit consists of primarily fine grained clay and clay loam deposits. At the recharge site these fine grained deposits are approximately 70 feet in thickness. Groundwater in this unit is high in total dissolved solids and nitrates from irrigation percolation. The permeability of the Upper Unit averages 2,000 gallons
Figure 1 Recharge and Recovery Site

per day per square foot. (Ken Schmidt and Associates, 1987). The Middle Alluvial Unit extends from approximately 300 feet to 1,000 feet in depth and consists of clay, sandy clay, mixtures of fine sand and silt and a few thin sand strata. Approximately 40% -- 50% of the deposits are sand and gravel. Deeper ground water in the Middle Alluvial Unit is generally suitable for public supply. The permeability of this Unit averages 140 gallons per day per square foot. (Ken Schmidt and Associates, 1987). The Lower Alluvial Unit extends from approximately 1,000 feet in depth to 2,000 feet in depth and consists of fine grained material with less than 10% sand and gravel deposits.
The volume of reclaimed water sent to the percolation ponds is measured by an ultra-sonic flow meter and recorded. The recharge site has a permit capacity of 3,314 acre-feet (AF) per year (one acre-foot = 325,851 gallons), and has experienced an average percolation rate of 0.24 feet per day. In 1989, the Town was credited with recharging 739 AF; in 1990 the Town recharged 1,660 AF, in 1991 1,667 AF, in 1992 1,787 AF, in 1993 1,109 AF, in 1994 2,257 AF, in 1995 2,477 AF, and in 1996 2,900 AF (approximately) were recharged. The total volume recharged is 14,596 AF, or 4,756 million gallons. Evapotranspiration from vegetation and evaporation from the surface of the ponds are taken into consideration to determine the actual amount of water recharged. The volume recharged and recovered is reported to the Arizona Department of Water Resources (ADWR) annually.

RECOVERY OPERATIONS

Since 1990 the Town has recovered 920 AF of recharged water from the shallow recovery wells (G-7 and G-8) for delivery to two (2) water-ski housing subdivision lakes. In November 1992, the Town completed the construction of a municipal supply well within the hydrologic impact area of the recharge project. On September 16, 1992 the Town was issued an ADWR Recovery Well Permit for recovery of up to 1,550 AF (488.8 million gallons) per year of water from this well for domestic and irrigation uses. The quality of water produced by this well equals or exceeds the quality of all other wells in the Town’s system. For example, nitrate, hardness and TDS are critical factors; these parameters measure 0.5 mg/L, 154 mg/L, and 710 mg/L respectively for this well.

GROUND WATER MONITORING

Water levels are measured and water samples are obtained monthly from eight monitor wells. These wells are shown on Figure 1. Seven (7) of the monitor wells (G-1, 2, 4, 7, 8, 9 and 10) owned by the Town penetrate 250 feet into the Upper Alluvial Unit. One well is owned by a local construction firm (Well 12bcb) and penetrates the Middle Alluvial Unit. Ground water in the Upper Unit moves toward the north north-west at a gradient of approximately 3 feet per mile. Monitoring results are illustrated on Table 1 by representative data from Well G-4, providing upgradient ground water quality, and Well G-7, providing downgradient ground water quality.
TABLE 1
Water Quality Results (January 21, 1997)

<table>
<thead>
<tr>
<th>PARAMETER</th>
<th>WELL G-4 UPGRAIDENT</th>
<th>WELL G-7 DOWNGRADIENT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nitrate (N)</td>
<td>6.7</td>
<td>3.2</td>
</tr>
<tr>
<td>Major Cations (mean-mg/L)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sodium</td>
<td>318</td>
<td>376</td>
</tr>
<tr>
<td>Calcium</td>
<td>153</td>
<td>44</td>
</tr>
<tr>
<td>Magnesium</td>
<td>38</td>
<td>15</td>
</tr>
<tr>
<td>Potassium</td>
<td>13</td>
<td>6</td>
</tr>
<tr>
<td>Major Anions (mean-mg/L)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Chloride</td>
<td>480</td>
<td>520</td>
</tr>
<tr>
<td>Sulfate</td>
<td>209</td>
<td>111</td>
</tr>
<tr>
<td>Bicarbonate Alkalinity</td>
<td>372</td>
<td>158</td>
</tr>
<tr>
<td>Dissolved Metals (mg/L)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Arsenic</td>
<td>0.004</td>
<td>0.018</td>
</tr>
<tr>
<td>Barium</td>
<td>0.056</td>
<td>0.047</td>
</tr>
<tr>
<td>Cadmium</td>
<td>&lt;0.0005</td>
<td>&lt;0.0005</td>
</tr>
<tr>
<td>Lead</td>
<td>&lt;0.002</td>
<td>&lt;0.002</td>
</tr>
<tr>
<td>Mercury</td>
<td>&lt;0.0002</td>
<td>&lt;0.0002</td>
</tr>
<tr>
<td>Selenium</td>
<td>&lt;0.005</td>
<td>&lt;0.005</td>
</tr>
<tr>
<td>Silver</td>
<td>&lt;0.01</td>
<td>&lt;0.01</td>
</tr>
<tr>
<td>Total Chromium</td>
<td>&lt;0.004</td>
<td>&lt;0.004</td>
</tr>
<tr>
<td>pH (mean)</td>
<td>7.5</td>
<td>7.6</td>
</tr>
<tr>
<td>Conductivity</td>
<td>2270</td>
<td>2010</td>
</tr>
<tr>
<td>Purgeable Halocarbons (mg/L)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bromodichloromethane</td>
<td>&lt;0.0005</td>
<td>&lt;0.0005</td>
</tr>
<tr>
<td>Chloroform</td>
<td>&lt;0.0005</td>
<td>0.0006</td>
</tr>
<tr>
<td>Dibromochloromethane</td>
<td>&lt;0.0005</td>
<td>&lt;0.0005</td>
</tr>
<tr>
<td>Bromoform</td>
<td>&lt;0.0005</td>
<td>&lt;0.0005</td>
</tr>
<tr>
<td>Volatile Organic Compounds (VOCs)</td>
<td>ND</td>
<td>ND</td>
</tr>
</tbody>
</table>

NOTE: ND = Not Detected

Wells G-7, G-8, and G-10 serve a dual role of monitoring and recovery, and are equipped with 250 GPM pumps. They have 8-inch diameter steel casings that are perforated from 150 to 250 feet in depth. The specific capacity of these three (3) wells is about 200 gallons per minute per foot of drawdown. The static water level in Wells G-1, 2, 4, 7, 8, 9 and 10 ranges from 125 to 130 feet below land surface and is highly dependent on the operation of the two (2) high production capacity SRP irrigation wells located one-quarter mile to the west and south of the site. The Aquifer Protection Permit issued for the Underground Storage and Recovery Project has monitoring requirements for both plant effluent and all monitoring wells. The frequency of sampling is monthly, quarterly, and yearly depending upon the parameter monitored.
WILDLIFE HABITAT ENHANCEMENTS

Typically, recharge ponds do not provide much value to wildlife for breeding, roosting, or cover, and the near absence of vegetation surrounding the ponds limits the diversity of species using the site. Consequently, the Town has incorporated elements into the recharge ponds to enhance wildlife habitat and provide public use facilities. The Town received two (2) Heritage Fund Grants ($120,000.00) from the Arizona Department of Game and Fish to help fund design and implementation of the habitat enhancement project. The project included construction of an observation platform, ramada, and access ramp adjacent to the recharge ponds built in 1993; interpretive signage; roosting and nesting structures; the planting of marsh, riparian, and upland vegetation; and construction of a public parking area adjacent to the fire station. (Jones & Stokes Associates, Inc. 1993) The recharge ponds provide valuable riparian habitat for wildlife resting and foraging, and the Maricopa Audubon Society has identified 140 bird species utilizing the site. In the Spring of 1994, emergent marsh vegetation was planted on the south and west sides of the permanent pond. Riparian vegetation (cottonwoods, willows, arrow weed, mule fat) was also planted along the levees of the recharge basins in the Spring of 1994, 1995, and 1997. The planting was accomplished with enthusiastic volunteer effort from the community, Boy Scouts, schools and the Audubon Society.

In addition to its primary function of recharge, the facility is used by the community in a number of diverse ways. The Town has constructed roosting islands in several of the basins and planted a grain crop on adjacent land for water fowl forage. The site is continuously being used by local Boy Scouts in their efforts to earn their Eagle Scout badges. Local schools and colleges use the facility for science and research projects. Most of the on-going habitat enhancement effort is performed by volunteers. The public has pedestrian access to the perimeter of the recharge facility and the site is regularly visited by birders. In 1994, this project received a Governor's Pride Award for environmental leadership and is one of two (2) urban sites used for introduction of rehabilitated animals back into the wild.

Because the existing recharge facility will not accommodate the ultimate effluent flow (11.0 MGD) from the WRF, another recharge facility is currently being designed. The new facility will be located on 110 acres immediately behind the new Maricopa County Regional Library that will be constructed on the south east corner of Greenfield and Guadalupe Roads. The library has been designed to provide an excellent view of the new facility, which is being designed as an educational park, and will incorporate a recovered water urban fisheries lake and other park-like amenities in addition to the recharge basins. This new
facility will recharge tertiary treated water and will be enhanced similarly to the existing facility to attract a wide variety of wildlife. The facility will be open to the public every day from dawn to dusk, and connects two of the largest trail systems in the metropolitan area.

CONCLUSION AND SUMMARY

The Town’s policy of 100% reuse of its reclaimed water resources is being achieved with a combination of direct reuse and indirect reuse (ground water recharge, storage and recovery). This is helping the Town achieve the State of Arizona ground water management goals and provide a safe water resource for current and future Town residents. The recharge process consists of percolation basins that are cycled through wet and dry periods and plowed (ripped) to enhance percolation. The quality of upgradient ground water and downgradient ground water are monitored by eight (8) shallow wells. Monitoring results have shown a general improvement of shallow ground water quality as a result of the recharge project. In particular, nitrate levels had shown a trend of improvement since recharge began in 1989. Denitrification facilities have been constructed to ensure reclaimed water total nitrogen is below 10 mg/L. Expansion of the recharge facility in 1993 included elements to enhance wildlife habitat and educational/viewing facilities for the public. Direct injection of effluent through a modified dry-well was begun in January, 1997, and has proven thus far to be an economical means of recharge. The success of the existing recharge facility and the need for additional recharge space has led the Town to develop a second recharge facility that will be larger and more accessible to the public.
REFERENCES


THE ARIZONA WATER BANKING AUTHORITY

Storing Colorado River Water for Arizona’s Future

Water has been called the “lifeblood of the West” and Arizona has been at the forefront of developing sound water management policy, implementing water conservation measures and appropriately planning for the use of this valuable resource. For over 70 years, Arizona leaders have worked to ensure that Arizona’s communities have dependable, long-term water supplies to sustain quality of life and to allow for future growth. From securing the State’s fair share of Colorado River water and gaining congressional authorization of the CAP to crafting the 1980 Groundwater Management Code, their foresight and planning have provided the water supply that has served and will continue to serve our growing communities and maintain a high quality of life in Arizona for years to come. Full utilization of the state’s Colorado River entitlement has been the key to Arizona’s water management policy for decades.

Under normal conditions, 2.8 million acre feet of Colorado River water is available for use by Arizona. It is anticipated deliveries to Arizona users on the Colorado River will be approximately 1.3 maf, leaving approximately 1.5 maf available for delivery through the CAP. According to the Arizona Department of Water Resources, under present development scenarios and without considering the impact of the Arizona Water Banking Authority, Arizona will not fully utilize its share of Colorado River water until the year 2030. Between
now and then, the accumulated amount of Arizona’s entitlement left in the River could be as high as 6 maf.

Recognizing that the full utilization of the CAP is critical to Arizona’s water future, early in the 1990’s, Governor Symington and the State Legislature in keeping the tradition of foresight with regards that future began to confront the issues relating to the under utilization of Arizona’s Colorado River entitlement. To address these issues, Arizona’s leaders created the following entities: the Governor’s Task Force on CAP Issues; the Governor’s CAP Advisory Committee; and the Joint Legislative Committee on Colorado River Issues. Among the products of these efforts was the creation of the Arizona Water Banking Authority.

**Background**

In 1922, with the signing of the Colorado River Compact, the waters of the Colorado River were divided equally between the Upper and Lower Basin states. With the Upper Basin (Wyoming, Utah, Colorado, New Mexico) and Lower Basin (Arizona, California and Nevada) each receiving the right to use 7.5 million acre feet (maf) from the Colorado River annually. In 1928 with the passage of the Boulder Canyon Project the water available to the Lower Basin states was further divided amongst those states (Arizona - 2.8 maf, California - 4.4 maf and Nevada - .300 maf).
In 1963 the U.S. Supreme Court confirmed Arizona's annual entitlement to 2.8 maf of Colorado River water in the landmark case of Arizona vs. California 373 U.S. 546 (1963).

After the confirmation of Arizona's share of Colorado River water by the U.S. Supreme Court, Arizona's leaders went to work in the congress to secure funding for a project to bring a portion of Arizona's water to the population bases of Central Arizona.

Central Arizona Project

Subsequently, the United States Congress passed the Colorado River Basin Project Act in 1968, authorizing construction of the Central Arizona Project (CAP) aqueduct, the largest public works project in the U.S. at the time. The Act included construction of the CAP aqueduct and all associated pumping plants and siphons, the New Waddell Dam, and the raising of Roosevelt Dam.

In 1971 the Arizona State Legislature authorized creation of the Central Arizona Water Conservation District (CAWCD) as a three county (Maricopa, Pinal, Pima) district. The CAWCD operates canal and is responsible for the repayment of approximately $2 billion of the $4 billion construction costs for the CAP Project. Water was first delivered to the Phoenix metropolitan area in 1985, to the Pinal County area in early 1987, and finally to the Tucson area in 1992.
CAP deliveries increased every year from 1985 to 1990. However, in 1991 the CAP experienced a sharp decrease in municipal and industrial (M&I) and agriculture orders for CAP water, from 745,000 acre feet (af) in 1990 to only 420,000 af in 1991. The greatest amount of decline occurred because of decreased orders by agricultural districts. Many of the newly formed irrigation districts, along with the agricultural users paying the property taxes to support these districts, were in poor financial condition and CAP water proved cost prohibitive.

Governor’s Task Force On CAP Issues

Faced with the growing underutilization of Colorado River water, mainly due to low agricultural demand for CAP water, Governor Fife Symington met with agricultural, municipal and industrial water users to begin exploring possible solutions. Out of those discussions came appointment of a 16 member Governor’s Task Force on CAP Issues charged with developing recommendations to be submitted to the Governor for consideration. Task Force membership included a broad representation of municipal, agricultural, financial, legal, Indian and governmental interests from regions of the state affected by the CAP.

The Task Force held eleven meetings between February and July that were all very well attended by Task Force members and interested parties. The primary objectives of the Task Force were to address the issues of lost benefits to the State and the perceived jeopardy of Arizona’s entitlement due to the under-utilization of Arizona’s share of Colorado River water.
As deliberations of the Task Force began, it quickly became apparent that the largest impediment to increased use of CAP water by agricultural users was the high cost of the water. Several options were examined in an attempt to help offset these high costs, including increased property taxes, marketing of electric power and possibly leasing a portion of Arizona’s Colorado River entitlement to California and/or Nevada.

The Task Force ultimately developed recommendations that focused on increasing the use of CAP water by the agricultural sector, including: 1) evaluating irrigation districts ability to repay certain CAP costs; 2) possible restructuring of distribution system debts; 3) payment of delinquent CAP assessments on state land; 4) and potential relief of certain provisions of the 1982 Reclamation Reform Act (RRA). Additional recommendations included increased usage of CAP water by M&I; the intrastate marketing of CAP agricultural and M&I priority water; need for the resolution of Indian water rights claims; and the need for the generation of additional revenues through power marketing agreements and/or the interstate marketing of a portion of Arizona’s Colorado River entitlement to reduce CAP water costs.

Although no definitive solutions were reached, much of the work done by the Task Force provided the groundwork for the current successes of the CAP and the near full utilization of Arizona’s 2.8 maf entitlement projected for calendar year 1997. The Task Force recommendations were further developed by the Governor’s CAP Advisory Committee the next year.
After the September 1992 submission of the report of the Governor's Task Force on CAP Issues, it was quickly decided that additional information and options were needed to deal with the issues associated with the under-utilization of Arizona's share of Colorado River water. In mid-December of 1992, Governor Symington appointed the Governor's CAP Advisory Committee, charged with developing recommendations to assure the long-term viability of the CAP. The 34 member Committee was co-chaired by the Governor and Mark DeMichele, President and CEO of Arizona Public Service Company. Like the Task Force, the Advisory Committee was representative of political, municipal, agricultural, business, legal, Indian and environmental interests. The Committee was also geographically diverse with members from Maricopa, Pinal, Pima, Yavapai and Coconino Counties.

In forming the Advisory Committee, Governor Symington stated:

"The Central Arizona Project is Arizona's lifeline. It is our water supply for future growth and the underpinning of our progressive water management policies. It is critical for Arizona to pull together and develop a solution to make the Project work."

The Advisory Committee used five guiding principals in developing proposed recommendations: 1) protect Arizona's entitlement of Colorado River water to provide a secure long-term water supply; 2) ensure the financial integrity of the CAP; 3) identify how
the CAP may be used to enhance the state’s environment; 4) possibilities for use of CAP water to assist in Indian water rights settlements; and 5) utilize CAP water to meet the management goals and policy objectives of the state Groundwater Management Code. All five principals were ultimately included in final recommendations of the Committee and are largely reflected in subsequent legislation passed by the State Legislature.

Committee deliberations occurred between January and September of 1993. Public involvement was key in all workings of the Advisory Committee and the development of recommendations. A public involvement group was organized to review studies performed by the interagency study team, with participation by a wide range of potentially affected parties. In addition to the public involvement group, an Indian involvement group was created. This group held meetings with representatives of interested Indian communities to discuss Indian related issues. Indian participation was important because of the large CAP water service contracts held by Indian communities with the federal government.

A six-step investigation process was used in developing the final report and recommendations submitted by the Advisory Committee in October 1993.

Step 1 - Description of the CAP as of 1993; Step 2 - Likely future conditions without alternative action; Step 3 - Identification of problems, issues and concerns; Step 4 - Identification of solution elements; Step 5 - Working group analysis; Step 6 - Formulation of Recommendations.
The Governor’s CAP Advisory Committee examined a broad spectrum of issues related to state and federal water management policies and objectives relating to the operation of the CAP.

The Committee developed a set of policy issues with specific recommendations concerning the financial, water marketing, Indian and environmental issues surrounding the CAP. The overriding issue continued to be the protection of Arizona’s entitlement to Colorado River water to assure an adequate water supply for future economic development and growth. The following statement of policy was adopted by the Advisory Committee:

*The CAP was envisioned as the primary water management tool for the State to reduce its dependence on mined groundwater and to provide a renewable water supply for municipal, industrial and Indian related economic growth. The premise of the CAP would be a substitute for existing groundwater use in the agricultural sector was one of the fundamental justifications for the authorization and construction of the Project. Arizona continues to have a need for the full amount of CAP water but the current price of the water makes the agricultural sector’s conversion from groundwater to CAP water cost prohibitive. Without the agricultural component of CAP demand, it is likely the state will not fully utilize its entitlement in the near term.*

Additional recommendations included use of the CAP to assist in meeting water management
goals of the state’s Groundwater Management Code, including safe yield goals; using CAP to assist in Indian water rights settlements; encouraging CAWCD to consider a target pricing policy to increase CAP usage, while maximizing power marketing opportunities; development of an intrastate marketing of water program; studying the feasibility of banking unused Colorado River water in Arizona for use by California and Nevada in future years; and the creation of a state water bank to promote maximum utilization of Arizona’s share of CAP water.

Joint Legislative Committee on Colorado River Issues

In the Fall of 1995, the President of the Arizona State Senate and the Speaker of the Arizona House of Representatives created the Joint Legislative Committee on Colorado River Water Issues to examine ways to increase diversions and use of Arizona’s share of Colorado River water. Increasing use of Arizona’s allocation within Arizona was one of the key recommendations of the Governor’s CAP Advisory Committee and this Legislative Committee was formed to legislatively implement some of the recommendations of the CAP Task Force and Advisory Committee.

One of the prime recommendations of the Colorado River Issues Committee was an endorsement of a program to provide the necessary resources and organization to take currently unused Colorado River water and store it for future use by municipalities in Arizona. The Committee’s recommendation was translated into House Bill 2494, sponsored by Speaker
Mark Killian. HB 2494 was passed almost unanimously by the Legislature and signed by Governor Fife Symington on April 30, 1996, creating the Arizona Water Banking Authority and the Arizona Water Banking Authority Study Commission.

**Arizona Water Banking Authority**

The Arizona Water Banking Authority is a five person body charged with directing the activities of the AWBA. The Director of the Department of Water Resources chairs the Water Bank and members include the President of the Central Arizona Water Conservation District and three persons appointed by the Governor (of these appointments one person will represent CAP municipal and industrial water users, and one person will represent Colorado River water users along the River, and one person must be knowledgeable in water resource management issues). Additionally, the President of the Senate and Speaker of the House of Representatives each serve as or appoint one non-voting ex officio member to the Water Bank.

As indicated earlier, Arizona does not use its full 2.8 maf share of Colorado River water. Leaving a portion of Arizona’s water in the River is a lost opportunity. The Arizona Water Banking Authority seizes this opportunity and gives Arizona the capability to further secure the dependable water supplies necessary to ensure the state’s long-term prosperity.

The Arizona Water Banking Authority is envisioned to take temporarily underutilized Colorado River water and store it in Arizona for future use by Arizona communities in times
of shortages on the Colorado River or outages on the Central Arizona Project aqueduct by: 1) assuring adequate supply to municipal and industrial users in times of shortages on the Colorado River or disruptions of the CAP system; 2) meeting the management plan objectives of the state's groundwater code; 3) assisting in the settlement of Indian water rights claims; and 4) exchanging water to assist Colorado River communities.

Key benefits of the Arizona Water Banking Authority include:

**Drought Protection** - the AWBA will help protect communities dependent on the CAP by providing a stored reserve of water that can be tapped during times of drought on the Colorado River.

**Enhanced Water Management** - the AWBA provides the ability to replenish depleted groundwater aquifers with CAP water, thereby helping the State to meet its groundwater management goals and objectives.

**Indian Water Rights Settlements** - Indian tribes in Arizona have significant claims to water rights. Often the affected parties negotiate settlements to resolve these claims. The AWBA could provide another pool of water to be used in settlements. For instance, credits for stored groundwater could be transferred to a tribe as a component of a settlement.

**Statewide benefit** - Arizona communities along the Colorado River could benefit as well. For example, cities in Mohave County could acquire credits through the AWBA
for water stored in central Arizona and cash-in those credits by diverting water directly from the Colorado River.

**Interstate Water Transfers** - the AWBA could contract with similar authorities in California and Nevada to allow these states to annually acquire a portion of Arizona’s temporary surplus of Colorado River water. The contracting state would pay to store water in Arizona, helping to replenish Arizona’s aquifers, and in the future would be able to draw a similar quantity directly from the River. The program does not involve the sale of any future rights to water, only a specific quantity of unused water.

Funding for the Water Banks activities come from three sources and are deposited in the Water Bank Fund. Much of the money comes from existing revenue sources and from fees that will be charged to those benefiting directly from the stored water. Sources of money include:

- Fees for groundwater pumping currently collected within the Phoenix, Pinal and Tucson Active Management Areas. In the Phoenix AMA, Tucson AMA and most areas of the Pinal AMA pumping fees for water banking purposes would be set at $2.50 per acre foot beginning in 1997. For groundwater pumping in areas of the Pinal AMA not served by the CAP, the $2.50 fee would phase-in over seven years. Money from this source will be used to benefit the area in which it was collected.

- The CAWCD is authorized to levy a four cent ad valorem property tax in the CAP
service area. The CAWCD has the option to use this money for capital repayment of the CAP. However, if CAWCD determines they do not require the funds for their purposes they are required to deposit the funds in Water Bank Fund. In 1997 approximately 8 million will be deposited into the Water Banking Fund for the purchase of water for storage. Recognizing the long term nature of the Bank’s efforts, the tax was extended through 2016.

- A general fund appropriation based on the level of water storage the legislature and governor believe to be appropriate. In 1997, the legislature appropriated $2 million to the effort.

- Fees collected from the sale of stored water credits used for drought protection. Fees are charged only if the credits were originally paid for with general fund money.

- Money collected by the sale of stored water credits to out-of-state interests.

**AWBA Study Commission**

In addition to the Water Bank, the legislature created a Study Commission to investigate opportunities for additional water banking uses, identify mechanisms to help Indian communities with rights to Colorado River water participate in the program, and review the first year operation of the AWBA. The Commission will consist of the AWBA members and
two ex officio members as well as nine persons appointed by the director of Department of Water Resources. DWR is responsible for staffing of the Study Commission.

**Operating the Water Bank**

The Water Bank has been working diligently to store Arizona's annual Colorado River allotment in Arizona. This recharge by the Water Bank is not meant as a substitute for existing uses, but as means of utilizing Colorado River water that would otherwise have gone unused by Arizona.

The Water Bank meets monthly to keep the public apprised of the workings of the Water Bank. Meetings are held either at the Arizona Department of Water Resources or in communities around the state such as Tucson, Lake Havasu City and Yuma. The Bank also maintains an extensive mailing list.

The Bank is required to adopt an annual plan of operation which guide its operation throughout the year. In developing the its 1997 Plan of Operation, the Bank's staff met CAWCD who is responsible for delivering the water for the Bank and with many potential recharge entities including: all permitted irrigation districts in Maricopa, Pinal and Pima counties, the Salt River Project (SRP). An analysis was then made of the amount of potential recharge in each AMA/county and the amount of funds generated in each AMA/county by month to keep monies in the AMA/county of generation. The adopted by the Bank included both direct recharge at Underground Storage Facilities and indirect recharge at Groundwater
saving Facilities. An Underground Storage Facility is a facility in which water is diverted from the CAP aqueduct and placed in spreading basins, streambeds, or injection wells for direct recharge into an aquifer. A Groundwater Savings Facility is a facility usually an irrigation district that has a history of groundwater pumping and that are able to take CAP water “in lieu” of groundwater. The Bank provides the CAP water to the user and in return receives a long-term storage credit for the groundwater that is not pumped. The plan adopted by the Bank for 1997 will recharge approximately 360,000 acre feet of previously unused Arizona entitlement to Colorado River water.

Based on the requirements of the Plan of operation the Bank applied for and received nine Water Storage Permits. A Water Storage Permit is granted by DWR and is required before an entity can hold long-term storage credits. The Bank also entered into several three party water storage agreements among the Bank, CAP, and the holder of the Groundwater Saving Facility permit. In addition to the storage agreements the Bank entered into an agreement with SRP for the use of the direct recharge facility they operate in the Salt River

The Water Bank began delivering recharge water February 1997 and is scheduled to recharge over 100,000 af by June 30 and approximately 360,000 af by December 30, 1997. Including the Bank’s efforts the total consumptive use by the State of Arizona for 1997 is estimated to exceed 2.7 (maf)

Conclusion
The Arizona Water Banking Authority legislation is the type of flexible statewide policy that will help guide Arizona water planning into the next century. By storing substantial amounts of water in central Arizona, the Water Bank will help safeguard against future shortages on the CAP system, assist in meeting the goals of the Groundwater Code, and aid neighboring states without harming Arizona. The Water Bank will create Arizona’s "water savings account," helping to ensure that the water supplies future generations inherit from us are just as secure as those we inherited.
VADOSE ZONE RECHARGE WELLS:
FIELD EVALUATION OF AN INNOVATIVE DESIGN¹

Christine Close, Floyd Marsh, and Gary G. Small²

ABSTRACT

The City of Scottsdale (COS) is committed to the long-term need to store and reserve excess water supplies for the future. The Water Campus is being developed as part of that commitment. The Water Campus includes an advanced water treatment (AWT) plant, regional wastewater reclamation plant, and recharge and recovery system. The project will treat wastewater and redistribute reclaimed water to golf courses in north Scottsdale. Excess reclaimed water will be treated in the AWT plant to drinking water quality and recharged into the vadose zone.

Vadose zone recharge wells will be used for indirect injection of treated water at the Water Campus. The aquifer storage facility is master-planned to reach a maximum recharge capacity of 49,900 acre-feet per year in 2041. Three recharge well clusters were constructed for pilot testing to evaluate the recharge capacities of varying soil types and assess performance of several well designs. Pilot testing of the well designs began in 1993 and continues on a yearly basis as the water supply is available.

Aspects of the pilot testing included evaluation of multi-well operation, water quality changes, soil recharge capacity, and plugging potential. Five well designs were tested including recharging directly through the gravel pack within the well annulus. The design performance during pilot testing was used as the basis to recommend three standard well designs for full-scale implementation of the aquifer storage facility.

²Christine Close, E.I.T., HydroSystems, Inc., Tempe, AZ. Floyd Marsh is the Water Resources Director for the City of Scottsdale, Scottsdale, AZ, and Gary G. Small is a Registered Geologist and President of HydroSystems, Inc., Tempe, AZ.
INTRODUCTION

The Water Campus incorporates three recharge sites including the Well Clusters 1, 2, and 3 (East Pima site). The Water Campus is located on the northwest corner of Pima Road and Union Hills Drive (*Figure 1*). Well Cluster 1 includes the first four pilot vadose zone recharge wells constructed for this project. Well Cluster 1 is located to the east of Reservoir A in the northeastern portion of the site. Well Cluster 2 is located to the southwest of Well Cluster 1 and is bounded by the WAPA utility easement to the west and Union Hills Drive to the south. Well Cluster 3 is located near Union Hills Drive and 95th Street, approximately one mile east of Well Clusters 1 and 2.

DESIGN PERFORMANCE DURING RECHARGE TESTING

Four vadose zone recharge well designs plus the gravel pack of two wells have been pilot tested for the Water Campus. The pilot testing began in June of 1993 and has continued on a yearly basis as excess potable water supplies are available. A pilot test has been conducted on each of the eight existing wells for a period of at least 30 days with the exception of Vadose Zone Recharge Well 3-1 (RW 3-1). The first four vadose zone recharge wells, located at Well Cluster 1, consisted of different designs. The design differences included casing size, eductor system, and gravel pack material. The results of the pilot tests were used to enhance the designs and performance of additional pilot wells. Due to the multitude of pilot test data, the lessons learned from Vadose Zone Recharge Well 1-4 (RW 1-4) will be presented and how the incorporation of these lessons affected the performance of Vadose Zone Recharge Well 2-1 (RW 2-1).

RESULTS OF RW 1-4 PILOT TESTS

During the installation of the RW 1-4 eductor and transducer lines, a 4-inch eductor coupler broke free, causing the eductor and transducer lines to fall to the bottom of the well casing. A video survey showed that as the lines fell, they broke slots of the casing in a minimum of seven locations. The broken slots allowed the gravel pack material to enter the well casing. The damage resulted in a loss of 17 feet of casing from the bottom of the well including 10 feet of blank "reservoir" casing and seven feet of slotted casing. Also, the 33 feet above the lost sections of original casing were modified to repair damage caused by the falling lines.

The eductor design for RW 1-4 uses a single 4-inch diameter, PVC eductor line equipped with a 2.3-inch orifice plate set at the bottom of the eductor line 162 feet below
land surface. The orifice plate was used to maintain a positive pressure on the eductor line during recharge operation. A secondary eductor design for RW 1-4 also included an additional 4-inch eductor line set directly into the gravel pack at a depth of approximately 10 feet below land surface. This line allows for the collection of data on the effects of recharging directly into the gravel pack. Each eductor line is equipped with a manually-operated ball valve to allow for direct shut-off, if necessary.

Table 1 provides a summary of RW 1-4 test data. The first test of RW 1-4 was conducted from September 9, 1994 through October 14, 1994. RW 1-4 was operated at a variety of flow rates for durations of 5 to 24 days. During recharge at 300 and 400 gpm, the eductor line valve was not opened fully causing a false positive pressure reading on the eductor line. During this time, the soil formation was experiencing air entainment. When the flow rate was increased, the eductor line valve was opened fully. The well operated at 560 gpm for a curation of 24 days without indication of further air entainment occurring.

RW 1-3, located 75 feet to the south of RW 1-4, was operated during the RW 1-4 test period using microfiltered Central Arizona Project (CAP) water from the MEMTEC microfiltration unit. The microfiltration unit processes a backwash cycle every 15 minutes. This cycle contributes to air entainment due to the loss of positive pressure on the eductor line during the cycle.

The test results show that RW 1-4 experienced air entainment due to the improper valve operation. This resulted in a loss of recharge capacity while operating at 300 gpm. The results also show that no further air entainment occurred while operating at 560 gpm. At this time, it is not known if the air entainment caused by the MEMTEC microfiltration system extended to RW 1-4.

The second test of RW 1-4 was conducted from March 1, 1995 through April 20, 1995. This was the first pilot test conducted using the gravel pack in the annulus of RW 1-4. There was a drying period of 4½ months between well operations. The test was conducted at a flow rate of 300 gpm for 23 days and approximately 550 gpm for 27 days. At 300 gpm, the water level rose to 33 feet from the bottom of the well casing. Seven days after the flow rate was increased to 550 gpm, the water level began to rise. The water level rose at a rate of 0.85 feet per day. The well was shut down on April 20, 1995 after 27 days of testing at 550 gpm. The water level had increased from 60 feet to 78 feet during testing at 550 gpm. RW 1-4 operated at 300 gpm with the same efficiency as during the first test. This indicates that the idle time had no effect on the performance of the well. The water level increase while operating at 550 gpm indicates that air entainment was occurring from the start of testing at this flow rate. Operating RW 1-4 at 550 gpm using the gravel pack recharge method produced significant air entainment resulting in a loss of recharge capacity from 10.57 gpm/ft to 6.96 gpm/ft over the 27 day test period when compared to the first test.

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<table>
<thead>
<tr>
<th>Test Start Date</th>
<th>Pressure on Eductor Line, psi</th>
<th>Duration at Flow Rate</th>
<th>Flow Rate, gpm</th>
<th>Water Level Rise, ft</th>
<th>Recharge Capacity, gpm/ft</th>
<th>Idle Time between Test Periods</th>
</tr>
</thead>
<tbody>
<tr>
<td>9/9/94 First Test</td>
<td>&lt;0</td>
<td>6 days</td>
<td>400.0</td>
<td>33</td>
<td>12.12</td>
<td>First Test</td>
</tr>
<tr>
<td></td>
<td>&lt;0</td>
<td>5 days</td>
<td>300.0</td>
<td>33</td>
<td>9.09</td>
<td></td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>24 days</td>
<td>560.0</td>
<td>52</td>
<td>10.57</td>
<td></td>
</tr>
<tr>
<td>3/1/95 Second Test</td>
<td>23 days</td>
<td>14-18</td>
<td>550.0</td>
<td>60 to 78</td>
<td>9.16 to 7.05</td>
<td>4½ months</td>
</tr>
<tr>
<td></td>
<td>28 days</td>
<td>74</td>
<td>7.4</td>
<td></td>
<td></td>
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<tr>
<td>4/28/95 Third Test</td>
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<td>66</td>
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<td></td>
<td></td>
<td>8 days, Multiple well testing</td>
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<tr>
<td></td>
<td>25</td>
<td>63 days</td>
<td>600.0 to 550</td>
<td>73 to 107</td>
<td>8.2 to 5.14</td>
<td>8 months, 3 weeks</td>
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<tr>
<td></td>
<td>7</td>
<td>96 to 99</td>
<td>4.58 to 4.44</td>
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</tr>
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<td></td>
<td>1</td>
<td>110</td>
<td>5.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>96 to 98</td>
<td>4.16 to 4.08</td>
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<td></td>
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<td></td>
<td>11</td>
<td>103 to 114</td>
<td>4.08 to 3.68</td>
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<td></td>
</tr>
<tr>
<td>10/28/96 Fifth Test</td>
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<td>99</td>
<td>3.13</td>
<td></td>
<td></td>
<td>9 months, 2 weeks</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>310.0</td>
<td>3.02</td>
<td></td>
<td></td>
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</tr>
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<td></td>
<td>5</td>
<td>275.0</td>
<td>3.02</td>
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<td></td>
<td></td>
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<tr>
<td></td>
<td>8</td>
<td>290.0</td>
<td>3.88</td>
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<td></td>
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</tr>
</tbody>
</table>

The third test of RW 1-4 was started on April 28, 1995 which marked the beginning of the multiple well test periods (Table I). A combination of three recharge wells were operated during the multiple well tests. Flow rates were chosen based on water availability and operating requirements of the recharge wells. When RW 1-1, RW 1-3, and RW 1-4 were operated simultaneously, RW 1-1 had no apparent impact on RW 1-3 or RW 1-4. The maximum water level height reached in RW 1-4 during the multi-well test was 74 feet. The water levels in RW 1-3 and RW 1-4 declined throughout the test period. The 100 foot well spacing between RW 1-3 and RW 1-4 allowed sufficient operating space for RW 1-3 and RW 1-4 to recharge simultaneously.
The fourth test of RW 1-4 consisted of a second gravel pack recharge test (Table 1). The test was started on January 10, 1996 and continued to April 19, 1996. The well was operated at flow rates of 600 gpm, 550 gpm, 440 gpm, 400 gpm, and 420 gpm. The height of water in the well casing showed increasing trends under each flow condition. The water level rose at a rate of 0.58 feet per day. This rate of water level rise is less than the rate experienced during the previous gravel pack test period (0.85 ft/day). A moderate to high degree of air entrainment occurred in the soil formation surrounding RW 1-4 during the second gravel pack test. The results of this test period showed that no recovery of recharge capacity occurred during the drying period for this well. Air entrainment continued to occur throughout this test period resulting in a decline in recharge capacity from the third to fourth test periods (refer to Table 1).

A fifth test of RW 1-4 was conducted from October 28, 1996 through November 13, 1996 using the eductor line into the well casing. There was a drying period of 9½ months between well operations. The flow rate for the test duration was approximately 310 gpm. The height of water in the well ranged from 99 to 96 feet. The drying period did not have any influence on improving recharge capacity (refer to Table 1). Based on a comparison with the first test, RW 1-4 has experienced a decline in recharge capacity of 65 percent as a result of continued air entrainment. The results of the fifth test indicate that further air entrainment did not occur during this 16 day test.

The sixth test of RW 1-4 was conducted from November 18, 1996 through December 4, 1996. This was the third test using the gravel pack. The well was idle for 4 days before the start of the test period. RW 1-4 was operated at 275 gpm, 290 gpm, and 420 gpm. During this test period, RW 1-4 did not experience increasing water levels as recharge progressed. The performance of RW 1-4 during the third gravel pack test was similar to that of the third eductor line test (fifth test, previous paragraph) indicating that, given the impaired condition of RW 1-4, no further air entrainment was occurring regardless of recharge technique.

Several aspects of efficient operation of a vadose zone recharge well system were learned from the pilot recharge testing of RW 1-4 including:

- Proper operational parameters are necessary to maximize the performance and lifetime of the vadose zone recharge well system. Improper operating conditions, such as, recharging with zero pressure eductor line or throttled valve settings will result in cascading water which causes air entrainment.
- Air entrainment will reduce the efficiency and the effective operating life of the vadose zone recharge well system. At this time, idle time has not been shown to reverse the effects of plugging due to air entrainment.
- Well design is one of the contributing factors to obtaining the best performance of the vadose zone recharge well. A continuous casing string
maximizes the flow of recharge water into the gravel pack and the surrounding soils.

- The blank section placed at the bottom of the casing string which is used as a reservoir is necessary to maintain a positive operating pressure on the eductor line and helps dissipate the velocity of the recharge water as it exits the orifice plate in the eductor line.
- The orifice plate eductor design provides a method for maintaining a full water column during recharge (i.e., positive pressure at well head).
- RW 1-4 has experienced moderate to severe air entrainment due to cascading water during its initial test period, gravel pack test periods, and backwash cycle of the microfiltration unit.

RESULTS OF RW 2-1 PILOT TESTS

Compared with the RW 1-4 design, the design of RW 2-1 consists of a larger diameter PVC well casing, additional perforated PVC well casing, a 10-foot reservoir section of blank well casing with a PVC cap, the same size gravel pack with 200 pounds of dry chlorine, and a 4-inch diameter eductor line with 3-inch diameter orifice.

The first recharge test of RW 2-1 was conducted from January 22, 1996 through June 4, 1996 (Table 2). The test was evaluated in two parts ranging from January 22, 1996 through March 12, 1996 and from March 25, 1996 through June 4, 1996. During the first part of the test, RW 2-1 was equipped with a 4-inch diameter PRV. The valve restricted the recharge flow resulting in cavitation and increasing water levels within the recharge system. The existing PRV was upgraded from a 4-inch to 6-inch and the test was restarted. During the second part of the test, RW 2-1 operated at approximately 800 gpm with a water level rise of approximately 102 feet. At 800 gpm, the recharge capacity of RW 2-1 is 7.8 gpm/foot of water level rise.

The first tests at Well Cluster 2 resulted in design changes to accommodate the higher recharge rates obtained from the use of the 3-inch diameter orifice plate. To complete the pilot testing at Well Cluster 2, the design changes focused on the water supply line equipment.

The second test of RW 2-1 started on December 11, 1996 and continues into 1997. The source water for this test is a blend of microfiltered CAP water and potable water. There was a 6 month drying period between well operations. The flow rate ranged from 720 to 780 gpm. The water level in RW 2-1 ranged from 95 to 105 feet during the test, however, it did not experience an increasing trend. The recharge capacities were 7.7 gpm/ft and 7.5 gpm/ft at flow rates of 780 and 720 gpm, respectively. Figure 2 is the performance plot of RW 2-1 for this test period. The recharge capacities for the second
<table>
<thead>
<tr>
<th>Test Start Date</th>
<th>Pressure on Eductor Line, psi</th>
<th>Duration at Flow Rate</th>
<th>Flow Rate, gpm</th>
<th>Water Level Rise, ft</th>
<th>Recharge Capacity, gpm/ft</th>
<th>Idle Time between Test Periods</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/22/96</td>
<td>18</td>
<td>50 days</td>
<td>880 to 860</td>
<td>84 to 98</td>
<td>10.48 to 8.77</td>
<td>First Test</td>
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<tr>
<td></td>
<td>26</td>
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<td>860</td>
<td>98</td>
<td>8.77</td>
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<tr>
<td>3/25/96</td>
<td>9</td>
<td>21 days</td>
<td>800</td>
<td>102</td>
<td>7.8</td>
<td>12 days</td>
</tr>
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<td></td>
<td>3</td>
<td>51 days</td>
<td>760</td>
<td>93</td>
<td>8.17</td>
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</tr>
<tr>
<td>12/11/96</td>
<td>9 days</td>
<td>780</td>
<td>101</td>
<td>7.7</td>
<td>6 months</td>
<td></td>
</tr>
<tr>
<td></td>
<td>26 days</td>
<td>720</td>
<td>95.8</td>
<td>7.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>0-2</td>
<td>23+ days</td>
<td>655</td>
<td>94.8</td>
<td>6.9</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

‘Flow rate and water levels fluctuating during this time period.

test were consistent with those found during the second part of the first test (see Table 2, 3/25/96).

The pilot testing of RW 2-1 provided the following information:

- A recharge rate of 800 gpm was not expected at Well Cluster 2 and this resulted in upgrades to the water supply line equipment. The flow meter, backflow preventor, and PRV should be sized to accommodate the maximum anticipated recharge rates.
- Proper operating procedures minimized the effects caused by air entrainment.
- The well design contributed to the performance of RW 2-1.
- Idle time had no apparent impact on the recharge capacity indicating that RW 2-1 has experienced a minimal amount of plugging due to particle migration and air entrainment.
- RW 2-1 was shut down on May 12, 1997. This completed a five month period of continuous recharge without any increasing water level trends.
CONCLUSIONS

The uniqueness of the vadose zone recharge system made pilot testing a necessity. The extensive pilot testing program undertaken by the COS has been beneficial in targeting areas for design and operational improvements which may have otherwise been overlooked. The primary causes of air entrainment were found to be improper operating procedures, eductor design, and recharging under zero or negative pressure. Idle time between well operations was not shown to reverse the effects of air entrainment. Therefore, design options and operating procedures should be used which reduce plugging caused by air entrainment.