Design, Operation, and Maintenance for Sustainable Underground Storage Facilities

Subject Area:
Environmental Leadership

Design, Operation, and Maintenance for Sustainable Underground Storage Facilities

Awwa Research Foundation

About the Awwa Research Foundation

The Awwa Research Foundation (AwwaRF) is a member-supported, international, nonprofit organization that sponsors research to enable water utilities, public health agencies, and other professionals to provide safe and affordable drinking water to consumers.

The Foundation's mission is to advance the science of water to improve the quality of life. To achieve this mission, the Foundation sponsors studies on all aspects of drinking water, including supply and resources, treatment, monitoring and analysis, distribution, management, and health effects. Funding for research is provided primarily by subscription payments from approximately 1,000 utilities, consulting firms, and manufacturers in North America and abroad. Additional funding comes from collaborative partnerships with other national and international organizations, allowing for resources to be leveraged, expertise to be shared, and broad-based knowledge to be developed and disseminated. Government funding serves as a third source of research dollars.

From its headquarters in Denver, Colorado, the Foundation's staff directs and supports the efforts of more than 800 volunteers who serve on the board of trustees and various committees. These volunteers represent many facets of the water industry, and contribute their expertise to select and monitor research studies that benefit the entire drinking water community.

The results of research are disseminated through a number of channels, including reports, the Web site, conferences, and periodicals.

For subscribers, the Foundation serves as a cooperative program in which water suppliers unite to pool their resources. By applying Foundation research findings, these water suppliers can save substantial costs and stay on the leading edge of drinking water science and technology. Since its inception, AwwaRF has supplied the water community with more than $300 million in applied research.

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CONTENTS

vii

TABLES

FIGURES

FOREWORD

The Awwa Research Foundation (AwwaRF) is a nonprofit corporation that is dedicated to the implementation of a research effort to help utilities respond to regulatory requirements and traditional high-priority concerns of the industry. The research agenda is developed through a process of consultation with subscribers and drinking water professionals. Under the umbrella of a Strategic Research Plan, the Research Advisory Council prioritizes the suggested projects based upon current and future needs, applicability, and past work; the recommendations are forwarded to the Board of Trustees for final selection. The foundation also sponsors research projects through an unsolicited proposal process; the Collaborative Research, Research Applications, and Tailored Collaboration programs; and various joint research efforts with organizations such as the U.S. Environmental Protection Agency, the U.S. Bureau of Reclamation, and the Association of California Water Agencies.

This publication is a result of one of these sponsored studies, and it is hoped that its findings will be applied in communities throughout the world. The following report serves not only as a means of communicating the results of the water industry's centralized research program but also as a tool to enlist the further support of the nonmember utilities and individuals.

Projects are managed closely from their inception to the final report by the foundation's staff and large cadre of volunteers who willingly contribute their time and expertise. The foundation serves a planning and management function and awards contracts to other institutions such as water utilities, universities, and engineering firms. The funding for this research effort comes primarily from the Subscription Program, through which water utilities subscribe to the research program and make an annual payment proportionate to the volume of water they deliver and consultants and manufacturers subscribe based on their annual billings. The program offers a cost-effective and fair method for funding research in the public interest.

A broad spectrum of water supply issues is addressed by the foundation's research agenda: resources, treatment and operations, distribution and storage, water quality and analysis, toxicology, economics, and management. The ultimate purpose of the coordinated effort is to assist water suppliers to provide the highest possible quality of water economically and reliably. The true benefits are realized when the results are implemented at the utility level. The foundation's trustees are pleased to offer this publication as a contribution toward that end.

David E. Rager   Robert C. Renner, P.E.
Chair, Board of Trustees   Executive Director
Awwa Research Foundation   Awwa Research Foundation

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Equus Beds Groundwater Management District No. 2, McPherson County, Kansas
Edwards Underground Water District, San Antonio, Texas
Fort Dix, New Jersey
Town of Gilbert, Arizona
Las Vegas Valley Water District, Nevada
Nassau County, New York
Orange County Water District, California
Peace River/Manasota Regional Water Supply Authority, Florida
San Antonio Water System, Texas
City of Scottsdale, Arizona
City of Tucson, Arizona

EXECUTIVE SUMMARY

INTRODUCTION

Achieving a sustainable, reliable drinking water supply has emerged in recent years as an increasingly important goal, not only in the United States but also worldwide. This is being driven by population growth, increasing water demands, declining groundwater levels, contamination of water sources, greater awareness of adverse environmental impacts associated with current water supply practices, concern regarding the potential adverse impacts of global warming, and many other factors. Among the many ways that are being applied to achieve this goal, managed aquifer recharge (MAR) is proving to be viable and cost-effective. During times when excess water is available and of suitable quality, it is recharged into an aquifer using either infiltration through surface soils or recharge through wells. When the stored water is needed it is recovered. The aquifer is utilized as a very large storage reservoir, storing water for periods of as short as a few hours to as long as many years.

The storage capacity available in aquifers is large. About 98% of all of the fresh water globally is found in aquifers, and many of these are depleted, thereby providing storage capacity. Some aquifers that are full contain water of poor quality, however such aquifers may still be utilized for water storage, displacing the poor quality ambient groundwater and replacing it with fresh water.

Successful recharge requires the effective use of appropriate technology. During the last few decades many research and development projects have been conducted and other recharge projects have become operational. Advances have been made in the science of aquifer recharge, including the geochemistry, microbiology and also the hydraulics. Materials of construction have improved, and equipment has been steadily upgraded to meet evolving needs. The results provide a strong foundation for the successful implementation of aquifer recharge projects. To achieve success, it is necessary to understand the lessons that have been learned, taking advantage of good ideas that worked and hopefully not repeating the ideas that did not work.

RESEARCH OBJECTIVES

The Awwa Research Foundation (AwwaRF) funded this study with the goal of identifying technical variables that result in successful design, operation and maintenance of sustainable underground storage (SUS) facilities. Seven key objectives of the project are as follows:

1. Increase the available knowledge base of SUS facilities throughout the U.S. and provide easy access to that knowledge.
2. Survey underground storage facilities with a variety of geographical locations, capacities, geological conditions, and operational methods to ensure broad applicability.
3. Identify design and operational strategies that minimize the cost of operating the storage system, optimize the long-term quality of recovered water, and minimize potential impacts to surrounding areas.
4. Identify and evaluate sites where SUS performance failed to meet objectives.
5. Address the use of SUS to reduce the vulnerability of water facilities.

6. Create an easy-to-use, practical guidance document that will aid in the efficient design and operation of SUS facilities.
7. Develop an outreach program to distribute research findings to those who need it most.

APPROACH

Technical aspects of selecting, designing and managing a system for recharge of groundwater are presented in these guidelines, including surface infiltration and well recharge methods. These build upon prior work by Dr. Herman Bouwer (Artificial Recharge of Groundwater: Hydrogeology and Engineering in the *Journal of Hydrogeology* Vol. 10, p. 121–142, 2002) and by R. David G. Pyne, P.E. (*Aquifer Storage Recovery: A Guide to Groundwater Recharge Through Wells, Second Edition*, 2006). Dr. Bouwer prepared Chapter 1, "Design, Operation, and Maintenance of Surface Recharge Systems," while Chapter 2, "Design, Operation, and Maintenance of Well Recharge Systems," was prepared by David Pyne, who also served as Principal Investigator. The guidelines were prepared in draft form based upon these two sources and were then "field-verified" by ASR Systems' project team members through field visits to 18 operating recharge sites representing diverse geographic and technical conditions and a broad range of operating scales, plus evaluation of lessons learned from four failed recharge sites. Experience within the United States was supplemented by that from overseas. Chapters 1 and 2 were then revised to incorporate the "Lessons Learned" that are summarized in Chapter 3 for each of the visited sites.

In addition to Dr. Herman Bouwer and David Pyne, ASR Systems project team members include Dr. Jess Brown, Carollo Engineers, who served as Co-Principal Investigator. The final guidelines incorporate additional peer review by the Project Technical Advisory Committee, membership of which included Dr. Peter Dillon, CSIRO, Adelaide, Australia; Dr. A. Ivan Johnson, P.E., American Society of Civil Engineers, Arvada, Colorado; and Dr. Mario Lluria, Salt River Project, Arizona. Other contributing individuals and organizations included Dr. Mitchell J. Rycus, University of Michigan, who prepared Chapter 4, "Sustainable Underground Storage: A Strategy for Improving Reliability and Reducing Vulnerability of Water Systems;" Daniel St Germain, P.G., Malcolm Pirnie Inc., White Plains, New York; Tom M. Morris, ASR Systems, Las Vegas, Nevada; and Dr. Christopher J. Brown, P.E., LJHB Partners, Jacksonville, Florida. The resulting document is hopefully a useful tool in the quest to achieve sustainable water supplies through underground storage.

Chapter 1 provides a brief introduction to recharge systems, including surface infiltration, vadose zone infiltration, wells and combination systems. Technical and scientific aspects of successful surface recharge are then presented, such as use of infiltrometers to determine infiltration rates, soil clogging, effect of water depth on infiltration rates, mounding of water levels and effect of mounding on infiltration rates. Design and management of infiltration basins is then addressed, including resolution of common problems. Sustainability of surface recharge systems is discussed, followed by consideration of the role of aquifer recharge in water reuse.

Chapter 2 presents the design of wells, wellheads and wellfields for aquifer recharge, highlighting those technical aspects that are in many ways unique for successful implementation of well recharge systems. Materials of construction are considered, along with a discussion of well casings and screens. Selection of ASR storage intervals, mechanical integrity testing of well casings, and horizontal directionally-drilled ASR well concepts are then presented. Design of ASR wellheads and downhole equipment addresses each of the key aspects of equipping wells for successful operation, such as downhole flow control, pipeline flushing and waste flow discharge, trickle flows,

corrosion control, air and vacuum relief, flow measurement, disinfection and pH adjustment, pump selection and many other aspects. Wellfield design includes consideration of dispersive and advective mixing between stored water and ambient groundwater vertical stacking of storage intervals, observation wells, recovery efficiency, target storage volume, well plugging and redevelopment, well maintenance and rehabilitation, and several other aspects of successful wellfield design.

In addition to discussion of surface and well recharge methods (Chapters 1 and 2), the report includes a concise summary of the most important lessons learned from the 22 operating and failed recharge sites that were visited (Chapter 3). It also includes a proposed analytical approach (Chapter 4) that may be applied for water utilities to reduce their vulnerability to service interruption and thereby enhance their system reliability. This would be achieved through development of sustainable underground storage (SUS) as an important new component of their operations, joining water supply, treatment and distribution. A discussion of "Knowledge Gaps, Emerging Issues, and Research Needs" is presented in Chapter 5.

The Appendix includes case studies for the 18 operating and four failed SUS facilities that were visited as part of this project. These are presented on the attached CD-ROM, providing useful perspectives regarding how different water utility systems have approached the need for Sustainable Underground Storage (SUS). Each case study is approximately 10 to 15 pages. Case studies are provided for the following locations, four of which included both surface and well recharge facilities:

Equus Beds Groundwater Management District No. 2, McPherson County, Kansas
Orange County Water District, southern California
Scottsdale Water Campus, Scottsdale, Arizona
Calleguas Municipal Water District, Thousand Oaks, California
Centennial Water & Sanitation District, Highlands Ranch, Colorado
Peace River/Manasota Regional Water Supply Authority, Florida
Las Vegas Valley Water District, Nevada
City of Beaverton, Oregon
San Antonio Water System, Texas
Nassau County Department of Public Works, New York
Central Avra Valley Storage Recovery Project, Tucson, Arizona
C-111 Infiltration Basins, Homestead, Florida
Fort Dix, New Jersey
Edwards Underground Water District, San Antonio, Texas
Urrbrae Wetland ASR Project, South Australia
Northwest Hillsborough County ASR Project, Florida
East Meadow, Cedar Creek, New York
Bay Park, New York

A current inventory of operating SUS facilities in the United States is summarized at the end of this Executive Summary (Table ES.1). A more complete listing of inventory details and contact information is included in the Appendix on the CD-ROM. A reasonable effort has been made to ensure that significant facilities are included with this inventory, however, it is probable that other surface recharge and well recharge sites are in operation, while still others are being placed into operation.

For the inventory of SUS facilities, in some cases it was necessary to try to differentiate between planned SUS projects and other projects for which SUS is an incidental benefit, such as

stormwater drainage wells that are prevalent in some states, and reclaimed water injection/disposal wells. The level of pretreatment prior to recharge varies, reflecting differences in the regulatory framework in different states; differences in source and receiving water quality; differences in the location where compliance with drinking water standards is measured; and age of the project. Some regional stormwater drainage well programs have been operational for up to century. Some of these have been "grandfathered" under current regulations and would have difficulty getting permitted as new projects today. "Best practices" for recharge pretreatment and water management are evolving. Furthermore, best practices for one part of the world may be different than for another part of the world, reflecting differing needs, constraints and opportunities.

A wide variety of water measurement units are utilized in different parts of the United States and in other countries. A Table of Conversion Units is presented at the end of this report. To facilitate reading, single measurement units are generally presented, particularly those most commonly used in the United States.

FUTURE RESEARCH

Research is needed in five general areas:

1. Water Quality.

   Five potential research topics are presented addressing the most significant knowledge gap, namely, the need for an improved understanding of subsurface geochemical, microbial and physical processes that contribute to changes in water quality.
2. Facilities Design and Operation.

   Ten proposed research topics are presented, the goal of which is to achieve the greatest amount of long-term value from a small recharge area. This is in response to the rapidly increasing difficulty for obtaining suitable sites for SUS facilities.
3. Economics.

   A nationwide database is needed, presenting recharge economics using a common method for comparison of multiple SUS data sources. Currently the plethora of measurement units and assumptions hinders true comparison of data from different locations and sources.
4. Regulatory Framework.

   Probably the greatest constraint upon achieving widespread successful and effective application of SUS facilities is the regulatory framework, which varies widely between states in the U.S.A., and also within different parts of many states. The regulatory framework is evolving. Of considerable value would be an update of the regulatory framework in each state, and development of an updated model SUS regulatory framework that incorporates a risk-based regulatory program, achieving a reasonable balance of risk, benefit and cost with full consideration of the needs, constraints and opportunities in each state.
5. Water System Reliability Improvement.

   The reliability improvement and vulnerability reduction concepts presented in this report, and applied to one representative site, need to be applied to several other existing or proposed SUS sites, thereby demonstrating the relative value of SUS as one of the many "vulnerability reduction" strategies that are typically considered.

Table ES.1
Inventory of sustainable underground storage operational sites
Prepared for Awwa Research Foundation, Project 3034, April 2007
Prepared by ASR Systems, LLC, Gainesville, Florida

Well Site Number	State	Location	Organization	Recharge Water Source	Surface Recharge Method					Operating Since Year	Wetted Area (acres)	Recharge AFY
					Canals	Ditches	Channels	Basins	Other			
United States												
1	AZ		Arizona Sierra Utilities	surface water					*	1987		
2	AZ		Arizona Sierra Utilities	surface water					*	1987		
3	AZ	Granite Reef USP	Salt River Project, Phoenix	surface water				*		1990		
4	AZ	Green Valley	Green Valley Wastewater Tr. Facility	reclaimed water			*	*				
5	AZ	Peoria	City of Peoria	reclaimed water				*				
6	AZ	Phoenix	CAP Agua Fria, Tonopah, Marana sites	surface water			*	*				
7	AZ	Phoenix	Flushing Meadows	reclaimed water				*				
8	AZ	Prescott	City of Prescott	reclaimed water				*				
9	AZ	Prescott	City of Prescott	reclaimed water				*				
10	AZ	Town of Gilbert	Neely Road; Riparian Reserve at Water Ranch	reclaimed water				*		pre-1998		
11	AZ	Tucson	CAP Pima Mine Rd Recharge Project	surface water				*		1998	14	10,000 60,000 to 80,000
12	AZ	Tucson	Central Avra Valley Storage/Recovery Project	surface water				*		2001		
13	AZ	Tucson	Tucson Water, Sweetwater site	reclaimed water				*		1986		
14	AZ	Window Rock, Blue Canyon Dam	Navajo Nation Dept. of W. Management	surface water				*		1985		
15	AZ	Butlers Valley	Butlers Valley Demo. Project	surface water				*		1987		
16	AZ	Butlers Valley	Butlers Valley Demo. Project	surface water				*		1987		
17	AZ		Vidler Recharge Project	Colorado River water				*				
18	CA	Carson	Recharge basins	drinking/reclaimed								
19	CA	Fresno	Recharge basins					*				
20	CA	Inland Empire Utilities District	Surface spreading	drinking/reclaimed								
21	CA	Kern County	Kern Water Bank	surface water				*				
22	CA	Los Angeles County PW Dept	Recharge basins					*				
23	CA	Orange County Water District	Several basins in Anaheim area	suface/reclaimed water				*				
24	CA	San Bernadino	Rapid infiltration basins	reclaimed water				*				

(Continued)

Table ES.1(continued)

Surface Recharge Sites (continued)

Well Site Number	State	Location	Organization	Recharge Water Source	Surface Recharge Method					Operating Since Year	Wetted Area (acres)	Recharge AFY
					Canals	Ditches	Channels	Basins	Other			
25	CA	San Jose	Recharge basins					*				
26	CO	Greeley	Central Colorado W. Conservancy District		*	*	*	*		pre-1985	100	3,000
27	FL	Reedy Creek Improvement District	Disney World	reclaimed water				*		1995		
28	FL	South Florida WMD	C-111 infltration basins, S. Florida	surface water				*				
29	FL	Statewide	Thousands of stormwater retention basins	storm water				*		1985		
30	FL	West Palm Beach	Recharge ponds next to wetlands	reclaimed water				*		2000		
31	IN	City of Elkhart	Christiana Creek Diversion	surface water		*	*			1957		
32	KS	1) Dodge City, 2) Grant County	SW Kansas GW Mgmt District No. 3	1) reclaimed 2) Playa L.				*				
33	KS	Hodgeman County, Pawnee R.	Big Bend GW Mgmt District No. 5	surface water			*					
34	KS	Janzen Recharge, Scott County	W. Kansas GW Mgmt District No. 1	surface water					*	1981		
35	KS	McPherson County	Equus Beds GW Mgmt District No. 2	surface water				*				
36	KS	multiple sites	NW Kansas GW Mgmt District No. 4	surface water					*			
37	Ml	Kalamazoo	Station 9, Station 24	surface water		*				1958	3	
38	NE	Hall County, Central Nebraska	Wood River Recharge Basins	surface water from Platte R. atrazine monitoring								
39	ME	Little Blue Nat. Res. Dist, Davenport	Bruning Recharge Dan	surface water					*		1,574	
40	NE	N. Nat. Res. Comm, Lincoln	4 reservoirs	surface water					*	1982		
41	NJ	Fort Dix	12 recharge basins	reclaimed water				*		1995		
42	NJ	Vineland	Landis Sewerage Authority	reclaimed water				*				
43	NY	Nassau County, Long Island	2,000 to 3,000 stormwater recharge basins	surface water				*		1930		3,500
44	OH	City of Cuyahoga	recharge local wellfield next to river	surface water	*			*		1971		
45	OK	Altus	Lugert-Altus Dam	surface water	*	*				1947		
46	TX	San Antonio	Edwards Underground Water District	surface water			*				123	5,407
47	VA	Eastern Shore	Various small farms and golf courses	stormwater diversions				*				

(Continued)

Wellfield Recharge Sites (continued)

Well Site Number	State	Location	Operating Since Year	Recharge Water Source	Total Recharge or Recovery Capacity (MGD)	Number of Wells	Type of Well	
							ASR	Recharge/ Injection
1	AZ	Chandler Ocatillo	2002	reclaimed		4	*	
2	AZ	Chandler Tumbleweed	1999	reclaimed	10	5	*	
3	AZ	Fountain Hills	2001	reclaimed	2	3	*	
4	AZ	Gilbert		reclaimed			*	
5	AZ	Glendale		reclaimed		4	*	
6	AZ	Honeywell, Phoenix		reclaimed			*	
7	AZ	Peoria						*
8	AZ	Phoenix SRP	1995?	surface water		1		*
9	AZ	Scottsdale Water Campus	1998	reclaimed	21	55		*
10	AZ	Sun Lake City	1998	reclaimed	2.4	3		*
11	AZ	Vidler Recharge Project		surface water			*	*
12	CA	Calleguas Municipal Water District	1992	drinking water	65	26	*	
13	CA	Camarillo	1992	drinking water	2	1	*	
14	CA	Foothills Municipal Water District	1994	drinking water		1	*	
15	CA	Goleta Water District	1978	drinking water	5	6	*	
16	CA	Livermore	1997	drinking water	3.5	1	*	
17	CA	Los Angeles County PWD				100		*
18	CA	Monterey Peninsula WMD	2001	drinking water	2.5	1	*	
19	CA	Orange County WD	1975	drinking water	12	31		*
20	CA	Oxnard, City of	1991	drinking water	4	1	*	
21	CA	Pasadena, City of	1992	drinking water	16	6	*	
22	CA	Pleasanton, Zone 7		drinking water		1	*	
23	CA	Rosevllle	2005	drinking water	4.9	1	*	
24	CA	Santa Barbara. City of	1967			41		*
25	CA	Santa Clara County Water District, San Jose	1999	drinking water	1.1	1	*	
26	CA	Victor Valley Water District, Victor Valley, California	2003	drinking water	4.3	4	*	
27	CO	Castle Pines North, CO		drinking water				

(Continued)

Table ES.1(continued)

Wellfield Recharge Sites (continued)

Well Site Number	State	Location	Operating Since Year	Recharge Water Source	Total Recharge or Recovery Capacity (MGD)	Number of Wells	Type of Well ASR	Type of Well Recharge/ Injection
28	CO	Centennial Water & San. District, Highlands Ranch	1993	drinking water			*	
29	FL	Boynton Beach, City of	1993	drinking water	5	2	*	
30	FL	Cocoa, Florida	1987	drinking water	101	10	*	
31	FL	Collier County, Corkscrew		drinking water	8	5	*	
32	FL	Collier County, Marco Island	1998	surface water	4	3	*	
33	FL	Delray Beach, City of	2001	drinking water	2.8	1	*	
34	FL	Lee County, North Reservoir	2000	drinking water	0.4	1	*	
35	FL	Lee County, Olga WTP	2001	drinking water	0.5	1	*	
36	FL	Manatee County, Florida	1983	drinking water	4	2	*	
37	FL	Miami-Dade Water & Sewer Dept.	1998	surface water	15	3	*	
38	FL	Palm Bay (formerly Port Malabar), Florida	1989	drinking water	1	1	*	
39	FL	Peace River/Manasota Regional Water Supply Authority, Florida	1985	drinking water	17	21	*	
40	FL	Punta Gorda, City of	2001	drinking water	3	3	*	
41	FL	Tampa, City of		drinking water	8	8	*	
42	FL	West Palm Beach		surface water	8	1	*	
43	IA	Des Moines Water Works	1998	drinking water	1	1	*	
44	ID	Micron Technologies. Boise		drinking water		1	*	
45	ID	United Water, Boise		drinking water		2 or 3	*	
46	KS	Wichita. City of		surface water	1.5	1	*	
47	NJ	Brick Township Municipal Utilities Authority, Brick	1996	drinking water		1	*	
48	NJ	Evesham Township Municipal Utilities Authority, Evesham		drinking water			*	
49	NJ	Gordons Corner	1971	drinking water	3	2	*	
50	NJ	Matchaponix	1989	drinking water	1.5	1	*	
51	NJ	Monroe South		drinking water			*	
52	MJ	NJAWC Western Divistion, Murray Avenue		drinking water			*	
53	NJ	Swimming River		drinking water			*	

(Continued)

Table ES.1(continued)

Wellfield Recharge Sites (continued)

Well Site Number	State	Location	Operating Since Year	Recharge Water Source	Total Recharge or Recovery Capacity (MGD)	Number of Wells	Type of Well	
							ASR	Recharge/ Injection
54	NJ	Toms River		drinking water			*	
55	NJ	Wildwood	1968	drinking water	4	4	*	
56	NV	Las Vegas Valley Water District	1988	drinking water	157	56	*	
57	NV	North Las Vegas, City of		drinking water		6	*	
58	OK	Oklahoma City, Blaine Recharge Demo Project	1972	surface water		10		*
59	OR	Beaverton	1998	drinking water	5	4	*	
60	OR	City of Portland	2003	drinking water		2	*	
61	OR	Pendleton, City of	2003	drinking water		2	*	
62	OR	Salem	1996	drinking water		6	*	
63	OR	Tigard, City of	2003	drinking water		1	*	
64	OR	Tualatin Valley Water District, Beaverton	1999	drinking water		2	*	
65	SC	Beaufort-Jasper Water and Sewer Authority	1999	drinking water		3	*	
66	SC	Grand Strand WSA, Myrtle Beach	1990	drinking water	6	6	*	
67	SC	Kiawah Island Utility, Kiawah Island	2001	drinking water	3	2	*	
68	SC	Mount Pleasant Waterworks & Sewer Commission	1995	drinking water		3	*	
69	TX	El Paso,Texas	1985	reclaimed	5	11	*	
70	TX	Kerrville	1996	drinking water	2.5	2	*	
71	TX	San Antonio Water System	2005	drinking water	30	17	*	
72	UT	Brigham City		drinking water			*	
73	UT	Jordan Valley Water District	1993	drinking water		1	*	
74	UT	St. George		drinking water			*	
75	VA	Chesapeake	1992	drinking water	3	1	*	
76	WA	Lakehaven Utility District, Federal Way	1990s	drinking water		1	*	
77	WA	Seattle Water Department	1992	drinking water		2	*	
78	WA	Walla Walla	2000	drinking water	4	2	*	
79	WI	Oak Creek		drinking water		1	*	
80	Canada	Kitchener-Waterloo		drinking water		6	*	

Please provide update information to dpyne@asrsystems.ws

CHAPTER 1
DESIGN, OPERATION, AND MAINTENANCE OF SURFACE RECHARGE SYSTEMS

INTRODUCTION

Artificial recharge systems are engineered systems where surface water is put on or in the ground for infiltration and subsequent movement to aquifers to augment groundwater resources (Figure 1.1). Other objectives of artificial recharge may be to reduce seawater intrusion or land subsidence, to store water, to improve the quality of the water through soil-aquifer treatment or geopurification, to use aquifers as water conveyance systems, and to make groundwater out of surface water where groundwater is traditionally preferred over surface water for drinking. Infiltration and artificial recharge are achieved by ponding or flowing water on the soil surface with basins, furrows, ditches, low dams in streambeds etc. (Figures 1.2 through 1.6); by placing it in infiltration trenches or shafts or wells in the vadose zone (Figure 1.7); or by placing it in wells for direct injection into the aquifer Other forms of groundwater recharge include natural, enhanced, induced, and incidental recharge.

Natural recharge is how natural (meteoric) groundwater is formed as the difference between water inputs into the soil (precipitation and infiltration from streams, lakes, or other natural water bodies) and outputs (evapotranspiration plus runoff). Natural recharge typically is about 30–50% of precipitation in temperate humid climates, 10–20% of precipitation in Mediterranean type climates, and about 0–2% of precipitation in dry climates (Bouwer 1989, 2000a, 2000b, 2000c, and references therein; Tyler et al. 1996). Natural recharge rates are reflected by groundwater ages, which vary from a few hours or days in wet-weather springs or very shallow groundwater in high rainfall areas, to tens of thousands of years or more in dry climates with deep
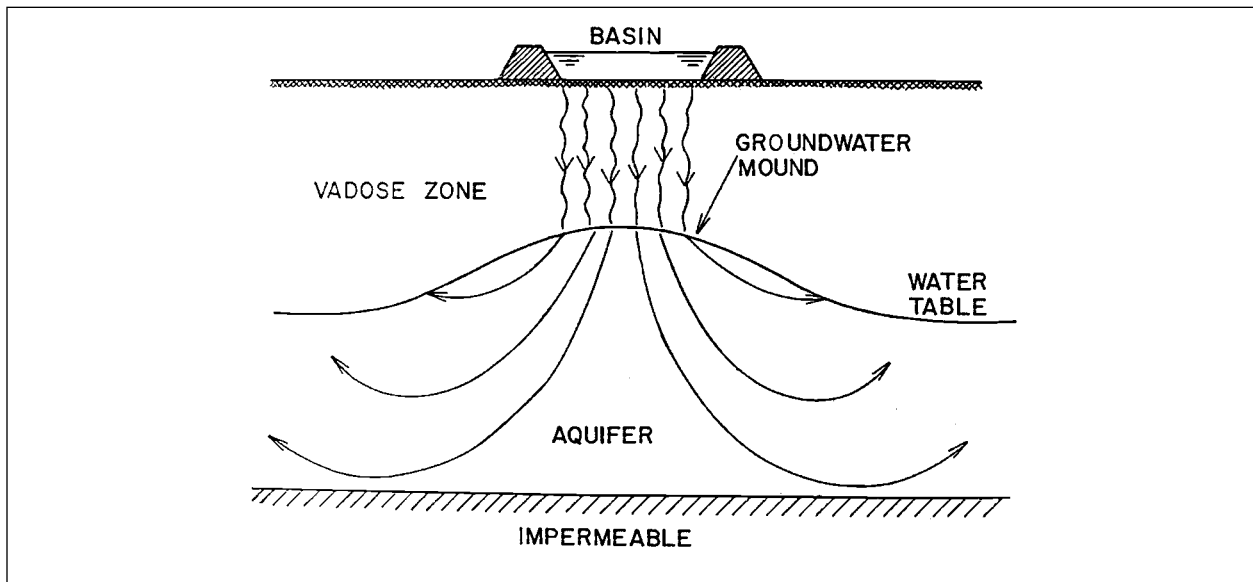
Figure 1.1 Section showing a typical groundwater recharge system with infiltration basin, unconfined acquifer, and groundwater mound below the basin

1

Figure 1.2 Engineered rectangluar infiltration basins near Palm Springs, California, using Colorado River water

Figure 1.3 Infiltration basin in dunes of western Netherlands showing natural shore line with bank vegetation

groundwater levels (Tyler et al. 1996) or in confined aquifers at considerable distance from their outcrops where they are recharged. Groundwater is an extremely important water resource for it comprises more than 98% of all the world's liquid fresh water (Bouwer 1978, and references therein).

Enhanced recharge mainly consists of vegetation management to replace deep-rooted vegetation by shallow-rooted vegetation or bare soil, or by changing to plants that intercept less precipitation with their foliage, thus increasing the amount of water that reaches the soil. In wooded areas, this is achieved, for example, by replacing conifers with deciduous trees (Querner 2000).

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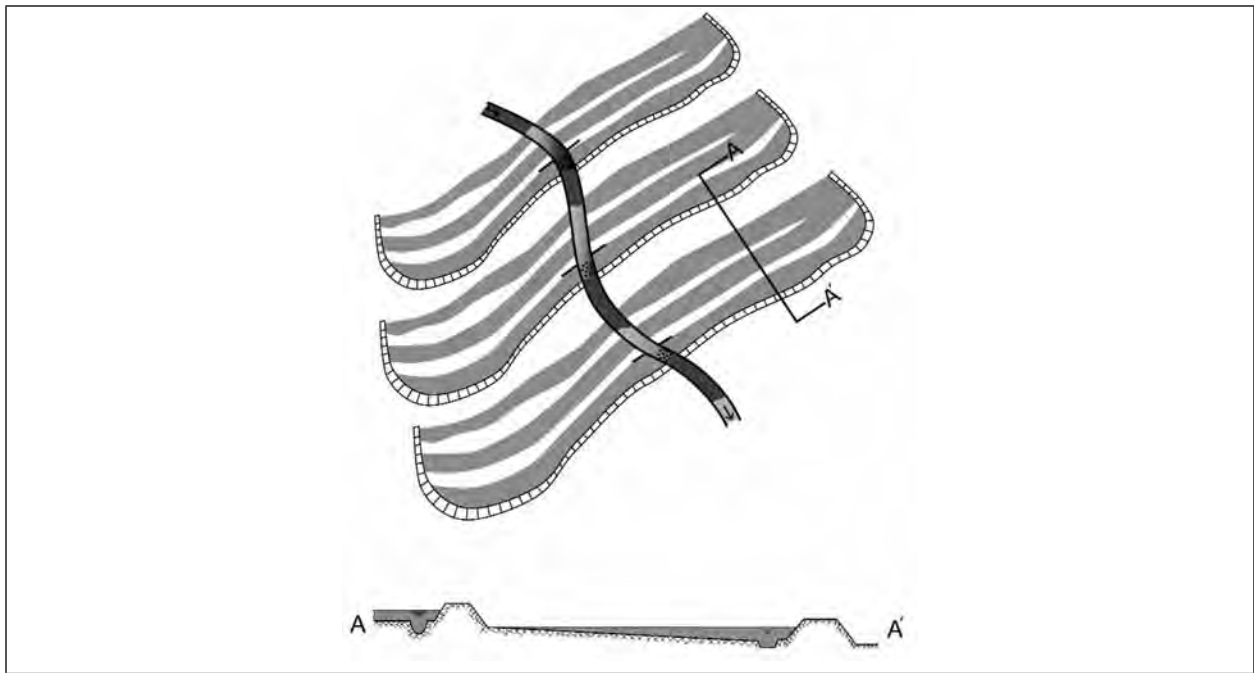
Figure 1.4 Infiltration basins on sloping land with berms on the contours and supply channel down the slope
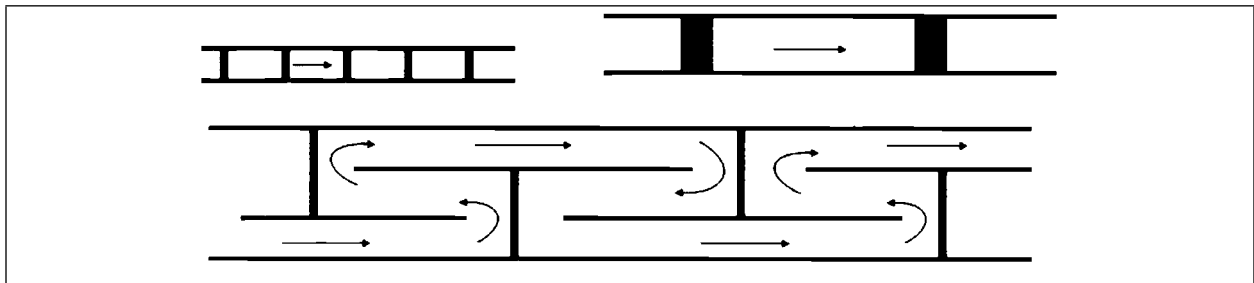
Figure 1.5 Plan views of in-channel infiltration systems with low weirs in narrow, steep channel (top left); bigger dams in wider, more gently sloping channel (top right); and T-levees in wide, flat channels (bottom)

Induced recharge is achieved by placing wells relatively close to streams or rivers, so that more river water is "pulled" into the aquifer as groundwater tables near the streams are lowered by pumping the wells. The main objective of these "bank filtration" systems is often to get pretreatment of the river water as it moves through the river bottom materials and the aquifers before it is pumped up for conventional drinking-water treatment and public water supply. Bank filtration is used particularly where river water is contaminated or where the public prefers groundwater over surface water (Kühn 1999).

Incidental recharge is caused by human activities that are not intended for recharge of groundwater as such. These activities include sewage disposal by septic-tank leach fields or cess pits, and drainage or deep-percolation from irrigated fields. Such drainage of irrigation water is

3

Figure 1.6 Photos of in-channel infiltration systems with low weirs in narrow, steep channel (top left); a 2 m high concrete dam in a wide Moroccan wadi (top right); a low dam in a more gently sloping channel (bottom left); and T-levees in wide, flat channels (bottom right)

necessary to prevent salt accumulation in the root zone. In dry climates, drainage is achieved by applying more irrigation water than needed for crop water use (evapotranspiration or consumptive use). Because the salts and other chemicals in the irrigation water are then leached out of the root zone with much less water than the irrigation water applied, the salt content in the leaching or drainage water is much higher than in the irrigation water itself. This condition, as well as the presence of agricultural and other chemicals in the deep-percolation water from irrigated fields, degrades the quality of the underlying groundwater (Bouwer 2000b; Bouwer et al. 1999). Irrigation water often has total dissolved salt contents in the range of 100–1000 mg/L and more.

Another form of incidental recharge is obtained with urbanization, where most of the land is covered with streets, driveways, roofs, and other impermeable surfaces that produce more runoff and have much less evapotranspiration than the natural surfaces. This recharge could be significant in semi-arid areas, where rain typically falls in small amounts that do not penetrate the soil very deeply, so that most of the water evaporates. With urbanization, however, more runoff is produced, which can be collected for on-site storage and artificial recharge, or it flows naturally to ephemeral streams where it infiltrates into the soil and moves down to the groundwater (Lerner 2002).

Artificial recharge is expected to become increasingly necessary in the future as growing populations require more water, and more storage of water is needed to save water in times of water surplus for use in times of water shortage. The traditional way of storing water has been

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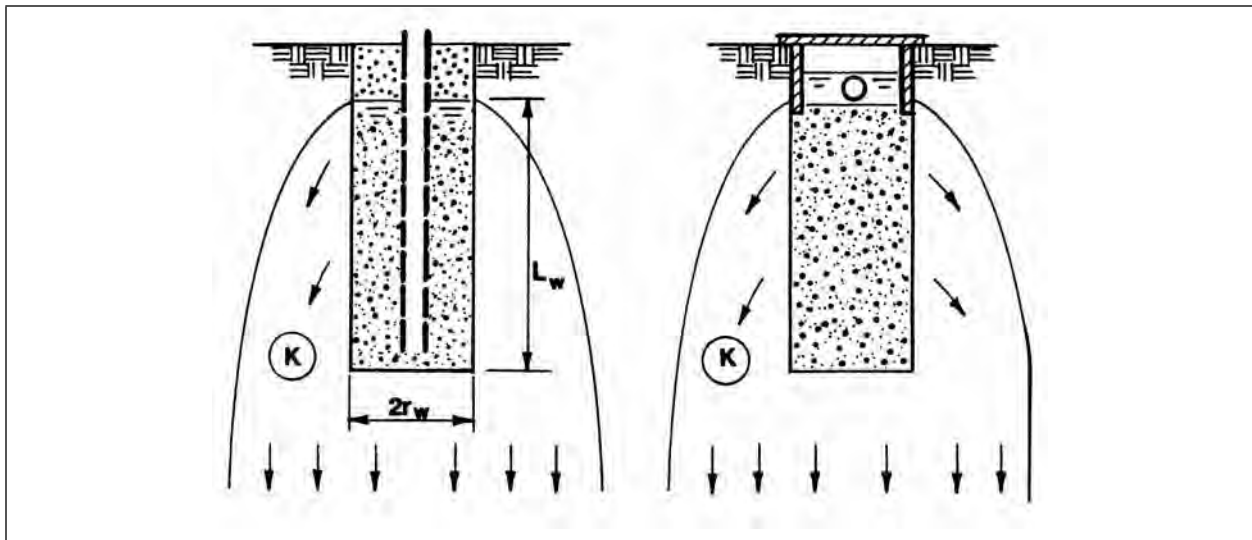
Figure 1.7 Sections showing vadose-zone recharge well (left) with sand or gravel fill and perforated supply pipe, and recharge trench (right) with sand or gravel fill, supply pipe on top of fill, and cover. Arrows represent downward flow in wetted zone with hydraulic conductivity *K*.

with dams. However, good dam sites are becoming scarce. In addition, dams have various disadvantages, such as evaporation losses (about 2m/yr in warm, dry climates); sediment accumulation; potential of structural failure; increased malaria, schistosomiasis, and other human diseases; and adverse ecological, environmental, and socio-cultural effects (Devine 1995; Knoppers and van Hulst 1995; Pearce 1992). New dams are often more difficult to build because of high cost and public opposition. Consideration is being given to destroying some dams, which is no easy task, especially if they are fairly large and have a lot of appurtenances like water intakes, shore developments, etc. (Tatro 1999). Dams interfere with the river ecology and may flood sensitive areas (cultural, religious, archeological, environmental, recreational, scenic, etc.). People living on the reservoir area of new dams have to relocate. Dams are not sustainable because eventually most if not all silt up, and because of evaporation they are not effective for long-term storage of water (years or decades). Such long-term storage may become increasingly necessary as increases of carbon dioxide and other greenhouse gases in the atmosphere cause global climatic changes that increase the probability of extremes in weather, such as more frequent drought and excessive rain. These conditions, along with increasing populations, increases the need for storing excess water in wet periods to meet water demands in dry periods.

Underground storage via artificial recharge has the advantage of essentially zero evaporation from the aquifer. Often economic and other aspects of recharge are also favorable. For these reasons, the practice of artificial recharge is rapidly increasing in many parts of the world. These aspects are discussed by Asano (1985) and in the proceedings of international recharge symposia that were held in California in 1988; in Florida in 1994 (proceedings of both symposia available from the American Society of Civil Engineers in Reston, Virginia, USA); in Amsterdam in 1998 (Peters 1998); Adelaide in 2002 and in Berlin in 2005. All three of the latter proceedings are available from Balkema Press. The last international recharge symposium took place in Phoenix, Arizona, in November 2007.

5

Water sources for artificial recharge through surface methods include water from perennial or intermittent streams that may or may not be regulated with dams, storm runoff (including from urban areas), aqueducts or other water conveyance facilities, irrigation districts, drinking water treatment plants, and sewage treatment plants. Artificial recharge of undesirable groundwater is expected to play an increasingly important role in water reuse, because it provides "soil-aquifer treatment," or geopurification of the effluent as it moves through soils and aquifers. Recharge also eliminates the undesired connection between the sewage treatment plant and the water supply system where municipal waste water is used to augment drinking-water supplies. This factor makes potable water reuse aesthetically much more acceptable to the public. Recharge also makes water reuse possible where religious taboos exist against certain direct uses of "unclean" water, as in Islamic countries (Ishaq and Khan 1997; Warner 2000), and in New Zealand, where the Maoris require wastes to pass through soil before they enter streams or lakes.

Of all the water in the world, 97% is salt water in the oceans (Bouwer 1978, and references therein). Of the remaining fresh water, two-thirds is in the form of ice in arctic and mountainous regions. Of the remaining liquid fresh water, less than 2% is surface water in streams and lakes, and much of that is fed by groundwater. More than 98% of the world's liquid fresh water thus is groundwater. Not only is groundwater the dominant water resource, aquifers also offer vast opportunities for underground storage of water through managed recharge and for conjunctive use of surface water and groundwater.

ARTIFICIAL RECHARGE SYSTEMS

Surface Infiltration

Surface infiltration systems for artificial recharge are divided into in-channel and off-channel systems. In-channel systems consist of dams placed across ephemeral or perennial streams to back up the water and spread it out, thus increasing the wetted area of the streambed or floodplain so that more water infiltrates into the ground and moves down to the groundwater (Figure 1.6). Some dams consist of low weirs spaced a small distance apart; others are larger dams spaced a greater distance apart (Figures 1.5 and 1.6). An option for consideration, particularly in areas with dry or almost dry river beds, is to divide the natural river channel into two parallel channels, one of which is used to convey residual streamflow while the other is being groomed and operated to maximize recharge.

Larger dams often need considerable spillway capacity to pass large flows. Sometimes they have a sacrificial section that washes out during high flows and is replaced when the flood danger is over. Steel weirs, earth dams, concrete dams, or inflatable rubber dams are used. The latter are big cylindrical bladders injected with water or air; air is generally preferred. Air pressures are relatively low (about 10 psi, or 1.5 kPa). When inflated, some water can spill over the dam, but for the big floods they are deflated to lay flat on their foundation. Where channels have small slopes and water depths, water is spread over the entire width of the channel or floodplain by placing T- or L-shaped earthen levees about 1 m high in the channel (Figures 1.5 and 1.6). These levees are pushed up by bulldozers using natural streambed sands. When the levees are washed out by high flows, they are restored by bulldozers. Off-channel surface recharge systems consist of specially-constructed infiltration basins (Figures 1.1 through 1.3), lagoons, old gravel pits, flood-irrigated fields, perforated pipes, or any other facility where water is put or spread on the ground for infiltration into the soil and movement to underlying groundwater.

6

Providing an in-channel capture basin just upstream of the hydrologic basin to be recharged provides some retention and settling of flood water for later slow release.

Water sources for in- and off-channel recharge systems should be of adequate quality to prevent undue clogging of the infiltrating surface by deposition and accumulation of suspended solids (sediment, algae, sludge); by formation of biofilms and biomass on and in the soil; by precipitation of calcium carbonate or other salts on and in the soil; and by formation of gases that stay entrapped in the soil, where they block pores and reduce hydraulic conductivity. Gases sometimes also accumulate under the clogging layer, where they form a vapor barrier to downward flow. The source of these gases can be dissolved air in the infiltrated water. The air goes out of the solution (1) as the water pressure drops from a pressure head equal to the water depth above the soil surface to a negative pressure head in the unsaturated zone below the clogging layer; or (2) where the soil or aquifer is warmer than the infiltrating water itself. Also, gases are formed by microbiological activity in the soil, such as nitrogen gas produced by denitrification and methane produced by methanosarcina and other methanogens in the Archeabacteria group.

Clogging of the infiltrating surface and resulting reductions in infiltration rates are the bane of all artificial recharge systems (Baveye et al. 1998; Bouwer. Ludke, and Rice 2001; Bouwer and Rice 2001). Pretreatment of the water to reduce suspended solids, nutrients, and organic carbon, and regular drying of the system to permit drying and cracking of the clogging layer and physical removal of the clogging layer may be necessary to minimize clogging effects. However, even when suspended solids, nutrients, and organic carbon are mostly removed from the water, clogging still is likely to occur because of microbiological growth on the infiltrating surface (Baveye et al. 1998). For example, such cloggings have been observed in laboratory infiltration studies in the dark with high- quality tap water (Bouwer and Rice 2001).

Surface infiltration systems normally require permeable surface soils to get high infiltration rates and to minimize land requirements. Where permeable soils occur deeper down and the less permeable overburden is not very thick, the overburden can be removed so that the basin bottom is in the more permeable material. Vadose zones should be free from layers of clay or other fine- textured materials that unduly restrict downward flow. Such layers can form perched groundwater that waterlogs the recharge area and reduces infiltration rates. Perched water can also form on aquitards where aquifers are semi-confined. Aquifers should be unconfined and sufficiently transmissive to accommodate lateral flow of the infiltrated water away from the recharge area without forming high groundwater mounds that interfere with the infiltration process. Also, soils, vadose zones, and aquifers should be free from undesirable contaminants that can be transported by the water and move to aquifers or other areas where they are not wanted.

Movement of infiltrated water through the vadose zone and aquifer is an effective way for further treating effluent from secondary sewage treatment plants for water reuse. This so-called soil aquifer treatment (SAT) removes essentially all biodegradable organic compounds, suspended solids, phosphorous, heavy metals, and microorganisms (viruses, bacteria, protozoa, helminths). Nitrogen is removed microbiologically by denitrification of nitrate and by anaerobic oxidation of ammonium (anammox process). Both processes require anaerobic conditions as occur deeper in the vadose zone and in the aquifer itself, and possibly in anaerobic micro sites that may occur in the upper vadose zone. Denitrification is heterotrophic and thus requires biodegradable organic carbon as an electron acceptor. This may be a limiting factor since most of the biodegradable organic carbon is degraded and essentially used up in the upper 3 ft or so of the vadose zone, leaving little or no organic carbon for the aquifer. The recently discovered anammox process is autotrophic, requires no organic carbon and, hence, can occur in the aquifer itself. Anammox is

7

most effective if nitrate and ammonium nitrogen are both present in about equal proportions ($5NH_4$ and $3NO_3$ stoichiometrically). If nitrogen removal in SAT systems is to be maximized, the sewage treatment plant should leave some dissolved biodegradable organic carbon in the effluent. Also, the basins should be flooded and dried so as to produce the right combination of aerobic and anaerobic conditions to stimulate nitrification in the upper vadose zone and denitrification and anammox in the lower vadose zone and aquifer. Field experiences already have shown that frequent short flooding periods (3 days flooding and 4 days drying) give essentially complete nitrification in the vadose zone and no removal of nitrogen whereas longer flooding periods (9 days flooding and 12 days drying) remove more than 50% of the total nitrogen in the effluent applied, with the residual nitrogen all as nitrate.

While SAT greatly reduces concentrations of organic compounds, residual TOC values of about 2 mg/L have been observed. With the present interest in pharmaceuticals and endocrine disruptors, more research on the composition of the final TOC is needed. Because the final concentration of these chemicals will be low, they are of concern but not alarming. SAT also enables blending of the effluent water with natural groundwater and thus enables indirect potable water reuse. The benefits of underground storage, SAT, and recovery for potable water reuse make artificial recharge of groundwater a very desirable technique for municipal water and wastewater management. Guidelines for the design and management of such systems have been developed in California. If, for example, the effluent has had primary and secondary treatment, plus disinfection, the infiltration rates in the recharge basins are less than 24 ft/day, and the depth to groundwater is more than 20 ft, the wells for pumping water for potable use should then be at least 1000 ft away from the basins, and they should pump a blend of at least 80% natural groundwater and not more than 20% reclaimed water for which the underground travel time from basin to well should be at least one year.

Vadose-Zone Infiltration

Where sufficiently permeable soils and/or sufficient land areas for surface infiltration systems are not available, groundwater recharge can also be achieved with vertical infiltration systems like trenches or wells in the vadose zone. Recharge trenches are dug with a backhoe and they are typically less than about 1 m wide and up to about 5 m deep (Figure 1.7). They are backfilled with coarse sand or fine gravel. Water normally is applied through a perforated pipe on the surface of the backfill, and the trench is covered to blend in with the surroundings. For example, a layer of topsoil for grass or other plantings can be placed on top of the backfill to blend in with landscaping, or concrete slabs or other paving are added where the area is paved. Sand-filled ditches have been tested in agricultural areas in Jordan to intercept surface runoff for deeper infiltration into the vadose zone (Abu-Zreig et al. 2000).

Vadose-zone wells (also called recharge shafts or dry wells) are normally installed with a bucket auger, and they are about 1 m in diameter and as much as 60 m deep (Figure 1.7). The wells are backfilled with coarse sand or fine gravel. Water is normally applied through a perforated or screened pipe in the center. Free-falling water in this pipe should be avoided to avoid air entrainment in the water and formation of entrapped air in the backfill and the soil around the vadose-zone well. To do this, water is supplied through a smaller pipe inside the screened or perforated pipe that extends to a safe distance below the water level in the well. Pipes with various diameters also can be installed. Water is then applied through the pipe that gives sufficient head

8

loss to avoid free-falling water. Also, a special orifice type of valve can be placed at the bottom of the supply pipe that can be adjusted to restrict the flow enough to avoid free-falling water.

The main advantage of recharge trenches or wells in the vadose zone is that they are relatively inexpensive. The disadvantage is that eventually they clog up at their infiltrating surface because of accumulation of suspended soils and/or biomass. Because they are in the vadose zone, they cannot be pumped for "backwashing" the clogging layer, or redeveloped or cleaned to restore infiltration rates. To minimize clogging, the water should be pretreated to remove suspended solids. For recharge trenches, pretreatment is accomplished in the trench itself by placing a sand filter with possibly a geotextile filter fabric on top of the backfill. Where this would reduce the flow into the backfill too much, the recharge trench could be widened at the top to create a T-trench with a larger filter area than the surface area of the trench itself. Economically, the choice is between pretreatment to extend the useful life of the trench or vadose-zone well, and constructing new ones. The old trenches or wells can then be abandoned, or they can continue to be used to take advantage of whatever residual recharge they still give. If the clogging is predominantly organic, some recovery in infiltration capacity may be achieved by very long drying or "resting" periods, perhaps on the order of a year or so.

Wells

Direct recharge or injection wells are used where permeable soils and/or sufficient land area for surface infiltration are not available, vadose zones are not suitable for trenches or wells, and aquifers are deep and/or confined. Truly confined aquifers are still rechargeable by lateral displacement of ambient groundwater away from the recharge well. Such aquifers also accept and yield water by expansion and compression of the aquifer itself and, particularly, of interbedded clay layers and aquitards that are more compressible than the sands and gravels or consolidated materials of the aquifer. However, excessive compression of aquifer materials by overpumping is mostly irreversible (Bouwer 1978, and references therein). Recharge may also be possible through semi-confining layers. This could, however create quality deterioration in the lower aquifer if the groundwater above the aquitard is of low quality due to irrigation, septic-tank leach fields, and other incidental recharges.

In the USA the water used for well injection is usually treated to meet drinking-water quality standards for two reasons. One is to minimize clogging of the well-aquifer interface, and the other is to protect the quality of the water in the aquifer, especially where groundwater is pumped from other wells in the aquifer for potable uses. Where groundwater is not used for drinking, water of lower quality can, in some areas, be injected into the aquifer. For example, in Australia stormwater runoff and treated municipal waste water effluent are injected into brackish aquifers to produce water for irrigation by pumping from the same wells. Clogging is then alleviated by a combination of low-cost water treatment and well redevelopment, and groundwater quality is protected for its declared beneficial uses (Dillon et al. 1997; Dillon and Pavelic 1996). These aquifer storage recovery operations have been successfully going on since 1993 in South Australia, and the number and size of the projects are expanding, using limestone, fractured rock, and alluvial aquifers.

Unconsolidated aquifers tend to be relatively coarse textured (sands and gravels) and are saturated; these materials do not give the same quality improvement of the recharge water as the finer-textured, unsaturated soils below surface and vadose-zone infiltration systems. Also, the water used for well injection in the USA is often chlorinated and has a chlorine residual of about

9

0.5 mg/L when it goes into the recharge well. Thus, whereas secondary sewage effluent can readily be used in surface infiltration systems in many parts of the United States, effluent for well injection should at least receive tertiary treatment (sand filtration and chlorination). This treatment removes remaining suspended solids and protozoa, like giardia, and cryptosporidium and parasites, like helminth eggs, by filtration; and bacteria and viruses by chlorination, ultra violet irradiation, or other disinfection. In USA the tertiary effluent is often further processed with membrane filtration (microfiltration and reverse osmosis) to remove any pathogens that may have escaped the tertiary treatment, and also nitrogen, phosphorus, organic carbon, and other chemicals. Dissolved salts also are almost completely removed. With all these removals, clogging problems still commonly occur when this water is used for groundwater recharge through wells. Geochemical compatibility between the recharge water and the existing groundwater (carbonate precipitation, iron hydroxide formation, mobilization of mineral chemicals, etc.) must also be considered. In Australia, where stormwater has been seasonally injected into aquifers, pathogen attenuation rates in aquifers are adequate for irrigation use and generally also meet local requirements for potable use of recovered water (Dillon and Pavelic 1996).

While clogged recharge wells can be redeveloped and rehabilitated with conventional techniques, a better approach is to prevent serious clogging by frequent pumping of the well, for example, about 15 min of pumping once, twice, or three times per day. This frequent "backwashing" of the clogging layer, which, of course, requires a dedicated pump in the well, often prevents serious clogging. As a matter of fact, the frequent backwashing may eliminate the need for membrane filtration. In one project near Phoenix, Arizona, USA, for example, well recharge with sewage effluent after primary and secondary treatment, nitrification-denitrification, filtration, and UV disinfection has shown no signs of clogging in the three years of operation of recharge wells that were pumped for about 30 minutes three times a day (Goldman 2001). Similarly in Australia the redevelopment is tuned to the sediment and organic loading of the well with very positive results (Dillon and Pavelic 1996). Thus, frequent pumping of injection wells may be more effective than membrane filtration treatment of the water to prevent well clogging. Typical backflushing frequencies are usually weekly to monthly, particularly with recharge of water from drinking water sources.

Combination Systems

Whenever possible, surface infiltration systems are preferred, because they offer the best opportunity for clogging control and the best soil-aquifer treatment if quality improvement of the water is of importance. If permeable soils occur at ground surface or within excavatable depth, the water can directly move into the coarse soils. However, where deeper fine-textured layers significantly restrict the downward movement of the water to the aquifer, and perched ground-water rises too high, surface infiltration can still be used if vertical infiltration systems are installed through the restricting layer (Figure 1.8). The upper parts of the systems then drain the perched groundwater while the lower parts provide infiltration and recharge of the aquifer. If the bottom of the restricting layer is not too deep (less than 3 m, for example), trenches can be used to drain the perched water and send it down to the aquifer. For deeper restricting layers (up to about 40 m) vadose-zone wells can be used whereas conventional wells can be used where the restricting layers are beyond reach of bucket augers (Figure 1.8). The wells would then be screened above and below the restricting or confining layer.
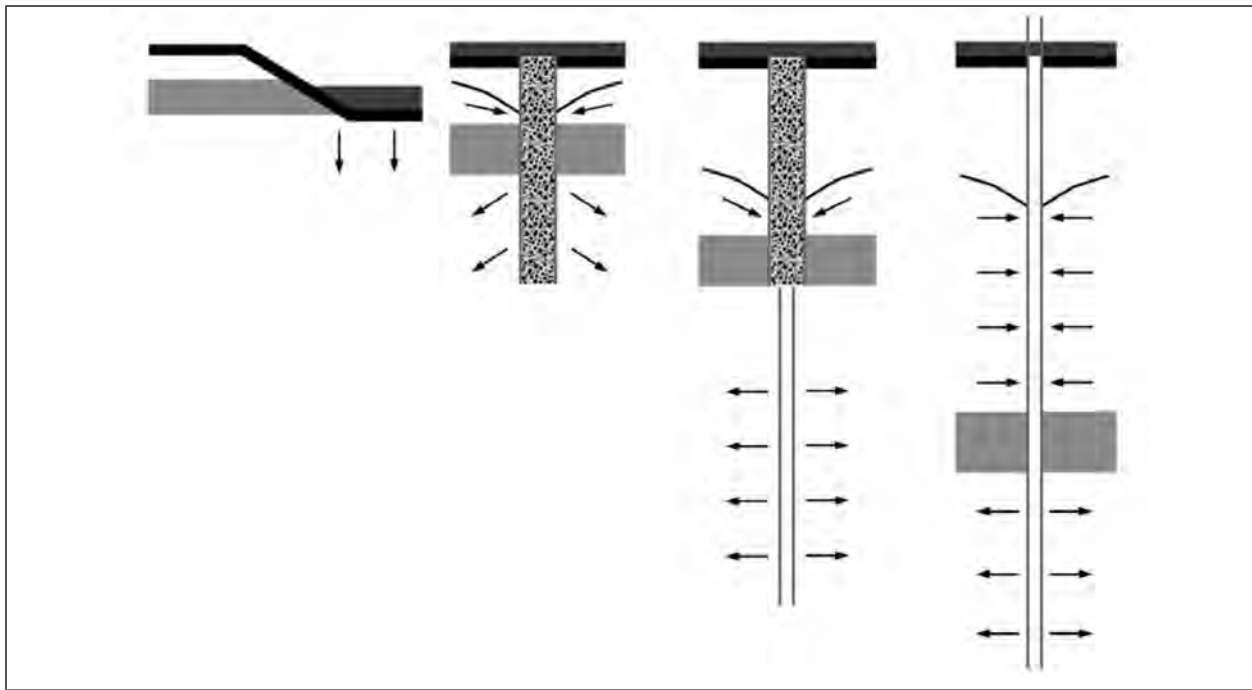
Figure 1.8 Surface infiltration systems with restricting layer (hatched) and perched groundwater draining to unconfined aquifer with trench (left), vadose-zone well (center), and aquifer well (right)

The advantage of the systems shown in Figure 1.8 is that the water has been prefiltered through the soil and the perched groundwater zone, so that its clogging potential is significantly reduced where it enters the aquifer. Even then, if the lower part of the system extends into the aquifer, as in Figure 1.8, it would probably be good practice to regularly pump the well. Water-quality issues also must be considered, particularly where the water above the restricting layer is of lower quality than that in the aquifer itself.

The principle of draining perched groundwater for recharge of underlying aquifers with systems such as shown in Figure 1.8 has not been adequately tested in the field. Thus, pilot testing of these systems should always be done to see if they work satisfactorily and how they should best be managed before large projects are installed and considerable amounts of money are invested. Pilot testing also is desirable for the simpler systems of basins, trenches, vadose-zone wells, and aquifer wells, because how these systems perform and how they should be designed and managed depends very much on local conditions of soil, hydrogeology, climate, and water quality. The golden rule in artificial recharge is to start small, learn as you go, and expand as needed.

DESIGN AND MANAGEMENT

Pre-Investigations

Where demands for groundwater exceed natural recharge, and where artificial or managed recharge is considered the best solution, the first steps are to assess availability of suitable water resources, soils, vadose zones, and aquifers, using soils and geologic maps, talking with local well

11

drillers, and doing some field investigations at promising sites. A few short deep trenches should be dug with a backhoe to examine the upper 10 or 15 feet of the vadose zone to make sure that there are no fine textured or cemented layers that would impede downward flow of water. Depths to groundwater, water quality, and groundwater conditions (confined or unconfined), should also be known. Pump or slug tests should be performed at selected sites to determine transmissivity and storage coefficient of the aquifer if not already known.

If no fatal flaws are detected, infiltration rates of the various soils should be measured with single cylinder infiltrometers that have a diameter of about 2 ft. Correcting the results for flow divergence in the soil and for limited depth of wetting then gives an estimate of the final infiltration rate for vertically downward flow in the soil. Very importantly, to eliminate effects of ionic composition of the water on the infiltration rate, the tests should be done with the same water as will be used in the actual project, especially if the soil contains clay. Infiltration rates typically range from 2 inches/day for fine soils with clay to about 30 ft/day for clean sands like dune or beach sand. However, higher infiltration rates also mean faster build up of clogging layers, especially when the water contains suspended solids. Thus, infiltration rates for recharge basins in these coarse soils may be considerably less than those from short infiltrometer tests. The results of these infiltration studies will give an idea of the recharge area needed to infiltrate a certain design flow, or, if available land is limited, how many acre feet/day can be recharged for a given land area. Next a few pilot tests should be done on the more promising sites to get a better idea of infiltration capacities. The pilot test basins should be about 10×10 ft in size and the water depth should be maintained at about 8 inches. The pilot tests should be done for at least a year to see how infiltration rates respond to clogging of the soil surface and to drying and cleaning this surface for the different seasons. In keeping with the philosophy that full scale projects should always be built in phases, starting small, learning as they proceed, and expanding as needed, a larger test basin of about 150×150 m should also be installed so that the results of the smaller plots can be tested on a larger area. The presence of low permeability layers deeper in the vadose zone that can form perched groundwater mounds can then also be detected. If again there are no fatal flaws, preliminary plans for the full scale recharge project can be made. This includes layout of basins and their flooding and drying periods.

The average infiltration for one flooding and drying cycle is called the hydraulic loading rate. Thus, if the infiltration rate during flooding of a basin is 2 ft/day and the basin is dry half the time (two weeks flooding and two weeks drying, for example), the hydraulic loading rate is 1 ft/day or 365 ft/year. One acre of basin area can then infiltrate 1 acre ft per day or 365 acre ft per year. The hydraulic loading rate includes drying times and thus gives an idea of how much land would be required for a given recharge capacity.

Infiltration Rates

Surface infiltration systems require permeable soils and vadose zones to get the water into the ground and to the aquifer, and unconfined and sufficiently transmissive aquifers to get lateral flow away from the infiltration system without excessive groundwater mounding. Thus, soil maps and hydrogeologic reports are used to do the first screening and to select promising sites. Depending on the relative amounts of clay (<2 μ), silt (2–50 μ) and sand (>50 μ), the textural classification of soil can be evaluated from the soil-textural triangle (Figure 1.9) prepared by the Soil Survey Staff of the U.S. Department of Agriculture (USDA) and published in 1951. Typical hydraulic conductivity values of the various soils are:

12
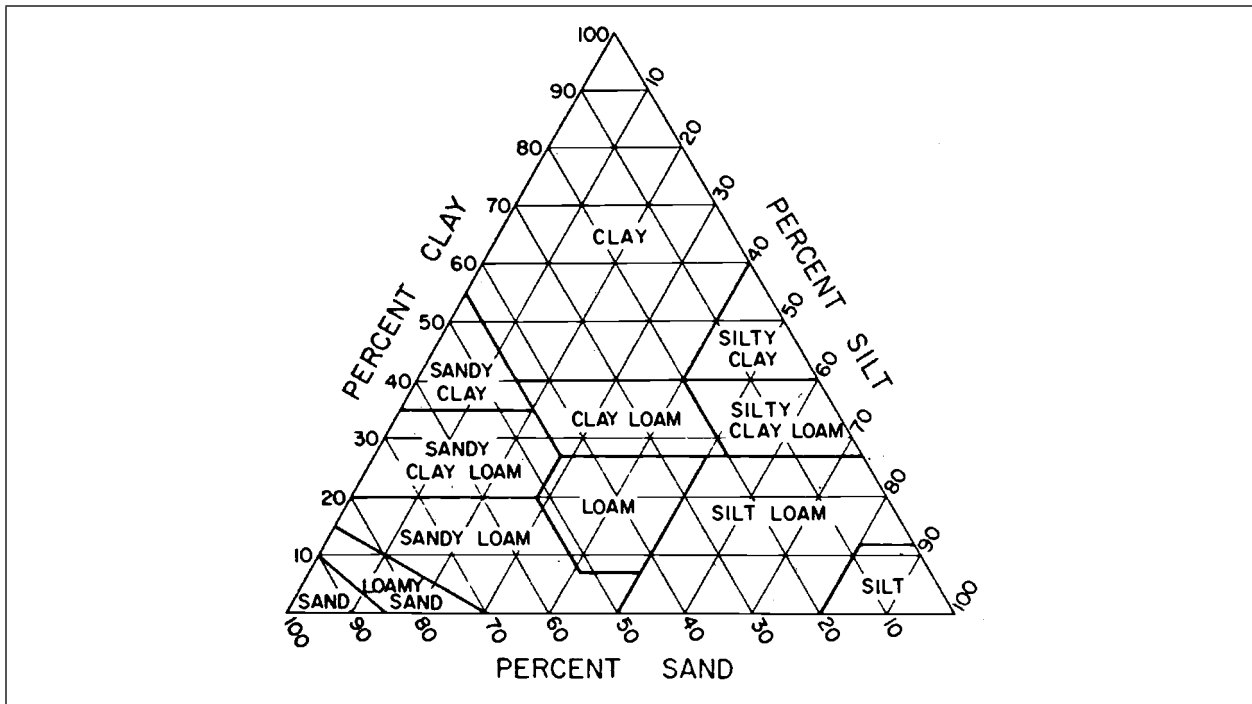
Figure 1.9 Triangular chart showing the percentages of sand, silt, and clay in the basic soil texture classes (*Source:* USDA 1951)

clay soils	<0.1 m/d
loams	0.2 m/d
sandy loams	0.3 m/d
loamy sands	0.5 m/d
fine sands	1.0m/d
medium sands	5.0 m/d
coarse sands	>10.0 m/d

If the soil contains gravel, the bulk hydraulic conductivity of the soil-gravel mixture can empirically be estimated as (Bouwer and Rice 1984b).

$$K_b = \frac{K_s e_b}{e_s} \tag{1}$$

where K_b = bulk hydraulic conductivity of soil-stones mixture
K_s = hydraulic conductivity of the soil fraction alone
e_b = bulk void ratio of soil-stones mixture
e_s = void ratio of soil alone

This equation applies to a continuous soil matrix with the stones embedded therein. The stones are impermeable bodies in a soil matrix, which reduces its hydraulic conductivity relative to that of the soil without stones.

13

$$v_i = K \frac{H_w + L_f - h_{cr}}{L_f}$$
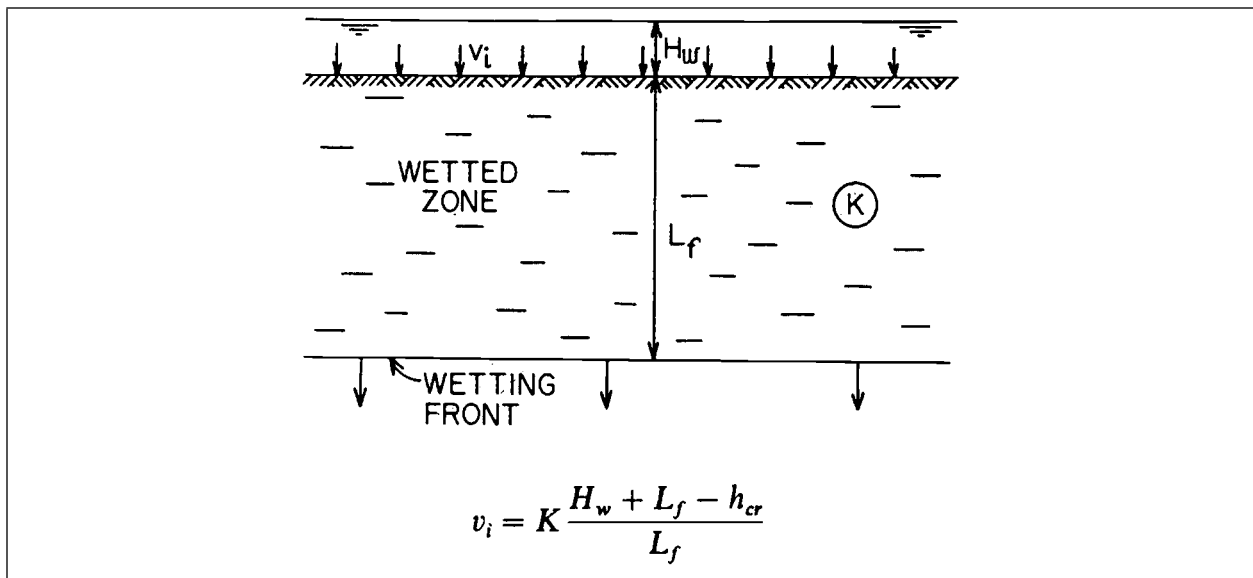
Figure 1.10 Section showing geometry and symbols for Green and Ampt piston-flow model of infiltration

Applying Darcy's equation to a soil after it has been flooded with water (Figure 1.10) yields (Bouwer 1978, and references therein)

$$V_i = K \frac{H_w + L_f - h_{we}}{L_f} \tag{2}$$

where
V_i = infiltration rate
K = hydraulic conductivity of wetted zone
H_w = water depth above soil
L_f = depth of wetting front
h_{we} = capillary suction or negative pressure head at wetting front

This is the Green-and-Ampt equation for infiltration into a flooded soil. It was developed almost 100 years ago and is based on the assumption of piston flow (Green and Ampt, 1911). Diffusion-based and empirical equations have also been developed (Bouwer 1978, and references therein). The term v_i is the volumetric infiltration rate per unit of surface area. It can be visualized as the rate of decline of the water surface in an infiltration basin if inflow is stopped and evaporation is ignored. Because the wetted zone is not completely saturated but contains entrapped air, its K is less than K_s at saturation, about $0.5K_s$ for sandy soils and $0.25K_s$, for clays and loams (Bouwer 1978).

The value of h_{we} is taken as the water-entry value of the soil. This parameter is the negative pressure head, where water displaces most of the air on the curve relating soil-water content to soil-water pressure of the soil. Typical values of h_{we} are (in cm water; Bouwer et al. 1999):

14

coarse sands	–5
medium sands	–10
fine sands	–15
loamy sands-sandy loams	–25
loams	–35
structured clays	–35
dispersed clays	–100

Equation (2) shows that when the soil is first flooded, L_f is small and V_i is high. However, as the wet front moves downward and L_f increases, the ratio in equation (2) approaches a value of one, and the infiltration rate becomes numerically equal to K of the wetted zone.

Infiltrometers

After soil and hydrogeologic surveys have identified potentially suitable sites for artificial recharge of groundwater with surface infiltration systems, "wet" infiltration tests should be performed to see what kind of infiltration rates can be expected, so that the land area needed for a certain volumetric recharge rate, or the recharge rate that can be achieved with a certain land area, can be estimated. Infiltration tests typically have been done with metal cylinder infiltrometers about 30 cm in diameter. However, use of such small infiltrometers often seriously overestimates the large-area infiltration rates because of lateral flow (divergence) below and around the cylinder due to lateral capillary suction in the soil (Bouwer 1960 and 1986, Bouwer et al. 1999). Double ring or "buffered" infiltrometers do not compensate for divergence, 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, 3×3 m bermed areas, where divergence or "edge" effects are relatively less significant. However, these tests are laborious and they also require large volumes of water, because it may take more than a day to reach or approach "final" infiltration rates. A better approach is 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 (for example, 1 to 5 hrs, depending on the soil). The extent of lateral wetting is measured using a shovel, and the depth of wet-front penetration is measured using a shovel or auger, or it is estimated from total accumulated infiltration and fillable porosity. The resulting infiltration data are then corrected for water depth in cylinder, limited depth of soil wetting below cylinder, and divergence outside cylinder to get an estimate of the long-term infiltration rate for a large inundated area (Bouwer et al. 1999). This rate should be about equal to the hydraulic conductivity K of the wetted zone.

The infiltrometers that have been used for this procedure are single steel cylinders 60 cm in diameter and 30 cm high with beveled edges (Figure 1.7). A piece of 5×10 cm lumber is placed on top of the cylinder and the cylinder is driven straight down with a sledge hammer to a depth of about 3–5 cm into the ground. The soil is packed against the inside and outside of the cylinder with a piece of 2.5×5 cm wood 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 to be used in the recharge project, to avoid errors due to effects of water quality on status of clay (coagulated or dispersed; Bouwer 1978). A plate or flat rock is placed on the soil inside the cylinder for erosion prevention when adding the water. The cylinder is filled to the top, and clock time is recorded. The water

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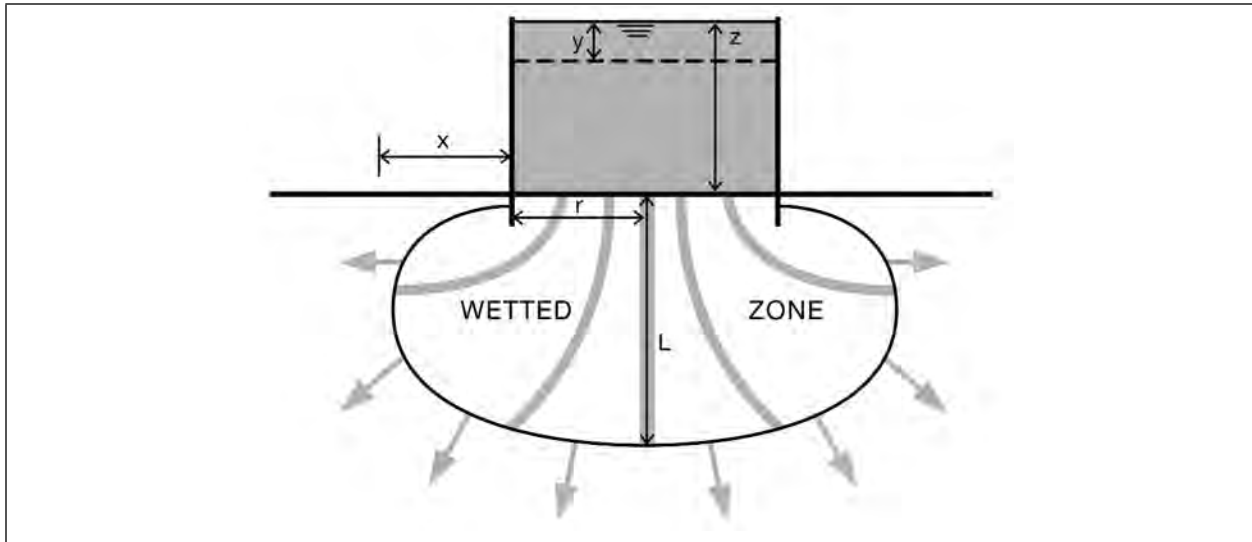
Figure 1.11 Geometry and symbols for single-ring infiltrometer

level is allowed to fall about 5–10 cm. The decline is measured with a ruler, clock time is recorded, and the cylinder is filled back to the top. This procedure is repeated for about 6 hours or until the accumulated infiltration has reached about 50 cm, whichever comes first. The last decline y_n is measured and clock time is recorded to obtain the time increment Δt_n for y_n. A shovel is used to dig outside the cylinder to determine the distance x of lateral wetting or divergence from the cylinder wall (Figure 1.11).

The depth L of the wet front at the end of the test is calculated from the accumulated declines y_t of the water level in the cylinder as

$$L = \frac{y_t \pi r^2}{n\pi(r+x)^2} \tag{3}$$

where n is the fillable porosity of the soil. The value of n can be estimated from soil texture and initial water content. For example, n is commonly about 0.3 for dry uniform soils, 0.2 for moderately moist soils, and 0.1 for relatively wet soils. Well-graded soils have lower values of n than uniform soils. The value of L can also be determined by augering or digging down immediately after the test to see how deep the soil has been wetted. This method works best if the soil initially is fairly dry, so that there is good contrast between wet and dry soil, and if there are not very many rocks. Applying Darcy's equation to the downward flow in the wetted zone then yields

$$i_w = \frac{K(z + L - h_{we})}{L} \tag{4}$$

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 can be estimated from the data listed below Equation 2. Because K is now the only unknown in Equation 5, it can be solved as:

16

$$K = \frac{i_w L}{(z + L - h_{we})} \qquad (5)$$

This calculated value of K is 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, as mentioned previously.

If the K-values calculated with Equation 6 are sufficiently large for an infiltration system, the next step is to put in some test basins of about 0.2 ha for long-term flooding, to evaluate clogging effects and the potential for infiltration reduction by restricting layers deeper down, and to have confidence in the scaling up from essentially point measurements of infiltration rates with cylinders to full-scale projects that may have 10–100 ha of recharge basins. Good agreement has been obtained between predicted infiltration rates (K in Equation 6) and those of larger basins. For example, six infiltrometers installed in a field west of Phoenix, Arizona, gave an average K of 40 cm/d as calculated with Equation 6. Two test basins of 0.3 ha each in the same field yielded final infiltration rates of 30 and 35 cm/d (Bouwer et al. 1999). If the infiltrometer tests give infiltration rates that are too low for surface infiltration systems, alternative systems such as excavated basins, recharge trenches, recharge shafts or vadose-zone wells, injection wells or aquifer storage recovery (ASR) wells can be considered.

The above cylinder infiltrometer 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 declines with vernier equipped hook gages are not necessary. The main idea is with cylinders to account somehow for divergence and limited depth of wetting, rather than applying a flat reduction percentage to go from short-term cylinder infiltration rates to long-term large-area infiltration rates, as has sometimes been done only to give overestimated reductions of basin infiltration rates. Because of spatial variability, cylinder infiltration tests should be carried out at various locations within a given site. Finally, the resulting average infiltration rates should never be expressed in more than two significant figures.

Where soils are stony, cylinder infiltrometers may be difficult to install. Sometimes, removing stones with a pick outside the cylinder as it is driven down and filling the empty spaces with a fine, powdery soil, is possible. If not, alternative procedures are to use larger, bermed test areas of at least 2×2 m for infiltration measurement, or to take a disturbed sample of the soil material between the stones and measure its hydraulic conductivity in the laboratory. Void ratios of the soil alone and of the bulk stony soil mixture are then estimated or determined, so that K_b of the stone-soil mixture is calculated with Equation 1 to give an idea of basin infiltration rates (Bouwer and Rice 1984b).

Uniform coarse sands as can be found in some glaciated areas (eskers, kames, outflow channels) can have very high hydraulic conductivities, well in excess of 10 m/day. This enables application of the recharge water with low pressure perforated pipe on the land surface (Figure 1.12), as is done in Finland to avoid construction of infiltration basins and other earth moving in environmentally sensitive areas. To prevent development of wetlands type vegetation around the pipes, they are intermittently operated or moved around.

17

Figure 1.12 Low-pressure perforated pipe for aquifer recharge in Finland

Soil Clogging

The main problem in infiltration systems for artificial recharge of groundwater is clogging of the infiltrating surface (basin bottoms, walls of trenches and vadose-zone wells, and well-aquifer interfaces in recharge wells), and resulting reduction in infiltration rates. Clogging is caused by physical, biological, and chemical processes (Baveye et al. 1998). Physical processes are accumulation of inorganic and organic suspended solids in the recharge water, such as clay and silt particles, algae cells, microorganism cells and fragments, and sludge flocs in sewage effluent. Another physical process is downward movement of fine particles in the soil that were in the applied water or in the soil itself, and accumulation of these fine particles at some depth where the soil is denser or finer, and where they form a thin subsurface clogging layer. The depth of this layer ranges from a few mm to a few m or more, depending on the soil profile. In the soils literature, this fine-particle movement and accumulation deeper down are called "wash out-wash in" (Sumner and Stewart 1992). Fine soil particles also form surface crusts when infiltration basins are dry and the soil is exposed to rainfall.

Biological clogging processes include accumulation of algae and bacterial flocs in the water on the infiltrating surface, and growth of micro-organisms on and in the soil to form biofilms and biomass (including polysaccharides and other metabolic endproducts) that block pores and/or reduce pore sizes. Chemical processes include precipitation of calcium carbonate, gypsum, phosphate, and other chemicals on and in the soil. Sometimes, these precipitations are induced by pH increases caused by algae as they remove dissolved CO_2 from the water for photosynthesis. Bacteria also produce gases (nitrogen, methane) that block pores and accumulate below clogging layers to create vapor barriers to infiltration. Gas is also formed in soil below recharge basins or in trenches or wells when the recharge water contains entrained or dissolved air and/or is cooler than the soil or aquifer itself. The water then warms up in the soil or aquifer, air goes out of solution and forms entrapped air which reduces the hydraulic conductivity. For well injection, this process is called air binding. Entrapped air also forms in response to decreases in water pressures, for example, when water moves through a clogging layer in a recharge basin and the pressure head is reduced from the

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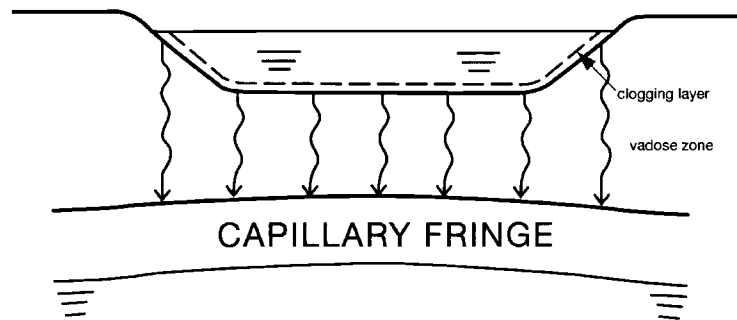
Figure 1.13 Infiltration basin, clogging layer, and capillary fringe above water table

water depth above the clogging layer to a negative pressure or suction in the unsaturated soil below the clogging layer. Well recharge also can cause precipitation of iron and manganese oxides or hydroxides as dissolved oxygen levels change, and solution and precipitation of calcium carbonate due to changes in pH and dissolved carbon dioxide levels. Conversely, dissolution of calcite by injected water has been found to increase hydraulic conductivity of limestone aquifers used for aquifer storage and recovery (Dillon and Pavelic 1996; Pyne 1995).

Because infiltration rates vary inversely with water viscosity, temperature also affects infiltration rates. In areas with large differences between winter and summer temperatures, viscosity effects alone can cause winter infiltration rates to be as low as about half of those in summer. Thus, if recharge systems need to be based on a certain capacity, they should be designed on the basis of the winter conditions, when water is coldest and infiltration rates are lowest. On the other hand, biological activity and the clogging that it causes may be highest in the summer. All these effects are hard to predict and the best way to get adequate design and management information for a full-scale project is by installing a few pilot basins of at least about 20×20 m and operating them for groundwater recharge for at least a year to measure seasonal effects.

Because clogging layers are much less permeable than the natural soil material, they reduce infiltration rates and become the controlling factor or "bottleneck" in the infiltration process (Figure 1.13). When the infiltration rate in surface systems becomes less than the hydraulic conductivity of the soil below the clogging layer, this soil becomes unsaturated to a water content whereby the corresponding unsaturated hydraulic conductivity is numerically equal to the infiltration rate (Bouwer 1982). The resulting unsaturated downward flow is then entirely due to gravity with a hydraulic gradient of one. The thickness of clogging layers may range from 1 mm or less (biofilms, thin clay and silt layers or "blankets") to several cm and dm or more for thicker sediment deposits. Because clogging layers are the rule rather than the exception, flow systems below surface infiltration systems typically are as shown in Figure 1.13, for vadose zones without restricting layers that otherwise could cause perched water to rise too close to the basin bottom.

The infiltration rate V_i for the basin in Figure 1.13 can be calculated by applying Darcy's equation to the flow through the clogging layer as

$$V_i = K_c \frac{H_w - h_{ae}}{L_c} \tag{6}$$

19

where K_c = hydraulic conductivity of the clogging layer
L_c = thickness of clogging in layer
H_w = water depth above clogging layer
h_{ae} = air entry value of vadose zone soil

Equation 6 applies to large shallow basins. For small and/or very deep basins, infiltration through the banks may be significant and can be calculated similarly.

Since the clogging layer is often very thin, from less than 1 mm to about 1 cm, its actual thickness and hydraulic conductivity are difficult to determine. For this reason, K_c and L_c are lumped into one parameter L_c/K_c, with the dimension of time (usually days) and called the hydraulic resistance R_c, which is the number of days it takes for a unit infiltration amount to move through the clogging layer at unit head loss. For a given system, R_c is calculated with Equation 7 from measured values of V_i and head loss across the clogging layer, using a tensiometer to measure h_{ae}. Also, h_{ae} can be estimated as $2h_{we}$, using the values shown below Equation 2. The air-entry value is more appropriate than the water-entry value in this case, because infiltration usually starts with a clean bottom condition, which causes the upper wetted zone initially to have positive water- pressure heads and, hence, to be near saturation. As the clogging develops and Vi decreases, the wetted zone becomes increasingly unsaturated as water contents decrease to produce hydraulic conductivities that are numerically equal to infiltration rates. Thus, air is displacing water in this case, and the air-entry value is the more appropriate value to use for the pressure head below the clogging layer (Bouwer 1982). Air-entry and water-entry values are parameters in hysteresis-affected water-content characteristics. These are curves relating water content to (negative) water-pressure head. Because of hysteresis, they are different for drying and wetting of the soil (Bouwer 1978, and references therein).

Clogging is controlled by reducing the parameters that cause clogging. For surface water, this typically means pre-sedimentation to settle clay, silt, and other suspended solids. This effect is accomplished by dams in the river or aqueduct system (which would also regulate the flow) or by passing the water through dedicated pre-sedimentation basins prior to recharge. Coagulants like alum and organic polymers are used to accelerate settling. Water inlets for the basins should be designed to avoid soil erosion with concrete splash pads or rock piles. Basin banks must be lined or vegetated to minimize erosion by rainfall runoff. For well recharge, sand or membrane filtration may also be necessary. Algae growth and other biological clogging in basins are reduced by removing nutrients (nitrogen and phosphorus) and organic carbon from the water. This reduction is also important where trenches, shafts, or wells are used for recharge with sewage effluent or effluent-contaminated water. Disinfection with chlorine or other disinfectants with residual effects reduces biological activity on and near the walls of the trenches, shafts, or wells and, hence, reduces clogging. Clogging rates increase with increasing infiltration rates, because of the increased loading rates of suspended solids, nutrients, and organic carbon on the surface.

Because of this, increasing the injection pressures in recharge wells that show signs of clogging actually hastens the clogging process. Regular pumping of recharge wells and periodic redevelopment of the wells controls and delays clogging, but maybe not "forever." The longest operating ASR wells in the United States are at Wildwood, NJ. Backflushed daily, they have been in operation since 1968 but recently have experienced a reduction in capacity. Increasing the water depth in recharge basins or the injection pressure in recharge wells also compresses the clogging layer, which reduces its permeability and, hence, the infiltration rates (see section "Effect of water depth on infiltration"). Despite pre-treatment of the recharge water, clogging still occurs due to growth of algae and autotrophic bacteria, dust being blown into the basin, and other factors.

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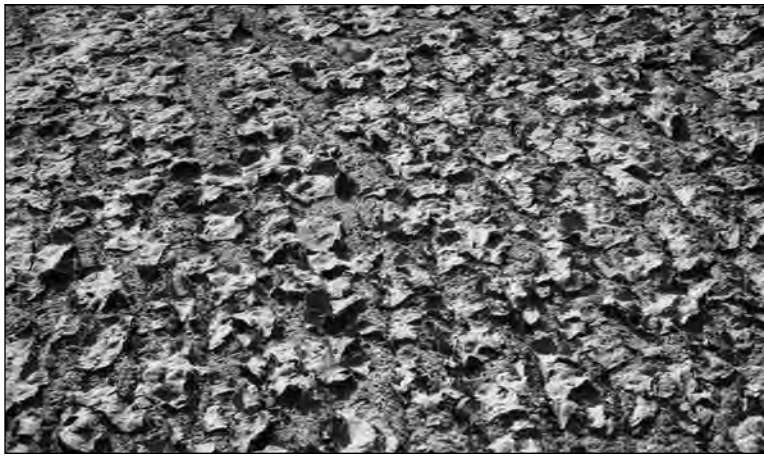
Figure 1.14 Dried and shrunk clogging layer of organic solids in sewage effluent

For surface infiltration systems, clogging is mostly controlled by periodically or regularly drying the basins or other infiltration facility, thereby letting the clogging layer dry, decompose, shrink, crack, and curl up (Figure 1.14). This procedure is generally sufficient to restore infiltration rates to satisfactory values. If clogging materials continue to accumulate, they must be periodically removed at the end of a drying period. This removal is done mechanically with scrapers, front-end loaders, graders, or manually with rakes. After removal of the clogging material, the soil should be disked or harrowed to break up any crusting that may have developed at or near the surface. Disking or plowing clogging layers as such into the soil without first removing them gives short-term relief, because eventually fines and other clogging materials accumulate in the topsoil and the entire disk or plow layer must be removed. Disking or harrowing may have to be followed by smoothing and lightly compacting the soil to prevent fine particle movement through the loose soil and accumulation of the fine particles on the underlying undisturbed soil when the soil is flooded again. This smoothing can be done by rolling or by dragging a pole or other implement over the soil. If routine drying and cleaning fail to restore infiltration rates, and there is no problem with high or perched groundwater levels, the problem may be deeper in the soil profile due to soil compaction and/or accumulation of fine particles on denser or finer soil. Compaction should always be minimized during construction of basins when cuts and fills need to be made. Harrowing should then be avoided and when the earth moving is finished the soil profile should be ripped with road-construction type rippers that go to a depth of about 1 m or 3 ft. Ripping tends to move stones up in the soil profile, which may eventually interfere with infiltration and disking of the soil. Therefore, ripping should not be done too frequently. For good quality surface water with very low suspended-solids contents and coarse soil materials in the recharge basin, drying and cleaning may be necessary only a few times a year or even less frequently. Where soils are relatively fine textured or have many stones, clogging control becomes a major challenge. Where the water is very muddy or where inadequately treated sewage effluent is used, drying and cleaning may be needed after each flooding period.

Pre-sedimentation is especially important where recharge water is obtained from streams with variable flows and as much water as possible needs to be used for groundwater recharge. Maximum volumes of water then may need to be captured during periods of high flow. For this purpose, deeper-than-normal infiltration basins are constructed to capture and store as much flood

21

flow as possible for subsequent infiltration and groundwater recharge. However, flood waters tend to carry more sediment, which settles out in the deep basins. Because of particle segregation according to Stokes Law, the sediment layer then has its coarsest particles on the bottom and its finest particles on top. This arrangement greatly reduces infiltration rates, especially if there have been repeated inflows of muddy water into the basin. Such inflows tend to create multi- layered clogging layers on the bottom with particle-size segregation in each layer (Bouwer, Ludke, and Rice 2001). Thus, the best way to utilize flood waters for artificial recharge is to capture and store these waters in deep basins or reservoirs that provide pre-sedimentation but are not expected to give high infiltration rates. Clear water is then taken out of these reservoirs and placed into shallow infiltration basins that can be readily dried and cleaned to maintain high infiltration rates (Bouwer and Rice 2001).

Another potential source of sediment in water for infiltration systems is rainfall runoff and erosion from the banks of the infiltration basins themselves. This is controlled by lining the banks with plastic or cement, or by a vegetative cover (natural or installed).

Hydraulic Loading Versus Infiltration Rates

In almost all cases, continuous flooding causes a clogging layer to form on the bottom of the basin or other surface infiltration facility causing infiltration rates to significantly decrease. Thus, regular drying is necessary to restore infiltration rates by drying and cracking of the clogging layer. If this is not sufficient, the dry flakes of the clogging layer must be removed by sweeping, raking, or scraping before flooding is resumed. This "down" time of the infiltration is taken into account by expressing the infiltration capacity of a system as a hydraulic loading rate, which is the total infiltration per flooding period divided by the combined length of the flooding and drying period. Thus, if the average infiltration during a flooding period is 4 ft/day, and the basins are operated on a schedule of 10 days flooding and 10 days drying, the long term hydraulic loading rate is 2 ft/day or 730 ft/yr. If a recharge project then must handle 73,000 ac.ft/yr (about 65 mgd), it must have at least 600 acres of recharge basin area. Seasonal effects also need to be considered, because hydraulic loading rates in winter are often less than in summer, due to cooler water with higher viscosity and to slower drying and infiltration recovery. On the other hand, biological activity and bio-clogging are more intense in the summer.

The intrinsic permeability (Bouwer 1978) shows that the product of K and viscosity of the water is constant for a given porous medium. Thus, K decreases inversely with increasing viscosity of the water. Since the viscosity of water increases with decreasing temperature, infiltration rates and hydraulic loading rates for a project in Nevada USA in winter were only about half as much as those in summer. For average water temperatures of 25°C and 5°C, the corresponding viscosities are 0.85 and 1.45 centipoises, respectively (Bouwer 1978). The ratio between these viscosities is 1.7, which is less than the observed ratio of about 2 for the hydraulic loading rates in the Nevada project. This difference is probably because drying periods for infiltration rate recovery are longer in winter when evaporation rates are lower, thus also decreasing hydraulic loading rates in that season. Temperature effects on infiltration must be taken into account in planning and managing surface infiltration systems for groundwater recharge. This is especially true for projects that use sewage effluent and must be able to absorb a certain flow. The total infiltration area should then be designed with enough safety or reserve capacity so that it can accept the desired flow when it is cold.

Infiltration and hydraulic loading rates are site specific and are best evaluated on pilot basins or on actual systems. Schedules of flooding, drying, and cleaning, and disking or other

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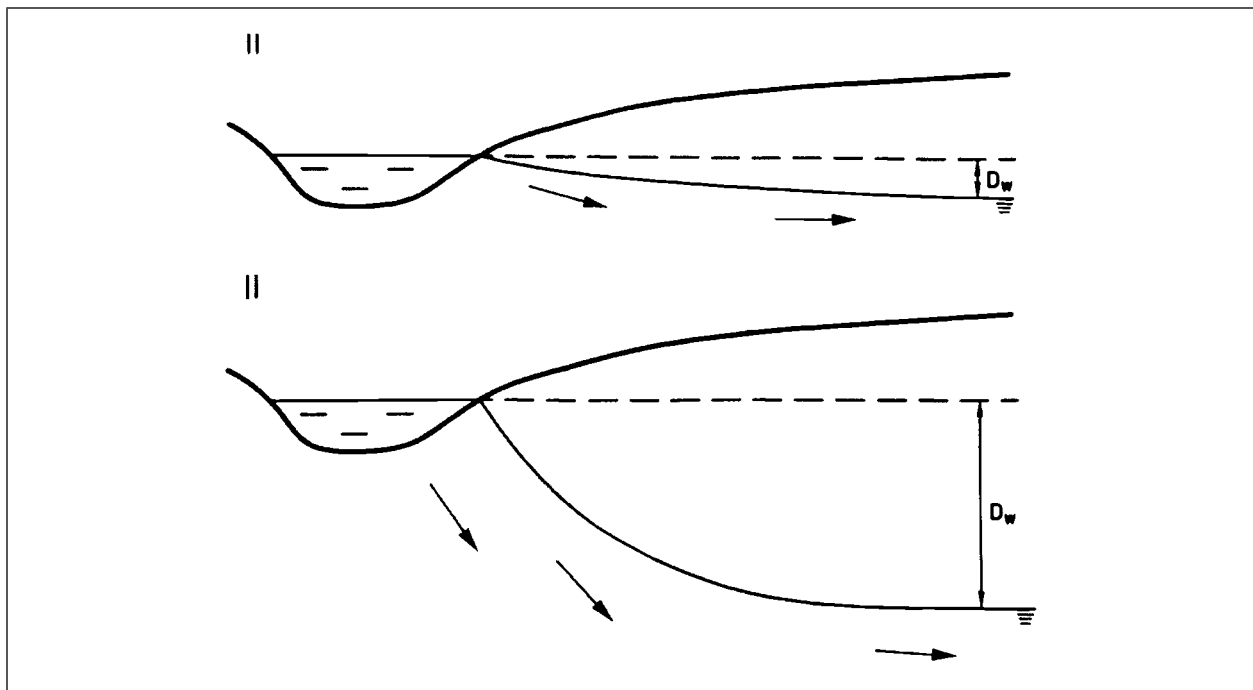
Figure 1.15 Section showing recharge basin with shallow water table and lateral flow in aquifer controlled by slope of water table (top), and with deep water table with downward flow below basin controlled by gravity (bottom)

tillage for optimum hydraulic loading are developed by trial and error. Experienced operators know that different infiltration basins in the same project often show different clogging and infiltration behavior and different responses to drying and cleaning. For this reason, multi-basin recharge projects should be designed so that each basin is hydraulically independent and can be operated according to its own best schedule. Hydraulic loading rates for systems in warm, relatively dry climates with good-quality input water and operated year round typically are about 30 m/yr for fine textured soils like sandy loams, 100 m/yr for loamy sands, 300 m/yr for medium clean sands, and 500 m/yr for coarse clean sands.

Annual evaporation rates from wet soil surfaces and free water surfaces commonly range from about 0.4 m/yr for cool, wet climates to 2.4 m/yr for warm, dry climates. Thus, evaporation losses are quite small compared to hydraulic loading rates. This makes groundwater recharge attractive for storing water, including long-term storage or water banking, because evaporation of groundwater from an aquifer is essentially zero, unless it is within reach of tree or plant roots (Bouwer 1975, 1978).

Effect of Water Depth on Infiltration

If no clogging layer exists on the bottom of an infiltration basin and the basin is "clean," the water table below the basin will connect with the water level in the basin, so that the basin and the aquifer are in direct hydraulic connection (Figure 1.15). If the depth D_W of the water table below the water level in the basin at some distance from the basin is relatively small, the flow away from the basin is mostly lateral and is controlled by the slope of the water table. On the other

23

hand, if the water table is deep and D_W is relatively large, the flow from the basin is mostly downward and controlled by gravity. Thus, if the water depth in the basin is increased, D_W also is increased. The resulting effect on infiltration is then significant if D_W is small, but negligible when D_W is already large. For example, if D_W in the system shown in Figure 1.15 is 3 m and the water depth in the basin is increased by 1 m, the infiltration flow increases by 33%. If, on the other hand $D_W = 30$m, the same 1m increase in the basin water depth will increase infiltration by only 3.3%.

If the basin is clogged, the infiltration flow is controlled by the clogging layer, and the vadose zone below the basin is unsaturated, as shown in Figure 1.13 (Bouwer 1982). In that case, infiltration rates increase almost linearly with water depth, as indicated by Equation 7, if nothing else changes. However, an increase in the water depth in a basin compresses the clogging layer, which then becomes less permeable. In that case, infiltration rates do not increase linearly with water depth and sometimes actually decrease, as has been observed in practice (Bouwer and Rice 1989). This compression is due to the fact that increasing the water depth in the basin with unsaturated flow below the clogging layer increases the head loss through this layer and, hence, the intergranular pressure in the clogging layer, which leads to compression of the layer in accordance with soil consolidation theory (Bouwer and Rice 1989). Compressible clogging layers like organic (sludge, algae) deposits or loose clay or "mucky" layers have greater compression and greater permeability reductions than less-compressible clogging materials like silts or fine sands.

Secondary effects also aggravate clogging. For example, if the water depth increases without a corresponding increase in infiltration rate, the rate of turnover of the water in the basin decreases. This causes suspended unicellular algae such as *Carteria klebsii* to be exposed longer to sunlight, which increases their growth rate, and hence, increases the algal filter cake or clogging layer on the bottom as more algal cells are physically strained out by the soil. Also, a high algal concentration on the bottom and in the water increases the pH of the water due to uptake of dissolved CO_2 for photosynthesis by the algae. This increase causes calcium carbonate to precipitate out and accumulate on the bottom, thus further aggravating the clogging problem and causing infiltration rates to decline even more. These processes explain why increasing the water depths in infiltration basins to overcome infiltration reductions by clogging has actually caused further reductions in infiltration rates, to the surprise and dismay of operators who thought that providing more "head" on the clogging layer would compensate for infiltration reductions by clogging layers. For this and other reasons, such as easier and quicker drying of basins for restoring infiltration rates, shallow recharge basins with water depths of about 0.5 m or less are generally preferred over deep basins. Also, in shallow basins with clear water, a mat of filamentous algae may develop on the bottom. The oxygen produced by the algae during photosynthesis can then stay entrapped in the algae mat (Figure 1.16), causing fragments and flakes of the mat to break loose and float to the water surface where they move to the downwind side of the basin and can be removed.

Effect of Artificial Recharge on Groundwater Levels

Rises in groundwater levels below infiltration systems, or mounding, can occur in two ways: perched mounding and aquifer mounding. If layers exist in the vadose zone whose hydraulic conductivities are less than the infiltration rate in the recharge basin, water accumulates above these "perching" layers to form "perched" groundwater. This perched groundwater then rises until it develops enough head and lateral spread on the perching layer for the flow to go

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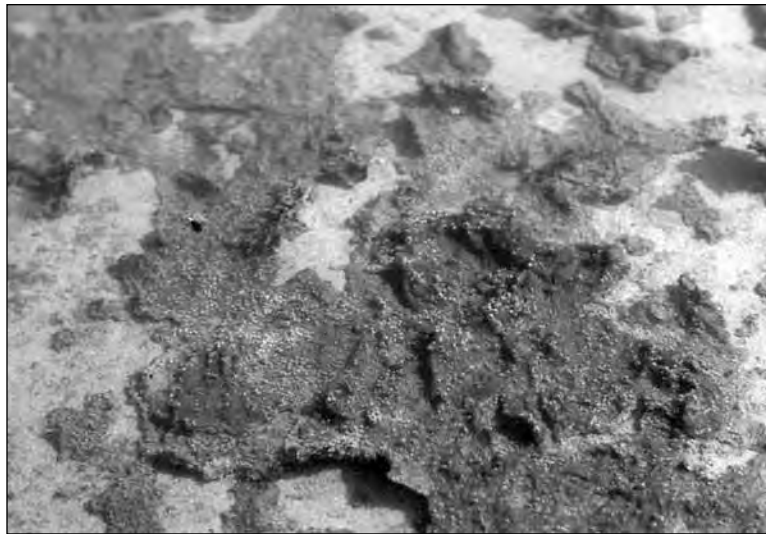
Figure 1.16 Bottom layer of filamentous algae breaking up and floating to the surface
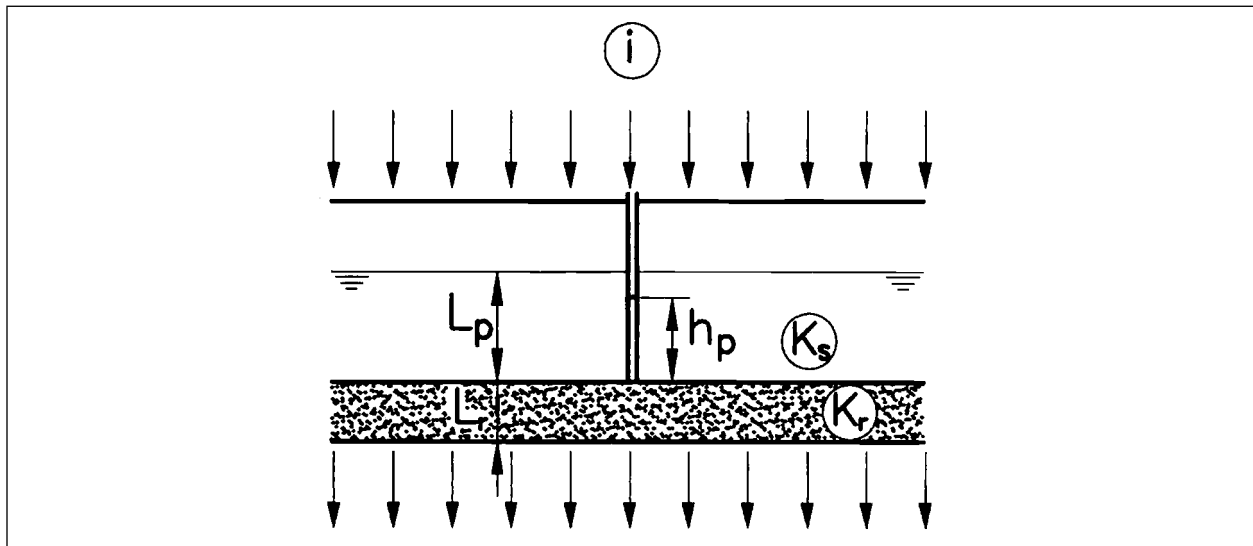
Figure 1.17 Section showing geometry and symbols for infiltration system with perched groundwater above a restricting layer with hydraulic conductivity K_r

through the perching layer at the same rate as it arrives from above, and the perched water table reaches an equilibrium position. For large recharge areas, this process can be considered a one-dimensional flow system (Figures 1.17 and 1.18). Applying Darcy's equation to the vertically downward flow in the perched groundwater above the restricting layer and through the restricting layer itself gives two equations with two unknowns (Bouwer et al. 1999), which when solved for the equilibrium height of the perched groundwater mound yields

25

Figure 1.18 Polishing pond and four infiltration basins of 10 acres each for secondary sewage effluent in Phoenix, Arizona

$$L_p = L_r \frac{\frac{V_i}{K_r} - 1}{1 - \frac{i}{K_S}} \tag{7}$$

where L_p = equilibrium height of perched mound above restricting layer
L_r = thickness of restricting layer
V_i = infiltration rate and downward flux through soil and restricting layer
K_r = hydraulic conductivity of restricting layer
K_s = hydraulic conductivity of soil above restricting layer.

Often, V_i is much smaller than K_s because surface soils tend to be finer textured than deeper soils, or a clogging layer is on the bottom of the infiltration system that reduces infiltration rates. Also, V_i often is considerably larger than K_r. For these conditions, Equation 8 can be simplified to

$$L_p = V_i \frac{L_r}{K_r} \tag{8}$$

which is useful to see if perching could be a problem.

In stratified soils, slowly permeable perching layers commonly consist as discontinuous layers or lenses, which cause a circuitous downward flow with lateral components. Also, for long, narrow recharge basins or recharge "strips," lateral spread and perching mounds above deeper restricting layers (Bouwer 1962) are often significant. If so, the vertical fluxes decrease as they cross the restricting layer over a larger horizontal area than that of the infiltration system itself. This condition also reduces the height of the perched mounds. In these circumstances, predicting

26

heights of perched groundwater mounds with Equation 8 or 9 overestimates mound heights. For example, analyses with an electrical resistance network analog showed that infiltration from a long and rectangular recharge basin of width *W* produced a perching mound on a slowly permeable layer deeper in the vadose zone that was about 0.4*W* high and 2*W* wide at its base (Bouwer 1962, 1978).

Numerous papers have been published on the rise of groundwater mounds on aquifers in response to infiltration from a recharge system, and some also on the decline of the mound after infiltration has stopped (Glover 1964; Hantush 1967; Marino 1975a, 1975b; Warner et al. 1989). The usual assumption is a uniform isotropic aquifer of infinite extent with no other recharges or discharges. One of the difficulties in getting meaningful results from the equations is getting a representative value of aquifer transmissivity. The most reliable transmissivity data come from calibrated models. Next in reliability are values obtained from Theis-type pumping tests, step-drawdown, and other pumped well tests, and slug tests (in decreasing order of "sampling" size).

Slug tests (Butler 1997), although simple to carry out, always have the problem of how to get representative areal values from essentially point measurements (the usual scaling-up problem). Averages from various tests often substantially underestimate more regional values (Bouwer 1996, and references therein). Also, results from slug tests on newly-drilled holes (sometimes only for slug testing and future monitoring) are commonly influenced by residual drilling mud around the screened section of the well and, hence, underestimate the hydraulic conductivity. Piezometers at two different depths in the center of a mound enable the determination of both vertical and horizontal hydraulic conductivity of an aquifer already being recharged with an infiltration system, through model simulation (Bouwer, Rice, and Escarcega 1974).

In deep or thick unconfined aquifers, streamlines of recharge flow systems are concentrated in the upper or "active" portion of the aquifer, with much less flow and almost stagnant water in the deeper or "passive" portion of the aquifer. Use of transmissivities of the entire aquifer between the water table and the impermeable lower boundary for mound calculations then seriously underestimates the rise of the mound. Older work (Bouwer 1962) with resistance-network analog modeling showed that for long rectangular recharge areas or recharge strips and a very thick aquifer, the thickness of the active, upper portion of the aquifer is about equal to the width of the recharge area. This thickness should then be multiplied by *K* of the upper aquifer to get an "effective" transmissivity for mounding predictions. Also, if the Hantush or another equation is used to calculate long-term mound formation, as for water banking in areas with deep groundwater levels, larger transmissivity values should be used to reflect the increase in transmissivity as groundwater levels rise. Otherwise, the Hantush equation overestimates the mound rise.

The best way to get representative transmissivity values for artificial recharge systems is to have a large enough infiltration test area or pilot project that produces a groundwater mound, and then to calculate the transmissivity from the rise of that mound using, for example, the Hantush equation (Equation 9). The fillable porosity to be used in the equations for mound rise is usually larger than the specific yield of the aquifer, because vadose zones often are relatively dry, especially in dry climates, and if they consist of coarse materials like sands and gravels. The fillable porosity should be taken as the difference between existing and saturated water contents of the material outside the wetted zone below the infiltration system. The Hantush equation (Figure 1.19; provided with kind permission of Springer Science and Business Media) is:

27
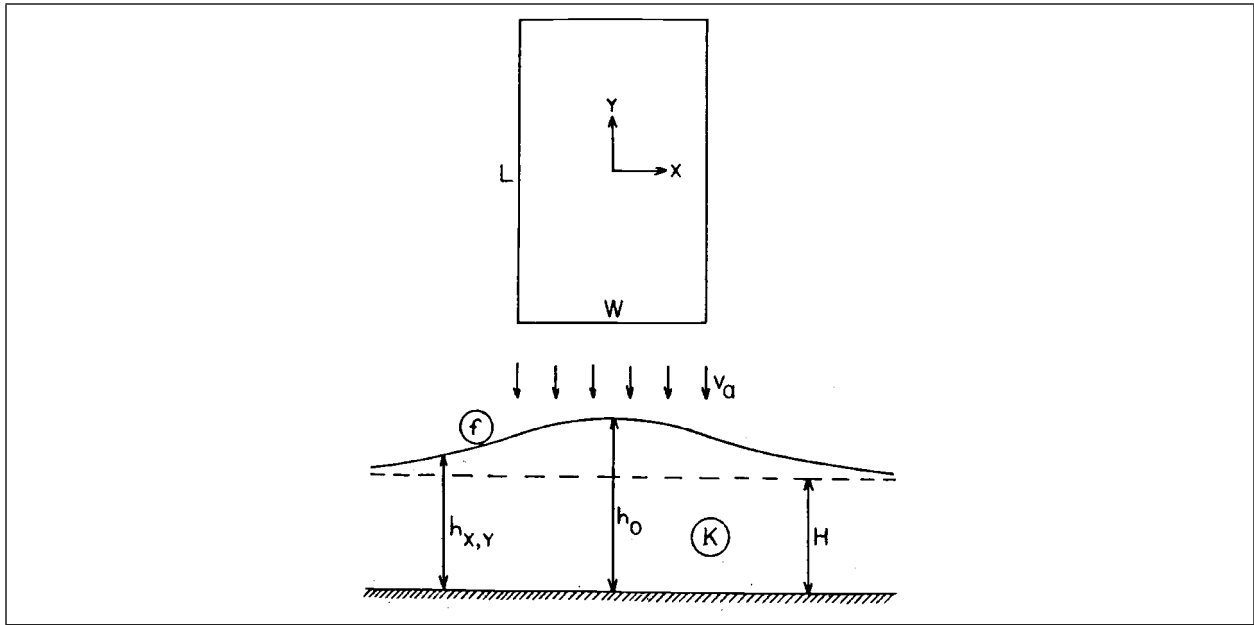
Figure 1.19 Plan view (top) and section (bottom) of infiltration and recharge system showing geometry and symbols for Hantush equation

$$
h_{x,y,t} - H = \frac{V_\alpha t}{4f} \left\{ F\left[\left(\frac{W}{2}+x\right)n, \left(\frac{L}{2}+y\right)n\right] \right. \tag{9}
$$

$$
+ F\left[\left(\frac{W}{2}+x\right)n, \left(\frac{L}{2}-y\right)n\right]
$$

$$
+ F\left[\left(\frac{W}{2}-x\right)n, \left(\frac{L}{2}+y\right)n\right]
$$

$$
\left. + F\left[\left(\frac{W}{2}-x\right)n, \left(\frac{L}{2}-y\right)n\right] \right\}
$$

where $h_{x,y,t}$ = height of water table above impermeable layer at x, y, and time t (Figure 1.19)
H = original height of water table above impermeable layer
V_α = arrival rate at water table of water from infiltration basin or basins
t = time since start of recharge
f = fillable porosity ($1 > f > 0$)
L = length of recharge basin or recharge project area (in y direction)
W = width of recharge basin or recharge project area (in x direction)
$n = (4t\,T/f)^{-1/2}$

$$
F(\alpha, \beta) = \int_0^1 erf(\alpha\tau^{-1/2}) erf(\beta\tau^{-1/2})\,d\tau
$$

28

where $\alpha = (W/2 + x)n$ or $(W/2 - x)n$ and $\beta = (L/2 + y)n$ or $(L/2 - y)n$. Values of F(α, β) were tabulated by Hantush. They are reproduced in Table 1.1.

Values of V_α, L, and W should be selected as they occur at the water table. If extensive perching and lateral flow occur in the vadose zone, V_α is less than the average infiltration rate of the recharge area, and L and W are larger than the actual dimensions of the infiltration system. Usually, however, V_α is taken as the infiltration rate for the entire recharge area (taking into account "dry" areas between basins), and L and W are taken as the dimensions of the entire recharge project. The Hantush equation is also used to calculate mound rises farther away from the recharge area, up to distances of about 0.5 W and 0.5 L to avoid negative terms in the error function of Equation 10. For predicting water-table effects farther away from the project, the Theis equation is used. Computer models like MODFLOW (McDonald and Harbaugh 1988) are used to include other regional recharge inputs and pumped-well outputs for the aquifer.

Of most interest to operators and managers, often, is the long-term effect of recharge on groundwater. Appropriate questions include: where will the groundwater mound be 10, 20, or 50 years from now; how much water can be stored or "banked" underground; will the whole area become waterlogged; and how must the water be recovered from the aquifer to prevent waterlogging of the recharge area and adjacent areas? Some water-banking projects have been installed or are planned in the desert valleys or basins of southern California and Arizona. Because these regions will then have basins for recharge and wells for pumping groundwater, an interest also exists to collect the natural recharge that is occurring in those areas, so that more water is pumped out of the aquifer than is put in with artificial recharge. This plan has sparked intensive interest in estimating natural recharge rates, which in these dry climates is very small (Tyler et al. 1996) and only a fraction (maybe about 1%) of a very small precipitation (about 10 cm/yr, or even less). To avoid depletion of the groundwater, excess pumping of groundwater should not exceed natural recharge rates. Monitoring will be necessary to make sure that undesirable depletions or other groundwater level responses do not occur.

A quick idea about ultimate or quasi-equilibrium mound heights for water banking or other recharge projects is obtained from a steady-state analysis, where the mound is considered to be in equilibrium with a constant water table at some depth and at a large distance from the infiltration system. The constant "far-away" water table can be established by groundwater pumping, discharge into surface water like rivers or lakes, or some other "control." Also, the farther away from the recharge area, the slower the water table rises. Thus, when groundwater levels are far enough away from the recharge area, they can be considered essentially stable. Steady-state equations were then developed for two general geometries of the entire recharge area:

1. The basins form a long strip with a length of at least 5 times the width, so that after long times it still performs about the same as an infinitely long strip (Glover 1964); and
2. The basins are in a round, square, or irregular area that can be represented by an equivalent circular area (Bouwer et al. 1999). For the long strip (Figure 1.20), the groundwater flow away from the strip is taken as linear horizontal flow (*Dupuit-Forchheimer flow*).

Below the infiltration area, the lateral flow is assumed to increase linearly with distance from the center. The lateral flow is then assumed to be constant between the edge of the recharge system at distance $W/2$ from the center and the constant-control water table at distance L_n from the edge (Figure 1.21). This set of conditions yields the equation

29

Table 1.1
Values for solution of Hantush equation (Equation 10)

Values of the function $F(\alpha, \beta)$ in Eq. (10) for different values of α and β

α \ β	0.02	0.04	0.06	0.08	0.10	0.14	0.18	0.22	0.26	0.30	0.34	0.38	0.42	0.46	0.50	0.54	0.58	0.62
0.02	0.0041	0.0073	0.0101	0.0125	0.0146	0.0184	0.0216	0.0243	0.0267	0.0288	0.0306	0.0322	0.0337	0.0349	0.0361	0.0371	0.0380	0.0387
0.04	0.0073	0.0135	0.0188	0.0236	0.0278	0.0353	0.0416	0.0470	0.0518	0.0559	0.0596	0.0628	0.0657	0.0683	0.0705	0.0725	0.0743	0.0759
0.06	0.0101	0.0188	0.0266	0.0335	0.0398	0.0509	0.0602	0.0684	0.0754	0.0817	0.0871	0.0920	0.0963	0.1001	0.1035	0.1065	0.1091	0.1115
0.08	0.0125	0.0236	0.0335	0.0425	0.0508	0.0652	0.0776	0.0884	0.0978	0.1060	0.1133	0.1197	0.1254	0.1305	0.1350	0.1389	0.1425	0.1456
0.10	0.0146	0.0278	0.0398	0.0508	0.0608	0.0786	0.0939	0.1072	0.1183	0.1290	0.1381	0.1461	0.1532	0.1595	0.1650	0.1700	0.1744	0.1783
0.14	0.0184	0.0353	0.0509	0.0652	0.0786	0.1025	0.1232	0.1414	0.1573	0.1714	0.1839	0.1949	0.2048	0.2135	0.2212	0.2281	0.2343	0.2397
0.18	0.0216	0.0416	0.0602	0.0776	0.0939	0.1232	0.1490	0.1716	0.1916	0.2094	0.2251	0.2391	0.2515	0.2626	0.2724	0.2812	0.2890	0.2959
0.22	0.0243	0.0470	0.0684	0.0884	0.1072	0.1414	0.1716	0.1984	0.2222	0.2433	0.2621	0.2789	0.2938	0.3071	0.3189	0.3295	0.3389	0.3472
0.26	0.0267	0.0518	0.0754	0.0978	0.1188	0.1573	0.1916	0.2222	0.2494	0.2737	0.2954	0.3147	0.3320	0.3474	0.3612	0.3735	0.3844	0.3941
0.30	0.0288	0.0559	0.0817	0.1060	0.1290	0.1714	0.2094	0.2433	0.2737	0.3009	0.3252	0.3470	0.3665	0.3839	0.3995	0.4134	0.4257	0.4368
0.34	0.0306	0.0596	0.0871	0.1133	0.1381	0.1839	0.2251	0.2621	0.2954	0.3252	0.3520	0.3761	0.3976	0.4169	0.4341	0.4495	0.4633	0.4756
0.38	0.0322	0.0628	0.0920	0.1197	0.1461	0.1949	0.2391	0.2789	0.3147	0.3470	0.3761	0.4022	0.4256	0.4466	0.4654	0.4823	0.4973	0.5108
0.42	0.0337	0.0657	0.0963	0.1254	0.1532	0.2048	0.2515	0.2938	0.3320	0.3665	0.3976	0.4256	0.4508	0.4734	0.4937	0.5119	0.5281	0.5427
0.46	0.0349	0.0683	0.1001	0.1305	0.1595	0.2135	0.2626	0.3071	0.3474	0.3839	0.4169	0.4466	0.4734	0.4975	0.5191	0.5385	0.5559	0.5715
0.50	0.0361	0.0705	0.1035	0.1350	0.1650	0.2212	0.2724	0.3189	0.3612	0.3995	0.4341	0.4654	0.4937	0.5191	0.5420	0.5626	0.5810	0.5975
0.54	0.0371	0.0725	0.1065	0.1389	0.1700	0.2281	0.2812	0.3295	0.3735	0.4134	0.4495	0.4823	0.5119	0.5385	0.5626	0.5842	0.6036	0.6209
0.58	0.0380	0.0743	0.1091	0.1425	0.1744	0.2343	0.2890	0.3389	0.3844	0.4257	0.4633	0.4973	0.5281	0.5559	0.5810	0.6036	0.6238	0.6420
0.62	0.0387	0.0759	0.1115	0.1456	0.1783	0.2397	0.2959	0.3472	0.3941	0.4368	0.4756	0.5108	0.5427	0.5715	0.5975	0.6209	0.6420	0.6609
0.66	0.0394	0.0773	0.1136	0.1484	0.1818	0.2445	0.3020	0.3547	0.4027	0.4466	0.4865	0.5227	0.5556	0.5854	0.6122	0.6354	0.6582	0.6778
0.70	0.0401	0.0785	0.1154	0.1509	0.1849	0.2488	0.3075	0.3612	0.4104	0.4553	0.4962	0.5334	0.5672	0.5977	0.6254	0.6503	0.6728	0.6929
0.74	0.0406	0.0796	0.1171	0.1531	0.1876	0.2526	0.3123	0.3671	0.4172	0.4630	0.5048	0.5429	0.5774	0.6087	0.6371	0.6627	0.6857	0.7064
0.78	0.0411	0.0806	0.1185	0.1550	0.1900	0.2559	0.3166	0.3722	0.4232	0.4699	0.5125	0.5513	0.5865	0.6185	0.6475	0.6736	0.6972	0.7184
0.82	0.0415	0.0814	0.1198	0.1567	0.1921	0.2589	0.3203	0.3768	0.4286	0.4760	0.5192	0.5587	0.5946	0.6272	0.6567	0.6834	0.7074	0.7291
0.86	0.0419	0.0822	0.1209	0.1582	0.1940	0.2615	0.3237	0.3808	0.4333	0.4813	0.5252	0.5653	0.6017	0.6348	0.6648	0.6920	0.7165	0.7386
0.90	0.0422	0.0828	0.1219	0.1595	0.1957	0.2638	0.3266	0.3844	0.4374	0.4860	0.5305	0.5711	0.6080	0.6416	0.6721	0.6996	0.7245	0.7469
0.94	0.0425	0.0834	0.1228	0.1607	0.1971	0.2658	0.3292	0.3875	0.4411	0.4902	0.5351	0.5762	0.6136	0.6476	0.6784	0.7063	0.7316	0.7543
0.98	0.0428	0.0839	0.1236	0.1617	0.1984	0.2676	0.3314	0.3902	0.4442	0.4938	0.5392	0.5807	0.6184	0.6528	0.6840	0.7123	0.7378	0.7608
1.00	0.0429	0.0842	0.1239	0.1622	0.1990	0.2684	0.3324	0.3914	0.4457	0.4955	0.5410	0.5827	0.6206	0.6552	0.6865	0.7150	0.7406	0.7638
1.20	0.0437	0.0858	0.1263	0.1654	0.2030	0.2740	0.3396	0.4001	0.4558	0.5070	0.5540	0.5969	0.6362	0.6719	0.7044	0.7339	0.7605	0.7846
1.40	0.0441	0.0866	0.1275	0.1669	0.2049	0.2767	0.3431	0.4043	0.4608	0.5127	0.5603	0.6039	0.6438	0.6801	0.7132	0.7432	0.7704	0.7949
1.80	0.0444	0.0871	0.1283	0.1680	0.2062	0.2785	0.3454	0.4071	0.4641	0.5165	0.5645	0.6086	0.6489	0.6856	0.7190	0.7494	0.7769	0.8018
2.00	0.0444	0.0871	0.1284	0.1681	0.2064	0.2787	0.3457	0.4075	0.4645	0.5171	0.5651	0.6092	0.6495	0.6863	0.7198	0.7502	0.7778	0.8027
2.20	0.0444	0.0872	0.1284	0.1682	0.2065	0.2788	0.3458	0.4076	0.4646	0.5171	0.5653	0.6094	0.6497	0.6865	0.7200	0.7505	0.7781	0.8030
2.50	0.0444	0.0872	0.1284	0.1682	0.2065	0.2788	0.3458	0.4077	0.4647	0.5172	0.5653	0.6095	0.6498	0.6867	0.7202	0.7506	0.7782	0.8032
3.00	0.0444	0.0872	0.1284	0.1682	0.2065	0.2789	0.3458	0.4077	0.4647	0.5172	0.5654	0.6095	0.6499	0.6867	0.7202	0.7506	0.7782	0.8032

α \ β	0.62	0.66	0.70	0.74	0.78	0.82	0.86	0.90	0.94	0.98	1.00	1.20	1.40	1.80	2.00	2.20	2.50	3.00
0.02	0.0387	0.0394	0.0401	0.0406	0.0411	0.0415	0.0419	0.0422	0.0425	0.0428	0.0429	0.0437	0.0441	0.0444	0.0444	0.0444	0.0444	0.0444
0.04	0.0759	0.0773	0.0785	0.0796	0.0806	0.0814	0.0822	0.0828	0.0834	0.0839	0.0842	0.0858	0.0866	0.0871	0.0871	0.0872	0.0872	0.0872
0.06	0.1115	0.1136	0.1154	0.1171	0.1185	0.1198	0.1209	0.1219	0.1228	0.1236	0.1239	0.1263	0.1275	0.1283	0.1284	0.1284	0.1284	0.1284
0.08	0.1456	0.1484	0.1509	0.1531	0.1550	0.1567	0.1582	0.1595	0.1607	0.1617	0.1622	0.1654	0.1669	0.1680	0.1681	0.1682	0.1682	0.1682
0.10	0.1783	0.1818	0.1849	0.1876	0.1900	0.1921	0.1940	0.1957	0.1971	0.1984	0.1990	0.2030	0.2049	0.2062	0.2064	0.2065	0.2065	0.2065
0.14	0.2397	0.2445	0.2488	0.2526	0.2559	0.2589	0.2615	0.2638	0.2658	0.2676	0.2684	0.2740	0.2767	0.2785	0.2787	0.2788	0.2788	0.2789
0.18	0.2959	0.3020	0.3075	0.3123	0.3166	0.3203	0.3237	0.3266	0.3292	0.3314	0.3324	0.3396	0.3431	0.3454	0.3457	0.3458	0.3458	0.3458
0.22	0.3472	0.3547	0.3612	0.3671	0.3722	0.3768	0.3808	0.3844	0.3875	0.3902	0.3914	0.4001	0.4043	0.4071	0.4075	0.4076	0.4077	0.4077
0.26	0.3941	0.4027	0.4104	0.4172	0.4232	0.4286	0.4333	0.4374	0.4411	0.4442	0.4457	0.4558	0.4608	0.4641	0.4645	0.4646	0.4647	0.4647
0.30	0.4368	0.4466	0.4553	0.4630	0.4699	0.4760	0.4813	0.4860	0.4902	0.4938	0.4955	0.5070	0.5127	0.5165	0.5169	0.5171	0.5172	0.5172
0.34	0.4756	0.4865	0.4962	0.5048	0.5125	0.5192	0.5252	0.5305	0.5351	0.5392	0.5410	0.5540	0.5603	0.5645	0.5651	0.5653	0.5653	0.5654
0.38	0.5108	0.5227	0.5334	0.5429	0.5513	0.5587	0.5653	0.5711	0.5762	0.5807	0.5827	0.5969	0.6039	0.6086	0.6092	0.6094	0.6095	0.6095
0.42	0.5427	0.5556	0.5672	0.5774	0.5865	0.5946	0.6017	0.6080	0.6136	0.6184	0.6206	0.6362	0.6438	0.6489	0.6495	0.6497	0.6498	0.6499
0.46	0.5715	0.5854	0.5977	0.6087	0.6185	0.6272	0.6348	0.6416	0.6476	0.6528	0.6552	0.6719	0.6801	0.6856	0.6863	0.6865	0.6867	0.6867
0.50	0.5975	0.6122	0.6254	0.6371	0.6475	0.6567	0.6648	0.6721	0.6784	0.6840	0.6865	0.7044	0.7132	0.7190	0.7198	0.7200	0.7202	0.7202
0.54	0.6209	0.6364	0.6503	0.6627	0.6736	0.6834	0.6920	0.6996	0.7063	0.7123	0.7150	0.7339	0.7432	0.7494	0.7502	0.7505	0.7506	0.7506
0.58	0.6420	0.6482	0.6728	0.6857	0.6972	0.7074	0.7165	0.7245	0.7316	0.7378	0.7406	0.7605	0.7704	0.7769	0.7778	0.7781	0.7782	0.7782
0.62	0.6609	0.6778	0.6929	0.7064	0.7184	0.7291	0.7386	0.7469	0.7543	0.7608	0.7638	0.7846	0.7949	0.8018	0.8027	0.8030	0.8032	0.8032
0.66	0.6778	0.6953	0.7110	0.7250	0.7375	0.7486	0.7584	0.7671	0.7748	0.7816	0.7846	0.8064	0.8171	0.8243	0.8252	0.8255	0.8257	0.8257
0.70	0.6929	0.7110	0.7272	0.7417	0.7546	0.7660	0.7762	0.7852	0.7932	0.8002	0.8034	0.8259	0.8370	0.8445	0.8454	0.8458	0.8460	0.8460
0.74	0.7064	0.7250	0.7417	0.7566	0.7698	0.7816	0.7921	0.8014	0.8096	0.8168	0.8201	0.8434	0.8549	0.8627	0.8636	0.8640	0.8642	0.8642
0.78	0.7184	0.7375	0.7546	0.7698	0.7834	0.7956	0.8063	0.8159	0.8243	0.8317	0.8351	0.8591	0.8710	0.8789	0.8799	0.8803	0.8805	0.8805
0.82	0.7291	0.7486	0.7660	0.7816	0.7956	0.8080	0.8190	0.8288	0.8374	0.8450	0.8485	0.8731	0.8853	0.8935	0.8945	0.8949	0.8951	0.8951
0.86	0.7386	0.7584	0.7762	0.7921	0.8063	0.8190	0.8302	0.8402	0.8491	0.8569	0.8604	0.8855	0.8980	0.9065	0.9075	0.9079	0.9081	0.9081
0.90	0.7469	0.7671	0.7852	0.8014	0.8159	0.8288	0.8402	0.8504	0.8594	0.8674	0.8710	0.8966	0.9094	0.9180	0.9191	0.9195	0.9197	0.9197
0.94	0.7543	0.7748	0.7932	0.8096	0.8243	0.8374	0.8491	0.8594	0.8686	0.8767	0.8803	0.9064	0.9195	0.9282	0.9294	0.9298	0.9300	0.9300
0.98	0.7608	0.7816	0.8002	0.8168	0.8317	0.8450	0.8569	0.8674	0.8767	0.8849	0.8886	0.9151	0.9284	0.9373	0.9384	0.9389	0.9391	0.9391
1.00	0.7638	0.7846	0.8034	0.8201	0.8351	0.8485	0.8604	0.8710	0.8803	0.8886	0.8924	0.9191	0.9324	0.9414	0.9426	0.9430	0.9432	0.9433
1.20	0.7846	0.8064	0.8259	0.8434	0.8591	0.8731	0.8855	0.8966	0.9064	0.9151	0.9191	0.9472	0.9614	0.9709	0.9722	0.9726	0.9728	0.9729
1.40	0.7949	0.8171	0.8370	0.8549	0.8710	0.8853	0.8980	0.9094	0.9195	0.9284	0.9324	0.9614	0.9759	0.9858	0.9871	0.9875	0.9878	0.9878
1.80	0.8018	0.8243	0.8445	0.8627	0.8789	0.8935	0.9065	0.9180	0.9282	0.9373	0.9414	0.9709	0.9858	0.9959	0.9972	0.9977	0.9979	0.9980
2.00	0.8027	0.8252	0.8454	0.8636	0.8799	0.8945	0.9075	0.9191	0.9294	0.9384	0.9426	0.9722	0.9871	0.9972	0.9985	0.9990	0.9992	0.9934
2.20	0.8030	0.8255	0.8458	0.8640	0.8803	0.8949	0.9079	0.9195	0.9298	0.9389	0.9430	0.9726	0.9875	0.9977	0.9990	0.9995	0.9997	0.9998
2.50	0.8032	0.8257	0.8460	0.8642	0.8805	0.8951	0.9081	0.9197	0.9300	0.9391	0.9432	0.9728	0.9878	0.9979	0.9992	0.9997	1.0000	1.0000
3.00	0.8032	0.8257	0.8460	0.8642	0.8805	0.8951	0.9081	0.9197	0.9300	0.9391	0.9433	0.9729	0.9878	0.9980	0.9933	0.9998	1.0000	1.0000

Source: Hantush 1967.

30

Figure 1.20 Flushing Meadows Project in Salt River bed west of Phoenix, Arizona

Figure 1.21 Section showing geometry and symbols for groundwater mound below a long infiltration area (strip) of width *W*

$$H_c - H_n \;=\; \frac{iW}{2T}\left(\frac{W}{4} + L_n\right) \tag{10}$$

for the ultimate rise of the groundwater mound below the center of the recharge strip when equilibrium exists between recharge and pumping from the aquifer a distance away (Bouwer et al. 1999). The symbols in this equation are (Figure 1.21):

> H_c = height of groundwater mound in center of recharge area
> H_n = height of groundwater table at control area
> i = average infiltration rate in recharge area (total recharge divided by total area)
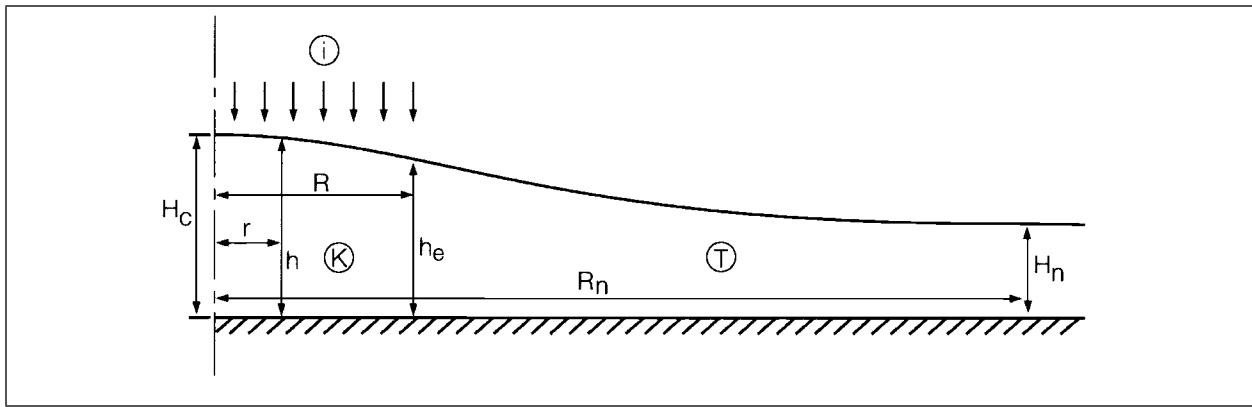> W = width of recharge area

31

Figure 1.22 Section showing geometry and symbols for groundwater mound below round infiltration area of radius R

L_n = distance between edge of recharge area and control area
T = transmissivity of aquifer

For a more round or square type of recharge area (Figures 1.18 and 1.22), the groundwater flow is radially away from the area. The equilibrium height of the mound below the center of the recharge system above the constant groundwater table at distance R_n from the center of the recharge system can be calculated with radial flow theory (Bouwer et al. 1999) as

$$H_c - H_n = \frac{iR^2}{4T}\left(1 + 2\ln\frac{R_n}{R}\right) \tag{11}$$

where R is the radius or equivalent radius of the recharge area, R_n is the distance from the center of the recharge area to the water-table control area (Figure 1.22) and the other symbols are as previously defined.

Equations 10 and 11 thus are used to predict the final mound height below a recharge area for a given elevation of the control water table at distance R_n or L_n from the recharge area. As indicated for the Hantush equation, the value of T in equations 10 and 11 must reflect the average transmissivity of the aquifer at the ultimate equilibrium mound height.

If the calculated ultimate mound height is too high, R_n or L_n must be reduced by groundwater pumping from wells closer to the recharge area, or H_n must be reduced by pumping more groundwater. Equations 10 and 11 then indicate where groundwater should be recovered and to what depth groundwater levels should be pumped to prevent water tables below the recharge areas from rising too high. Ultimate mound heights can also be reduced by making the recharge area longer and narrower, or by reducing recharge rates by using less water for recharge or by spreading the infiltration facilities over a larger area. Limits to groundwater rises can also be required to protect third party interests, like basements, cemeteries, gravel pits, pipe lines, old trees, low areas, and others that would be damaged by high groundwater levels.

The rise of the groundwater mound below the recharge area can be calculated with the Hantush equation, as discussed above, which gives the rise in relation to time and distance from the recharge area, including the center. The aquifer should be sufficiently transmissive to prevent

32

undue rise of the mound below the infiltration area that could reduce infiltration rates by "drowning" the recharge basins. Also, groundwater table rise away from the recharge system should not be so high that it unfavorably affects gravel pits, landfills, basements, pipe lines, and other "third party" interests.

A simple way to calculate or predict groundwater mounding below basins or other surface infiltration systems for groundwater recharge is to calculate ultimate mounding heights for an assumed equilibrium condition where the flow in the aquifer away from the recharge area is taken as steady horizontal flow to a constant water table some distance away. Such a constant water table may be due to groundwater pumping, to a stream or lake into which the groundwater discharges, or it could be taken far away (a mile, for example) where the water table rise is so little compared to the mound below the center of the basin area, that the water table there can be considered constant. The steady-state equation then gives the height of the water table at any point between the recharge area and the point where the water table is taken constant. The advantages of this approach are that it is simple and gives the rise of the water table due to the recharge project alone and not to other groundwater accretions. This may be important for legal aspects. Another possibility would be to model the whole region with all its groundwater discharges and recharges and aquifers, and see what the overall effects are on groundwater levels in the region.

Mound rise for a given capacity of a recharge project is higher when the basins are close together in a square or a round area than when they are in a long narrow area like a linear or strip arrangement. Also, spreading the basins out over a larger area will cause lower mound rise than when they are all together in one bloc.

Effects of Groundwater Levels on Infiltration Rates

Often, bottoms and banks of infiltration basins are covered with a clogging layer that controls and reduces infiltration rates so that the underlying soil material is unsaturated (Figure 1.13). The water content in the unsaturated zone then establishes itself at a value whereby the corresponding unsaturated hydraulic conductivity is numerically equal to the infiltration rate, because the downward flow is due to gravity alone and the hydraulic gradient is one (Bouwer 1982). The unsaturated zone breaks the hydraulic continuity between the basin and the aquifer, so that infiltration rates are independent of depth to groundwater, as long as the water table is deep enough so that the top of the capillary fringe above the water table is below the bottom of the basin. This capillary fringe is commonly about 30 cm thick for medium sands, more for finer sands or soils, and less for coarse sands. Thus, a conservative conclusion is that as long as the water table is more than about 1m below the bottom of a basin where infiltration is controlled by a clogging layer on the bottom, infiltration rates are unaffected by changes in groundwater levels. If the water table rises, infiltration rates start to decrease only when the capillary fringe reaches the bottom of the basin. They then continue to decrease linearly with decreasing depth to groundwater below the water level in the basin, until they become zero when the water table has risen to the same elevation as the water surface in the basin. The transition between hydraulically disconnected water-table conditions (Figure 1.8) to hydraulic connection (Figure 1.15) was modeled by Dillon and Liggett (1983), who observed that infiltration rates decline significantly due to hydraulic connection as the water table rises and intersects the basin or stream.

Where the water for recharge is exceptionally clear and free from nutrients and organic carbon, temperatures are low, and soils are relatively coarse, infiltration proceeds for a considerable time without development of a clogging layer on the bottom. In that case, direct hydraulic
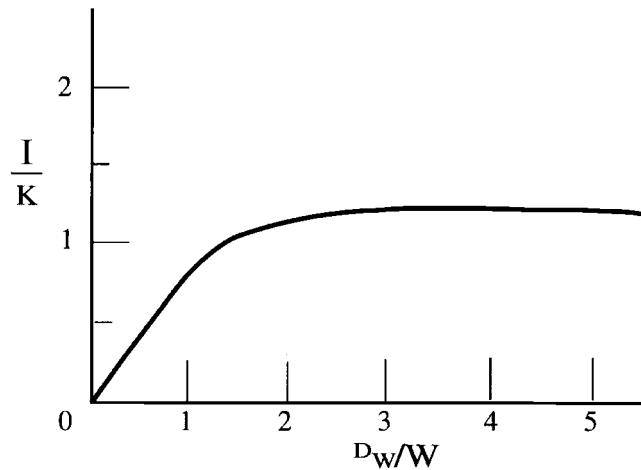
33

Figure 1.23 Dimensionless plot of seepage (expressed as *I/K*) versus depth to groundwater (expressed as D_w/W) for clean stream channel or long infiltration basin with no clogging layer on bottom

continuity exists between the clean basin and the aquifer with the water table joining the water surface in the basin (Figure 1.15). Groundwater levels are then characterized by the depth D_W of the water table below the water-surface elevation in the basin. D_W should be taken at a sufficient distance from the recharge area such that groundwater levels are relatively unaffected by the recharge flow system (Figure 1.15). In previous work, this distance was arbitrarily taken as 10 times the width of the basin or recharge system (Bouwer 1969). If D_W is relatively large, the flow below the recharge system is mainly downward and controlled by gravity (Bouwer 1969, 1978) so that the hydraulic gradient is about one for some distance below the bottom (Figure 1.15). In that case, infiltration rates are essentially unaffected by depth to groundwater. However, if groundwater levels rise and D_W decreases, the flow from the recharge basin becomes more lateral until eventually it is completely controlled by the slope of the water table away from the basin (Figure 1.16; Bouwer 1969, 1978). Modeling these flow systems on an electrical resistance network analog has shown that the change from gravity-controlled flow to flow controlled by slope of the water table occurs when D_W is about twice the width W (or diameter) of the recharge system (Bouwer 1990). This relation is shown in Figure 1.23, where I is the infiltration rate per unit area of water surface in the basin and K is the hydraulic conductivity in the wetted zone or aquifer. Thus, as long as $D_W < 2W$, infiltration rates decrease almost linearly with decreasing D_W and reach zero when $D_W = 0$ (Figure 1.15).

However, if $D_W > 2W$, infiltration rates are essentially constant and about equal to the theoretical maximum value when $D_W = 4$, regardless of the actual value of D_W. These relations apply to uniform, isotropic underground formations. Anisotropic or stratified situations need to be considered on a case-by-case basis. In the USA, legal interpretation of groundwater and surface-water interactions do not always conform with hydrologic relations (Bouwer and Maddock 1997).

Infiltration rates in clean basins (no clogging layers) thus are more sensitive to depth to groundwater than rates in clogged basins. Clogged basins are the rule rather than the exception, and groundwater mounds can rise much higher there than below clean basins before reductions in infiltration rates occur. Sometimes, maximum permissible mound heights are dictated by circumstances

34

other than their effect on infiltration rates, such as presence of sanitary landfills, underground sewers or other pipelines, basements (especially deep basements of commercial buildings), cemeteries, deep- rooted vegetation like old trees that may die when groundwater levels rise too high, and other situations where high groundwater levels are not wanted.

An interesting and unusual application of aquifer recharge through infiltration basins to achieve groundwater mounding as a primary goal is the C-111 project in south Florida, presented subsequently in Chapter 3 and the Appendix. The goal of this project is to create a mound in the water table at the eastern edge of the Everglades National Park, thereby preventing loss of water through subsurface eastward flow to beneath adjacent urban areas, where the water table is maintained at a lower elevation. A series of these infiltration basins is planned along the eastern edge of the Park. During 2004 the first four basins recharged 142,000 AF. An interesting secondary goal of this project is to reduce the volume of poor quality water that is discharged to Biscayne Bay during wet weather periods, causing adverse estuarine ecosystem impacts. Typical depths to static water level are zero to three feet.

Vadose-Zone Wells

Vadose-zone wells, also called dry wells or recharge shafts, are boreholes in the vadose zone, usually about 10–50 m deep and about 1–2 m in diameter (Figure 1.7). They are commonly used for infiltration and "disposal" of storm runoff in areas of relatively low rainfall that have no storm sewers or combined sewers. Temporary surface storage must be provided where the runoff rates exceed the infiltration rate of the dry wells. This can be achieved by putting the dry wells in low areas that become flooded when it rains, like a park, or sport fields. The areas can be naturally depressed, excavated, or flat and surrounded by a berm or levee. Dry wells normally are drilled into permeable formations in the vadose zone that can accept the runoff water at sufficient rates. Where groundwater is deep (for example, 100–300 m or more), dry wells are much cheaper than recharge wells and, hence, it is tempting to use dry wells for groundwater recharge instead of aquifer wells. Such vadose-zone wells are similar to recharge pits or recharge shafts, which also have been used for recharge of groundwater. To get adequate recharge, vadose-zone wells should penetrate permeable formations for a sufficient depth. On the other hand, where recovery wells pump the water from the aquifer, the pumped wells could also be used for recharge, so that vadose-zone wells may not be necessary (see section on aquifer storage recovery wells). Also, where groundwater levels are very deep and vadose zones relatively dry, considerable volumes of water are needed to wet the vadose zone before water arrives at the aquifer.

The main problem with vadose-zone wells is, of course, clogging of the wall of the well and the impossibility of remediating that clogging by pumping or redeveloping the well since it is in the vadose zone and groundwater cannot "backwash" the clogging material. Also, energy-based cleaning techniques like surging and jetting cannot be used, because vadose-zone wells are filled with sand or gravel. Thus, clogging must be prevented or minimized. This goal is achieved by protecting the water in the well against slaking and sloughing of clay layers in the vadose zone that could make the water in the well muddy, causing clay to accumulate and form a clogging layer on the more permeable soil material where most of the infiltration takes place. This slaking is minimized by filling the well with sand and using a perforated pipe or screen in the center to apply the water for recharge. Placing plastic sheets or geotextiles in the well against the zones with clay layers also may be effective. Also, the water must be treated before recharge to remove as many clogging agents as possible, including suspended solids, assimilable organic carbon,

35

nutrients, and microorganisms. Micro filtration may be necessary where low quality water such as sewage effluent is used. Disinfection to maintain a residual chlorine level may also be helpful. If clogging still occurs (and long-term clogging is always a possibility), it may be mostly due to bacterial cells and organic metabolic products like polymers on the wall of the well. Thus, while such clogging cannot be remediated by pumping, cleaning or redevelopment, a very long drying period could produce sufficient biodegradation of the clogging material to restore the vadose-zone well for another episode of recharge, albeit at reduced rates.

Because recharge with aquifer wells or vadose-zone wells may be more expensive than with surface infiltration systems, rigorous economic analyses are necessary to develop the best system. Factors to be considered include the cost of vadose-zone wells vs. aquifer wells, their recharge capacities and the number of wells needed, their useful lives, maintenance and/or replacement costs, and the cost of necessary pretreatment of the water. Contaminated vadose zones usually preclude the use of vadose-zone wells.

Recharge rates for vadose-zone wells in uniform soil materials are calculated from Zangar's equation for reverse auger-hole flow (Bouwer 1978). For a typical vadose-zone well geometry, with groundwater levels significantly below the bottom of the well and a water depth in the well of at least five well diameters, this equation can be simplified to

$$Q = \frac{2\pi K L_w^2}{\ln\left(\frac{2L_w}{r_w}\right) - 1} \tag{12}$$

where Q is the recharge rate, K is the hydraulic conductivity of the soil material, L_w is the water depth in the well, and r_w is the radius of the well (Figure 1.7). L_w should be at least 10 r_w in order for the equation to be valid. The proper value for K is difficult to assess, because the wetted zone is not always saturated and the streamlines have horizontal and vertical components, which complicates matters for anisotropic soils. The best way to evaluate K for use in Equation 12 is from test wells in the vadose zone.

More research is needed on vadose-zone recharge wells to develop an optimum design for well capacity, clogging control (including pretreatment and superdisinfection), useful life, and minimum long-term cost of recharge per unit volume of water. Superdisinfection consists of maintaining such a high residual disinfectant level in the recharge water that microbiological activity cannot occur in the well itself but takes place farther away in the vadose zone or aquifer, where the disinfectant is dissipated and biological activity can occur. The expectation is that this activity would then be far enough away from the well so that it occurs over a large enough area to prevent development of a clogging zone. Instead, it could develop a biofilter zone, which could even improve the quality of the recharge water going through it. More research is necessary to see if this approach is possible and how it could be managed for optimum recharge capacity and water-quality improvement. Ultimately, the usefulness of vadose-zone wells or trenches depends on their useful lives and the cost of recharge per unit volume of water added to the aquifer.

Seepage Trenches

Where permeable surface soils are not available but permeable strata occur within trench-able depth (about 2–5 m, for example), drilled vadose-zone wells are probably not necessary and

36

seepage trenches (also called infiltration trenches) are likely to be more cost effective (Figure 1.7). The trenches are backfilled with coarse sand or fine gravel, water is applied to the surface of the backfill, and the trench is covered to keep out sunlight, animals, and people (Hantke 1983) and to make the trenches "invisible" by giving them the same surface condition as the surrounding area. Clogging is reduced by use of geotextiles on or in the backfill to filter the water and by placing plastic sheets against clay zones in the trench to prevent sloughing of the clay and mud from entering the trench. As with vadose-zone and aquifer wells, the water for seepage trenches must have a very low suspended-solids content. Using a simple conversion from radial flow from a vertical line source to parallel flow from a vertical plane source, the recharge rate for seepage trenches is estimated to be about 20% of Q calculated with Equation 12 for a vadose-zone well. This recharge rate then applies to a trench width and length section equal to the diameter of the well (i.e., $2r_w$) and a trench-water depth equal to the water depth in the well. Thus, if a dry well 1 m in diameter, 10 m deep, and filled with water to the top infiltrates 1,000 m^3/d, a 10 m deep trench, 1 m wide, and full of water infiltrates about 200 m^3/d per m length of trench. As with surface infiltration systems, experimental vadose-zone wells or trenches should always be installed in new areas where there is no previous experience with these systems to see how they perform and how they should be designed and managed (including pre-treatment of the water) for optimum performance in a full-scale system.

Clogging Potential for Injection/Recharge Wells

To predict the clogging potential of the water for well injection, three main clogging parameters have been identified (Peters and Castell-Exner 1993): the membrane filtration index (MFI), the assimilable organic carbon content (AOC), and the parallel filter index (PFI). These parameters can also be used for evaluating water for vadose-zone wells and trenches. The MFI describes the suspended-solids content of the water in terms of the slope of the straight portion of the curve in a plot of time/volume versus volume in a membrane filter test, using for example, a 0.45μ millipore filter. Thus, the dimension of the MFI is time/volume^2; for example, s/L^2.

Assimilable organic carbon is determined microbiologically by plating out and incubating a water sample for growth of bacteria of the type Pseudomonas fluorescence, counting the bacterial colonies, and expressing the results in terms of the carbon concentration of an acetate solution producing the same bacterial growth. AOC may be less than 1% of dissolved organic carbon (DOC). AOC levels in the recharge water should be below 10 μg/L to avoid serious clogging of the well if no chlorine is added to the water. If a residual chlorine level is maintained prior to recharge, higher AOC levels probably are tolerable. Rather than AOC, biodegradable organic carbon or BDOC is often preferable as a biological clogging parameter, especially for higher organic carbon concentrations. BDOC is easier to determine than AOC, because BDOC is based on degradation of organic carbon by passing the water through laboratory soil columns or in batch tests with soil slurries.

The PFI is determined by passing the recharge water through columns filled with the appropriate aquifer material. The flow rates per unit area through the columns are then maintained at much higher values than the infiltration rates per unit area of the aquifer around the well. Thus, clogging occurs faster in the columns than in the well, and the PFI serves as an early warning of clogging to come for the recharge well so that preventive or remedial action can be taken early.

Experience has shown that MFI, AOC, and PFI are useful parameters for comparing relative clogging potentials of various waters, but that they cannot be used to predict clogging and

37

declines in injection rates for actual recharge wells, which also depend on well construction and aquifer characteristics. Thus full-scale studies on recharge test wells are still necessary to determine feasibility and design and management criteria for operational recharge wells. Practical aspects such as a varying flow in the water-supply pipes to the recharge project and associated possibility of fluctuating suspended-solids contents in the water also play a major role in well clogging. The suspended-solids fluctuations can be caused by formation of biofilms in the pipelines during periods of low flow, and by erosion of the biofilms during high flow. Treatment of the water at the recharge site to remove suspended solids prior to well injection may then be necessary.

Increasing injection pressures to overcome clogging effects generally is not successful and often actually hastens the clogging process by compressing the clogging layer in the same way as discussed in the section "Effect of water depth on infiltration." Even if the clogging layer is not compressed by the higher injection pressures and if injection rates are indeed increased, the higher infiltration rates in the well then increase production of pore-clogging biomass by higher loading rates of nutrients and organic carbon. They also increase physical clogging by higher loading rates of suspended solids. Increased injection rates by increasing injection pressures often are relatively short-lived solutions.

Aquifer Storage Recovery Wells

A new and rapidly-spreading practice in artificial recharge is the use of ASR wells (Pyne 1995, 2005), which are a combination of recharge and pumped wells. ASR wells are discussed in greater detail in Chapters 2 and 3. They are used for recharge when surplus water is available and for pumping when the water is needed.

ASR wells have certain unique features that differentiate them from production wells or injection wells. When completed, the wells may often be similar; however, the design process is different, and the end results may or may not be the same, depending upon conditions at each ASR site. The principal factors differentiating ASR wells from other types of wells are that ASR wells are designed for both recharge and production of water, and in many cases are designed for use in non-potable aquifers for water storage. The choice of materials for the well casing, screen and pump requires special attention for ASR wells due to the higher potential for corrosion and plugging resulting from these two factors.

ASR wells typically are used for seasonal storage of finished drinking water with a residual chlorine level in areas where water demands are much greater in summer than in winter, or vice versa, and where surface storage of water is not possible or too expensive. The winter surplus is then stored underground with ASR wells, which are pumped in summer (or vice versa) to augment the production from the water treatment plant. Typically the only treatment of the water pumped from the wells is chlorination. ASR wells make it possible to design and operate water treatment plants for close to average daily demand. The use of ASR wells to store seasonal surplus water and meet seasonal peak demands is less expensive than the use of treatment plants and surface reservoirs with capacities based on peak demands without ASR wells.

DESIGN AND MANAGEMENT OF INFILTRATION BASINS

The main objective of design and management of infiltration basins for recharge of groundwater is to minimize development of clogging on and in the soil. Such clogging reduces infiltration rates, sometimes very severely. Clogging layers develop as clay and other fine particles

38

suspended in the water settle out in the basin and accumulate on the bottom and banks to form a thin layer, often less than a few mm thick, of very low hydraulic conductivity. As the clogging layer develops, it restricts the infiltration more and more until eventually all the pressure head due to water depth in the basin is dissipated through the clogging layer. The flow in the underlying soil material then becomes so small that it becomes unsaturated and continues as such to move to the underlying groundwater. Since the entire head due to water depth is then dissipated across the clogging layer, the intergranular pressures in the clogging layer increase. This, in turn, compresses the clogging layer and makes it even less permeable. In addition to fine soil particles, clogging is aggravated by precipitation and settling of calcium and magnesium carbonates and phosphates, and by algal cells which also absorb CO_2 from the water. Such absorption causes the local pH to increase to the point where carbonates and phosphates can precipitate and further aggravate the clogging process. Microbiological activity also occurs in the clogging layer. This produces biofilms and insoluble polysaccharides which can further block soil pores. Also, bacteria can produce gases like methane and free nitrogen, which block pores and can accumulate below the clogging layer to form a vapor barrier to infiltration. The clogging process cannot be eliminated but it can be reduced, mainly by pre-sedimentation in deeper desilting ponds where flocculants are added to the water and the clear water is skimmed off to go to the infiltration basins.

Infiltration basins should be constructed in such a way that there is no soil erosion from the banks or around the inlet(s) because that would muddy the water and contribute to soil clogging. Thus, banks must be vegetated or lined with concrete or plastic or otherwise stabilized, and the water should enter the basin over a concrete slab with rocks or coarse gravel around it for energy dissipation. Other than that, basins can be triangular, square, rectangular, round, oval, or in a free form shape. The latter may be used where the project should be attractive with landscaping and vegetation for bird life and public enjoyment.

Maximum hydraulic loading probably is achieved with shallow basins with a water depth of less than about 1 ft. This would allow rapid draining and drying of the basin after the inflow is stopped for infiltration recovery. Also, infiltration rates in deep basins may be less than in shallow basins because the larger water depths compress the clogging layer on the basin bottom which increases its hydraulic impedance, and, hence, decreases the infiltration rates. Shallow basins with small water depths (less than 1 ft for example) also have higher turnover rates for the water, which reduces the time that suspended algae are exposed to sunlight and, hence, decreases the growth of algae in the water that can contribute to the soil clogging process. Thus, contrary to what intuitively would be expected, shallow basins tend to give higher infiltration and hydraulic loading rates than deep basins. Thus, community friendly infiltration systems like free form lakes come at a price of reduced hydraulic capacity and, hence, greater land requirements. A good solution may be to have a combination system that has both: engineered basins for maximum infiltration and a park with a few lakes for community benefits and some infiltration.

Sometimes the upper soil layers are less permeable than the layers further down. It may then be advantageous to remove the less permeable surface material so that the basin bottom is in the coarser material and will have higher infiltration rates. The maximum economic depth of the excavation depends on how much it costs and on the value of the expected increase in hydraulic loading rate. In Arizona, a maximum limit of 5 ft is sometimes used. Where the land is fairly smooth and flat, basins can be constructed by excavation and use of the soil materials for construction of berms. Where the land is flat and sloping, berms could be placed on the contours to back the water up, as in rice paddies.

Flooding and drying periods for maximum hydraulic loading are determined by experience. If the basin is flooded too long, infiltration rates toward the latter part of the flooding period can become quite small. It then takes too long for the water to disappear into the ground after the inflow is stopped for a drying period, unless, of course, the basin can be drained into another pond that already has had a long enough drying period for infiltration rate recovery and needs to be filled again. If this is not possible, it may take such a long time for the water in a deep basin to completely disappear for a drying period that the water must be pumped out. For these reasons, deep basins like old gravel pits may not be suitable for groundwater recharge. However, they are excellent for storage of excess water or as a pre-sedimentation facility for reducing the suspended solids content of the raw water, which may be necessary when captured during flood flows when the water can be quite muddy.

The best way to achieve optimum lengths of flooding and drying periods for maximum hydraulic loading is through experienced operators who know their systems and how each basin performs. Because of inherent soil heterogeneities, basins in a given system differ in their performance and may require different schedules of operation, i.e. flooding and drying periods and need for cleaning to remove clogging layers and subsequent disking. For this reason, each infiltration basin should be hydraulically independent with its own water supply and drainage facility so that each basin can be operated with its own best schedule of flooding, drying, and cleaning. Experienced operators tell us that each basin has its own personality and should be treated accordingly.

After a drying period and removal of dry clogging material by scraping or shaving the bottom with scrapers or graders, or by raking, recharge basins are usually disked to break up any crusts or compacted layers that may still exist. After disking, the soil surface may have to be smoothed and slightly compacted by, for example, dragging a telephone pole over it. This avoids sloughing of the soil ridges formed by the disking and resulting muddying of the water as it advances over the soil. The suspended fine particles could then enter the loose soil and accumulate on the undisturbed, denser soil below the depth of disking to form a mini-clogging layer that restricts infiltration rates. More research about this is needed.

Drying periods should be long enough for the soil surface to look dry for at least a week. If there was a clogging layer on the soil, this layer should then be well cracked and curled up like potato chips. These chips should never be disked or otherwise worked into the soil where they can cause deeper clogging. Rather, they should be removed with graders, scrapers, front end loaders, rakes, brooms, etc. Removal may not be necessary after every drying period but if infiltration rates begin to recover less and less during drying, it is time for removing the chips or flakes. If infiltration rates after a drying and cleaning period are still below normal, ripping the soil to a depth of about 0.5 to 1 m with construction type rippers may be necessary. However, frequent ripping is not suitable for stony soils because it will move the stones up in the soil profile and eventually to the surface, which then makes this surface difficult to clean.

Vegetation in the basins should not be allowed to develop, because it contributes to vector problems and interferes with cleaning and disking of the basins. Vegetation can be controlled mechanically, chemically, biologically and by periodically increasing the water depth in the basin to drown emerging weeds.

Because the performance of infiltration basins is so site specific and hard to predict for a full-scale project, large projects should never be designed and constructed all at once. The golden rule in recharge system design is to start small, to learn as you go, and to expand as needed.

RECHARGE EFFICIENCY

Evaporation is the main water loss in artificial recharge with surface infiltration systems. For year around operation of basins, water losses are due to direct evaporation from the water surface during flooding and evaporation from the soil during drying. Since the soil during most of the drying period is still moist, the evaporation from the soil will be about the same as that from a free water surface, which is about 5 to 8 ft per year in the warm and dry southwestern U.S.A. and about 1 to 2 ft per year in cooler climates. Since hydraulic loading rates for continuously operated recharge systems are on the order of 100–600 ft/yr, the evaporation losses are quite small and water recovery efficiencies are 90% or more.

Another non-recoverable loss of water is the volume needed to create a wetted zone in the vadose zone below the infiltration system so that the infiltrated water can move to the underlying aquifer. This is a one-time "investment" of water in the system that can be spread out over the useful life of the project and, hence, is negligible. It may be significant, however, for projects in warm dry climates with deep water tables and only occasional infiltration of small amounts of water, like flow in normally dry desert washes.

Another water "loss" occurs if groundwater after recharge moves away into areas over which the recharging agency has no control. On a regional basis, however, such movement of groundwater does not constitute a loss. Thus, putting water underground via artificial recharge is a very effective way for storing water in times when supplies exceed demands for use in times when supplies are less than demands.

PROBLEMS AND TROUBLESHOOTING

The most common problem in artificial recharge is inadequate hydraulic capacity, due to under design and/or poor management of the basins or other infiltration facilities. This is especially critical where sewage effluent is used for recharge and the basins must infiltrate the total effluent flow to achieve zero discharge. To avoid such surface discharge of sewage effluent, the total basin surface area should be selected on the conservative side so as to have plenty of capacity to handle unusual events like more effluent flow than expected, or extended cool, rainy periods when the basins dry slowly and complete infiltration recovery is not possible. What happens then is that when the basins must be filled again, the infiltration rates are still low and decline even further, which in turn reduces infiltration even more, etc. The excess effluent flow eventually must then be bypassed and directly discharged into a river or other surface fixture. Situations like this can be avoided by making the total infiltration area large enough to have some spare capacity for infiltration during periods of slow drying and poor infiltration recovery. Having a little surplus basin area is much better than having not enough when infiltration rates no longer can absorb the entire sewage flow, and the basins essentially become polishing ponds for temporary storage where the effluent just sits with very little infiltration into the ground. To avoid these situations, basins or other surface infiltration systems must be designed with plenty of extra basin area to handle the flow during times of poor infiltration recovery in the basins. This is another reason for starting small, learning as you go, and expanding when needed.

Uncontrolled growth of vegetation in the basins also reduces infiltration because the plants clog the soil profile with their thatch and root systems, and they make drying and cleaning of the basin bottom soils very difficult. They also aggravate vector problems.

41

SUSTAINABILITY

While natural recharge of groundwater with atmospheric precipitation unquestionably is sustainable, this may not be true for artificial recharge where infiltration rates are orders of magnitude higher and water qualities a lot lower, especially where sewage effluent is used. More needs to be known about movement of colloidal or other fine particles in the underground environment. Do they move through the soils or can they slowly accumulate in soil pores, especially at soil discontinuities, and decrease permeability? In some old systems, infiltration rates have shown some unexplained gradual declines. Where sewage effluent is used for SAT, some chemicals may precipitate or otherwise accumulate in the soil and eventually reduce its permeability. Biodegradation of readily degradable organic compounds in concentrations above degradation threshold values should be sustainable. Some of the quality improvements in SAT may also occur by sorption to soil particles, which is not sustainable. On the other hand, sorption to soil of organic compounds at concentrations below threshold values in the water could enhance their biodegradation because organic carbon concentrations in the adsorption layer are higher than in the water itself, thus maintaining SAT benefits. At the present state of knowledge, it can only be postulated what might happen. Considerable field and laboratory work is needed to monitor what actually happens over long time scales. Such research is necessary to ascertain long term and sustainability aspects of artificial recharge and soil-aquifer treatment. All that can be said at this time is that artificial recharge and SAT systems can be expected to have a very long useful life.

ADDITIONAL SUGGESTIONS BASED ON EXPERIENCE AT OPERATING SURFACE RECHARGE SYSTEMS

For design of surface recharge basins it is beneficial to link a series of basins in a chain, if this is possible. Progressively cleaner water would then flow toward the final basin. This approach tends to result in higher overall system recharge performance. Diversion channels or pipes are needed so that each basin can be easily removed from the chain of basins for maintenance purposes, without disturbing operation of the remaining basins in the chain. Flow should be maintained through each of the basins so that no dead-end sinks occur in the system. This may require provision of an outflow from the final basin in the chain, returning the water to a discharge sink, an open stretch of dry creek bed, or a return to the river from which the water was originally diverted. Consideration should also be given to providing two parallel diversion structures from the river source, allowing diversion structure maintenance and repairs without having to take the entire recharge system offline.

Maintaining increased infiltration rates in basins has been achieved at the Orange County Water District, California, using automated basin cleaning vehicles. These move around the basin floor, separating the sand from the silt and other particulates, and pump the debris out of the basin, replacing the sand.

Another approach to increasing infiltration rates should also be considered, by which subsurface collector pipes, or bank filtration wells, are provided close to the source water supply. These remove the silt from the water more effectively than through using settling basins. The higher quality water can then be piped to off-channel basins for recharge, thereby reducing the clogging rate and the cleaning frequency of the basins.

The surface recharge system should be designed with sufficient control to allow emergency closure of the river intakes at such times as the source water quality is unacceptable for

42

recharge. This could include use of an inline water quality probe, including turbidity as one of the measured constituents, so that diversion structures can be automatically closed. Alternatively the site may have 24-hour operator observation and control, or perhaps a notification system so that operating staff can be called to the site to close the diversion structure when a storm is expected. During storm events the sediment load in the water may not settle in the desilting system that is designed for use with normal diversion flows. This would result in clogging of the recharge basins. Highly turbid recharge water quickly reduces infiltration rates.

Over a period of several decades of operation basin elevations may decline as a result of periodic maintenance to remove clogging layers over the sand. It is important to offset this tendency by either bringing in new sand to the site or by washing the removed material to separate out the silt and other debris, returning the sand to the basin.

For prevention of insects, deeper water that is moving has lower potential for larvae development. Shallow water with slow movement increases the potential for development of flies and mosquitoes.

During conceptual design and layout of surface recharge facilities, careful consideration should be given to the aquifer hydraulic response. It may be beneficial and cost-effective to convey a portion of the recharge water a considerable distance to separate sites so that recharge is distributed more evenly across a hydrologic basin, and closer to where the major pumping centers are concentrated.

Recharge basins may be operated to achieve a constant rate of flow into each basin, allowing water levels in the basin to vary up or down, or they may be operated to maintain a constant head, with a varying flow rate. The latter approach is more easy to monitor, operate and maintain.

Allowing natural vegetation to grow on the basin floors engenders mixed opinions regarding whether this aids or hinders recharge performance. In southwestern states this is believed to hinder performance, and probably slightly increases evapotranspiration losses. At the East Meadow surface recharge site in Nassau County, Long Island, New York, this is believed to have improved performance by creating shallow root channels in the basin floors.

Sometimes there exists a need to utilize deep recharge basins for multiple purposes, including public recreation use. This is possible, however it has drawbacks. The basin water level may need to be maintained for boating. Fish detritus and other bacteriological organisms supporting a healthy fish environment increase the basin clogging rate. Draining the basin for maintenance purposes to sustain recharge requires costly fish kills, removal and restocking.

Flood control basins may be conjunctively utilized as infiltration basins. This may be possible for much of the year, providing aquifer recharge along the stream channel. However during the storm season, the recharge ponds would be drained to allow increased capture of storm runoff.

Where deep recharge basins have relatively steep walls, such as may occur with use of old gravel and sand quarries for recharge, consideration should be given to regrading the walls so that they have a more gentle slope. This will increase the surface area available to support infiltration and therefore should increase subsequent percolation rates. It should also facilitate maintenance operations.

43

Figure 1.24 Recreational lake in Santee, California, east of San Diego using sewage effluent after it has had soil aquifer treatment by subsurface flow in a streambed

ROLE OF RECHARGE IN WATER REUSE

Planned water reuse is expected to become increasingly important, not only in water-short areas where sewage effluent is an important water resource, but also where streams or other surface waters (including seawater at popular beaches) need to be protected (Bouwer 1993; 2000a). Sewage treatment for planned water reuse is often cheaper than the treatment for discharge into surface water that is necessary to protect in-stream and downstream users of that water against unacceptable pollution. Planned water reuse requires treatment of the effluent so that it meets the quality requirements for the intended reuse. Due to treatment costs, economic feasibility, and aesthetics, the treated sewage effluent most commonly is used for nonpotable purposes such as agricultural and urban irrigation (golf courses, sports fields, recreational and decorative lakes, Figure 1.24), power-plant cooling, industrial processing, construction, dust control, fire protection, toilet flushing (mostly in commercial buildings but also more and more in private homes), and environmental purposes (wetlands, riparian habitats, perennial streams, wild-life refuges). Unplanned or incidental use of sewage effluent for drinking or public water supplies goes on all over the world as municipalities share the same river for drinking water and sewage disposal (Crook, MacDonald, and Trussell 1999).

Planned reuse for potable purposes is still rare but is expected to increase in the future (McEwen and Richardson 1996; National Research Council 1994; Crook, MacDonald, and Trussell 1999). Water reuse and recycling also will probably be an important aspect of demand management in integrated water management (Bouwer 2000a). Inclusion of a groundwater recharge and recovery cycle in the reuse process has several advantages, such as storage to absorb seasonal or longer-term differences between supply of effluent and demand for reclaimed water, quality improvement of the effluent water as it moves through soils and aquifers (soil-aquifer treatment or geopurification), favorable economics, aesthetic benefits, and better public acceptance of water reuse. The latter is especially important for potable reuse, where the recharge cycle breaks up the undesirable pipe-to-pipe connection that has been the bane of several proposed potable-water reuse schemes (Crook, MacDonald, and Trussell 1999). Recharge and soil-aquifer

44

treatment also make water reuse more acceptable in countries where a religious taboo exists against the use of "unclean" water (Ishaq and Khan 1997; Warner 2000).

If the recharge is via basins or other surface infiltration facility, the sewage effluent typically is first given primary and secondary treatment, and disinfection with chlorine (National Research Council 1994). Primary effluent can also be used (Lance, Rice, and Gilbert 1980; Carlson et al. 1982; Rice and Bouwer 1984), and some projects use tertiary effluent, where the sewage after secondary treatment is filtered through sand or other granular medium and then chlorinated or otherwise disinfected. Primary treatment is a mechanical process that removes everything that floats and sinks. Secondary treatment is a biological process where bacteria degrade organic compounds in aerated tanks (activated sludge process) or trickling filters. Tertiary treatment consists of sand filtration and disinfection; and advanced treatment refers to all other treatment steps such as lime precipitation, nitrification-denitrification, activated carbon filtration, and membrane filtration, such as reverse osmosis.

Often, water-quality improvement is the main objective of recharge with sewage effluent. For this reason, the systems are usually no longer called recharge systems, but soil-aquifer-treatment (SAT) systems or geopurification systems (Bouwer and Rice 1984a). SAT typically removes essentially all the suspended solids and micro-organisms (viruses, bacteria, protozoa like giardia and cryptosporidium, and helminth eggs). Nitrogen concentrations are greatly reduced by denitrification and possibly also by the recently-discovered process of anaerobic oxidation of ammonia (anammox; Van de Graaf et al. 1995; Gable 2003; Kuenen and Jetten 2001). Dissolved organic carbon also is greatly reduced, typically from a range of 10–20 mgL to 2–5 mgL. Most phosphates and metals are also removed from the water, especially in calcareous soils, but the they accumulate in the underground environment (Bouwer and Rice 1984a).

To stimulate denitrification in the soil system, the infiltration basins should be operated to bring nitrate and organic carbon together under anaerobic conditions. For secondary effluent where usually nitrogen is mostly in the ammonium form, frequent short flooding periods (2 days flooding and 5 days drying for example), can be expected to give essentially complete nitrification of ammonium to nitrate and leave not enough organic carbon for subsequent denitrification in the aquifer. In that case, almost all the nitrogen in the effluent water will show up as nitrate as it joins the aquifer. If long flooding periods are used, the N in the effluent remains mostly in the ammonium form as it reaches the aquifer where it may be partially removed by anaerobic oxidation via the anammox process (Van de Graaf et al. 1995; Gable 2003). However, if intermediate flooding and drying periods are used like, for example, 9 days flooding and 12 days drying (values selected to operate on three-week cycles), there is both nitrification and denitrification resulting in nitrogen removal of about 70% (Bouwer et al. 1980). The best way to find the optimum combination of flooding and drying periods for maximum nitrogen removal in a given recharge system is by experimenting with various combinations of these periods. If the ammonium in the infiltrating water is observed to be mostly converted to nitrate in the aquifer, the flooding periods were too short and the drying periods too long. If, on the other hand, the ammonium in the infiltrating water stays about the same when it joins the aquifer, the flooding periods were too long. The challenge is to find the optimum combinations from maximum nitrogen removal and hydraulic loading.

Recovery wells for pumping water after SAT from the aquifer can be located so that they pump 100% reclaimed water (Figure 1.25) and prevent the spread of reclaimed water into the natural groundwater outside the portion of the aquifer dedicated to SAT. Alternatively, the wells can be located to pump a mixture of reclaimed water and natural groundwater.
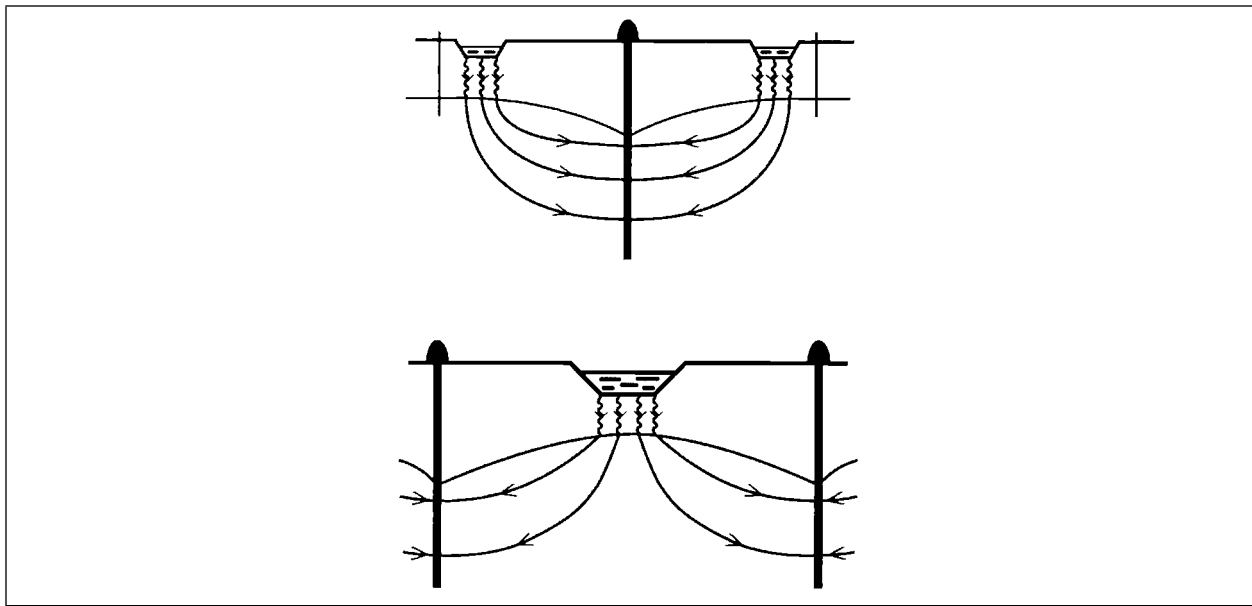
45

Figure 1.25 Section showing recharge and recovery SAT systems with infiltration areas (indicated by vertical arrows) in two parallel rows and line of wells midway in between (top), and in center area surrounded by a circle of wells or in a long strip with wells on both sides (bottom) of the infiltration system

Water from wells such as shown in Figure 1.25 is essentially pathogen free and, hence, can be used for essentially all non-potable purposes, such as irrigation of lettuce and other crops consumed raw, parks, playgrounds, golf courses, fire protection, toilet flushing, etc.

The main concern when this water is used for drinking is the presence of residual organic carbon, which consists of a broad spectrum of mostly synthetic organic chemicals in very small concentrations (E.J. Bouwer et al. 1984), some of which are carcinogenic or may have other adverse health effects. To protect the public health, California has set an upper limit for the total organic carbon (TOC) content of the water after SAT that is due to the sewage effluent. The upper limit for TOC is the recycled water percentage divided by 0.5, expresses in mg/L. This distinction is made because some natural groundwaters have natural TOC contents of more then 1 mg/L, due to humic and fulvic acids or other "natural" organic compounds. To keep the sewage-derived TOC in the well water from systems like those shown in Figure 1.25 to less than 1 mg/L, the effluent can be treated with reverse osmosis or carbon filtration in the sewage treatment plant before groundwater recharge. Another solution is to use systems as in Figure 1.25 (bottom) with enough blending with natural groundwater that enters the wells from the opposite side of the infiltration area or from greater depth to reduce the sewage TOC in the well water by dilution to less than the limit. To achieve this, California has developed guidelines for potable use of water from wells in aquifers that are recharged with sewage effluent, as shown in Table 1.2. These guidelines and the percentages of reclaimed water in the well water are based on TOC removal in the SAT system in order to keep the well-water TOC of sewage origin below the limit. Results from two major health-effects studies on morbidity and mortality in populations receiving water from systems as in Figure 1.25 in their public water supply indicated no adverse health effects (Nellor, Baird, and Smith 1984; Sloss et al. 1996).

46

Table 1.2
Proposed California guidelines for potable use of groundwater

	Type of Recharge	
Contaminant Type	Surface Spreading	Subsurface Injection
Pathogenic Microorganisms Treatments	Turbidity < 2 NTU	Turbidity after membranes < 0.5 NTU
	Chlorine CT > 450 mg-minutes/L	Same
	Virus removal > 5 logs	Same
	Total coliform < 2.2 MPN/100 mL	Same
Retention Times	6 months	12 months
Horizontal setbacks	500 feet	2000 feet
Control of Nitrogen Compounds	<5 mg/L prior to spreading or	<5 mg/L prior to injection
	< 10 mg/L based on 10 years' experience	Same
Control of Regulated Compounds	Meet drinking water primary and secondary MCLs	Same
Control of Nonregulated Compounds	TOC prior to spreading <16 mg/L	TOC prior to reverse osmosis treatment <16 mg/L
	TOC prior to vadose zone (in ≤ 0.5mg/L/RWC, where RWC = Recycled Water Contribution (percentage of recycled water spread)	TOC prior to injection ≤ 0.5 mg/L/RWC
	Extensive monitoring— unregulated chemicals, selected notification levels, pharmaceuticals, endocrine disrupting chemicals and other chemicals	Same
Recycled Water Contribution	Maximum is 50%, unless other criteria are satisfied	Same

Source: California Department of Health Services 2004.

An emerging concern is the possibility that the sewage-effluent TOC also includes pharmaceuticals and hormones or hormonally active compounds (endocrine disrupters) whose underground fate and health significance presently are poorly understood (Daughton and Jones-Lepp 2001). Another form of groundwater recharge with sewage effluent is the incidental recharge obtained where sewage effluent is used for irrigation. Because the treatment requirements for irrigation reuse are not as strict as for potable reuse, irrigation is likely to become an increasingly significant user of sewage effluent, especially in dry climates where irrigation is essential for agricultural production and urban green areas (landscaping, recreational and athletic areas, private

47

yards, etc.). For sustainable irrigation, the salts and other chemicals in the irrigation water must not be allowed to accumulate in the root zone of the crops or plants, but must be leached out of the root zone with natural rainfall as, for example, with the winter rains in Mediterranean-type climates, or with extra irrigation water where natural rain is insufficient. The amount of extra irrigation water needed for leaching is controlled by the salts in the irrigation water and the salt tolerance of the plants (Tanji 1990). Typically, the leaching requirement is about 10% of the irrigation amount needed for crop consumptive use (evaporation from the soil plus transpiration from the plant, or evapotranspiration, ET). This requirement corresponds to an irrigation efficiency of about 90%, meaning that of the water applied, 90% is used for ET and 10% for leaching salts and other chemicals out of the root zone. Such high irrigation efficiencies can be achieved with sprinkler or drip systems. Most agricultural irrigation systems use flooding methods like borders or furrows, which have lower irrigation efficiencies, often about 50–80%. This method leaves plenty of water for leaching and maintaining salt and chemical balances in the root zone. Irrigation efficiencies of 100% theoretically would only be sustainable without rainfall if distilled water were used for irrigation.

At an irrigation efficiency of 80%, chemicals brought in with the irrigation water are leached out with 20% of the irrigation water. Thus, concentrations of salts and other chemicals not taken up by the plants or biodegraded or immobilized in the soil profile are 5 times higher in the leachate than in the irrigation water. This leachate, also called drainage or deep percolation water, then moves down to the groundwater, where in the long term it is likely to cause serious quality degradation (Bouwer 2000b, 2004; Lemly 1993). For example, assume that a 6-month summer crop in a warm, dry climate needs 1 m water for ET. At an irrigation efficiency of 80%, the irrigation amount thus must be 1.25 m for the growing season, of which 0.25 m leaches through the root zone and moves to underlying groundwater. Assuming a water content of 15% in the vadose zone, the actual downward velocity of the water is about $0.25/0.15 = 1.7$ m per 6 months (assuming no movement during the rest of the year). Thus, if the groundwater is at a depth of 30 m, it would take the water about $30/0.85 = 35$ years to move to the groundwater. Assuming a fillable porosity of 10% and vertical stacking of the deep percolation water above the groundwater, this deep percolation water adds a layer of 2.5 m per year with low quality water to the aquifer. If the irrigation water has a salt content of 500 mg/L, the deep percolation water would have a salt content of 2,500 mg/L. Significant rainfall will, of course, reduce this figure, which assumes no deep percolation from rainfall. If sewage effluent is used for irrigation, other chemicals in the leachate may include disinfection byproducts; natural and synthetic organic compounds like pharmaceuticals, hormones, and others (Lim et al. 2000), nitrate; and humic and fulvic acids that were already in the sewage effluent, plus those that were formed by decaying plant materials.

These humic and fulvic acids could then form disinfection byproducts when the groundwater is pumped up again and chlorinated for drinking. Eventually, membrane filtration like reverse osmosis (RO) may have to be used to lower the salt concentrations in the upper groundwater to drinking-water levels. Membrane filtration would also remove other contaminants like nitrate, pharmaceuticals and other synthetic organic compounds. Some chemicals, however, may not be entirely removed by RO.

Because of this incidental recharge, irrigation with sewage effluent thus may cause worse contamination of aquifers in the long run than artificial recharge with sewage effluent. In the latter case, hydraulic loading rates are much higher than evaporation rates and, hence, essentially no increases in chemical concentrations occur in the water moving down to the groundwater. Where land above potable aquifers is irrigated with sewage effluent, water and salt balances should be

48

evaluated to predict possible long-term groundwater impacts. Groundwater monitoring will then be necessary to see what actual effects such irrigation will have on underlying groundwater to determine what should be done to avoid or minimize adverse effects.

SUMMARY—SURFACE RECHARGE

Storing more water underground will become increasingly necessary as populations and water demands increase, climates change, and surface storage may not be an option because of lack of good dam sites, adverse environmental effects, evaporation losses, and vulnerability to sabotage or destruction. Almost all of the world's liquid fresh water already occurs as groundwater formed by atmospheric precipitation. The natural recharge then is the difference between precipitation and the sum of evapotranspiration and surface runoff. Where groundwater is withdrawn faster than replenished by nature, groundwater levels will drop and eventually wells will go dry. The solution then is to pump less groundwater and/or add more water to the aquifer via artificial recharge to attain sustainable groundwater levels through optimum conjunctive use of surface water and groundwater. Artificial recharge is especially useful for long term storage or "banking" of water because evaporation losses are essentially zero if the groundwater table is well below the vegetation roots. Artificial or "managed" recharge is achieved by spreading surface water on the land, where it infiltrates into the soil and moves down to join the aquifer. This will require permeable soils and vadose zones, and aquifers that are unconfined. More infiltration can also be achieved in stream channels by placing low dams in the channel to back the water up and spread it out, or with levees in the channel to spread the flow over the entire width of the channel. Off-channel systems consist of basins created by berms or by excavation. Where permeable soil strata are too deep for surface basins, trenches or dry wells can be dug to expose deeper and more permeable soil materials. Surface water also can be placed directly into aquifers through aquifer wells, but usually this requires pre-treatment of the water to essentially drinking water standards so as to avoid degradation of the natural groundwater and to minimize clogging of the well wall. Even then, regular pumping of the well will still be necessary to control such clogging.

Almost always a clogging layer with very low permeability will develop on the infiltrating surface of a recharge system. Such a clogging layer consists of sediment and other particles in the recharge water that accumulate on the soil surface. Biological activity and chemical precipitation in the clogging layer make it even less permeable and severely reduce infiltration rates. Thus, recharge basins or other surface recharge systems must be regularly dried to restore infiltration capacity. Drying clogged infiltration surfaces alone may not always be sufficient, especially if the clogging layers are relatively thick. Although they tend to shrink, crack, and curl up, they flatten out when flooded again and still reduce infiltration. Thus, after several flooding and drying cycles the basins should be cleaned again by removing the clogging layer at the end of a drying period with scraping or raking techniques followed by a light disking to break up any surface crusts that may have developed on the soil. Disking the entire clogging layer into the soil is not recommended since this would eventually create a thicker clogging layer that is harder to remove.

The optimum combination of flooding and drying cycles and cleaning for maximum long term infiltration depends on local conditions of soil, water, and climate and must be evaluated by experience. Typical flooding and drying periods for recharge with secondary effluent in south central Arizona are on the order of 2 weeks flooding and 10 to 20 days drying. In cooler climates with better quality water, flooding periods may be on the order of a month or year or more, again depending on how infiltration rates hold up. Where sewage effluent is used, flooding and drying

49

periods may have to be scheduled to create optimum conditions for maximum nitrogen removal by denitrification or anaerobic oxidation of ammonium (anammox process) in the soil or aquifer.

In addition to permeable surface soils, artificial recharge systems require vadose zones (unsaturated zones between land surface and groundwater table) that are sufficiently permeable to transmit the infiltrated water down to the aquifer. Clay layers or other layers of low hydraulic conductivity in the vadose zone can restrict the downward movement of water and form a perched groundwater mound in the vadose zone that should not rise so high that it backs up the infiltration flow from the basin. Thus, the vadose zone should be examined for restricting layers and also for landfills or other pollution sites that could adversely affect the quality of the downward moving water. The receiving aquifer should be checked to make sure that it is sufficiently transmissive so that the recharge water can move away laterally without building up a groundwater mound that is so high that it "drowns" the infiltration system. Also the rise in groundwater levels away from the recharge system should not be so high that it floods basements, damages underground pipe lines, threatens cemeteries, deep rooted plants like old trees, and other third party interests. Simple equations have been developed for calculating the rise of groundwater mounds above restricting layers in the vadose zone and on the aquifer below and away from the infiltration system. The basins may have to be arranged to form a long narrow strip rather than be concentrated in a more square or round area. Spacing the basins further apart also reduces mounding heights.

In addition to the technical and hydrologic aspects of design and operation of artificial recharge systems, there are also public, environmental, community, legal, regulatory and economic aspects to be considered. Public acceptance and participation are very important, especially where reclaimed water is used. Weeds and mosquitoes need to be controlled. In-channel recharge systems may interfere with other uses of the stream. Keeping the public informed and considering public concerns and wishes are important aspects, especially where the recharge is done in populated areas. For a more complete description of all these aspects, reference is made to *Standard Guidelines for Artificial Recharge of Ground Water* (ASCE 2001).

The underground environment in which artificial recharge takes place is not easily characterized with great accuracy and bio-chemical processes in the vadose zone and aquifer are difficult to predict. Thus, it makes good sense to start with a small or pilot project before a big project is designed and constructed. The golden rule in artificial recharge is to start small, learn as you go, and expand as needed to avoid surprises later on.

CHAPTER 2
DESIGN, OPERATION, AND MAINTENANCE OF
RECHARGE WELL SYSTEMS

INTRODUCTION

Aquifer recharge to achieve sustainable underground storage is most cost-effectively implemented through surface infiltration systems, however in many areas of the world the conditions are not right for this recharge method. Hydrogeology may be inappropriate, such as through the presence of thick clay layers just below the land surface, high water tables that would cause recharge water levels to rise above land surface, or insufficient land availability at reasonable cost. For these areas well recharge is usually a viable method to achieve the same goals.

Well recharge was initially conducted using injection wells extending below the static water level for the aquifer being recharged. As discussed in Chapter 1, injection wells tend to plug due to physical, geochemical and microbial processes occurring in the aquifer close to the borehole or screen. Periodic redevelopment of these injection wells by backpumping was found to be necessary in order to maintain their recharge capacity. Subsequently aquifer storage recovery (ASR) well technology was developed, using the same well for both recharge and recovery. The pump used for recovery is also used periodically to backflush the well for a few minutes or hours to maintain its recharge capacity and thereby overcome the fundamental problems associated with injection well plugging. Due to the demonstrated advantages of the newer approach, ASR technology has generally replaced injection well technology for achieving the goals of aquifer recharge in new locations. For this report, use of ASR technology is assumed. The same guidance generally applies for injection wells and other well recharge approaches. In areas where existing injection well diameters are too small to accommodate pumps capable of effective backflushing, air lifting of the wells has been used to achieve the same objectives.

Other recharge well methods have also been developed. As discussed in Chapter 1, vadose zone wells have been operating for many years in Scottsdale, Arizona, recharging very highly treated wastewater into shallow augured recharge shafts, thereby avoiding the much higher cost of drilling deep recharge wells to the local fresh water aquifer, for which static water levels are at depths of several hundred feet. Advances in vadose zone well technology have been implemented to steadily improve the performance of these wells and increase their service life since it is not possible to redevelop them when they plug.

In Australia research and demonstration projects are underway to investigate the viability of ASTR technology (aquifer storage transport recovery) to recharge stormwater that has received wetland pretreatment. Water is recharged into an ASR well, conveyed underground a short distance to a second recovery well from which it is pumped for irrigation and other purposes. Recovered water quality is being monitored to determine the potential future viability of this method for producing water suitable for potable purposes, and the associated design, operation and maintenance criteria that would apply. This is similar to the "dual infiltration well" technology that has been used in the Netherlands for several decades to provide disinfection of potable water by soil aquifer treatment in local sand aquifers, conveying water from an injection well to a recovery well at a distance of about 100 m.

Common to each of these well recharge methods are certain technical issues that apply to the design, operation and maintenance of the facilities if they are to achieve sustainable underground

51

storage goals. These are addressed in the remainder of this chapter, based upon experience gained at more than 70 ASR wellfields in the United States as of 2005, plus many injection wells and a few vadose zone wells, supplemented by valuable overseas experience. Rather than providing a complete treatise on the design, operation and maintenance of wells and wellfields for all purposes, guidance is offered regarding those specific features that are considered unique for successful and sustainable underground storage and recovery of water through wells.

A three-phased general approach is recommended for development of well recharge facilities:

- Phase I   Preliminary Feasibility Assessment
- Phase II   Design, Construction and Testing of Full Size ASR Well
- Phase III   Expansion to Meet System Needs

The first phase is of vital importance, providing a firm foundation for technical development of the project and also for regulatory approval. Typically the first phase identifies and prioritizes project objectives; evaluates trends and variability in water supply, water demand and water quality; estimates storage volume requirements; considers many hydrogeologic and groundwater quality issues, including a preliminary geochemical assessment and a local well inventory; presents a conceptual ASR facilities location, development plan and preliminary cost estimate; and addresses legal, regulatory and institutional issues. The scope for the first phase is tailored to meet local requirements and, in some cases where adequate data is available, may include groundwater modeling. Upon completion, a report is prepared that carefully documents the feasibility assessment, providing a well-thought-out plan for ASR development and also providing a support document for approval of needed permits.

Where ASR risk is deemed low, such as in areas with already operating ASR wellfields nearby, the scope may be scaled back or perhaps included as an initial task in Phase II. Sometimes it may be completed as a one- or two-day workshop that is documented in a technical memorandum before proceeding with final design of facilities. On the other hand, where no prior local ASR experience is available, or where risk is otherwise deemed to be greater, the program may be expanded into more than three phases, each of which balances further capital investment with increasing confidence in ultimate success.

Details regarding the first phase feasibility assessment are available (Pyne 2005), however they are beyond the scope of the current project, which addresses design, operation and maintenance considerations. The second phase is the subject of this report. Well recharge facilities need to be designed at full size since testing results are scale-dependent. Small diameter test wells tend to lead to inadequate ASR performance, negating the initial cost savings if the technology is then shown to provide little value. If the feasibility assessment is completed properly, the probability of constructing full size ASR facilities in Phase II that are subsequently deemed to be successful is greater than 95%, based on experience to date. Simulation modeling of aquifer hydraulics or water quality changes is best conducted as part of the second phase, following construction and testing of the wells.

52

DESIGN OF ASR WELLS

Well Casing

ASR wells generate rust from steel casings to a greater extent than either production or injection wells. This is due to the increased surface area subject to wetting and drying during recharge and recovery, the presence of dissolved oxygen in the recharge water at most sites, and also due to recharge of water that typically contains a small chlorine residual, not only during recharge periods but also during extended storage periods. The potential for generation of rust is even greater for brackish water storage zones. The rust flows down the well during recharge, contributing to plugging of the well.

Solids present in the recharge water are usually more significant causes of ASR well plugging than rust. However, for low permeability aquifers, the increase in plugging potential due to rust can be unacceptable in some cases, particularly where frequent backflushing to waste would be an operating problem to be avoided if possible. During recovery or backflushing redevelopment, the rust combines with other solids carried into the well during recharge and is pumped from the well, either to waste or into the wellhead piping system.

For ASR wellfields located at water treatment plants, it is not uncommon for the water pumped from the well at the beginning of recovery, or during periodic backflushing during recharge, to be conveyed back to the treatment process for retreatment. Duration of this backflushing may typically range from about 10 minutes to 2 or more hours. Once the rust, fine particles from the formation, byproducts of subsurface microbial activity and other particulates have been flushed from the well, the water then can be diverted directly into the treated water distribution system following disinfection, or recharge can commence again.

For ASR wells located other than at water treatment plants, the only option is to waste this water to a nearby drainage system, recharge pit or sewer line. The pumping rate at such times may be slightly greater than the design recovery rate for the well since the pump is usually pumping against a lower head than normal, therefore producing more water. This is good in that it helps to purge material from the well; however, disposal of water for an extended period at such rates is sometimes a problem. In residential areas lacking storm drainage networks, ASR backflushing operations can cause temporary, localized flooding of streets. Homeowner opposition can arise due to inconvenience and apparently wasted water. In other areas with adequate drainage networks, regulatory opposition may be encountered due to the ultimate discharge into a receiving stream or other water body for this water, which in some cases is initially colored and may contain considerable solid material. That this event may occur infrequently during initial testing, and then perhaps once or twice per year, is of little assistance. Failure to periodically backflush the well to waste leads to increased well plugging, possibly higher energy costs during recharge, and increased difficulty in eventually unplugging the well.

One solution to a rust problem is to utilize casing material that will not contribute to the production of rust. Use of such materials is also appropriate for situations where the storage zone geochemistry outside a steel casing may rapidly corrode the casing. Since ASR wells are often constructed in deep, brackish aquifers, this situation is not uncommon. Polyvinyl chloride (PVC) casing offers many advantages in situations where required casing depth, wall thickness and diameter are within the range of readily available materials. Where a steel casing is required, epoxy coating can substantially reduce or eliminate the surface area of steel that is subject to rusting. Stainless steel, either 304 or 316, or other corrosion resistant alloys may also be utilized.

53

All of these approaches have been used successfully in operational ASR systems. Recently fiberglass casings have also been utilized successfully for production wells in Florida for which the required casing diameter exceeded that available in PVC, so it is likely that fiberglass casings will soon be utilized more widely for ASR wells. The first fiberglass-cased ASR well is in operation at Bolivar, Adelaide, Australia. At the present time it is common in the United States to use conventional carbon steel casing and accept the rust production as a long-term operating issue to be addressed later, if necessary.

Further discussion of each of these casing material options follows later in this chapter. First, however, it is important to consider a basic approach for the selection and design of ASR well casing, including diameter, materials of construction, and strength (yield strength and collapse resistance).

Diameter

The well casing must have an inside diameter to accommodate the required pump, based upon the pumping rate and the revolutions per minute (rpm) of the pump. Table 2.1 lists nominal pump bowl diameters from 4" to 24", along with the minimum recommended casing inner diameter (ID) and maximum pump capacity for different speed pumps.

For ASR wells, inner casing diameter should be no different than for normal production wells except perhaps when one or more injection tubes are used for recharge and are installed inside the casing. It is advisable to have extra space inside the casing to allow easy entry and withdrawal of the pump, injection tubing, air line, and electrical cable for submersible pumps. Having sufficient casing diameter minimizes the potential for damage to the electrical cable during installation of submersible pumps. For wells anticipated to be equipped with deep set vertical line shaft turbine pumps, larger casing diameters are often provided to overcome lost effective well diameter from borehole bends or dog-legs. Alternatively special drilling precautions are followed to ensure that the hole is reasonably straight, such as use of long, fluted drill collars and use of high speed gyroscopic surveys to confirm pilot hole and borehole alignment.

Most ASR wells have at least a 12-inch inner casing diameter. Where there is any question regarding the ability of the installed casing to accept the pump and any associated downhole equipment without becoming stuck, it is prudent to lower a dummy or simulated pump assembly canister that is equal or slightly greater in dimensions than the downhole equipment (pump bowls, submersible cable, column flanges, control lines for any downhole control valve, water level transducer sounding tubes, etc.) to be installed. If the dummy can be placed into the casing and easily removed, then the pumping equipment can be installed with reasonable confidence. If this precaution is not taken, and if the casing is crooked or egg-shaped, or if the diameter of the downhole equipment is very close to the inside dimensions of the casing, then the problem may not become apparent until quite late in the wellhead construction process, at potentially substantial expense. Squeezing too much into an undersized casing can create problems.

Due to the increased cost associated with larger casing diameters, the downhole control valve discussed subsequently in this chapter has rapidly gained in acceptance since it provides needed control of injection flows without requiring larger casing diameters. This valve facilitates ASR in situations where existing wells with small diameter casings are retrofitted to ASR operations.

An option that may be appropriate for some ASR sites is to provide a casing that is larger diameter from the ground surface to below the anticipated pump setting. A smaller diameter casing extends from below the pump setting to the screen or open hole at the bottom of the well.

54

Table 2.1
Casing inner diameter and pump capacity

Nominal Pump Bowl Diameter (in)	Minimum Recommended Casing ID (in)	Maximum Pump Capacity (gpm)	Pump (rpm)
4	5 to 6	100	3,600
5	6 to 7	250	3,600
6	7 to 8	400	3,600
7¼	8 to 9	700	3,600
8	9 to 10	1,000	3,600
		600	1,800
10	11 to 12	2,500	3,600
		1,500	1,800
		700	1,200
11	12 to 13	4,000	3,600
12	13 to 15	2,500	1,800
		1,500	1,200
14	15 to 17	3,500	1,800
		2,500	1,200
16	18 to 19	5,000	1,800
		3,500	1,200
20	22 to 23	7,000	1,200
		5,000	900
24			

Courtesy of Hank Baski.

The pump is then set in the larger, upper casing section. This design is particularly appropriate for deep, productive aquifers, reducing well construction costs while maintaining high well yields.

Steel and stainless steel casings are manufactured to outside diameter dimensions. For instance, a 10" casing means that it has a nominal or approximate inside diameter of 10" but has a fixed outside diameter of 10¾". The inside dimension is smaller if there is a heavier wall thickness. Table 2.2 illustrates the available line pipe and oilfield casing outside diameters.

Oilfield casing comes in various grades or yield strengths, as discussed in the following section. Line pipe is normally designated as a Schedule. For instance, Schedule 40 is very common and there are tables that provide the wall thicknesses for the various schedules of pipe. It is important to note that most line pipe has a wall thickness allowance of ±12.5%. With modern manufacturing methods the wall thickness can be very carefully controlled. Therefore manufacturers make the pipe with the thinnest reasonable wall thickness possible. This means that for all welded line pipe one can assume that the wall thickness is at least 10% less than the wall thickness in published tables listing schedules and wall thicknesses. Very little seamless steel or stainless steel is used in line pipe well casing.

Table 2.2
Common pipe/casing outside diameters

Line Pipe (in) Steel and Stainless Steel	Oilfield Casing (in) Steel
4½	4½
	5½
5⁹⁄₁₆	
	6
6⅝	6⅝
	7
	7⅝
8⅝	8⅝
	9⅝
10¾	10¾
	11¾
12¾	
	13⅜
14	
16	16
18	
	18⅝
20	20
22	
24 to 60	

Oilfield casings have published wall thicknesses, however they are expressed in terms of casing weight in pounds per foot. The calculated wall thicknesses from the published API tables are what is actually produced. In addition the API tables list ultimate ratings for the casings which include internal pressure, collapse pressure, pipe wall tension load and threaded connection tensile load.

PVC pipe is manufactured with various wall thickness designations including SDR (standard dimension ratio), schedule and internal psi pressure rating. For ASR well purposes it is most useful to use the SDR system, which is the number obtained by dividing the outside diameter of the pipe by the wall thickness. This means that the smaller the number, the thicker the pipe. SDR 17 is commonly utilized for ASR well casings using PVC pipe.

Fiberglass pipe is referenced by a nominal inner diameter (ID) and the outer diameter (OD) changes as the wall thickness becomes larger. Since fiberglass pipe is built around a steel mandrel, the wall thickness is built up in layers of fiberglass to achieve the desired collapse strength.

56

Materials of Construction

PVC Casing

PVC well casing is utilized frequently in the water well industry, for smaller, shallower wells less than about 1000 ft deep. Many ASR wells are constructed with PVC casings due to their favorable corrosion resistance relative to carbon steel cased wells. For larger, deeper wells, greater care is required to ensure satisfactory construction. PVC casing outer diameters are available by special order up to 900 mm (36 inches). Introduction of mechanical joint casing connections has greatly simplified the installation process, thereby reducing both risk and cost. However these casings are currentlyavailable only in outer diameters up to 450 mm (17.4 inches).

Probably one of the deepest, large diameter PVC casings for any well is at Gloucester, Virginia. 17.4" outer diameter casing was set to a depth sufficient to accommodate the pump. Below that depth a 12" PVC casing was set to 1,380 feet. A stainless steel wire-wrapped well screen was used below that depth. The entire casing and screen assembly was installed at one time, gravel packed and cemented. Ocean Reef, Key Largo, Florida, has a 17.4 inch outer diameter PVC casing set to 1,050 feet. In general, setting a PVC casing to a depth of about 152 m (500 feet) is not too difficult. Below that depth, the technology exists to successfully construct the well; however time, skill, and care are required to avoid casing or hole collapse.

For most ASR systems, PVC casing diameters in the range of 175 to 450 mm (6.9 to 17.4 inches) will be appropriate, either for ASR wells or for monitor wells. These are outside diameter (OD) measurements for the pipe and do not include the couplings. Wall thickness for PVC casings is thicker than for steel casings, therefore corresponding inside diameters for nominal pipe sizes tend to be smaller. For the range of outer diameters referenced above, the inside diameters range from 6.0 to 15.2 inches.

Schedule 80 PVC casing is sometimes selected, particularly for smaller casing diameters; however, this is a standard wall thickness and therefore provides decreasing resistance to collapse with larger diameters. Selection of casing according to a standard dimension ratio (SDR) number, such as SDR 17, ensures consistent collapse strength regardless of casing diameter. For SDR 17, the wall thickness is $1/17$ times the outside diameter of the casing. For 450-mm (17.4-inch) OD casing, which has been used for many ASR wells, SDR 17 would have a 26 mm (1.024 inch) wall thickness. A corresponding Schedule 80 nominal pipe size of 18 inches would have a wall thickness of 0.938 inches while a 16 inch Schedule 80 PVC casing would have a wall thickness of 0.844 inches. The inside diameter is not always round, varying for a 17.4" OD PVC casing between 15.079 and 15.352 inches.

Depending on the pipe size, couplings can add up to about 40 mm (1.5 inches) outer diameter to the casing string, unless the outside diameter of the couplings provided by the manufacturer is less. For the 17.4" SDR 17 PVC casing, the outside diameter of the coupling is 475 mm (18.9 inches). Since it is important to be able to introduce a 50 mm (nominal 2-inch, actual 2.375 inch outer diameter) tremie line into the annulus around the PVC casing, the hole into which the casing is run should be at least 150 mm (6 inches) greater diameter than the casing diameter.

Installation of long runs of PVC casing in a well is comparable to threading a wet spaghetti noodle down the barrel of a rifle. This is in part a function of the density of PVC material which is approximately the same as the density of drilling mud, giving it a tendency to almost float in a mudded hole. It may be necessary to push the casing down the hole prior to cementing,

57

particularly if the drilling fluid weighs more than about 12 lbs/gallon, or has a specific gravity greater than about 1.44. It is therefore advisable to provide a sufficient annular space in order to facilitate installation.

The compressive strength of the PVC casing is weakest during cementing of the casing, when the heat of hydration can be sufficient to raise the casing temperature and thereby reduce its strength. For this reason, it is advisable to cement the casing in stages. The first stage is typically introduced under pressure from the bottom of casing upward within the annulus between the casing and the borehole wall, whereas subsequent stages are tremied into place within the annulus from land surface. Typical stages are in the range of 150 to 200 feet, two stages per day. Circulating water or adding ice within the casing while the cement is curing for each stage is another option. Use of pozzolan cement, or cement-bentonite mixtures that have a lower heat of hydration is also effective.

Other measures that have been considered but are not known to have been applied are to fill the PVC casing during cementing with a heavy mud to reduce the hydrostatic pressure differential, or to provide dual packers on the inside of the casing, pressurized between the packers, for internal support as the casing is cemented in stages. Combining these precautions usually ensures satisfactory well completion. Control of temperatures and pressures is important. For 17.4" SDR 17 Certa-Lok PVC casing, the manufacturer's estimate of "Resistance to Hydraulic Collapse Pressure" at room temperature is 224 psi, without any safety factor. However heat of hydration during cementing can increase temperatures to above 66°C (150°F), substantially weakening the pipe. All of this extra care increases well construction time and consequently cost. The extra cost, combined with the increased installation difficulty, are the principal reasons why deeper, larger ASR well casings have traditionally utilized materials other than PVC. However, successful experience with deep, large-diameter PVC casings for ASR wells is becoming more widespread.

It is important that the hole to be cemented stays open during the successive cementing stages. Settling of drilling mud and subsequent hole collapse may preclude staged cementing of the casing to ground surface in some situations where the casing installation and cementing operation requires excessive time. At extra cost, the risk of hole collapse during cementing can be offset through setting additional outer steel casings so that each cementing stage is likely to terminate within the next steel casing string.

The use of PVC casings at ASR sites has been effective, in conjunction with appropriate selection of wellhead piping materials, in keeping the production of particulates during recharge and recovery to an acceptable minimum. At most sites with PVC-cased ASR wells, water is clear and meets drinking water quality standards within about 10 to 20 minutes after the beginning of recovery.

Fiberglass Casing

For casing diameters larger than those available with PVC, fiberglass is a viable and cost-effective option. Until recently the principal constraint upon use of fiberglass for ASR well casings has been uncertainty regarding whether the couplings could provide sufficient tensile strength for well installation, and also adequate mechanical integrity to maintain pressure or vacuum during well testing. A typical requirement could be that the well inner casing should maintain a predetermined applied pressure or vacuum for an hour with either a cement plug in the bottom of the casing, or a downhole packer, with acceptably small loss of pressure or vacuum. This has recently been demonstrated successfully iin Florida with several large diameter (24 inch)

fiberglass cased wells, so it is anticipated that future high capacity ASR wells for which a 17.4" OD PVC casing is too small, will increasingly utilize fiberglass (FRP) casings.

Special consideration is required during casing design to ensure that the inside and outside coatings on the fiberglass provide adequate abrasion resistance, considering the subsequent raising and lowering of drill bits, drill strings, screen and gravel pack installation, well development equipment and procedures, and the abrasion that may reasonably be expected as a result of starting and stopping the pump. Selection of a resin liner composition approved for use in water wells, and with an internal thickness of about 0.1 inches (2.5 mm), is one approach. Concern has also been expressed regarding potential release of glass fibers into the water as a result of long term exposure to water and periodic well redevelopment operations.

Fiberglass casing, though not commonly used for water wells, has good corrosion resistance, reasonable collapse resistance and reasonable cost, however it may be considered somewhat fragile from a well construction standpoint. It is important to use a potable grade of fiberglass casing, suitable for drinking water use.

Steel Casing

Uncoated carbon steel casing is utilized in many ASR wells, similar to conventional water production wells. When existing production wells are to be retrofitted for ASR testing or operations, there is usually no choice in the well design or casing material. The only option usually available is cleaning the existing casing prior to ASR retrofitting. Occasionally it is possible to insert a liner in the casing.

Operating measures can be implemented to handle the solids produced from the steel casing during recovery and to minimize introduction of solids to the well during recharge. During initial recovery or backflushing operations, water can be returned to the water treatment plant for retreatment. It can be discharged directly to the storm drainage system, whether piped or by surface conveyance. It also may be discharged to a drywell, pit, or pond constructed adjacent to the ASR well with sufficient volume to contain all of, or a considerable portion of the water to be discharged to waste. This may reduce the peak rate of discharge to the local drainage system, if necessary, and will provide some settling and dilution of the initial solids pumped from the well.

One or more injection tubes are sometimes provided inside the casing to control cascading. Injection tubes may also be provided outside the casing, entering the casing below the static water level through a specially fabricated, short casing section. These tubes also serve to reduce entry of rust into the well during recharge since the surface area of exposed steel is reduced. Water is conveyed into the well through the tubes and therefore is less likely to scour rust from the casing. The tubes may be of steel, stainless steel, fibreglass or PVC. This requires sufficient casing and hole diameter to accommodate the pump and also the injection tube(s). An alternative approach is to recharge down the pump column, achieving the same objective. The potential for galvanic corrosion due to use of different materials of construction for well casing and injection tubes should also be considered.

When recharging through an existing well casing it is important to verify the downhole condition of the well. A video log of the casing and well bore or well screen will indicate whether the casing is corroded, encrusted or possibly blocked by an obstruction. It will also show the condition of the well screen and whether the bottom of the well has filled in to any significant depth. Such an inspection may also indicate the presence of a layer of oil floating on top of the

59

water column from an old oil-lubricated pump: the presence of filamentous bacteria, or other objects in the well.

Epoxy-Coated Steel Casing

A fusion-bonded, or liquid applied epoxy coating can be applied at the factory and is selected to meet applicable American Water Works Association (AWWA) standards for use with public drinking water systems. For butt-welded steel casing, the coating adjacent to each weld will be lost during construction. However, pre-installed welding collars can be used to reduce coating burn. For threaded and coupled casing, the coating would remain; however the benefits of complete reduction of surface area exposed to rusting may not be worth the additional cost of providing threaded and coupled casing. It is recommended that "friction" type casing wrenches should be used for threaded and coupled pipe.

Care is required to avoid damage to the coating during installation and well construction activities that occur after the casing is set and cemented in place. Rubber bumpers have been used successfully around the drill pipe to prevent such damage. Fusion bond coatings are also generally tougher than liquid applied coatings and typically hold up better to abrasions during construction.

If coated casings are used and attached to stainless steel well screens, the potential for galvanic corrosion of the steel casing is created. Galvanic corrosion of the steel casing will focus on imperfections or damaged areas of the casing coating and can result in very rapid localized corrosion in these areas. Localized holes in the casing can result in a very short time in areas near the screen. Coated pump columns can also be subjected to this corrosion and holes in the column can result in only a few years. For these reasons, it is best to use a dielectric coupling between the stainless steel screen and the coated steel casing.

Stainless Steel Casing

Although this is the most costly material for ASR well casing, it provides several advantages. These particularly include strength, corrosion resistance, abrasion resistance, service life and minimal wall thickness. Selection of the appropriate stainless steel alloy, usually 304 or 316, needs to be based on the water quality. The use of a low carbon grade of stainless steel also reduces carbide precipitation in the weld area and the associated decreased corrosion resistance.

Stainless steel well casings have been utilized for ASR wells in many locations. Although the cost per unit length of casing is relatively high, the overall impact upon capital cost of the well and wellhead installation is typically not that great. It may be offset by reduced operating costs since fewer particulates will need to be purged from the well during the beginning of recharge and recovery. Furthermore, elimination of corrosion products can reduce the required frequency of periodic backflushing operations during well recharge.

Frequently ASR wells are constructed in aquifers that are either brackish or contain otherwise poor quality water, however during normal ASR operations water quality inside the casing changes, becoming the same as the stored water quality, which is usually drinking water. Outside the casing and associated cement sheath the ambient groundwater quality remains unchanged. Where the ambient groundwater is aggressive and in contact with the casing, corrosion can occur, causing premature well failure. The additional investment required for a corrosion-resistant well casing, when amortized over the extended life of the well, can ensure a highly cost-effective well design.

Corrosion Resistant Alloys and Other Casing Materials

For situations where very high corrosion potential exists, consideration should be given to use of corrosion resistant alloys. These are more expensive relative to stainless steel and have not previously been utilized for ASR wells.

Yield Strength and Collapse Resistance

Regardless of the selection of casing materials, care is required to ensure that the casing strength is adequate to prevent collapse due to formation or cementing pressures, and to ensure that the tensile strength of the casing and couplings is not exceeded during casing installation. This is particularly true for PVC casings, which experience significant loss of strength due to the heat of hydration that occurs during cementing. This can lead to casing collapse if cementing is not conducted properly and in a series of stages. In general, for wells exceeding about 800 to 1,000 feet of depth, steel or stainless steel casings are normally utilized.

The yield strength of a material is designated in pounds per square inch (psi) and represents the elastic limit. Once the elastic limit is exceeded there is plastic deformation. A simple example of this is a straight piece of wire from a coat hanger. One can bend the wire a little bit and it will spring back to its original shape. If one bends it past the elastic limit, it will plastically deform and spring back, but will be bent rather than straight.

The collapse resistance of a casing is a function of its yield strength, SDR , and the ovality or eccentricity of the casing. Many casings are not truly round. As the wall of the casing becomes thicker it has an increasing collapse resistance. The ovality of the casing is defined by the major diameter minus the minor diameter divided by the average diameter. Ovality is important for determining casing collapse resistance.

Oilfield steel casing comes in various grades or yield strengths. A J55 casing means that the minimum yield strength is 55,000 psi. Other common grades are N80, P110 and V150. Steel line pipe normally has a yield strength from 30,000 to 35,000 psi. Off-the-shelf stainless steel pipe is annealed and has the same yield strength of 30,000 to 35,000 psi. Stainless steel is available with a yield strength up to 60,000 psi, by special order.

Various formulas have been developed for determining the collapse resistance of casing, the most common of which have been developed by Timoshenko. The Timoshenko elastic formula which does not incorporate eccentricity calculates a unique answer. The Timoshenko elastic formula that incorporates eccentricity solves a quadratic equation, resulting in two answers. The user must select one answer, however this is usually an obvious choice. An alternate approach is the API 5C3 method. The API method calculates several intermediate values which are used in three subsequent formulas. Each of these three formulas is valid for a certain range of casing dimension ratio (outer diameter/wall thickness). The dimension ratio is also calculated, allowing the user to easily select the correct answer for the casing in question. The input data for these three formulas include yield point (psi), outside diameter (inches), wall thickness (inches), nominal outside diameter (inches, same as outside diameter), "minus" OD tolerance (inches), and "plus" OD tolerance (inches). Once these input values are entered, results for these five methods can be calculated. There is often a wide variation in collapse resistance values calculated by the various methods, requiring the user to determine how conservative he or she needs to be.

This is not an exact science and so appropriate safety factors need to be used in addition to the calculations of ultimate collapse pressure, commonly expressed in terms of psi per foot of

61

depth. Typical criteria for determining the ultimate collapse resistance of casing is recommended to be from 0.4 to 1 psi per foot depth of casing. For the minimum collapse resistance one has only to consider the pressure exerted by a column of water, which is 0.433 psi per foot of depth. If a casing is pumped dry and the static water level outside the casing is at the ground surface, then a criterion of 0.433 times a minimum safety factor of 1.2 equals a required collapse resistance of 0.52 psi per foot of casing depth. If the borehole is of a competent or consolidated rock and the depth to static water level is deep, it may be possible to then have collapse resistance criteria of less than 0.4 psi per foot. On the other hand, in unconsolidated materials that may be subject to liquefaction, as in an earthquake-prone area, the weight of the earth's material is about 1 psi per foot of depth. With a safety factor of 1.5 the casing collapse resistance should be 1.5 psi per foot. This means that a 500 foot well would need a collapse resistance of 750 psi. These examples illustrate how important are the site-specific conditions for determining the collapse resistance of the well casing.

The collapse resistance of a casing is also dependent upon the type and quality of cementing that is subsequently completed. For example, it is possible to use PVC casing beyond its collapse resistance strength if the casing is cemented carefully. The exothermic curing reaction of the cement causes the temperature of the PVC and the cement to rise, weakening the casing. This can be counteracted in several ways, including circulating cold water and/or ice within the casing, and by cementing in stages. Another solution is to install inflatable packers inside the PVC casing and pressurize the zone between the packers to counteract the hydrostatic pressure of the cement. In this same fashion, using inflatable packers, a thinner wall stainless steel or fiberglass casing may be used in order to reduce well construction costs.

Joints

Various methods are available for joining sections of ASR well casing. Line pipe and stainless steel are typically welded together. This is generally done in two ways. The first, and preferable method, is a butt weld. The ends of the casing are beveled and a weld is made to the full thickness of the pipe. This is the strongest weld and requires the greatest skill on the part of the welder. It is important not to have the welding protrude inside the casing and also to have a 100%, or nearly 100% weld penetration. Pipe fitters and pipeline welders are skilled at making this type of weld. A second method for welding includes the use of a weld collar or ring. A collar that may be about 4" long is welded to the outside of one end of the casing such that there is a 2" overlap. This involves a fillet weld which only has about 60 to 70% of the strength of a butt weld. The reduced strength is due to the weld being in shear rather than the entire weld being in tension as in a butt weld. The weld collar makes it easier to lift the casings with elevators and functions as a lineup guide for adding the next piece of casing. The welding is then much easier using a fillet weld on top of the collar to the casing.

For many ASR installations the well construction cost is less than half of the total installation cost for the well, wellhead and associated facilities. Greater capital investment in the well casing can achieve significant long term economies in well performance and operation.

When there is a better understanding of the annual costs associated with periodic backflushing for well redevelopment, and for initial recovery to waste to remove solids, the tradeoff between investment in higher cost well casing and reduced operating costs will become clearer. Until that time, it seems wise to seek reasonable opportunities to minimize solids production. One

of these alternatives is to select an appropriate casing material like stainless steel, even at some increase in well construction cost.

Cost is not the only issue here. Operational complexity is an important factor. An ASR well in a low permeability, unconsolidated aquifer that requires backflushing every few days or weeks represents less of an operating problem if the backflushing water volume is minimized. Selection of an appropriate casing material can help to minimize the frequency, duration, and pumped volume of backflushing.

Stainless steel has an additional application for ASR well casings. If a well casing becomes damaged or has holes in it due to a variety of reasons, it is possible to repair the casing by inserting an expandable stainless steel sleeve or patch with an outer diameter slightly smaller than the internal diameter of the existing casing. This can be accomplished by plastically deforming such a sleeve with a swedge or an inflatable packer. This type of repair results in the smallest possible reduction in the internal diameter of the casing.

Couplings can be used; "threaded and coupled" (T&C) casing costs more but reduces the time required for installation. If couplings are used, a pipe joint compound is required. The type of pipe joint compound is important. It needs to seal the joints so that there is no leakage. It also needs to have anti-galling properties and be acceptable for potable water use. The best choice to consider is an epoxy-based pipe joint compound to ensure a leak-tight connection.

For well depths of less than about 500 feet, NPT (National Pipe Taper) or BSPT (British Standard Pipe Taper) threads can be used for threaded and coupled steel casing. For depths greater than 500 feet it is recommended that oilfield casing threads be used. Epoxy-coated line pipe should be threaded and coupled as the welding will destroy the epoxy coating at the joint inside the casing, creating an opportunity for focused and accelerated galvanic corrosion activity at the weld joint. Oilfield casing is normally threaded and coupled. Because of its tendency to become brittle as a result of welding, oilfield casing is generally not recommended to be welded. Oilfield casing threads are designed for hanging long casing strings in a well. The three most common threads are 8 round short, 8 round long, and buttress. In addition there are a number of premium or specialty threads that have been developed by various companies. NPT and line pipe threads are identical. Specifications for threads and thread compounds are available in various bulletins and other publications that are available for sale from the American Petroleum Institute (www.api.org).

Fiberglass pipe is normally threaded together and incorporates 1 or 2 O-rings. Recently a new type of quick connection has become available using splines and double O-rings in order to facilitate casing installation while ensuring its mechanical integrity and tensile strength.

PVC pipe can be joined with various methods. In smaller pipes bell and socket glued joints have been used. These have the disadvantage of having to wait for the glue to set so that the casing does not come apart during installation. Alternatively, rod and groove joint is available which incorporates O-rings for sealing and can normally support the weight of the PVC pipe. Threaded joints may also be used. The couplings can be of PVC or of a non-corrosive metal such as stainless steel.

Corrosion Potential

Steel casing has been used for many years in water wells. If the ASR well is completed in an aquifer containing fresh water, carbon steel casing may be considered, especially in large diameter, deep wells where stainless steel costs can be quite high. The highest rate of corrosion occurs in the well casing where the water level fluctuates. Corrosion products provide a food

source for bacteria, thereby contributing to well plugging. Corrosion can be reduced or eliminated by two methods.

Nitrogen gas can be introduced by a tube into the annulus between the pump column and the casing, displacing the air containing the oxygen that causes corrosion. With no oxygen there is no rusting. The flushing rate for the nitrogen gas would be determined by the rate at which the dissolved oxygen evolves from the water into the annular space. A second method would be to use a vacuum pump to provide a vacuum of about ¼ psi absolute. This would replace the air in the annulus with water vapor from the well water, removing the corrosive oxygen. Both of these methods are suitable for ASR wells with water levels that remain in the casing during recharge, storage and recovery periods, or rise above land surface during recharge. Although these approaches have been widely used for industrial process design, application within the water well industry is not known to date. If the well is under water table conditions, then the vacuum method may be preferred as it would remove the air from the unsaturated aquifer and replace it with water vapor. Because of the larger surface area in the unsaturated zone, the vacuum would also remove more dissolved oxygen from the water.

Dissolved oxygen may be removed from the recharge water by pretreatment, such as through chemical addition of sulfur dioxide. This was implemented at the Swimming River ASR site in New Jersey during ASR initial testing with the objective of dissolving the mineral siderite in the storage zone and thereby controlling iron concentrations in the recovered water. This approach was successful in removing dissolved oxygen, however a better alternative approach was later implemented to control the iron concentrations. Addition of sulfur dioxide reduces the pH of the recharge water to between 3 and 4, accelerating corrosion in steel cased wells, so it is not recommended for corrosion control. However other chemical pretreatment approaches are utilized in the oilfield industry to control downhole corrosion in steel cased wells carrying seawater for secondary recovery operations. Some of these approaches may be applicable in the water well industry.

For ASR sites storing high quality surface water, such as from rivers or lakes, recharge water dissolved oxygen concentrations may also be pretreated through bank filtration or through biological filtration in above-ground tanks, relying upon natural microbial and geochemical processes to remove or reduce the oxygen while at the same time removing particulates, micro-biota, total organic carbon and other constituents. This would then help to control corrosion in steel-cased wells.

Stainless steel has a much higher corrosion resistance than steel. The two most common grades of stainless steel are 304 and 316. Type 316 SS is more corrosion resistant and costs more. Type 304 has been successfully used, however if there are corrosive conditions such as stray electrical currents combined with chlorides in the recharge water, then 316L is recommended. The "L" stands for low carbon content and is available for both 304 and 316. Where chloride concentration exceeds about 500 mg/L or where dissolved oxygen concentrations are high, Type 304 SS has a tendency to corrode by pitting, particularly in the zone of fluctuating water levels in the casing.

For ASR wells where corrosion potential is unknown or is a serious concern, it may be appropriate to hang steel and stainless steel coupons in the well annulus. These can be retrieved periodically to monitor downhole corrosion, providing time for consideration of response measures if the need is indicated.

PVC is considered essentially non-corrosive for normal ASR applications. It is often used for ASR well construction in the United States and is also used for water well construction in Europe. Fiberglass casing is also very corrosion resistant. It tends to be more brittle than other

casing materials and, with time, some concern exists that the glass fibers may become weakened by the water. Both fiberglass and PVC casings are more susceptible than steel casings to damage caused by various well construction, service and maintenance operations.

Cementing

Cementing requirements for ASR wells are more stringent than for conventional water supply wells since pressures reverse during recharge and recovery, increasing the potential for fluid movement around the outside of the casing to land surface. Casings in ASR wells should be cemented from the bottom of casing to ground surface to ensure an adequate seal against flow movement outside the casing through possible channels opened during construction. This is usually accomplished by pumping the cement down the inside of the casing and up to the ground surface around the outside of the annulus between the casing and the borehole wall. Preferably this is accomplished in a single stage. If circulation to ground surface is not achieved, a tremie line from the surface is used to complete cementing. Upon completion of cementing, the cement plug is drilled out to penetrate the ASR storage zone.

In some cases a single string completion is utilized in which the borehole is drilled initially to the bottom of the aquifer, following which the casing and screen are lowered into the hole. Cementing of such a well requires special precautions to avoid cementing the aquifer. If the well is gravel packed, a layer of bentonite can be placed on top of the gravel pack, separating it from cement that is introduced from above using a tremie pipe. The gravel pack is typically continued several feet above the top of the well screen, providing a small reservoir of gravel that can settle down into the borehole around the screen if further screen and gravel pack development occurs during ASR cyclic operations. In at least one ASR well, gravel feed tubes have been provided so that the depth to top of gravel can be periodically checked and, if necessary, replenished. A rule of thumb that is recommended by Johnson Screens is to extend the gravel pack one quarter of the length of the total well screen interval above the top of the screen.

If the well is naturally developed, cement baskets or an external casing packer can be utilized to prevent cement from entering the aquifer down the annulus between the casing and the borehole wall. A tremie pipe can then be utilized to pump in the cement. Alternatively a temporary cement plug can be provided in the casing and screen string prior to installation in the borehole. Cementing then occurs through holes cut or provided in the well casing above the temporary plug, which is above the screen and annular seal.

Tremie pipes necessitate large and expensive annular spaces around well casings, typically about 6" larger diameter than the outer diameter of the well casing. This larger and costlier borehole diameter is one of the justifications supporting consideration of the use of pre-packed well screens or naturally developed screens. Once cemented, the additional borehole diameter also ensures separation of the casing from any corrosive fluids in the formations penetrated by the well.

At three known retrofitted ASR sites in existing uncemented (cable tool drilled) water supply wells, the pressures occurring during recharge caused upward flow around the outside of the casing. At one site, this created flow at the surface. At another site, the result was formation of a sinkhole adjacent to the well. The third site experienced downward movement of surficial sands into the underlying limestone production interval, causing a severe solids problem in the well. Cementing is normally a desirable and required regulatory practice to prevent production well contamination from adjacent land use activities; however, for ASR wells there are additional hydraulic reasons that apply due to their cyclic operation. Unlike conventional water supply wells,

the direction of water movement in an ASR well reverses during recharge and recovery, effectively developing out any fines that might otherwise seal a uni-directional flow path around the outside of a casing.

MECHANICAL INTEGRITY TESTING

Regulatory agency requirements at some ASR sites, particularly in Florida, have included the need for periodic mechanical integrity testing, otherwise known as "MIT Tests." The intention is to demonstrate that the well inner casing will not lose water to the surrounding formations during typical pressures occurring during ASR recharge and will not cause movement of groundwater into the well through the casing during recovery. Carbon steel-cased wells in particular tend to corrode with time due to chemical or galvanic corrosion, so a steel casing that is sound initially may eventually fail. Failure can potentially contaminate overlying aquifers or the ASR storage zone by providing a cross-connection through the well casing. When this occurs, mitigation measures include plugging and abandoning the well, or relining it with a smaller inner casing.

The MIT test is typically conducted on a well during construction, following cementing of the inner casing and prior to drilling out the cement plug. Alternatively the test may be conducted on the completed well after first installing a packer at the base of the casing. The casing is filled with liquid, pressurized to about 50 psi and then monitored for 30 minutes to one hour. If the pressure holds for that time period, within about 10%, the test is considered acceptable. If the pressure dissipates rapidly, indicating a leak, then the responsibility is on the drilling contractor to either repeat the test or to resolve the problem. Resolution can include plugging the well and reconstructing a new well.

For a steel-cased well that has been properly welded or threaded together, there is usually no difficulty passing the MIT test. For a PVC-cased well the pressure may decline due to slight expansion of the casing, even without a leak. Recognizing the negligible risk of aquifer contamination due to ASR operations, regulatory agencies in Florida have recently required and accepted MIT test results without imposing strict criteria for success. Initial test results are then available for comparison with repeat tests that will probably be required in later years. The subsequent tests are typically expensive, requiring removal of the pump from the well, setting of a packer in the casing, and then conducting the test. If the results from later tests are substantially different than results from the initial tests, a basis then exists for evaluating the extent of the problem and determining appropriate remedial measures.

SELECTION OF ASR STORAGE INTERVALS

Most ASR wells store water in artesian aquifers since groundwater velocities are lower, the potential for surface contamination is reduced, and recharge pressures can be above land surface. Deep aquifers with little natural recharge that are being "mined" quite often make excellent ASR storage zones. Some ASR wells are operating satisfactorily in water table aquifers, particularly in areas where depth to static water level is substantial.

For artesian aquifers, by definition the aquifer is already "full" of water. Recharge into such aquifers displaces the ambient groundwater away from the well, forming a "bubble" of recharged water surrounded by the ambient groundwater. Artesian pressures rise during recharge and recover close to original levels as soon as recharge ceases. During recovery the opposite effect occurs. A very small amount of storage is created by compressing the water and expanding the

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aquifer during recharge, however this is usually insignificant. Virtually all of the storage is created by lateral displacement of ambient groundwater, usually within a radial distance of a few hundred feet. The actual shape of the bubble depends upon lateral and vertical differences in hydraulic conductivity within the storage zone however, in concept at least, it is slightly oval, reflecting the local hydraulic gradient and anisotropy of the aquifer.

With either seasonal storage or long term storage ("water banking") the net effect upon water movement in the aquifer is negligible since the stored water volume will eventually be recovered. However if the storage duration is several years and the natural rate of groundwater movement is several hundred feet per year, the water that is recovered may not be the same molecules as the water stored. For an artesian aquifer with similar water quality to the recharge water this may not be a significant local or regional issue, however if a significant water quality difference exists between the stored water and the ambient groundwater, this "loss of stored water" can be important. Most ASR storage zones exhibit at least one water quality constituent that is different between the recharge water and the ambient groundwater, requiring treatment before potable use. Hence ASR storage zones tend to be in artesian aquifers where the regional movement of stored water is slow. Where unconfined aquifers are utilized for ASR storage, with associated higher natural groundwater velocities, the aquifers are typically fresh so that water quality changes are acceptable. Recharge then raises groundwater levels, often contributing to the achievement of regional water management goals.

In consolidated formations that can produce water without pumping sediment or solids, an open hole well completion is low cost and very effective. Typical formation lithologies are sandstone and karst or fractured limestone. For such formations the sandstone primary porosity or limestone secondary porosity typically provides most of the water storage volume for ASR wells, utilizing the void spaces between the sand grains in the sandstone, or the intergranular or vuggy porosity in the limestone. Secondary porosity due to fracturing and solution features typically provides most of the water conveyance to and from the ASR well during recharge and recovery. For basalt-consolidated aquifers, the relatively productive intervals between the successive lava flows provide virtually all of the porosity. In unconsolidated formations, screen intervals must be selected using naturally-developed or gravel pack well completion. Whether the ASR storage zone is consolidated or unconsolidated, certain factors need to be considered during selection of the appropriate storage interval.

The simplest case is one in which the ASR storage zone under consideration contains water of similar quality to that which will be recharged, and has no potential geochemical problems. In such a case, the ASR well design will tend to be similar to a conventional production well design. If screened, the screen length may tend to be slightly longer to maximize recharge efficiency and to minimize the rate of plugging. If open hole, the hole length will tend to fully penetrate the production interval for the same reason.

For the more common case where the storage zone is brackish or contains water of such quality that mixing is to be minimized, the selection of the storage interval requires greater care. Thin intervals that have excellent vertical confinement are best suited for minimizing mixing. For the Marathon ASR test well, Florida, which successfully stored treated drinking water for emergency water supply purposes, storage was in a confined sand production interval 11 m (40 feet) thick and containing seawater with a TDS concentration of 39,000 mg/L. In less extreme cases of water quality difference, thicker storage intervals with less confinement may be sufficient to provide the desired recovery efficiency.

Where the choice of storage intervals is limited, well and wellfield design can to some extent adapt to the limitations imposed by nature. Multiple wells, or horizontal directionally-drilled wells can provide ASR development of a zone that has sufficient storage volume capacity but low yield to individual wells. This could be due to low transmissivity, shallow depth, subsidence concerns, or potential upconing of brackish water from below the storage zone during ASR recovery. The cost of additional wells is frequently small when compared to the cost of alternative water storage or other sources of peak water supply.

For the unusual situation where several potential storage zones are available, it is appropriate to consider the volumes and rates required for storage and recovery, selecting the storage interval that best matches ASR objectives. In some cases, storage in multiple intervals will be appropriate, each utilizing separate ASR wells at the same site.

Where the storage zone has great thickness or poor confinement and contains poor quality groundwater, acceptable recovery performance sometimes may be achieved by operating at high rates and long durations during recharge. The volume stored then may be sufficient to displace the poor quality water away from the well, both vertically and laterally, so that a useful recovery volume can be achieved during each subsequent recovery season. This may take several annual cycles of operation, each showing an increase in recovery efficiency. Alternatively, a large initial storage volume may be recharged following construction. This may be considered as the formation of a buffer zone, analogous to initial filling of a surface reservoir. Once the buffer zone is formed, or the "surface reservoir" is filled, ASR operations at the ultimate recovery efficiency can proceed. This is the "Target Storage Volume" concept discussed in greater detail below.

The recovery efficiency attainable will depend upon the hydraulic and water quality characteristics at each site. While 100% recovery efficiency is a reasonable target and is obtained in most cases of storage in brackish aquifers, lower recovery efficiency may occur in some situations. An economic analysis will then indicate whether the lost investment in the water not recovered is more than offset by the value of the water recovered when needed. Usually this is the case since the water is usually stored at times of the year when marginal costs of water production (cost per unit volume) are low, including only electricity, chemicals and residuals disposal. When the water is recovered the marginal value is usually quite high, reflecting local alternatives for supplemental peaking water supply.

The most complex issues pertaining to storage zone selection are with aquifers, or portions of aquifers, that offer geochemical challenges. One solution is to design the ASR well to case out production intervals that may contribute severe geochemical problems, if this can be achieved without losing much of the potential production capacity of the well.

Typically, the detailed information needed to make a reasonable judgement regarding well design to avoid geochemical problems can only follow coring, core analysis, geophysical logging, and water quality testing in selected depth intervals. In the absence of these data, it is difficult to know which intervals are contributing the undesirable water quality. Consequently, the design of the second and subsequent ASR wells may benefit from experience gained with the first such well at any new site. In such circumstances initial construction and testing of an exploratory well is appropriate.

Where multiple storage intervals are potentially available at a site, it is tempting to construct a single cluster well, with separate recharge tubes or casings extending to each interval. Confining layer sections between the ASR storage zones would then be grouted to maintain the seal. In reality this has generally proved to be a poor idea due to pipeline corrosion causing leakage between aquifers. Furthermore this tends to lead to casings which are too small in diameter to accommodate a

68

pump, thereby creating plugging problems that are not easily resolved. It is much better to construct a separate, properly designed ASR well for each storage interval.

SCREEN DESIGN

In unconsolidated or partially consolidated formations, well screens are utilized to keep the aquifer interval open and to control pumping of sand. Four categories of well screens may be considered:

- Wire-wrap screen, which is constructed using a rod base, wrapped with a wedge wire to form a continuous, non-clogging slot opening. This is the best screen design for sand control, whether for naturally-developed or gravel-packed well design. Where a more rugged design is needed, the wire-wrap screen jacket is installed over a perforated pipe base. This rugged design is frequently used in horizontal oilfield completions.
- Pre-packed screens, utilizing two wire-wrapped screens with the annulus between them filled with a gravel pack material.
- Openings of various kinds formed in a pipe. These openings can consist of louvers, punched bridge slots, torch cut perforations, milled slots, and other types of openings. These are almost always used with a gravel pack design as it is difficult to construct them with a consistent and accurate narrow slot opening. These types of screens are seldom utilized in a naturally-developed well, except where very coarse material is encountered.
- Wire-wrap screen, which is constructed using a rod base, wrapped with a wedge wire to form a continuous, non-clogging slot opening. This is the best screen design for sand control, whether for naturally-developed or gravel-packed well design. Where a more rugged design is needed, the wire-wrap screen jacket is installed over a perforated pipe base. This rugged design is frequently used in horizontal oilfield completions.

When a well screen is installed without a gravel pack it is considered to be a naturally developed well. The slot size selected for the screen can be selected using several different criteria. A common approach is to use a slot size that allows 60% of the aquifer material to pass through the screen. In some cases it may be appropriate to consider allowing passage through the screen of as low as 20% to 40% of the formation material. In aquifers that are extremely heterogeneous with coarse layers interspersed with fine layers, it may be necessary to allow a much lower percentage to pass through the screen, as a result of which the fine-grained material will determine the slot size. Alternatively the screen may have different slot sizes at different depths, depending upon the material encountered. In extremely fine-grained materials slot sizes might be as small as 0.005 inches (0.127 mm). In most cases the slot size of a screen in a naturally developed well does not exceed 0.100 inches (2.5 mm) with the larger slot sizes being more common in river alluvium, glacial outwash, and alluvial fans in the western United States and smaller slot sizes being more common for the Atlantic Coastal Plain and the Caribbean Gulf Coast states. Larger slot sizes have more open area but provide little marginal advantage in well yield, and the collapse strength of the screen is reduced.

Table 2.3
Open areas of screens

Screen Diameter	Slot Size	Continuous Slot		Louvered (maximum open area)		Bridge Slot		Mill Slotted (vertical)		Plastic Continuous Slot		Slotted Plastic	
		in²/ft	%	in²/ft	%	in²/ft	%	in²/ft	%	in²/ft	%	in²/ft	%
4" ID	20	44	25	—	—	—	—	—	—	22	13	—	—
	60	90	52	—	—	19	12	8	5	52	30	18	11
8" ID	30	80	25	—	—	—	—	—	—	57	18	26	8
	60	135	41	10	3	17	6	15	5	93	29	47	14
	95	165	51	15	5	—	—	23	7	—	—	—	—
12" ID	30	77	16	—	—	12	3	—	—	—	—	—	—
	60	135	28	20	4	33	7	21	5	—	—	52	11
	95	182	38	30	7	—	—	32	7	—	—	—	—
	125	214	45	39	9	68	14	43	9	—	—	—	—
16" ID	30	97	16	—	—	16	3	—	—	—	—	52	9
	60	169	28	24	4	35	6	27	5	—	—	—	—
	95	228	38	35	6	—	—	41	7	—	—	—	—
	125	268	45	47	8	78	13	55	9	—	—	—	—

Source: Driscoll 1986.

Gravel pack wells were initially utilized because of their low cost and also a requirement to reduce sand pumping while using wide slot screens made from torch cut perforations, louvers, mill slots and punch openings. The gravel pack has to be well-rounded and typically of silica sand. There are various design criteria for the grading of the gravel pack which can vary depending upon the types of aquifers encountered and empirical sizing based upon historical success. The slot size is normally designed to hold back 90% of the gravel pack material. Wire-wrap screens are also very successful with a gravel pack envelope due to their large percentage open area. Wire wrap screens typically have an open area from 10% to 50% whereas louvers, perforations, torch slots, etc., can have open areas from 2% to 10%. Table 2.3 (Driscoll 1986) shows open areas of selected types of screens and slot sizes.

Discussion has occurred during past years regarding whether the screen and gravel pack design for an ASR well in an unconsolidated aquifer should be any different than for a normal production well. Normal ASR operations can be viewed as a long term cyclic redevelopment of the well screen, with consequent increased potential for movement and settling of the surrounding formation. A case can be made that the screen slot size for an ASR well should be slightly larger than for a normal production well, and the gravel pack as thin as practicable, such that during pumping and redevelopment, the gravel pack will clear more readily of any particulates introduced during recharge. If this approach is taken, it is necessary to add one or two gravel tubes from the ground surface to the top of the gravel pack, and to periodically confirm the depth to the top of the gravel. Gravel is then added as necessary to make up for gravel pack material washed through the screen during recovery. Otherwise, the formation material may collapse around the screen during ASR operational changes from recharge to recovery. A helpful tip is to use a

70

non-ferrous gravel feed tube to prevent the original gravel from being rust-cemented to the inside of the mild steel tube, rendering it ultimately useless for either observation or gravel re-fills.

Some uncertainty will always exist as to whether provision of one or more gravel tubes will adequately protect the well against collapse. The best solution is to design the screen identical to that for a production well so that there is no increased likelihood of gravel pack movement through the screen. Whether used for production or for ASR, a sand-producing well is an operating problem to be avoided.

Both wire-wrapped and louvered screens have been utilized in ASR wells. Wire-wrapped, continuous slot screens are more efficient and have been commonly utilized throughout the world. Louvered screens are particularly common in the southwestern states, at least partly reflecting their greater strength and ruggedness for the deep well settings and greater depths to static water level often prevalent in this region. Slotted screens have probably also been utilized for ASR wells, particularly where existing wells have been retrofitted to ASR purposes, however no specific locations for such wells are known.

It is important for an ASR well to be more efficient compared to a conventional production well. Conventional production wells tend to utilize shallower, higher yielding, fresh aquifers for water supply purposes. To avoid wellfield interference in these shallower, fresh aquifers, ASR wells often tend to use deeper, thinner, confined aquifers, sometimes with poor water quality, but providing acceptable hydraulic characteristics for recharge, storage and recovery. Where deeper confined aquifers are fresh they are often "mined" first to recover the good quality water, following which ASR wells utilize the storage volume resulting from the mining operations. Maximizing recharge and recovery rates at a single ASR well site can provide considerable overall economies in addition to reducing the requisite number and cost of well sites. The extra capital cost of providing a well with increased efficiency is more than offset by the long term performance savings.

Where there is a concern regarding strength of the wire-wrapped well screen as opposed to a louvered screen, features can be incorporated into the screen design to improve its strength, such as extra strength rods and wrap wire, or pipe based screen.

It is important to consider the compressive, tensile and collapse strength of any screen, particularly during installation and well development. For single string completions where the casing and screen are lowered into a well as a single, connected unit, the screen must be designed to withstand the compressive load from the casing unless the casing and screen are hung in the well during installation of the gravel pack and subsequent cementing. The tensile strength of the screen must be sufficient to allow the weight hanging on any particular screen joint. For both tensile and compressive loads, safety factors of from 2 to 4 have been used. The screen must also be able to resist collapse due to radial hydrostatic or formation pressures. Various criteria have been used, ranging from 0.2 to 0.5 psi per foot of depth. For example, a screen set at 1000 feet utilizing 0.2 psi per foot of depth would result in a collapse rating of 200 psi. In earthquake-prone areas the collapse rating might approach 1 psi per foot of depth, especially if liquefaction may occur.

To date, the Las Vegas, Nevada, and Glendale, Arizona, ASR wellfields are the only known applications of constructing new ASR wells in an alluvial aquifer using natural screen development instead of a gravel pack. At the Las Vegas site, many difficulties were encountered with this method due to the geologic nature of the aquifer. The semi-consolidated aquifer material did not fully collapse back upon the screen to form a continuous natural filter pack, leaving voids around the screen. All development methods were unsuccessful since any energy applied was redirected

71

up and down the bore hole following the path of least resistance. During recovery, the four wells constructed in this manner produced considerable amounts of sand at startup and continued to produce small amounts of sand during extended operation, necessitating the use of sand separators to remove sand from the recovered water. Comparisons indicated that the efficiency of such wells was substantially less than adjacent wells drilled using conventional production well completion methods with a screen and gravel pack. The results of the Las Vegas research indicate that the formation must be able to fully collapse back onto the screen to make naturally developed wells successful. Addition of a formation stabilizer between the screen and the formation is often utilized for this purpose.

With the advent of horizontal directionally-drilled (HDD) wells for both water supply and ASR purposes, it is anticipated that interest will grow in the concept of constructing naturally developed ASR wells. Installing a gravel pack in a horizontal well has yet to be demonstrated in the water supply industry although it is a known practice in the petroleum industry. The naturally developed screen will have a smaller slot size than a gravel packed screen. It is anticipated that for horizontal directionally-drilled ASR wells, a reduction in screen transmitting capacity will not be a significant constraint since screen length and diameter are more likely to be determined based upon other criteria.

Although efficient screen design is important, effective screen development is critical for achieving high efficiency for any well. Removing the drilling mud from the well using chemicals to break down the mud plus mechanical surging and jetting is a normal procedure. Getting sufficient energy through the screen to remove the drilling mud and, for naturally developed wells, to collapse the surrounding formation around the screen, is often a challenge, particularly when the screen slot size is small. For some applications use of biodegradable drilling mud will be a good solution. For other applications it will be necessary to surge and pump large volumes of water backward and forward through successive segments of the screen for an extended period such as several days or more in order to properly develop the well.

Pre-packed well screens are available to overcome the challenge of gravel pack installation in a horizontal well, or to utilize in lieu of a tremied gravel pack in a vertical well. The gravel pack material is confined between two stainless steel screens, providing great strength and also abrasion resistance. Concern exists that the surrounding formation may not be effectively developed through such a screen using conventional surge block and jetting tools since much of the development energy would be dissipated within the screen. However results to date with use of prepacked screens in naturally developed wells have been good. Advantages include a thin filter pack to develop through; a tremie pipe is not needed for addition of lost gravel pack; a smaller hole is required; the mud damage to the aquifer is closer to the screen and therefore more removable, and installation of a gravel pack is not needed. It should be noted that these advantages also apply to naturally developed screens that are not pre-packed.

For HDD wells in confined, thick, unconsolidated, brackish aquifers it is anticipated that the well screen may in many cases be best located near the top of the aquifer. During recharge the fresh water to be stored will tend to stay near the top of the aquifer, floating on top of the underlying brackish water. During recovery the HDD well will skim this water from the top of the aquifer, with minimum localized drawdown impact that could induce upconing of more dense, brackish water from below. If the interval with highest hydraulic conductivity is located near the base of the brackish storage zone instead of near the top, an alternate approach would be to utilize the higher hydraulic conductivity zone for screen location but to create a larger buffer zone around

72

the well or to trickle flow recharge water during extended storage periods. This may tend to counteract the effects of density stratification.

SCREEN MATERIAL

Well screens should be constructed of non-corrosive materials. Most ASR wells are constructed of stainless steel (SS) screens, with 316 stainless steel being more corrosion resistant than 304 stainless steel. However the unit cost of 316 SS pipe is currently (January 2005) about 55 to 60% more expensive than 304 SS while for stainless steel screens the cost differential is about 25 to 32%.

Carbon steel is generally not satisfactory for ASR well screens due to the higher potential for corrosion and deposition, which would tend to seal the screen openings. PVC and fiberglass screens are available in various configurations such as slotted pipe and a wire-wrap design.

SCREEN DIAMETER

Aquifer hydraulic characteristics, screen transmitting capacity and screen length are significant factors determining the well yield. Screen diameter is typically established during well design to achieve a reasonable range of target yields that can be sustained by the aquifer. Screen diameter may also be controlled by the pump bowl diameter for situations where the pump is set within the screen interval. The screen slot size is determined once the hole is constructed and the lithology and grain size distribution of the formation to be screened has been analyzed.

The inside diameter of a screen should be such that friction loss is not excessive for water flowing along the length of the screen. For vertical wells a velocity not to exceed about 5 feet per second might be considered. Otherwise water will preferentially flow into the top of the screened section during ASR recharge, and similarly be recovered from the top of the screened section during recovery.

For horizontal wells the friction loss along the full length of the screen is an important design criterion, requiring calculation of the head loss. It also requires comparing the increased well construction cost to provide a larger diameter and longer screen, with the associated differential in drawdown, and possibly the differential in land subsidence potential from one end of the screen to the other. This is particularly important in aquifers that are not preconsolidated and therefore are more susceptible to subsidence. In deeper aquifers preconsolidation tends to reduce subsidence potential. For cemented or consolidated aquifers, subsidence is typically not a problem.

WELL DEVELOPMENT

Key elements of well development are not unique to ASR wells, however there is an underlying emphasis upon achieving a level of well efficiency that is greater than is commonly achieved in normal production wells. Well efficiency is defined as the theoretical drawdown divided by the actual drawdown. There is a need to achieve high well efficiency because ASR wells tend to utilize deeper, thinner, confined aquifers, ranging from consolidated to unconsolidated. It is important to take additional measures to boost the recharge and recovery rates in these wells to compensate for those aquifers that are thin and have lower hydraulic conductivities.

73
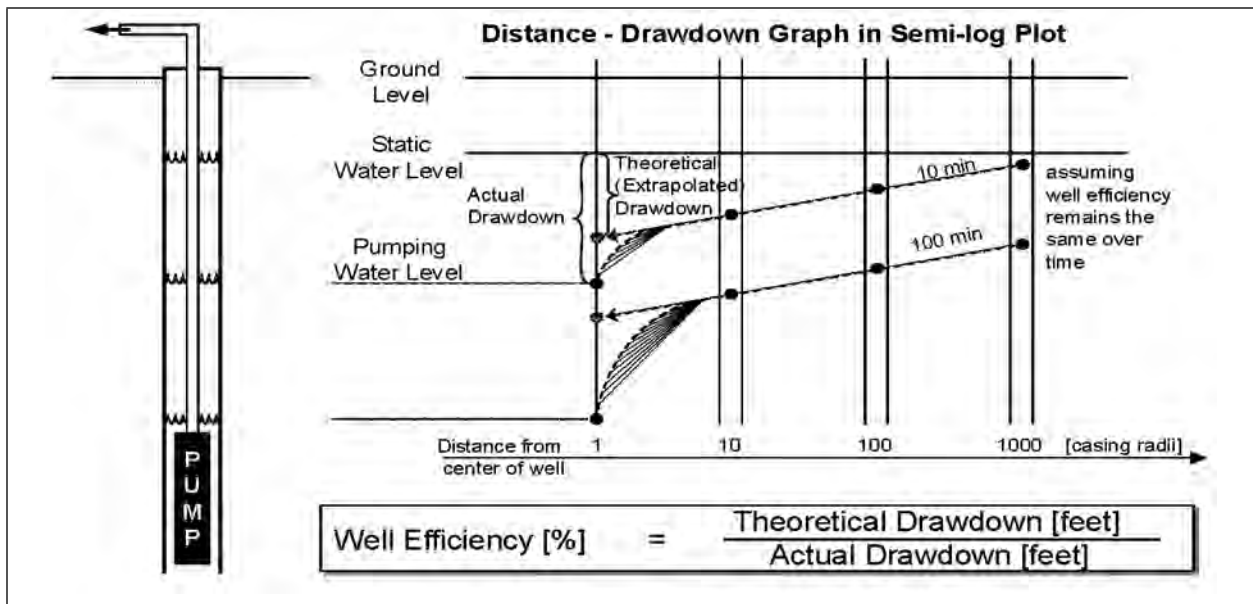
Courtesy of Hank Baski.

Figure 2.1 Definition of well efficiency

Well efficiencies typically range from 40 to 80 percent and can exceed 100 percent in some situations where the effective hydraulic radius of the well is greater than the physical radius of the screen or hole. This requires that there be no mud damage remaining. Although this is not a well development issue, well efficiency can be adversely affected if the well screen or open hole only partially penetrates an aquifer. Higher well efficiency leads to lower pumping costs and higher recharge rates. Figure 2.1 illustrates how well efficiency may be determined based upon a 24 hour pump test where drawdown measurements can be obtained from two observation wells located approximately 50 feet and 500 feet from the pumping well. If a single observation well is used, it should be located 1.5 to 2 times the aquifer thickness from the pumping well. Well efficiency changes with duration of pumping so it is customary to establish a target well efficiency, such as 75% or greater, associated with a given duration of pumping, such as 24 hours.

A related issue is the location of the pump intake within the screen to reduce total drawdown. For a well with uniform production per foot of screen, location at approximately the middle of the screen tends to achieve more of a balanced distribution of head loss from the top to the bottom of the screen, compared to the normal approach of having the pump intake above the top of the screen. The result is a reduction in total drawdown, thereby improving well efficiency. Otherwise the highest entrance velocities will tend to be at the top of the screen, diminishing toward the bottom of the screen. Increasing the screen diameter and thereby reducing entrance velocities tends to minimize this problem, at some increase in well construction cost.

Well development measures typically include mechanical methods such as surging, pulsing and bailing; airlift development; high pressure jetting with water or air; sonic jetting; and addition of chemicals to disperse the drilling mud and clays. Other measures that have also proven effective include long term surging and pumping to move large volumes of water back and forth through the well screen, sometimes for periods of several weeks for a long screen. The effectiveness of a well development method may be judged according to the amount of sediment and fines

74

produced. If clear water is produced, no improvement in well efficiency is being made. This does not necessarily mean that the well is efficient since the development method may not be effective.

For most well development procedures it is important to start well development with a gentle process and with low energy. This avoids the potential for locking up the formation and also reduces the potential for collapsing a screen. As the water moves more freely into the screen during the well development process, higher energy levels are used. It is important to look at well development from an energy viewpoint. Higher energy rates in shorter intervals normally result in more effective well development. Energy may be in the form of mechanical, such as used in surging; pneumatic, as used in air-lifting; chemical, as used in dispersing agents to break up drilling mud, or acidization of carbonate aquifers; hydraulic energy, as used in jetting; heat, as used to speed up chemical reactions, kill bacteria, and make chemicals more effective; and pulsing or vibrational energy, as used in sonic, ultrasonic, explosive or other pulsing methods.

In the future it is likely that combinations of well development methods may be utilized, such as airlifting plus surge blocks; or jetting plus surge blocks, plus pumping, plus chemicals, plus sonic.

Periodic backflushing of an operational ASR well is typically required. By controlling recharge pressures so that they do not become excessive, it is usually possible to remove accumulated particulates from the ASR well during recharge periods. This pumping period, typically 10 minutes to 2 hours' duration, effectively redevelops the well and maintains its recharge capacity.

Figure 2.2 shows the distribution of head loss through the screen, the gravel pack and the developed and undeveloped portions of the surrounding aquifer for best case and worst case conditions of well development. Clearly it is desirable to design and construct ASR wells to keep the fines from penetrating sufficiently deep into the aquifer, or under such recharge pressure, so that they cannot be readily removed by pumping and well development.
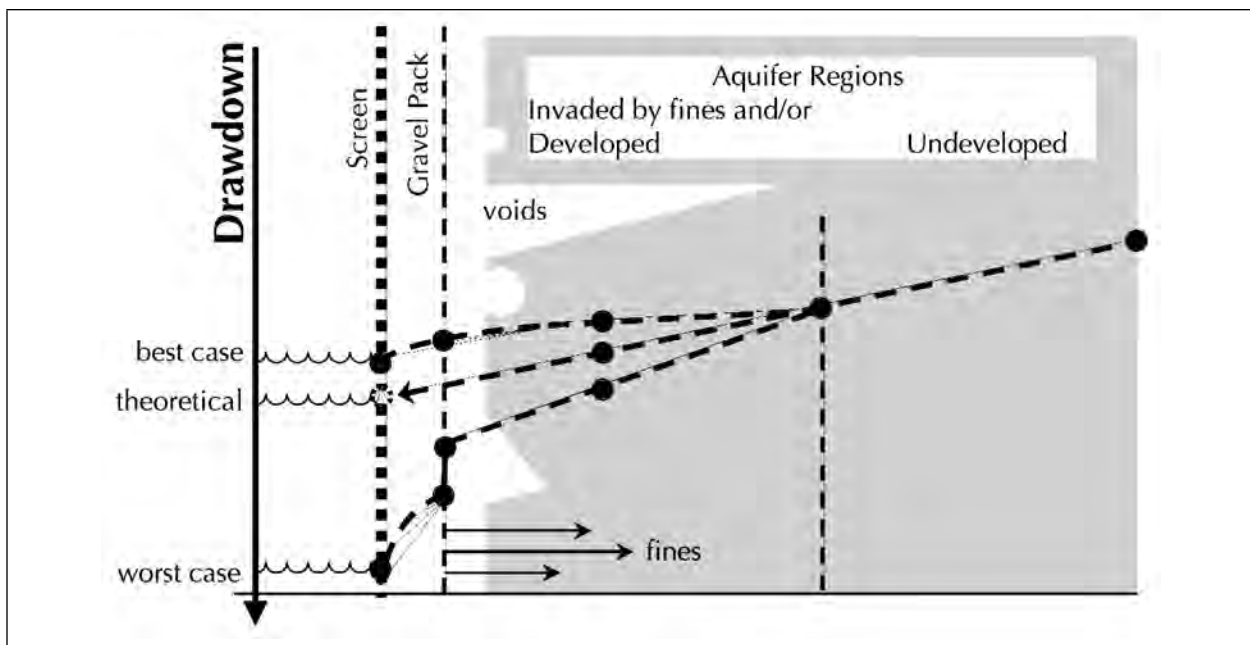
Courtesy of Hank Baski.

Figure 2.2 Head loss from screen, gravel pack, and developed and undeveloped aquifer regions

HORIZONTAL DIRECTIONALLY-DRILLED ASR WELLS

No HDD ASR wells currently exist, however it is anticipated that within the next few years this well design will become more common. The principal advantage is the ability to achieve higher flow rates from a single well, particularly in relatively thin, shallow aquifers with drawdown limited by either hydraulic or subsidence constraints. The longer well screen may be several hundred feet to over 1000 ft long, effectively developing a much greater yield with the same amount of drawdown compared to a vertical well. Theoretical yields may be as much as three to eight times the yield of a corresponding vertical well, or more. In practice the relative increase in yield is probably less but still significant, more than offsetting the increased well construction cost.

A second and potentially important application of HDD technology for either ASR or water supply wells is in areas where upconing of brackish water from beneath a vertical well may limit the rate or duration of groundwater production. This has been a challenge at a few ASR sites in aquifers with poor underlying confinement, and where the storage zone overlies a saline aquifer. Installing an HDD well instead of a vertical well may be able to achieve the desired recharge and recovery flow rates with minimal upconing of brackish water since drawdowns are minimized.

The underlying HDD technology already exists, having been developed in the petroleum industry in the 1940s, adapted to the utility pipeline industry in the 1970s, and adapted to the environmental remediation industry in the 1990s. Adaptation for ASR and water supply purposes will entail refinements of existing technology to provide for larger well diameters, greater yields and shallower depths than those typically encountered in the petroleum industry where this technology started.

Water utility experience to date includes probably fewer than ten relatively small diameter wells and relatively shallow depths, typically less than 200 feet. Since about 1992 horizontal wells have provided small water supplies, about 40 gpm, from two wells for Squaw Valley, California. These are 5" wells drilled at a slight incline less than 200 feet deep into the side of a mountain, intersecting groundwater flow in the fractured and weathered bedrock. Other such wells exist. Probably one of the first high capacity HDD wells was a 12-inch directionally-drilled horizontal collector well constructed along the bank of the Raccoon River at Des Moines, Iowa in 2000. That well produces about 1,700 gpm. Three earlier HDD water supply wells were successfully constructed at Aiken, South Carolina in the early 1990s, using 8" casing and 5" screen. Since then HDD wells have been constructed for water supply purposes in San Antonio, Texas; Antelope Hills, Colorado; Castle Pines, Colorado; Nekoosa, Wisconsin, and possibly other locations in the United States. HDD wells have also been constructed for dewatering purposes in California and for grouting of dams in various locations.

The need for HDD well construction is becoming evident in many parts of the country as a result of the rapidly increasing complexity and cost of obtaining suitable sites for water supply wells. Environmental constraints upon groundwater production combined with legal and regulatory constraints upon well construction and water use have placed a significant premium on the siting of wells. In urban areas suitable sites are often difficult to find due to urban development and limited infrastructure to convey water at high rates to and from a site, and to dispose of waste flow discharges. Wherever it is possible to construct one or more HDD wells on a single site, with each well potentially achieving several times the production rate of a vertical well, the benefit will probably in many cases substantially exceed the associated additional cost.

Several other types of horizontal wells are in use. This brief discussion of HDD technology would be incomplete without some mention of an alternative approach that involves construction of vertical shafts intersecting subsurface horizontal tunnels from which collector wells may be radially jacked into place. Such a system is planned for Cincinnatti, Ohio, to be used as a bank filtration system for the City's raw water supply.

Similar systems exist at hundreds of sites in the United States and Europe, termed "horizontal collector wells" or "Ranney wells." These wells are typically used for water supply and bank filtration purposes, however in some cases they have also successfully been used for aquifer recharge. Recharge examples include Louisville, Kentucky; Canton, Ohio, and Manitowoc, Wisconsin. With these types of wells, a central caisson is constructed, typically about 13 ft to 20 ft (4m to 6m) inside diameter and up to approximately 150 ft (46m) deep, depending upon the local geologic setting. Construction material is usually reinforced concrete. Radial horizontal collectors up to approximately 300 ft (100m) long are then jacked or drilled into place, often extending beneath the water supply source. Individual well yields have ranged from 0.3 to 40 MGD (1 to 151 ML/day). This technology has been in use in Europe and the United States since the 1940s and possibly earlier.

Another category of horizontal wells includes infiltration galleries in shallow alluvial aquifers, which are constructed by digging a trench to a typical depth of about 20 feet and simultaneously installing a screen and gravel pack. When constructed close to a stream, such wells provide a bank-filtered supply of surface water that in many cases may be suitable for ASR recharge into a deeper aquifer. Effective seasonal management of the storage volume in the alluvial aquifer can augment the annual storage volume and potential recovery rate and duration from the ASR well.

DESIGN OF ASR WELLHEADS AND DOWNHOLE EQUIPMENT

Several features of ASR wellhead and downhole completion should be considered for efficient, long-term operation of an ASR system. That is not to say that disregard of these features will cause system failure. Rather, it is probable that consideration of these features will greatly enhance system performance. The various features are discussed in the following paragraphs.

MATERIALS OF CONSTRUCTION

As discussed previously in this chapter, materials of construction play an important part in ensuring efficient and cost-effective ASR system performance. For both wells and wellhead facilities, the same principles apply. Non-ferrous piping systems such as PVC, HDPE, stainless steel and cement (or fusion bond) lined ductile iron pipe are preferred for ASR wellhead piping, particularly in systems where aquifer permeability is low to moderate and some concern exists regarding plugging and redevelopment frequency. Where this is implemented, the volume of rust carried into the well during recharge and from the well during initial recovery will be greatly reduced. Furthermore, smooth piping will reduce the opportunity for solids entrapping near the wellhead. These factors, in turn, will tend to reduce the plugging rate, redevelopment frequency, and regulatory issues associated with disposal of backflush water to waste. Any increase in initial capital cost will offset higher long-term operating costs associated with handling of initially rust-colored, turbid recharge and recovery flows.

77

For downhole piping such as column or riser pipe, injection tubes, pressure transducer tubes and gravel feed tubes, PVC, stainless steel, hose or other non-ferrous materials are also recommended. Galvanized tubing should be avoided where pH is below nuetral, because of adverse experience with failure due to electrochemical corrosion. In particular, galvanized surfaces create a corrosion cell with non-galvanized surfaces, causing reduced service life and screen plugging. Where corrosion is anticipated, it can be partially inhibited with use of protective coatings and cathodic protection. However, this requires maintenance for continued operation and also requires additional space within the casing to accommodate the cathodic protection system.

In recent years synthetic flexible hose has become available for use as column or riser pipe for small-sized ASR installations. Hoses and hose connections are now available in diameters up to about six inches and have been used successfully in the mining industry for several years. These provide abrasion resistance, low friction loss, ease of installation and no corrosion potential. Fountain Valley, Arizona, utilizes hoses for their ASR well installations.

An important consideration relating to selection of PVC pipe or hose for use as column or riser pipe is the tendency of the pipe or hose to elongate due to pumping pressure, and also to rotate as a result of the pump starting and stopping. If other conduits or cables are attached to the column pipe, such as a submersible pump electric cable or a bubbler tube, they will also rotate with the column. In situations where the design does not anticipate this rotation, failure can occur. An inflatable packer can be used to control such rotation tendency. Using rigid column pipe, or restricting these applications to ASR wells with small flow rates, addresses these concerns. Mt. Pleasant, South Carolina, has two ASR wells that have operated successfully for many years with PVC column pipe. Recovery flow rate for the more productive well is 575 gpm. The pump and 25 HP submersible motor are set at a depth of 160 ft.

An important consideration related to materials of construction is the joint strength for downhole column pipe connections. A joint strength of four times the hanging weight is recommended. Joint strength is a function of the material yield strength, the remaining wall under any threads or grooves after allowing for corrosion losses, and the thread type. American Petroleum Institute (API) threads should be specified, not National Pipe Taper (NPT) threads, for settings over 500 ft. Steel threaded connections tend to gall. To prevent this, the threads may be coated or provided with a sealant. For stainless steel pipe they may be provided with threaded Nitronic 60 couplings or flanged couplings.

In general, use of non-metals, epoxy coatings, cement-lined piping, stainless steel, and special alloy materials of construction in ASR wells is usually a wise investment. The economic savings usually attributable to implementation of ASR justifies reasonable investment in the design of the well equipment and wellhead facilities.

PIPELINE FLUSHING AND WASTE FLOW DISCHARGE

Regardless of the materials of construction of the well and wellhead piping, the transmission and distribution system conveying water to the wellhead may have deposits of solid material trapped in places such as connections, valves, and fittings. Pressure surges and reversal of flow in this piping due to startup of recharge operations can re-suspend these particulates, transport them through the piping and carry them down the well, causing rapid plugging.

For this reason, it is advisable to flush the wellhead piping to waste immediately prior to recharge, with the flow in the direction of the recharge flows, and at a flow rate at least at the highest flow rate expected into the well. Wellhead piping must be designed with the ability to

78

discharge waste flows at high rates, frequently to a ditch or storm sewer line near the wellhead, or to the water treatment plant for retreatment. The duration of this flushing period may range from 10 minutes to as long as 2 hours or more, depending upon the site, the amount of solid material in the recharge water, and the length of pipeline. An air gap of at least 1.5 ft is usually provided in the waste discharge line. This will prevent potential contamination of the ASR well due to reverse flows during ASR recharge, caused by siphoning.

Similarly the ASR well should be pumped to waste for a few minutes prior to recharge, to purge any solids that may be present downhole. The duration of pumping can be determined through early operational experience and typically depends upon the materials of construction. For wells designed with PVC, fiberglass or stainless steel casings, the need for and duration of this procedure is probably reduced.

In many situations where the ASR well is supplied by a long, dead end transmission pipeline, disposal of the resulting water from line-flushing operations sometimes can be difficult due to the large volumes and high rates involved. Furthermore, disinfecting new pipelines by filling them with chlorinated water and then draining them to waste at a low flow rate does not remove solids from the pipeline. The solid material will be carried into the well during the first recharge operation. Mechanical cleaning of the pipeline, such as by "pigging," can be helpful to remove solids in such situations.

Where the ASR well is supplied by a new long cement-lined pipeline, high pH values may occur during initial recharge testing, due to grout and cement curing in the pipeline. At one site, this caused pH values to exceed 9.0 in the recharge water. Such an effect should be considered transitional until cement curing is complete.

Provision should be included in the wellhead design to isolate each well from the wellfield and flush it to waste at the wellhead so that remedial work can proceed while the remainder of the wellfield is in normal operation. Otherwise, the entire wellfield may need to be shut down during periodic backflushing operations. This is particularly important in situations where backflushing is expected to occur more frequently than a few times per year. For large ASR wellfields, it may be sufficient to be able to isolate groups of adjacent wells for backflushing operations.

TRICKLE FLOWS

Whether in the surface piping, wellhead piping, or well casing and screen, stagnant water is to be avoided. During periods of neither recharge nor recovery, it is advisable to maintain a trickle flow of chlorinated drinking water into the well. This can be provided through small-diameter tubing conveying typically 2 to 5 gpm (8 to 19 L/min) down the well. In addition to the tubing, a small flowmeter and a valve are suggested, bypassing water around any isolation valve at the wellhead that prevents recharge flows. The required rate of this trickle flow can be easily calculated by monitoring the rate at which a chlorine residual in the recharge water dissipates in the well or in the wellhead piping at the end of a recharge period. Typically, a residual is maintained for up to one or two days, occasionally longer. Maintaining a small chlorine residual in the well during storage periods, equivalent to that in drinking water, prevents bacterial growth in and adjacent to the well, thereby reducing the potential for bacterial plugging. A chlorine residual below about 1.5 mg/L should ensure that metal corrosion in the well is acceptably small. The need for a trickle flow to control subsurface microbial activity in the well may be related to latitude. Cooler water temperatures at more northerly latitudes reduce the microbial activity rate.

79

A second reason for providing a trickle flow of recharge water during idle periods is applicable particularly in very brackish and saline aquifers. Maintenance of freshwater in the casing permits use of pump materials that are less expensive than those that would be necessary for a well in which water quality can change with extended pumping from fresh to brackish or even to seawater.

For long lengths of transmission piping, the trickle flow at 2 to 5 gpm (8 to 19 L/min) may be insufficient to maintain a chlorine residual in the transmission piping to the well, in which case the residual is lost in the well. In such situations, one alternative is to provide a small chlorinated water feed using low-flow chlorination facilities provided at or near the wellhead for treatment of recovery flows. Another approach is to periodically slug the well with a large volume of recharge water during storage periods, sufficient to provide a residual of chlorinated water in the well and surface piping. The frequency of such an operation would be site specific, but probably every few days. For short storage periods, this practice may be considered acceptable. For longer potential storage periods, wellhead chlorination of recharge flows may be a better choice.

In applications involving a deep static water level it may be necessary to introduce the trickle flow into the well through small tubing or PVC pipe (1-inch) strapped to the riser column pipe. Experience in Las Vegas, Nevada showed the need for a small injection pipe to below the static water level. Water at a low flow trickling directly into large diameter well casings with deep water levels evaporates, leaving a scale of calcium carbonate on the casing wall with a thickness exceeding 3 mm. Sometimes extending over 100 meters (300 feet), the thick scale spalled off in sheets and had to be wire brushed every 5 to 7 years.

SAMPLING TAPS

Sampling taps should be provided on the side of the pipeline at the wellhead to permit sampling of both recharge and recovery flows. They should be suitable for collection of bacteriological samples and therefore should be non-ferrous material such as bronze. For the same reason, they should also be unthreaded at the discharge end, preventing ferrous or other contaminants that catch in the threads from contaminating the sample.

At some locations, special provisions may be necessary to convey sample flows away from the wellhead and piping in order to avoid ponding, iron staining, rusting or algae formation. A drain for a short distance is usually sufficient.

Care must be taken to ensure that the sampling tap is located at a point of positive pressure. During recharge, negative pressures can develop at the wellhead if flow is insufficient to maintain positive pressure in the well or injection piping. If the sample tap is located downstream of the last control valve at the wellhead and negative pressure develops, it will not be possible to obtain samples. Furthermore air would be pulled into the flow stream and would be conveyed into the aquifer. This can be resolved by installing the recharge sampling tap upstream of the last control valve on the recharge line.

Pressure gauges should not be connected to sampling taps. Where both are connected to the same tap in the pipeline, pressure gauge readings will tend to be erroneous during sample collection. Although proper valving can prevent this problem, it is wiser to avoid it by installing separate taps for each purpose.

If the recovered flows will be disinfected at or near the wellhead, or if other chemical addition is planned, such as pH adjustment, an additional sampling tap may be required sufficiently downstream of the chemical feed point so that a representative sample is obtained.

80

For some situations the possible production of sand in the recovered water is anticipated. For accurate sand testing, a port should be located within a half meter (18 inches) of the well discharge head at an elbow or tee. This location provides a high likelihood of turbulent water and a good representation of sand load and turbidity. Beyond this point, the water quickly stratifies, settling the sand to the bottom of the pipeline, making sand test data inaccurate. Equipping the port with a standard hose bib rotated 90 degrees allows for quick equipment installation and operation. A smooth end hose bib should be provided for microbial sampling. Provisions should be made for a drain to dispose of the continuous flow discharged from the testing equipment.

DISINFECTION OF RECHARGE AND RECOVERY FLOWS

Water recharged to and recovered from most ASR facilities meets drinking water standards and can be used following disinfection. However, there are certain disinfection considerations that should be considered during design.

Recovered flows are typically disinfected with chlorine. Chlorine gas is used in many applications because it is readily obtainable and transportable to most sites or is manufactured on site. Liquid chlorine such as sodium hypochlorite can be used; however, larger volumes are required and the disinfectant properties degrade with time. For either approach, adequate facilities for storing and handling the chlorine must be in place since it is highly toxic. These are usually defined in local and state regulatory requirements.

When chlorine gas is added to water, the pH will decrease to some extent, depending on the chlorine dosage, the alkalinity of the water, and the blend ratio between ASR recovered flows and those from the water treatment plant or other source. In some cases, the pH decrease can be sufficient to produce aggressive or corrosive water. While the probable extent of the pH decrease can be estimated during design, it is necessary to confirm the actual decrease following construction and operational testing. Provision should be included in the design to incorporate locations for chemical addition to raise the pH, if needed at a later date. Chemicals may include ammonia for chloramine formation, or base chemicals such as sodium hydroxide to raise the pH.

Nitrification tends to occur during ASR storage, converting ammonia in the recharge water to nitrate in the recovered water. During subsequent disinfection the chlorine demand of the recovered water will be impacted by several factors, including the residual ammonia concentration. The rate of nitrification will depend upon total organic carbon concentration, temperature and other factors, however during extended ASR storage for several weeks, ammonia reduction in excess of 50% may be expected. The nitrification process tends to lower the pH of the stored water. Where TOC levels are sufficiently high, such as with ASR wells storing highly treated reclaimed wastewater, denitrification has also been observed during aquifer storage, converting nitrate to nitrogen gas.

Upon completion of construction and testing of ASR facilities, field adjustment of disinfection dosages for chlorine and ammonia will probably be necessary so that recovered water can be added to the distribution system. This testing often includes determination of chlorine demand for the recovered water, along with analyses for total and free chlorine residual, total organic carbon, temperature, pH, total ammonia, nitrate, hydrogen sulfide and possibly heterotrophic plate counts. This list is usually in addition to whatever constituents may be required as tracers (i.e., conductivity, TDS, chlorida) so that dilution effects between recharge water and ambient groundwater can be distinguished. The disinfection system is designed based upon reasonable assumptions regarding chlorine demand. However these assumptions must then be verified in the field.

For drinking water stored and recovered from ASR wells it is usually only necessary to boost the disinfectant residual in the recovered water to meet regulatory criteria. Primary disinfection, including adequate contact time, is assumed to have been provided already at the water treatment plant.

DOWNHOLE FLOW CONTROL

This is one of the more important elements of ASR well equipment design and requires some care. Cascading occurs when the water level in the recharge piping does not rise to ground surface during recharge. Allowing water to cascade down the well can lead to plugging problems due to air-binding in the storage zone, and induced geochemical or bacterial activity. Cascading can also cause structural problems due to cavitation damage to pipes, valves, and fittings. Downhole flows need to be controlled in order to avoid these problems, each of which causes plugging of the ASR well. Plugging usually can be reversed; however, it requires considerable time and effort.

Water can be introduced into a well through the pump column, the annulus between the pump column and the casing, one or more injection tubes inside the casing, or some combination of these approaches. It can be introduced under pressure or under vacuum, and it can be controlled from the wellhead or from the bottom of the iinjection piping. One approach that is utilized frequently is to recharge through both the pump column and the annulus, providing operating flexibility and maximum recharge rates. Selection among these alternatives is based upon consideration of several factors, principal among which are the following:

- casing diameter
- static water level in the well
- type, size, and capacity of the pump
- specific capacity and specific injectivity of the well
- expected production rate and range of injection rates
- above-ground facility constraints and opportunities, such as presence of an adjacent ground storage reservoir

Some of this information may not be available at the time the design is completed, thus creating the need for a flexible design approach capable of accommodating a reasonable range of expected conditions. It is usually wiser to construct and test the ASR well to determine hydraulic performance characteristics before finalizing design of the wellhead facilities. This requires more time; however, it leads to better results. Provision of flexibility is still advisable since recharge rates can sometimes drop below planned rates and cause unplanned cascading.

For all well recharge methods to control downhole flow, the downhole restriction flow characteristics can be described by a simple equation:

$$Q = C_v H^{0.5}$$

where
Q = the flow rate in gallons per minute
C_v = a flow coefficient

H = the driving head, defined as the pipeline pressure plus the distance down to the static water level in the well, minus the friction loss of the water going down the column pipe.

For example, for a downhole restriction with a C_v of 100 and a driving head of 100 feet, the flow rate would be 1,000 gpm.

There are two basic approaches for downhole flow control: non-adjustable downhole flow restrictions and adjustable downhole flow restrictions. Both of these are addressed in the following pages.

Non-Adjustable Downhole Flow Restrictions

The use of a non-adjustable downhole flow restriction, especially if a vertical turbine pump column is used for recharge, is low cost if the injection pipe can be kept full of water. Also it is important that cascading and air entrainment during initial filling and at the end of a recharge period do not cause serious problems. There are several options for achieving this objective.

Annulus Recharge

Water may be recharged down the annular space between the well casing and the pump column. To maintain positive pressure at the wellhead and thereby prevent cascading, it is necessary to ensure that sufficient flows are always available for recharge. When recharge flows fall below this critical rate, cascading will occur and a vacuum will develop in the annulus and wellhead piping. Air will be drawn into any open air relief or vacuum breaker valves, any leaks in the upper portion of the casing or pump column or elsewhere in the wellhead assembly. This air will be carried down the well into the formation, where it will tend to plug the well. It can happen due to reduction in recharge flow rate or due to local or regional lowering of static water level in the storage zone.

A flexible solution is to seal the annulus at the wellhead and to ensure that any wellhead valves that are connected to the annulus are closed during recharge. In this way, recharge can occur regardless of wellhead pressure or vacuum, and under a full range of recharge rates, thereby maximizing recharge volumes. This is not an operator-friendly recharge approach since it is often too easy for wellhead air release/air vacuum valves to be left open accidentally, or to leak, causing air entrainment and resultant well plugging.

A disadvantage of this approach is that water flows over a substantial surface area of casing that is alternately wetted and dried. Therefore, annulus recharge in steel casings has a high potential for production of rust that can contribute to plugging the well during recharge and create regulatory problems during backflushing and initial stages of recovery. For new ASR wells, this can be avoided by selection of a non-ferrous casing or coating where possible.

A second disadvantage of this approach applies to retrofitting of existing wells for ASR purposes, particularly those with long casings into low or moderate permeability aquifers. Where the quality of well construction is unknown or suspected to be poor, it is possible that the casing joints or pump base may not be sealed adequately. Recharge would therefore entrain air even if the wellhead piping and control valves were sealed and closed, respectively. This can be checked by installing a temporary packer in the well and pressure-testing the casing with liquid to determine if it will hold a given pressure for at least 30 minutes. This is also referred to as "mechanical

83

integrity testing." Alternatively, a brief recharge test can be conducted at a low rate in the supposedly sealed annulus, sufficient to create a wellhead vacuum. Recharge is then shut off and the vacuum monitored to see if it will hold for 30 minutes.

A related issue is that pressure surges have been known to occur in the recharge piping of some ASR wells. In other ASR wells, recharge occurs at higher pressures anyway, to overcome high static water levels or to overcome density differences in saline aquifers. In such situations, the pump bases should be designed to withstand expected operating and transient pressures without leaking at the connection to the casing. A flanged connection between the top of casing and the pump base, machined to ensure flat, parallel surfaces and sometimes provided with a circular groove and o-ring, can provide the required degree of sealing.

Recharge down the annulus of wells equipped with submersible pumps requires care to ensure that the electrical cable port in the wellhead flange is adequately sealed to prevent air entry during vacuum recharge or to prevent leakage during pressure recharge.

Several variations on this annular recharge approach are possible. Recharge could occur down the annulus at sufficiently low velocity below the water level in the well that any entrained air has the opportunity to bubble out before reaching the formation. This approach was used successfully at Goleta Water District, California, during initial ASR operations approximately 20 years ago, however more recent experience at higher recharge rates at Goleta has led to air entrainment during recharge and foam in the recovered water. No known existing ASR sites currently utilize this low velocity approach. A downhole water velocity in the casing below the pump would have to be less than the air-bubble rise rate, or about 0.3 to 0.4 m/s (1 to 1.3 ft/sec) for air bubbles with diameters of 0.1 to 10 mm (0.004 to 0.4 inches).

Another variation is to cease recharge when cascading begins, whether due to static water level decline or due to reduction in recharge flows. Such an approach requires a degree of operating attention that is frequently not available. Larger ASR wellfields with computer-controlled operations and telemetered monitoring parameters can build this into their control systems; however, smaller systems are more likely to continue recharge regardless of whether cascading is occurring or not, with resultant reduction in recharge rates due to plugging by air bubbles and/or geochemical or microbial activity.

Methods to increase friction loss in the annulus occasionally have been considered or tried. These have included sizing the pump to minimize the annular space between the pump and the casing; addition of flanges at the couplings in the pump column; and other novel approaches such as adding floating objects such as ping pong balls in the annulus. Each of these approaches has the same drawback. It is sometimes difficult to place in the well, or retrieve from the well, a tightly fitting object as well casings are not always straight, plumb or round. Also, objects that are not tight-fitting will probably not provide much resistance to flow.

Recharge through the annulus entails careful consideration of the materials of construction for the casing and pump column in order to minimize corrosion. The surface area exposed to alternating wetting and drying can be substantial so carbon steel casings, in particular, are less suited to this recharge method.

Dual-discharge heads are designed to nest an inner and outer column pipe flowing to two distinctly different pipeline connections. The inner pipe represents the pump column and the outer pipe the injection column. The outer casing can be carried down to sufficient depth in the well to minimize the cascading effect during startup. These wellheads allow for simple connection of annular recharge with existing pumping equipment.

84

Several ASR sites utilize annulus recharge. Among these are Cocoa and Peace River, Florida, and Chesapeake, Virginia. The first two of these sites store drinking water in brackish aquifers where the depth to static water level is about 3 m (10 ft) below land surface, and 7 m (22 ft) above land surface, respectively. At Chesapeake, the aquifer is slightly brackish with a depth to water level of about 20 m (66 ft). The first two sites store water under slight pressure while Chesapeake recharges either under pressure or with a vacuum in the annulus, depending upon the flow rate. The Marathon ASR site in Florida utilized annulus recharge to a seawater aquifer with a static water level that varied above or below ground surface, depending upon the water density in the well.

Injection Tube Recharge

One or more injection tubes are sometimes used to control cascading during recharge. The small diameter tubes provide sufficient head loss at high flow rates that the water column is under positive pressure inside these tubes. For instance, a 2-inch (inner diameter) clean, new steel injection tube flowing at 18 L/sec (280 gpm) provides a friction loss of about 1 m for every meter of length. Table 2.4 shows friction losses for small diameter tubes at several flow rates, assuming a Hazen-Williams friction factor of 160, representative of new, smooth pipe. When available recharge flows exceed the capacity of one tube, a second tube may be opened. Two differently-sized tubes can cover a broad range of potential flow rates by operating separately or together. For different materials and different surface roughness conditions, higher friction losses occur, with consequent lower flow rates. For example, badly corroded steel pipe might have a friction factor of 60.

Several references provide tables listing the flow rates and friction losses for various pipe sizes and roughness characteristics, such as Driscoll, 1986.

Flow in such pipes may be calculated by the empirical Hazen-Williams equation:

$$Q = 1.318 C (R_h)^{0.63} S^{0.54} A$$

where
Q = the flow rate, in gallons per minute
R_h = the hydraulic radius of the pipe (area divided by perimeter), in inches
S = the slope of the energy grade line, or head loss caused by friction divided by the pipe length, dimensionless
A = the pipe cross-sectional area, in square inches
C = the roughness coefficient

The Hazen-Williams formula may be reformulated to estimate head loss per foot of pipe, as follows:

$$h = 10.44 \left[\frac{Q^{1.85}}{cd^{2.63}} \right]$$

where
h = head loss
d = pipe diameter, in inches

85

Table 2.4
Pipe friction losses

Pipe Diameter (inches)	Flow Rate (gpm)	Velocity (ft/sec)	Head Loss (feet/100 feet)	
			C = 120	C = 160
1.5	50	9	29	16
	100	18	104	61
	150	27	219	129
2	150	15	54	32
	200	20	92	54
	250	26	139	82
2.5	200	13	31	18
	300	20	66	39
	400	26	112	66
3	300	14	27	16
	400	18	46	27
	500	23	70	41
	600	27	98	57
	700	32	130	76
4	600	15	24	14
	800	20	41	24

An advantage of the use of an injection tube is that it is simple. Positive pressures can be maintained at the wellhead over a wide range of flows. Furthermore, the surface area in contact with the water is small, substantially reducing generation of rust during wet-dry cycles associated with recharge and recovery.

A disadvantage of this approach is that existing wells rarely have sufficient room within the annulus to add one or more injection tubes, in addition to the already existing pump column, power cable (for submersible pumps), air line or other water level measurement device. For new wells, the cost of oversizing the casing in order to provide sufficient room for the pump column, plus the injection tubes and water level measurement tube can be substantial, particularly for deep wells.

A second disadvantage is that if available recharge flow exceeds the injection tube capacity for some period of time, this additional flow cannot be recharged without manually adjusting the wellhead to utilize a second injection tube, if a second tube is provided. Conversely, if available flow falls below the capacity of the injection tube in use, cascading will occur, requiring that the injection tube and wellhead be designed to maintain a vacuum.

Injection tubes are typically small, tightly-jointed, and can maintain a vacuum. Over 225 injection wells in the Los Angeles salinity barrier have successfully used injection tubes for recharge for over 30 years. Many of these wells recharge under a vacuum. However none of them have pumps. When the salinity barrier wells become plugged, redevelopment requires disassembly of the wellhead, removal of the injection tubing from the well, special redevelopment procedures of the well screens, followed by re-assembly of the downhole and wellhead piping.

Injection tubes are most applicable in situations where operations are controlled by telemetry and where recharge flow variations can be met by adding or deleting ASR wells from operation. In this way, the relatively narrow flow range of individual wells is not a substantial constraint. Injection tubes are also most applicable where recharge is into fresh water aquifers, or brackish aquifers where a substantial target storage volume has already been developed so that there is less need to carefully balance recharge volumes between several ASR wells in a wellfield. Most utilities utilizing injection tubes will let recharge continue at variable rates, regardless of changes in available recharge pressure or vacuum.

Such an approach may require careful development of an operating plan for those situations where the storage zone is brackish or otherwise contains water of unacceptable quality. About a third of all ASR wells in the United States are in brackish aquifers, and many of the remainder are storing high quality water in aquifers that contain water quality with at least one constituent that would require treatment to achieve drinking water standards. Design of ASR wellfields usually should attempt to balance water storage among the ASR wells so that hydraulic interference does not move the storage bubbles away from each well. Adding or deleting ASR wells in such situations to respond to variations in recharge rates may contribute to reduced recovery efficiency unless the wells added and deleted are selected with care. This concern is reduced if the wellfield is configured in such a way that the storage bubbles from all the ASR wells have coalesced into a single large bubble that extends beyond the perimeter of the wellfield.

When injection tubes are used, they should extend below the lowest expected static water level in the well. They sometimes utilize an orifice plate at the bottom to increase friction loss. Changing the orifice plate consequently changes the head loss in the injection tube if this becomes desirable. To further dissipate head and to protect the well and screen from the effects of prolonged high-speed jetting, it may be desirable to install a short screen, shroud or bucket assembly at the base of the injection tube to deflect flows laterally or vertically.

As with other non-adjustable downhole flow restriction methods, injection tubes will allow cascading and air entrainment at the beginning and end of any recharge cycle, even if no cascading occurs during the cycle. Each site should be evaluated as to whether the volume of air carried into the aquifer is likely to be a problem.

An example of injection tube ASR applications is at Kerrville, Texas, where the injection tubes are stainless steel to a depth of 61 m (200 ft) in order to remain below the static water level in the well. This is the primary method of recharge; however, the well is also equipped for annulus recharge at higher flow rates.

Pump Column Recharge

Recharge through the pump column may occur with either vertical turbine lineshaft pumps or with submersible pumps.

Vertical Turbine Pumps

Recharge through vertical turbine pumps has been implemented at many ASR sites. The head loss generated by reverse water flow through the pump is usually sufficient to control cascading. Another way to look at this is to recognize that running water backwards through the pump bowls with the impellers locked causes substantial head loss. Several pump manufacturers have generated coefficients for injection, C_v, plus preliminary estimates of reverse flow rate

through their pumps when operating in an ASR recharge mode, however to date these flow rate estimates tend to be higher than subsequent actual performance. A non-reverse ratchet should be installed on the electric motor to prevent backspin of the pump and motor during recharge.

If head loss through the pump column and bowls is insufficient and cascading still occurs, a vacuum will develop in the upper part of the column. This may draw air into the well through the column coupling threads and also at the lineshaft stuffing box, particularly in existing wells where the condition of the pump column may not be known. The resulting cavitation may potentially damage the column and lineshaft. Vertical turbine line shaft pump columns typically use straight threads and various tapers of threads. These connections are subject to potential air leakage. Stuffing boxes also are designed to leak.

The potential development of a vacuum in the column is of particular importance for oil-lubricated pumps since the vacuum can draw oil into the recharge water, thereby contaminating the water. The best solution is to avoid this potential occurrence by not utilizing oil-lubricated pumps in ASR wells. Alternate methods include use of food- grade oil for lubrication, or making special precautions to avoid developing a vacuum in the pump column. Use of new column pipe is also advisable to minimize the likelihood of leaks at the threaded connections. Joints can be sealed with o-ring couplings or with copious amounts of thread sealer. If they are not sealed they will leak, regardless of whether they are new or old. Taper threads may also be utilized for shallow pump settings.

For vertical turbine pumps, cast iron discharge heads are available in various sizes, and fabricated steel heads can be made to accommodate most configurations. Standard cast iron discharge heads can be machine-surfaced to fit a steel sole plate grooved for an o-ring. Rubber gaskets can also be used. This approach may be useful for retrofitting an existing well for ASR purposes. It also provides a reasonable pressure seal for situations where recharge water levels may only rise a small distance above land surface during continued ASR operations or pressure surges.

Examples of recharge through the columns of vertical turbine pumps include Goleta Water District, California; Calleguas Municipal Water District, California; Las Vegas, Nevada; and one of the ASR wells at Kerrville, Texas. All four sites utilize at least one existing well retrofitted for ASR purposes.

Submersible Pumps

Water also may be recharged through the columns of submersible pumps, although this is much less common. These pumps typically include a check valve at the base of the column to prevent water from running backwards through the pump. This valve can be removed to provide for recharge; however, it is then necessary to provide a motor restart delay at the pump controls to avoid severe pump damage during power failures and emergency restarts, or during normal ASR operations.

Another consideration is that reverse spin of the motor will generate electricity. Most submersible pump motors are poly-phase motors that do not have an excitation circuit required to generate electricity. If such currents are generated, resistive loads wired to the motor leads at the motor starter could be used to dissipate this generated electrical energy. Recharge flow rates through the submersible pump should not exceed design production rates for the pump as excessive rotational speeds could develop.

88

Las Vegas, Nevada, experimented with injection through two types of high volume submersible pumps. In one application, all four of the pump motors experienced problems with bearings, and after repeated failures, the process was discontinued. Although 3,600 rpm bearings were used in place of the standard 1,800 rpm bearings, the pump was not designed to operate in a reverse flow direction for extended periods. The other submersible pump application has the highest quality pump and motor on the market and appears to be operating fine after 3 years of ASR service. However during injection, the ominous vibration at the well head of bearings rotating down at a depth of 100–150 meters suggests that this expensive pumping unit was wearing during injection.

Submersible motor thrust bearings, especially if water lubricated, should not be spun at excessively high rates, and slow start and slow stop will cause these bearings to fail. This can easily happen when injecting backwards through the pump. Bearings are needed which can operate under any condition. This typically includes an oil lubricated bearing, oil filled motor or bearing chamber.

The same concerns apply regarding development of a vacuum in a submersible pump column as for a vertical turbine pump. Cavitation can damage the upper part of the column, potentially leading to structural failure as well as drawing air into the column through threaded connections or the wellhead piping.

The type of well head seal will depend upon the type of pump in the well. A flanged surface plate should be used for submersible pumps. Alternatively, a blind flange bored and welded to the column pipe can be fitted to a ring flange on the well casing.

Adjustable Downhole Flow Restrictions

Downhole Control Valves on the Pump Column

During recharge, it is common for wellhead pressure to have a wide range unless downhole flow control is provided. Low pressure may be required to start flow into the well; however a full vacuum may develop rapidly at the wellhead when the water cascades into a sealed well with a few feet or more of depth to water level. In some deep wells the depth to recharge water level initially may exceed 1,000 ft. As recharge continues, mounding of the recharge water in the aquifer may combine with plugging to cause the recharge water level to rise above land surface. In a few applications, recharge pressure is then increased steadily to compensate for head losses attributable to plugging, up to the maximum available pressure from the source of supply. At that point, redevelopment is required to restore ASR capacity.

Flow control is usually provided to smooth out this extreme range in operating conditions. Flow control using injection tubes, the pump column or annulus recharge is limited by the fact that the flow restriction is fixed. Since 1992 adjustable downhole flow control valves have provided a greater degree of control over variable recharge flow rates and pressures.

At the start of a recharge cycle, the downhole valve installed in the pump column is closed and the column pipe is filled with water. The air in the pump column prior to recharge is expelled through an air release valve at the surface rather than being carried into the formation. The downhole flow control valve can then be slowly opened and adjusted to provide the optimum recharge operation, ranging from "drip-tight shut" to maximum flow. This is a major advantage compared to non-adjustable downhole flow restrictions. The flow rate or pressure can be controlled manually by above-ground valving and also may be controlled automatically through a SCADA system.

Downhole control valves are currently being used in wells with static water levels that are 1,500 ft and greater below land surface. Where the depth to static water level is less than about 30 ft, or where the recharge water level is at or above land surface, downhole flow control valves are not necessary. Along the coastlines and at some inland locations, static water levels tend to be relatively shallow. However in some of the inland basins, such as the Central Valley in California or the Denver Basin in Colorado, static water levels in deep sedimentary aquifers can be more than 1000 ft deep, prior to groundwater development. With groundwater pumping, depletion of these groundwater reservoirs is causing even greater depth to water levels.

During ASR recovery or backflushing, water levels will typically experience a broader range than for a normal water production well, because of slightly higher initial water levels and slightly to significantly lower pumping water levels at the end of recovery. This range sometimes can be a problem in sizing a pump to produce a desired flow rate since initial flows may be much higher than ultimate flows at the end of a recovery period. One solution is to provide a pressure control valve on the recovery piping so that the pump is always operating within an acceptable range of flow rates.

The broader ranges for operating water levels and pressures for ASR systems have to be considered during well and wellhead design to ensure that the system will operate properly over the expected range. Low pressures at the wellhead tend to cause very high flow rates that can cause operating problems, either in the distribution system during recharge or sometimes in the surface drainage system during backflushing to waste. Flow control is therefore required for many ASR systems.

A downhole water level control valve is typically operated by nitrogen, air, water, or oil pressure from the surface, in order to throttle recharge flows in ASR wells. With one approach the throttling mechanism is a reinforced rubber element which controls flow in an annular space in a valve that is connected in the pump column. It is usually installed 10 to 100 feet above the pump bowls and has the same length as a standard section of column pipe in vertical turbine pump installations with the same types of threads to facilitate installation. The valve diameter is less than the pump bowl diameter. The valve can be operated to maintain any particular desired well-head pressure. Other operating modes include maintaining a predetermined depth to water level during recharge, or a ground storage reservoir water level, or an injection flow rate. Several other variables are also considered during valve design and operation, including the valve opening and closure rate at the beginning and end of recharge, and the desired operating mode (open or closed) if the valve fails.

An alternate design is also available using a sliding sleeve valve that is remotely actuated to adjust the open area of several slots, thereby controlling flow and upstream pressure. Recently a third downhole control valve has become available for ASR purposes, using two hydraulic cylinders to move an internal sliding sleeve opposite a series of holes in an outer cylinder, with the holes arranged in a spiral pattern.

A significant advantage of the downhole control valve is its small diameter. Many existing small-diameter wells in locations with great depths to water level may be retrofitted to ASR purposes by recharging down the pump column without having to add injection tubes inside the casing to control flow rates.

Annulus Flow Control Packers

It is possible to install a flow-throttling packer to control recharge through the annulus. The packer would be installed below the static water level to control flow through the annular space between the pump column pipe and the inside of the casing. This would be particularly useful in large diameter wells in which the annulus could provide a higher recharge rate than the pump column. However no such ASR installations are known to exist.

Another application would be to control recharge and recovery for two or more aquifers completed with a single well with multiple screens. For two aquifers, two throttling packers would be installed on the column pipe, with slotted column pipe between the packers. Operation of these two throttling packers would permit simultaneous recharge or recovery from both aquifers, or recharge into one aquifer and subsequent recovery from the second aquifer. A three-aquifer system would require four packers. This general approach has been considered, but not yet applied, for storage of fresh water in highly saline aquifers with a substantial density difference between the stored water and the ambient groundwater. Recharge would occur through the entire aquifer thickness but recovery might occur only from the top of the aquifer. Similarly this approach might be applicable for ASR storage zones subject to upconing of brackish water from below during extended recovery periods.

This approach could reduce capital costs since one well, instead of two or three wells, could be utilized to achieve ASR objectives in multiple aquifer systems.

Combination Recharge Approaches

In general, pump column recharge is likely to provide the greatest degree of head loss for recharge flows while annulus recharge is likely to provide the least head loss. Where it is desired to maximize recharge rates, and water levels during recharge will be at or above land surface, annulus recharge may be most applicable. Where water levels during recharge will probably be below ground level, pump column or injection tube recharge may be most applicable to minimize air binding and cavitation damage. The greatest degree of operating flexibility probably is provided by a downhole control valve, particularly when automated to meet predetermined operating conditions.

Where a downhole control valve is not used, flexibility to utilize more than one method of recharge is sometimes useful, particularly in situations where a wide range of recharge flows or static water levels may be encountered, or where considerable uncertainty exists as to the ultimate operating conditions. For example, the ASR system at Chesapeake, Virginia, includes the provision to recharge down the pump column and also down the annulus, or both if high flows are available for recharge. At Kerrville, Texas, the ASR system includes one well equipped to flow down the vertical turbine pump column in a retrofitted production well while a new ASR well is equipped to recharge down the annulus or two injection tubes.

CORROSION CONTROL

Experience to date indicates that corrosion is not a universal problem for ASR wells. However, significant corrosion has occurred in some of them. The exact causes of this corrosion are not well understood because the affected wells did not have common construction features or similar water quality. Moreover, the corrosion problems have not occurred in sufficient quantities

91

that discernable patterns have been determined. In spite of this seemingly unpredictable situation, some general observations can be made regarding factors that may contribute to higher rates of corrosion in ASR wells.

Dissolved Oxygen

Water injected in ASR wells is likely to contain much higher concentrations of dissolved oxygen than naturally occurring groundwater. The oxidation process is one kind of corrosion, so it is logical to expect that increasing the available oxygen in the well could increase its rate of corrosion. Increasing the concentration of dissolved oxygen can increase corrosion in two ways: by increasing the oxidation rate of submerged carbon steel to produce general corrosion (thinning) or pitting; and by acting as a depolarizing agent in dissimilar metal couples to allow galvanic corrosion to occur at a higher rate. Evidence of dissolved oxygen corrosion is typically production of rusty water after a well is idle.

Ionic Composition

Injection water may undergo treatment processes that increase the ionic composition and content of the water. For example, chlorination and flocculation with alum usually result in increased chloride and sulfate concentrations in the water. These ions tend to produce more corrosive water as their concentrations increase, especially if dissolved oxygen is present. In addition, chloride concentrations exceeding about 200 mg/L can cause pitting in Type 304 stainless steel which is commonly used in well construction.

Dissimilar Metals

Modern prodution wells are usually constructed with steel casing materials and stainless steel screens and control surfaces. ASR wells frequently contain more stainless steel components than comparable prodution wells. Increasing the submerged surface area of stainless steel relative to ordinary steel can be expected to increase the corrosion rate of the steel, although it provides some degree of galvanic protection for the stainless steel. Dissolved oxygen likely accelerates this form of galvanic corrosion. This is more of a problem with well casing, for which dielectric couplings are used.

Microbiological Factors

Surface waters may contain microorganisms that also could cause corrosion when injected in an ASR well. Such microbiota are also present naturally in ASR storage zones. This type of corrosion is generally referred to as microbiologically-induced corrosion or MIC. It is most likely to occur in standing, un-disinfected water and is often suspected in cases where stainless steel is corroded in the absence of other obvious causes.

Electrical Grounding

Corrosion by electrical power grounding is usually a form of galvanic corrosion between steel well components and buried copper ground rods and wires around the wellhead. Electrical

grounding is necessary for safety. In most cases, the well is indirectly in metallic contact with the buried grounding system by grounded electrical motors. Grounding connectors may also be attached directly to the well. It is possible but unlikely in most situations that alternating current electricity plays a significant role in well corrosion. It can be argued that the electrical grounding of ASR wells is not significantly different from that for ordinary groundwater wells.

When excessive corrosion is found in an ASR well, an analysis should be made of the damaged area and an investigation made to determine the specific circumstances that contributed to the problem. Preventive advice is difficult to provide because of the lack of complete and definitive information for ASR wells that have actually been found with excessive corrosion. In general, planners of ASR wells should consider the performance history of the materials used in groundwater wells in the same area and take into consideration the additional corrosive factors of the injection water. Recent ASR wells have shown increased use of types of stainless steel that are resistant to higher chloride content of the water. Connections between dissimilar metals should be electrically isolated, or else the more active metal (normally carbon steel) should have additional thickness to account for galvanic corrosion.

A probable contributing factor to well corrosion is the existence of a natural "earth battery" due to both lateral and vertical differences in lithology in a well. Just as with corrosion due to dissimilar metals, electrical voltage differences occur due to adjacent dissimilar soils, such as clays, shales and sands. An indication of this voltage difference is measured during geophysical logging through use of a spontaneous potential log.

Sacrificial anodes may be appropriate for use in some situations, to counteract corrosion in downhole metallic components of the well installation. These are not known to have been applied to date in ASR wells.

A synergistic effect may occur in some ASR wells, with corrosion due to a combination of causes such as electrochemical activity plus the effect of chlorine gas released from recharge water and accumulating within the well casing above the static water level.

VENTING OF AIR IN PUMP COLUMN

Where the static water level is deep and the ASR storage zone is very productive, large diameter pump columns contain large volumes of air that may be carried into the formation at the beginning of recharge unless measures are taken to prevent the resultant potential plugging. One approach to controlling this problem that has proven successful at Calleguas Municipal Water District, California, is to first turn on the pump prior to recharge, thereby purging air from the column pipe and out through the air relief valve, with any residual air going to a nearby ground storage reservoir. Once a solid column of water is present in the column pipe and wellhead piping, the pump is turned off and the wellhead valves sealed so that a solid column of water flows back down into the well through the pump. This creates a vacuum in the pump column and injection is then started while the vacuum remains. Static water level at this site is at a depth exceeding 375 feet and the wells each produce at flow rates exceeding 1.5 MGD.

Many ASR wells do not have such a problem since the volume of air in the pump column is small and some of this is probably vented through the annulus air/vacuum relief valve at the start of recharge, particularly if the downhole water velocity in the casing at the beginning of recharge is less than the air bubble rise rate.

93

AIR AND VACUUM RELIEF

All ASR wells experience a greater degree of water level change than typical production or injection wells. This change in water level results in air being drawn into or released from the well during different phases of operation. Adequate venting on the casing and on the wellhead discharge piping should be provided in the form of air/vacuum relief valves. However, it is essential that these valves be closed during recharge to prevent entry of air during potential vacuum recharge. This is an important operating requirement, the omission of which can entrain substantial quantities of air and plug the well within minutes.

Air relief valves are usually designed to vent air under relatively high operating pressures. ASR wells usually recharge under much lower operating pressures at the wellhead. Sometimes under these lower pressures, the air relief valves will leak slightly. Provision for drainage of this leakage water away from the wellhead will avoid a problem that may be aesthetically unappealing (rust), inconvenient (ponding), or sometimes slippery and dangerous. An easy solution is to provide a soft rubber/low-pressure seat for the air release valve on the recharge piping.

The principal constraint upon utilizing wellhead pressures above land surface is the need to ensure that adequate drawdown can be achieved by pumping the well so that any solid particles forced into the aquifer during recharge can be dislodged during recovery. Where depth to static water level is substantial, this constraint will tend to control the recharge pressure and therefore the recharge rate.

PRESSURE SEALING WELLHEAD CONNECTIONS

With appropriate well and wellhead design there is no reason why an ASR well cannot recharge with pressure heads substantially above land surface. Recharge pressures have been noted as high as about 70 psi although this is unusual. Where recharge pressures occur above land surface it is essential to ensure that wellhead connections are tightly sealed and pressure tested. In particular the pump base for vertical turbine pumps and the electrical cable fittings for submersible pumps must be properly designed to withstand the planned pressures and any associated pressure transients. Water level access ports should be equipped with a lock-out to prevent accidental opening under pressure. The well casing should be equipped with a high quality pressure gauge to make head measurements since the actual water level is above the land surface.

For low pressures at the wellhead it is probably sufficient to provide a normal, cast wellhead flange, bolted to the top of casing flange with a rubber gasket. This should be sufficient for wellhead pressures up to about 10 psi. For higher pressures it is recommended to provide flanges that are machined flat and have been provided with an O-ring seal. The flanges should be installed perfectly level to avoid creating torque at the top column connection, which carries the full tensile load of the pump and column.

PRESSURE AND WATER LEVEL MANAGEMENT

Accurate pressure and water level measurement is important to ASR success. While recharge and recovery may occur without collection of these data, there would be no way to determine whether plugging is occurring until such time as the water level rise begins to inhibit recharge rates. By that time, the severity of plugging may preclude easy redevelopment by pumping. Instead, it may be necessary to pull the permanent pump and any additional tubing from

94

the well; clean the casing and screen with scraping, jetting, brushing, or other redevelopment methods while pumping the well with a temporary pump; maybe acidize or chemically treat the well and surrounding formation, and disinfect it prior to reinstalling the permanent pump. This is time-consuming, expensive, and risky since the recharge specific capacity may not be fully restored. More cost-effective would be periodic redevelopment by backflushing to maintain recharge capacity. The need for backflushing is usually determined based upon pressure and water level measurements.

Pressure Gauges

Pressure gauges should be both durable and accurate. Sealed cases filled with glycerin or silicone stand up well to harsh, outdoor conditions. The fluid-filled gauges also provide needle damping if vibrations or pressure spikes are present.

Pressure readings are useful in many places on ASR wellhead piping. Consideration should be given to installing taps for pressure gauges on the distribution system piping supplying the ASR well; upstream and downstream of any pressure control valves; upstream and downstream of any wellhead filters; and on the wellhead recharge and recovery piping. If vacuum or negative pressures occur, particularly at the wellhead, a combination vacuum/pressure gauge should be provided.

Gauges should provide the level of accuracy necessary for each location. Generally, a gauge with 0.5% accuracy is desirable for the wellhead but is not necessary at other locations.

To protect the gauges against damage during pressure surges, spikes, or fluctuations, dampening devices can be installed for each gauge. These range from a fitting provided by the gauge manufacturer to a simple, small petcock.

Water Level Measurement

A variety of systems are available for obtaining accurate measurement of water levels in a well, among which are the following:

- casing access tube for direct measurement with steel tape and chalk
- air lines and bubbler systems
- portable electronic sounders
- electronic pressure transducers

It is important to provide a direct means of measuring water levels through a casing access tube, even if other indirect means are also provided for convenience. The selection of the measurement system should reflect the probable frequency of water level measurement and other operational needs and opportunities.

Water levels fluctuate over a larger range in ASR wells, from recharge pressures or water levels attained at the end of the recharge period to drawdowns at the end of recovery or during backflushing to waste at high rate. The range sometimes can exceed the design range for pressure transducers, causing their failure. Typical transducer pressure ranges are 100 to 300 psi, although higher pressure transducers are available. Pressure transducers are also vulnerable to failure due to lightning strikes.

95

Air lines and bubbler systems work well and are often more reliable than pressure transducers. Air lines may be constructed of plastic tubing (nylon, polyethylene, PVC); or metal tubing (steel, stainless steel or copper). A small-diameter tube is installed in the well with the end of the tube submerged below the pumping water level. Air or nitrogen gas is injected down the tube at a low rate until the gas bubbles out of the end of the tube. The pressure required for the gas to bubble out of the end of the tube is equal to the depth the tube is submerged below the well water level. A small air compressor can purge up to about 75 m (230 ft) of air line, or a nitrogen bottle can meet higher pressure needs. It is possible to use two separate air lines with the appropriate valving for applications involving large water level changes.

Providing a small-diameter PVC casing access tube with a cap on the bottom and a perforated section near the top and bottom for manual measurements is highly advisable, regardless of the other in-place measurement methods such as a pressure transducer or air line. For recharge down the well annulus, this is probably the only way to measure water levels since cascading in the annulus will otherwise preclude accurate water level measurement. Cascading, whether under a vacuum or not, creates a column of water expanded with air bubbles so that it is difficult to determine the true water level in the well unless it is measured through a separate tube.

It is wise to provide redundancy in the ability to measure depth to water level. A common combination is a pressure transducer and also a bubbler tube. If the transducer fails then a backup water level measurement approach is available until such time as the transducer can be replaced. While the transducer is removed from the tube, a direct measurement of depth to water level using an electric tape or a steel tape can also be made through the dedicated tube for the pressure transducer. Experience from many ASR wellfields suggests that the substantial range in water levels between recharge and recovery, or possibly other downhole conditions, can be a problem for sustained operation of conventional pressure transducers in ASR wells. Proper transducer selection is required, including not only selecting an appropriate range of operating pressures but also ensuring adequate protection against corrosion.

FLOW MEASUREMENT

A very important aspect of ASR operations is measurement of flow rates and volumes of water recharged, recovered and discharged to waste. This is important for both technical and regulatory reasons.

Flowmeters used on ASR projects have included propeller, turbine, magnetic, venturi, and ultrasonic meters, as well as orifice plates, pitot tubes and other approaches. Selection of the appropriate flow meter should reflect project needs as well as meters currently in use at other locations operated by the same water agency or utility. Accuracy of these meters ranges from ±2% of actual flow, down to ±0.5% of actual flow.

Selection of the appropriate flowmeter range is important since it is quite common for recharge flows to vary over a broad range during initial testing and subsequent operations. An ASR system may be designed to recharge at a high rate. However, water may not always be available for recharge at this rate due to operational constraints such as increasing distribution system demands or maintenance of minimum distribution system pressures in the vicinity of the ASR well. The alternatives include continued recharge at whatever lower flow rate may available, or stopping recharge until flows are available at a rate within the range of the flowmeter. A flowmeter with an accuracy range of 10 to 120% of the design flow would probably be sufficient to permit continued recharge for most of the time until the system is switched over to recovery.

Added operational flexibility at the low end of the operational recharge flow range can extend the usefulness of ASR in situations where there is a need to store as much as possible of a limited supply of seasonally available water.

Flowmeter accuracy depends upon appropriate location in the wellhead piping, requiring an adequate distance of straight pipe upstream and downstream. For new ASR wellhead facilities, the flowmeter selected and the associated piping distances can be easily incorporated in the design. However, for retrofitting existing wells for ASR purposes it is frequently necessary to select a flowmeter type that will provide the desired accuracy within the piping distance available. Straightening vanes are sometimes used to straighten flow lines upstream and downstream of the meter.

For larger ASR systems, or for those involving automated control systems, it may be appropriate to obtain a certificate of proper flowmeter installation from the manufacturer.

For all ASR systems, it is advisable to provide dual flow measurement capability, at least for the duration of the test program. Meter failure or loss of calibration during the test program has occurred at several sites for a variety of reasons. Loss of calibration is difficult to detect at the time, and usually only becomes apparent late in the program when it is too late to repeat the tests. The resulting data can be difficult to interpret. It is desirable to have two different types of flowmeters, one of which is the primary meter. Any trend of increasing difference in measurements between the two meters would signal the need for calibration or meter replacement before proceeding further with the test program. These problems appear to be more common with propeller meters, which are used widely in the water industry. Having a standby propeller meter or replacement parts on hand can be helpful, available for rapid substitution if necessary. A venturi tube, calibrated orifice plate, or similar device incorporated in the wellhead piping can provide the backup flow measurement during testing and can be easily removed when the system changes into long term operation, if desired.

Bi-directional magnetic flow meters have been used at some ASR sites where it was desired to convey both recharge and recovery flows through the same pipe.

Flowmeters utilized on ASR systems should include totalizing measurement in order to monitor cumulative volumes during both recharge and recovery. This is typically provided with propeller type flowmeters, which are readily available, relatively inexpensive, and have been used widely on ASR projects. Propeller meters are usually accurate to within ±2% of actual flow rate.

Turbine meters are similar to propeller meters; however, they use a turbine instead of a propeller. The turbine spins at a higher velocity and consequently requires a more precise bearing and mechanism. For this reason, turbine meters are more sensitive to sand and particles in the water flow. Upstream screens should be installed with turbine meters. They typically provide higher accuracy and a wider operating range than a propeller meter. Typical accuracy is about ±1.5% of actual flow. Cost is usually about 30% greater than the corresponding propeller meter.

Venturi meters offer the advantage of having no moving parts. They place a smooth constriction in the flow stream and then measure the reduction in pressure at the throat of the constriction. The difference in pressure between the meter throat and the adjacent pipe is related to the flow rate. The actual meter tube usually can be installed between two pipe flanges and therefore requires little space. However, adequate upstream and downstream pipe distances must be provided and may be higher than for other meters. These meters result in relatively low head loss through the meter. They require a mechanism to read the differential pressure and a separate totalizer to integrate the flow signal. Reading the differential pressure requires a fairly sensitive gauge. Typically, a differential pressure transmitter is mounted at the venturi tube and sends a

97

signal to a remote flow rate indicator and totalizer. These meters are accurate within ±1% of full scale.

Magnetic flow meters also have no moving parts and have the advantage of compact size. The meter works by first creating a magnetic field in the pipe. When the water moves through the magnetic field, a voltage is induced that is proportional to the flow rate. Flow rate indicators and totalizers are available with either local or remote mounts. Magnetic meters are bi directional, with no loss in accuracy. The required upstream pipe distance is usually shorter, as a result of which these meters are particularly useful in retrofitting existing wells for ASR purposes. Additionally, the cost of these meters has reduced considerably over the past several years. They can be obtained with accuracies of ±0.5% of the actual flow rate. These meters are increasingly utilized for ASR design purposes.

Ultrasonic flow meters are portable and can be moved easily from one length of pipe to another. The meter mounts to the outside of an existing pipe and requires no moving or other parts in the water flow stream. They are also available in flanged body configurations. If bubbles are present in the water they may provide erroneous results. They operate by electronically measuring the time required for an ultrasonic signal to travel between two or three transducers mounted to the outside of the pipe. The difference in time between signals traveling upstream and downstream is proportional to the liquid velocity. The meters usually consist of several transducers that can be mounted in several configurations and record to a data logging microprocessor. Pipe material, diameter, wall thickness, and lining type and thickness must be known and entered into the microprocessor. As a result they can be difficult to get set up correctly. Ultrasonic flowmeters are well suited for checking the performance and accuracy of inline meters and can be obtained with an accuracy of ±1% of the actual flow rate.

Another primary or backup flow measurement option is to provide a tap in the wellhead piping so that a Pitot tube can be inserted, measuring the flow rate according to the differential pressure at various ports across the velocity profile of the pipe. This is typically about the same accuracy as an orifice meter, or about ±1% of the actual flow rate.

Several ASR wellfields incorporate measurement of the trickle flow directed into the casing during extended storage periods. Although the flow rate may be low, over a period of several months this can amount to a significant quantity of water. Monthly recording of cumulative trickle flow volume at each well is then manually added to the ASR storage volume records. Similarly flows discharged to waste are also measured at some sites, particularly where the discharge frequency and duration are more common and lengthy.

DISINFECTION AND pH ADJUSTMENT

Chlorine gas added to water will typically result in a decrease in pH. The magnitude of the decrease will depend upon the chlorine dosage and the alkalinity of the water. Where the recovered water will be blended with a much larger flow of water, the effect may be negligible. However, where little or no blending will occur prior to consumption, the pH drop following chlorination can be sufficient to produce aggressive water, capable of causing corrosion of pipes and fittings and associated "red water" complaints from consumers. The need for pH adjustment following recovery is usually determined following construction and initial testing of ASR facilities. Consequently, it is desirable to equip ASR wellhead facilities with locations for chemical addition, if later required.

98

Adjustment of pH also may be advisable for recharge flows. Where manganese is present in the storage zone, recharge at a pH of less than about 8.0 may tend to cause the manganese to go into solution during an extended storage period. Recovery of the stored water may then create a problem with excessive concentrations of manganese and associated black discoloration of wetted surfaces. Adjustment of the recharge water pH to levels of about 8.5 will help protect against recovery of water with high manganese concentrations.

Depending upon the potential for formation of disinfection by-products (DBPs) such as trihalomethanes and haloacetic acids when the recovered water is disinfected, it may be necessary to add ammonia to the recovered water to form a chloramine residual. Where ammonia is present in the recharge water, its presence in the recovered water should be tested before making a determination as to whether re-ammoniation is necessary. Typically, ammonia is reduced during aquifer storage. At two ASR sites a 50% reduction was noted within one month.

At a few ASR sites the recharge water quality may vary seasonally for a variety of reasons. At times low pH values may result in undesirable subsurface geochemical reactions such as dissolution of metals, or metal corrosion, while high pH values may risk calcium carbonate precipitation. At other ASR sites the recovered water may have a reduction in pH which, when combined with normal disinfection practices, causes the pH of the water going to the distribution system to be too low. For these situations it is necessary to provide a tap in the wellhead piping to provide pH adjustment of either the recharge or recovered flows. Adequate space is also needed for storage of whatever chemicals are required to achieve the pH adjustment, and the associated injection facilities.

PUMP CONSIDERATIONS

Differences Between Pump Design for ASR Wells and Water Supply Wells

Selection of a pump for an ASR well includes a few features not normally considered in pump selection for a normal production well. Pumping water levels in ASR wells may vary depending upon the degree of well plugging. Pump hydraulic characteristics should be selected so that operation occurs over a reasonable range around the design point for flow and head. Additional column pipe provides operating flexibility since the range of pumping water levels is usually not known until after a few cycles of operation. Net positive suction head (NPSH) and motor electrical horsepower should also be sufficient to match the full range of expected pumping water levels.

Pump Setting

To date, ASR wells have been equipped with vertical turbine, submersible, and horizontal centrifugal pumps. All have proven adequate for their specific applications. Vertical turbine pumps typically produce 200 to more than 5,000 gpm with horsepower ranging from 50 to 5,000 HP. Setting depths are up to about 1,000 ft and wire-to-water efficiencies range from 60 to 75%. Pump rotation is typically 1,750 rpm or less.

Narrow, long submersible pumps are available that were originally designed for oilfield applications and have been adapted to water supply purposes. With these pumps flows up to about 2,500 gpm can be achieved with a 10" nominal pump diameter, as shown in Table 2.1. An engineering and economic analysis is required to establish the most appropriate pump design for each

ASR well. In most cases this then determines the casing diameter. Submersible pumps typically produce 10 to 5,000 gpm and from 1 to 2,000 HP. Setting depths may exceed 2,000 ft with special construction and configurations, and wire-to-water efficiencies range from 50 to 65%. Submersible pumps are typically selected for quiet residential neighborhoods, very deep pumping levels, crooked wells, potential freezing conditions at ground surface, and well installations in buried vaults. Submersible pumps are used with rotation speeds of 3,500 rpm for lower flow applications or for higher flows using oil field configurations, requiring a smaller diameter motor and casing. However, high speed submersible pumps in conventional configurations generally provide a lower operating life in sizes about 200 gpm and above and should be used only when necessary. For ASR wells that recover water for only a few weeks per year, the service life of a submersible pump may be longer compared to a conventional water supply well that operates continuously. Similarly the slightly lower efficiency compared to a vertical turbine pump may not cause a significant increase in annual operating costs, for the same reason.

A differentiator between pump design for ASR wells and for other water supply wells is the common situation where a greater potential exists for downhole corrosion. In situations where storage zone permeability is very low, or plugging potential is deemed to be high, or discharge of initially turbid water would be a problem, additional corrosion protection measures are needed. Consideration should be given to coating the pump column pipe, bowls and impellers, both inside and outside, in order to reduce the surface area subject to rusting during alternate wetting and drying periods associated with recharge and recovery. Alternatively non-corroding materials of construction may be selected to achieve the same objective.

Normally, it is wise to use the same pump manufacturer as utilized for other wells and pumping installations operated by the owner of the ASR well. However, certain submersible pump manufacturers have indicated that they will not honor the pump warranty if the pump is used for injection. In this case, alternate manufacturers or types of pumps should be considered, or use of downhole control valves. As ASR applications have become more widespread, this reaction seems to have dissipated. Injection through a submersible pump entails removal of the check valve normally provided at the base of the pump column and ensuring that the pump bearings can operate without damage in reverse and over a wide range of speeds. With the check valve removed, there is greater risk of premature re-start of the pump after recharge or after a power failure, at a time when water is still draining down the column pipe. The resulting additional torque can damage the pump. For this reason, a restart delay may need to be provided to protect the pump. These risks can be managed effectively by providing a downhole control valve in the pump column so that recharge water is directed to the annulus instead of through the pump bowls.

A related consideration pertinent to the use of large submersible or vertical turbine pumps is that large motors should not be cycled on and off repeatedly without an intermediate period for heat dissipation. Turning these large motors on and off causes considerable wear and tear, which should be minimized. Each manufacturer will have its own criteria for acceptable pump operation. ASR well redevelopment and backflushing sometimes includes pumping the well to waste at a high rate for a few minutes, resting the well, then pumping the well again for a few minutes. This cycle is sometimes repeated one or two times to surge the well and thereby remove solids from the screen and gravel pack or the surrounding aquifer. The redevelopment operation may occur as frequently as every day or two, or as infrequently as once every year at the beginning of seasonal recovery. For vertical turbine pumps, such a redevelopment sequence is less of a problem. However, short cycle operation of large submersible pumps without a check valve for redevelopment and backflushing may be inadvisable, depending upon manufacturer requirements. If the

100

need for frequent cycling of large submersible motors becomes apparent, it may be advisable to reconsider well design and operation to reduce the generation of solids and the associated frequency of backflushing, or to pump the well for a longer period, with fewer pumping cycles, until clear water is achieved.

For vertical turbine pumps, a non-reverse ratchet should be included to prevent impeller rotation during recharge and also following pump shutoff. With a non-reverse ratchet, the torque on the impeller is typically in the same direction during recharge as during normal pump operation, so there is no tendency for the pump to unwind during recharge. However, the torque reverses again during column drain-down as the motor flywheel torque takes over. Without the non-reverse ratchet, situations can develop that can unwind the shaft.

Oil-lubricated vertical turbine pumps generally should be avoided in ASR wells where possible. Under certain operating conditions, the potential may exist for a vacuum to form in the column pipe where this is used for recharge. The oil would then be pulled into the recharge water, plugging the storage zone and contaminating the water. If the annulus is used for recharge, any floating oil in the annulus may be carried into the formation. The problem can be minimized through use of a separate injection tube. In vertical turbine pumps, shutting off the oil reservoir supply to the pump shaft at the beginning of each recharge period and opening it at the beginning of each recovery period can also work; however, this is somewhat risky as a long-term operational requirement since it would be easy to overlook the adjustment. A better way is to avoid the use of oil-lubricated pumps for ASR wells.

Many ASR wells have been provided with variable frequency drives, providing for adjustment of recovery rates over a wide range of approximately 50 to 120% of the design flow rate, if adequate horsepower is available. Experience to date with these systems suggests the need, particularly in more humid areas, to ensure a source of heat or air conditioning to prevent moisture buildup within the electronics due to condensation. If these VFDs remain energized and in use, they provide reliable service, however if they are turned off for an extended period, such as during several months of ASR recharge and storage, it is quite possible that the electronics will malfunction. Providing a small air conditioner within the VFD will prevent condensation and thereby help to ensure its reliability for ASR purposes. VFD design criteria, and associated costs, increase if the power supply to the ASR pump includes potential reliance upon an emergency generator.

Some ASR wells for Calleguas Municipal Water District, California, have been provided with two-speed motors to facilitate energy recovery during recharge. Another site has considered installing a two-speed motor to enable recovery at normal rates to meet distribution system diurnal variations in demand, and higher rates if needed to meet fire flow requirements.

Pump Types

Most ASR wells use centrifugal pumps, generally in the form of multi-stage vertical turbines and submersible pumps. In cases where the pumping water levels are shallow and within the suction lift range (about 25 feet), surface-mounted horizontal centrifugal pumps have been utilized.

An increasing number of ASR wells are located in parks and suburban residential areas where esthetics and noise considerations have become more important. In these situations submersible pumps are used in place of vertical turbines. In addition, pitless units and vaults are utilized to eliminate above-ground piping at the well site.

101

Axial, Mixed Flow, and Radial Pumps

Centrifugal pumps are of three general types, including axial flow, mixed flow and radial impeller configurations. Practically all of the ASR and larger water well pumps are of the mixed flow construction, however in cases where the water level is relatively shallow and high pumping rates are encountered, axial flow pumps are sometimes used. For deeper water levels and lower flow rates, radial flow pumps are utilized. Mixed flow impellers have three designs: open, semi-open, and closed. Most ASR pumps are of the closed impeller design as it is slightly more efficient and is used for pumping clean water.

Vertical Turbine Pumps

Vertical turbine well pumps were developed around 1900 and caused a major revolution in ground water supplies by enabling production of large quantities of groundwater from pumping levels deeper than 30 feet. They typically use an electric, vertical hollow shaft motor which drives the downhole bowl assembly through either an open line shaft or an enclosed line shaft. The line shaft has left hand threads which are tightened as the motor turns counter-clockwise looking down. The impellers then tend to turn the bowls in the same direction, which tends to tighten the right hand threaded pump column pipe from the bottom.

The open line shaft is lubricated by the water being pumped and utilizes rubber bearings every 10 feet. It is necessary to pre-lubricate these bearings before the pump is turned on to prevent excessive wear.

The enclosed line shaft can have brass/bronze bearings or wood bearings. For ASR and water supply wells a pharmaceutical grade mineral oil is used for lubrication of the enclosed shaft. With the wood-lined enclosed drive shaft water can be used as the lubricant.

Vertical turbine pumps usually are operated at 1,800 rpm. As production rates increase, consideration should be given to reducing to 1,200 or perhaps 900 rpm. Such lower speeds typically require larger bowl diameters, and therefore increased well construction costs. However the lower speed pumps tend to have a longer service life. 3,600 rpm vertical turbine pumps are seldom used and only have application at smaller production rates.

These rpm values for AC motors are typically for 60 cycles per second power supply. In countries where 50 cycles per second power supply is provided, rpm values are reduced by the ratio of 5/6.

Submersible Pumps

The horsepower limit for submersible pumps has increased in recent years to a maximum of approximately 2,000 horsepower. These pumps have become more popular as their reliability has improved, the larger horsepower motors have become available, and as noise and esthetic considerations have become more important for well and wellhead design. There are two classes of submersible pumps: a water well submersible and an oilfield submersible.

The water well submersible pump uses a vertical turbine bowl assembly and adapts an electric motor with water intake to the bottom of the bowl. For the normal ASR well this means that the impeller turns counter clockwise as one looks down the well. Historically this was necessary so that the rotation of the impeller would tighten, rather than loosen, the right-hand-threaded pump column pipes. By attaching an electric motor to the bottom of the bowl assembly,

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the rotational torque and especially the starting torque is in a clockwise direction looking down the well. This means that there is a tendency for the pump to unscrew the pump column pipes. For this reason it is important to properly torque the pump column pipes during installation to keep them from unscrewing.

Oilfield submersible pumps are of the 3,500 rpm class since they are designed for small diameter wells that are quite deep. In recent years these oilfield submersibles have been incorporated in deep water wells with high pumping lifts. For example, in the Denver, Colorado, area pumping lifts of 1,500 feet are common. The oilfield submersible then allows a smaller diameter well casing to be installed, with its associated cost savings. These pumps have a lower pump efficiency compared to the slower turning, larger diameter water well bowl assemblies. However for ASR wells that may recover water for only a few weeks each year, the increased annual operating cost of the higher speed pump may in many cases be acceptable compared to the savings in initial capital cost associated with construction of a smaller diameter well.

Some oilfield submersibles have the bowl assembly designed to operate in the clockwise direction looking down the well. This is favorable as the torque of the motor then tends to tighten the pump column pipe rather than loosen it.

Pump Materials of Construction

A variety of materials have been utilized in the construction of water well pumps, many of which have been used in ASR well pumps. Cast iron is common, since this material is readily available and easy to cast for the impellers and bowls. As pump settings became deeper and pressures increased, ductile iron became more appropriate for the impeller and bowls. Ductile iron has a yield strength approaching 100,000 psi and therefore provides high strengths at relatively low cost, though its corrosion resistance is not high.

Brass and bronze alloys are commonly used for the impeller in multi-stage mixed flow pumps. This material is more corrosion resistant than cast iron or ductile iron and serves as a bearing material against the cast iron or ductile iron bowls. There are many varieties of this type of material, including aluminum bronze, which is of high strength.

ASR wells tend to have more corrosive operating conditions than water supply wells, so an increase in the use of stainless steel is anticipated for both impellers and bowls in order to provide longer life and improved performance. Type 304 stainless steel is relatively common. Type 316L is more corrosion resistant and the "L" stands for low carbon. Either 304 or 316 can be low carbon, which reduces corrosion potential.

There are many types of corrosion resistant alloys available for constructing bowl assemblies. In general they are of the high nickel variety and can include Inconels and Hastaloy. These are expensive and are used only in more aggressive corrosion environments.

Plastics are used only in smaller pumps, such as 50 gpm or less, and can be low cost and corrosion resistant. These pumps are normally of the radial flow design with lower pumping efficiencies.

Many coatings are available for the impellers and bowls, one of the oldest of which is enamel or porcelain. This is quite effective in increasing efficiencies and reducing corrosion potential. New, high technology coatings are being developed, so it is anticipated that these will be used increasingly in future years to improve pump design, reduce cost and increase pump longevity.

Pump Bowl Diameter

The diameter of a vertical turbine pump bowl or a submersible pump bowl is a function of the pumping flow rate and the rpm of the impeller. Table 2.1 lists nominal pump bowl diameters from 4" to 24" and the minimum recommended casing inner diameter to accommodate pumps with different speeds. It is readily apparent, for example, that in order to reduce costs of a stainless steel cased well, a smaller diameter casing can be used that will require a 3600 rpm pump. For larger pumping rates, the only 3,600 rpm pumps are submersibles, generally of the oilfield variety that have been modified for water well use. In a 12" I.D. casing a 3500 rpm submersible pump is available with an outside diameter of 11" which can produce 4,000 gpm. With an 1,800 rpm submersible pump, which would be based upon a vertical turbine bowl coupled to a downhole submersible motor, only about half of this pumping rate can be obtained. However the 1,800 rpm bowl assemblies typically have a slightly higher bowl pumping efficiency.

In large diameter wells with pumping water levels generally less than 500 feet, vertical turbine pumps are commonly used. When pumping water levels are at 750 to 2,000 feet or more, then 3,600 rpm submersibles are commonly used.

In most cases it is advisable to have a diameter clearance of 2" between the outside of the bowl and the inside of the casing. On occasion this is reduced to 1" however this entails somewhat greater risk since not all wells are straight and not all casings are round. Furthermore bowl diameters may slightly exceed the dimensions listed by the manufacturer. In such situations it is often advisable to run into the completed well a dummy pipe with the same diameter and length as the pump, or slightly larger, to ensure that the pump will fit into the well.

Rubber centralizers may be added to the pump column, and to the pump intake on vertical turbine pumps, to help reduce the potential for long term casing damage resulting from the lateral movement of the pump during startup and shutdown.

Pump Drivers

Electric motors are most often used for ASR well pumps as they are quiet, convenient and virtually maintenance-free. Where electricity is not available, such as for farm irrigation wells, natural gas, propane or diesel engine drives are used to power the wells, providing either constant or variable speed. An engine-powered generator would be required for submersible pumps.

Engine drives are sometimes used for emergency backup power supply for ASR or water supply wells. A right angle drive is commonly used for engine-driven vertical turbine pumps. The ratio of the right angle drive is chosen to match the engine rpm to the pump rpm. An advantage of the engine drive is that it can also function as a variable rpm (revolutions per minute) drive to meet variations in pumping rate requirements.

Variable frequency drives (VFDs) have been refined in the past few years. They are now more reliable and provide for a much smoother startup and shutdown operation of a pump. VFDs are increasingly preferred for ASR wells. When an engine generator is utilized for emergency power supply for a well equipped with a variable frequency drive, care must be taken to select a more expensive VFD so that harmonics are controlled.

Submersible well pumps can typically operate with 460 volt, 3 phase power. For larger and deeper installations, the voltage is increased to 1,000 to 2,000 volts to reduce the amperage and therefore reduce the cable size, cable voltage losses and cost. The reduced cable size and

cable voltage losses can result in significant cost savings, especially when pumps are set at 1,000 to 2,000 feet and deeper, with a transformer at the wellhead.

Pump Column Pipe

The column pipe for an ASR well vertical turbine or submersible pump installation must be of a size so that frictional pumping losses are minimal. It must also be constructed of non-corrosive materials if the quality of the recharged or recovered water is corrosive.

Column Pipe Materials

The most common material for pump column pipe is mild steel, which is readily available and of relatively low cost. Yield strength is approximately 30,000 psi and it is available in various schedules, the most common being Schedule 40, which is considered a standard wall thickness. It is recommended that this not be used at depths greater than 500 feet. Oilfield casing, which has a yield strength of 55,000 psi or higher, is recommended for depths greater than 500 feet. This casing material is frequently used for deep set submersible pumps, and is also recommended as column pipe for deep set vertical turbines. Reference can be made to American Petroleum Institute (API) standards and specifications regarding the collapse, internal pressure and joint strength of the various sizes and wall thicknesses of oilfield casing.

PVC has occasionally been used for shallow submersible pump column pipe, particularly for small ASR wells. It is important to consider the direction of rotation of the submersible pump in order to prevent unscrewing or rotating of the joints. Furthermore the torque occurring at pump startup or shutdown can twist the power cable around the pump column. A VFD is highly desirable to reduce the starting torque on PVC pump column pipe.

Stainless steel is becoming more common for ASR pump column pipe. Yield strength is typically about 30,000 psi; however higher yield strengths are available at no increase in cost, typically with some loss in corrosion resistance. An important caution is: the threaded connections of stainless steel pipe tend to "gall." This is inherent for stainless steel material and results from the metal transferring between the coupling and the pipe such that once galling starts, the connection cannot be turned any further. This can occur either during makeup or breakout of a thread. The threads are ruined and must be replaced. Many years ago Armco Iron developed a stainless Nitronic 60 which has a much greater resistance to galling and is used as a coupling material. For example, for a 304 coupling connection to a 304 stainless steel pipe, a surface pressure of 2,000 psi can initiate galling. If a Nitronic 60 coupling is used with a 304 stainless steel pipe, galling initiates at 50,000 psi. Galling potential can be reduced with the application of an "anti-galling" thread lubricant. Even though these pipe lubricants reduce the galling tendency, the ideal anti-galling pipe lubricant has not been found to date, with the exception of metallic lead and lead carbonate (white lead), which are not used due to health requirements.

Fusion bond epoxy coated steel pipe is also used for pump columns. The epoxy protects the inside wall of the pipe, however this coating is easily damaged by the pipe wrenches utilized during column installation and removal. In areas where there exists a strong spontaneous potential (SP) from the ground surface to depth, this earth "battery" can cause corrosion of the pump column pipe, especially when combined with chlorides and oxygen in the water. This electro-chemical phenomenon can cause accelerated corrosion at points where the epoxy coating on the

105

outside of the column pipe has been damaged. The corrosion is accelerated as the electrical current is focused on very small areas and therefore the flux is much higher.

Galvanized pipe has been used for many years for small pump column pipes in applications requiring low production rates. Galvanized pipe is generally not available in large sizes. In areas where the water quality has a pH of 7 or higher, galvanized steel pipe may prove to be desirable. With a pH of less than 7, the zinc in the galvanizing is readily attacked and this option is not recommended.

Column Pipe Diameter

The inside diameter of the pump column pipe needs to be sufficiently large so that friction losses during pumping are acceptable. Typically these velocities are in the range of 5 to 10 feet per second. Velocities in excess of 2 to 3 feet per second are recommended so that any sand that may be produced from the well is lifted and discharged at the surface. If the column pipe is too large and sand grains are not pumped from the well, they can accumulate on top of the submersible check valve and prevent the check valve from opening. This will cause the pump to operate at shutoff head. Since no water will be produced the pump will overheat and be damaged.

Column Joint Strength Requirements

It is critical to consider the joint strength requirements for pump settings deeper than about 500 feet. For submersible pump installations up to 500 feet, National Pipe Taper (NPT) or British Standard Pipe Taper (BSPT) are common threads used for the pump column pipe. Vertical turbine pump settings up to 500 feet typically use the NPT or BSPT thread forms. The taper is less than the normal ¾" per foot on the diameter and can range from a straight thread to a taper up to ⅜" per foot on the diameter. A ³⁄₁₆" taper is common for the thread on the pipe. The coupling can have a straight thread or a taper to match that of the pipe. Because of the wide variety of vertical turbine pump column threads and the lack of published and readily available threading specifications, including tolerances, potential problems exist in the field. For pump settings deeper than 500 feet, pump column pipe joint strengths are very important. The joint strength is then a function of the material yield strength, wall thickness and thread pattern, or other method of joining.

Flanged column connections are common. In some cases the flange diameter determines the inner casing diameter to a greater extent than the pump bowl diameter.

OTHER ASR WELL SITE CONSIDERATIONS

Well Houses and Vaults

Most ASR wells are completed above ground with the casing extending at least one foot above land surface, or to above known flood levels. Usually the wellhead facilities are provided with a wellhouse. In a few situations the wellhead is below land surface in a covered vault. This is typically required in situations where the ASR well is located in a park or other valued natural setting. Some typical examples include one of the wells at the Seattle Highline ASR wellfield in Washington; the Tumbleweed ASR wellfield for the City of Chandler, Arizona; Centennial Water and Sanitation District, Highland Village, Colorado, and the Fountain Hills ASR wellfield, Arizona. The first three of these sites were concrete vaults constructed in place. The Fountain

Hills site, located on the north side of Phoenix, includes four ASR wells for which the wellheads are constructed in circular, pre-fabricated vaults. This latter approach is believed to have considerable long term potential for other ASR wells, for which wellheads could be prefabricated and shipped to the site for easy installation.

Pressure Control Valves

Pressure control valves may be required on either the recharge line, the recovery line, or both. This provides operational flexibility in situations where recharge pressures may fluctuate, where available recharge flow may be limited during certain hours of the day or months during the year, or where recovered flows may interfere with system head curves at certain times.

Pressure reducing valves (PRVs) have been used successfully at many ASR sites for controlling injection. Recharge facilities are designed to provide the highest flow at the highest available system pressure. The operational rate can then be adjusted downward by changing the injection pressure at the PRV, thereby maintaining pressures in the water distribution system or compensating for well plugging. In Las Vegas, Nevada, over 50 single-purpose injection wells are controlled by surface PRVs. There are ASR wells utilizing PRVs to control recharge in Kerrville, Texas, and Kitchener-Waterloo, Canada. ASR wells using adjustable downhole control valves may, in many cases, be able to eliminate the need for PRVs above ground.

Survey

A permanent survey benchmark should be provided at each ASR site, showing the elevation. This will provide a reference point for measurement of water levels.

Electrical Power at the Wellhead

ASR projects typically require substantial onsite testing during both day and night. Consequently, it is important to provide adequate lighting, not only for the ASR well, but also for any observation wells that will be measured or sampled at night. Electrical outlets at the site facilitate use of test and sampling equipment, laptop computers, power tools, and other activities, the need for which may not be apparent at the time the wellhead is designed.

Observation Well Equipment

If observation wells are to be sampled, consider how the samples will be taken and if dedicated pumps should be installed in these wells. Estimation of sample flow rates and volumes, and whether the pump will operate continuously or intermittently, will indicate the appropriate means for disposal of the pumped water. If monitoring of water levels is required in the same observation wells, consideration has to be given to the change in water levels caused by the sampling pump. Installing a small diameter sampling tube to the open hole or screened interval of the observation well can minimize the flow rate and volume required in order to quickly obtain a sample representative of downhole conditions.

107

Site Access

Adequate site access is important to facilitate delivery of chlorine cylinders or other disin-fection chemical supplies as well as pump trucks. Road access also should be provided so that cars can get to the wellhead, rather than just four-wheel drive or track vehicles. There needs to be space for equipment to pull the pump periodically, otherwise pump pulling costs can increase substantially.

Telemetry

Provision of telemetry for monitoring and control is frequently desirable, particularly with larger ASR systems, in order to reduce operational labor requirements. Not only does this facili-tate routine operations, but it can also simplify data collection, monitoring, and reporting require-ments. ASR systems are changed from recharge to recovery mode typically once a year. In addition, periodic backflushing during extended recharge periods requires changing from recharge to recovery and back again. Depending upon local hydrogeology, recharge water quality and materials of construction that may or may not generate rust, this may occur every few days to every few weeks. Adjustment of flow rates may occur more frequently, during both recharge and recovery. Telemetry control may include some or all of the following functions:

- pump on-off
- pump failure alarm
- recharge pressure control valve setting
- recovery pressure control valve setting
- water level in ASR and observation well
- chlorine residual
- flow rate (both recharge and recovery)
- valve operation, such as for gravity and pressure recharge, recovery to waste, recovery to the distribution system, and trickle recharge flow during storage periods.
- conductivity probe
- turbidity probe

The telemetry monitoring and control system should provide adequate capability for data storage and processing, and for preparation of reports to track cumulative storage volume, water levels, water quality, and operational performance. It should also include a physical or software lock to prevent inadvertent discharge of turbid water into the treated water distribution or collec-tion system upon initiation of recovery.

Telemetry systems work best when they operate continuously and are properly main-tained, similar to the variable frequency drives. Lack of operation for an extended period, such as for several months during ASR recharge and storage, can lead to moisture buildup in the elec-tronics due to condensation, causing a malfunction. Keeping these control systems energized and in operation is necessary to ensure reliable performance. Alternatively, air conditioning could be provided to ensure reliable operation.

The telemetry system should be capable of shutting down recharge or recovery operations rapidly if needed, such as during a major fire or pipeline break when all available water is suddenly required to maintain system pressures. It should also be capable of rapidly shutting

108

down recharge at times when the source water quality is unsuitable so that poor quality water is not placed into storage. This might be due to high levels of taste and odor, turbidity, regrowth of bacteria in the water conveyance system, or other such situations that occasionally arise.

At some larger ASR wellfields such as San Antonio Water System and Calleguas Municipal Water District, the SCADA system incorporates a "scheduler program" that is relied upon to guide ASR operations so that the most efficient well is brought online first and the least-needed well is brought on last. Such a program should also ensure that water storage is distributed appropriately among the ASR wells so that buffer zones around each well are maintained and that storage bubbles are not shifted laterally underground to any significant extent due to well interference. This is typically a transitional problem at the beginning of wellfield development in a brackish aquifer when cumulative storage volumes are relatively small. For a properly designed ASR wellfield the need for such careful control will reduce once the storage bubble becomes larger and extends beneath the entire wellfield.

For a fully-developed ASR wellfield in a hydrogeologic setting where regional aquifer water levels may be substantially impacted by recharge and recovery operations, it is important to consider where the pumps are operating on their pump curves, to ensure that overpumping does not occur when water levels are high.

Filtration

Filtration of recharge flows is not normally provided at ASR wellheads since the recharge water is usually treated drinking water. However particulates are often present in the recharge water, picked up in the transmission/distribution system between the water treatment plant and the ASR well. At one ASR site, Gordons Corner, New Jersey, recharge flows are passed through a gravity sand filter to improve recharge performance. It is likely that wellhead filtration will be needed for future ASR wells storing partially-treated surface water, and for some sites storing drinking water, particularly in fine-grained aquifers. The ASR wellfield at Marco Lakes, Collier County, Florida, utilizes a series of pressure filters to pretreat recharge water obtained from a surface water source. The ASR well at West Palm Beach recharges high quality surface water that is passed through a strainer prior to storage in a confined, karst limestone brackish-water aquifer. Strainers are also being provided for each of the ASR wells at the Calleguas Municipal Water District, California, to filter out pieces of cement lining from the tunnel and pipeline transmission system bringing water to the wellfield.

Degasification

No ASR wells to date are known to have included removal of entrained gases in the recharge water, however the need for this has been considered a few times in recent years. The cost and complexity of doing this may be relatively small. The primary benefit at some sites will be in the reduction of dissolved oxygen (redox potential) in the recharge water. This would reduce the potential for subsurface geochemical and microbial reactions that may otherwise release metals from the storage zone aquifer matrix or cause oxidation or precipitation. It would also reduce the potential for oxidation of the well casing for carbon steel-based wells. The net effect should be the same, whether this is accomplished by creating a vacuum in the well casing annulus, or by creating a nitrogen blanket in the annulus.

109

Degasification of recovered water has also been considered at some ASR sites. Whether air in the recovered water is the result of cascading water during recharge, air leaks in piping, or entrapment in a semi-confined aquifer due to rising water table levels, it has to be resolved in order to make beneficial use of the recovered water. If the event causing the air is noticed early, there is a chance that the well can be cleared by short term pumping to waste. If the cause is more severe and prolonged, there may be the need for extensive surge development or keeping the well out of service for an extended time. Expensive air stripping strategies may need to be employed in the most severe conditions where continuous production of "foamy" water occurs. This can be costly with the installation of stripping towers, a new transformer, a new booster pump to bring the water back to distribution system pressure, and added electrical operating costs. Air in the storage zone can cause geochemical precipitation and plugging due to enhanced microbial activity in the aquifer. These are all good reasons for avoiding design and operational measures that may introduce air into an ASR well. These problems can be readily alleviated for sites where recovered water is discharged to a ground storage reservoir.

pH Adjustment

At a few ASR sites the recharge water quality may vary seasonally for a variety of reasons. At times low pH values may result in undesirable subsurface geochemical reactions such as dissolution of metals, or metal corrosion, while high pH values may risk calcium carbonate precipitation. At other ASR sites the recovered water may have a reduction in pH which, when combined with normal disinfection practices, causes the pH of the water going to the distribution system to be too low. For these situations it is necessary to provide a tap in the wellhead piping to provide pH adjustment of either the recharge or recovered flows. Adequate space is also needed for storage of whatever chemicals are required to achieve the pH adjustment, and the associated injection facilities.

Standardized Well Equipment

As ASR wellfields expand in successive phases, adding new wells and associated wellhead facilities, it is important to consider standardization of well and wellhead equipment, casing size, piping and instrumentation. This will improve reliability and minimize down-time during recharge and recovery operations.

ENERGY RECOVERY

Where depth to static water level is substantial, the opportunity for energy recovery may be considered. Careful selection of the appropriate pump and bowl assembly is required in order to accommodate reverse rotation and power generation, as well as the pumping requirements in the ASR well. In particular, use of a two-speed motor probably would be required; a higher speed for pumping and a lower speed for power generation. A less desirable alternative is to design the system so that the pump motor is disengaged during recharge while a second motor is connected through a right-angle gear drive. Electrical modifications would be required for both methods.

The kilowatt output capability of a typical turbine is approximated by the following formula:

$$kw = (1.88 \times 10^{-4}) \times Q \times H \times \text{turbine efficiency}$$

where   Q = flow in gallons per minute
                                H = net head in feet

Typically, the expected energy produced by well pump/turbines of this type is approximately 30% of the well production brake horsepower. Limited experience to date with power generation during recharge of ASR wells suggests that this is cost-effective wherever a need for electricity exists nearby by the agency or utility owning the ASR wells. If the electricity is sold to the energy grid, the payment received likely is too low to justify the additional capital and operating expense incurred by the owner of the ASR wells. However this may change in the future, particularly if the produces energy is valued more highly.

DESIGN OF ASR WELLFIELDS

Design of an ASR wellfield differs from design of a production wellfield whenever mixing between stored and native water is to be minimized. Mixing can occur due to two primary situations: dispersion around each ASR well, and advective movement of stored water away from the well.

Where no significant difference in water quality occurs, or where the intended use of the recovered water is such that any mixing is acceptable, then conventional wellfield design procedures relating to spacing and arrangement of wells are applicable.

As with any wellfield, it is important to purchase sufficient land initially to encompass the current and projected buildout needs. In recent years, obtaining suitable well and wellfield sites has become increasingly challenging. When a suitable site is obtained, it is usually cost-effective to apply design approaches that maximize the associated yield, service life and efficiency of the wellfield facilities. Sufficient land must be acquired to provide for construction of replacement wells at a later date.

Wellfields are usually best located close to where the water is most needed during recovery. Recovery water demands are usually, but not always, greater, and of shorter duration, than recharge water flows. This provides the opportunity for conveyance of water during low water demand periods from the source location to the storage location. However the storage location can be elsewhere, including in a different aquifer or a different hydrologic basin, so long as the associated water conveyance cost and any associated water losses are deemed acceptable. Many communities, and at least one state, are storing water at suitable hydrogeologic locations far removed from their service area.

DISPERSIVE MIXING

Clustering of ASR wells provides the opportunity to create a bubble of stored water from the center of the bubble outward, thereby displacing poor quality native water away from the wellfield and minimizing trapped areas of this poor quality water. When designed and operated in this manner, ASR system performance can exceed that which would occur as a result of conventional wellfield design.

The difference lies primarily in the ASR well spacing, which tends to be closer than for conventional wellfield design. The spacing tends to be related more to the lateral extent of the stored water around each well at projected cyclic operational volumes, rather than short-term well interference effects during recharge and recovery. Higher pumping costs are incurred during ASR recovery periods compared to conventional wellfield design, due to increased well interference, however this usually small extra cost is more than compensated by the associated improvement in recovery efficiency associated with closer spacing of the ASR wells. For example, the ASR well-field for the City of Cocoa, Florida, includes six ASR wells around the periphery of the water treatment plant site on 25 hectares (60 acres) of land. The spacing between ASR wells averages about 183 m (600 ft), or approximately the theoretical radius of the stored water bubble around each well at its planned seasonal operating volume of about 61 Mm^3 (160 MG). This spacing is somewhat closer than would be appropriate for a conventional wellfield in the same aquifer. Native water at the ASR wellfield site in the storage zone beneath the water treatment plant has a chloride concentration ranging from about 400 to 1,200 mg/L and a total dissolved solids concentration of about 1,000 to 3,000 mg/L.

If ASR wells are spaced too closely together, the pumping water levels in the wells during recovery can be substantially lowered due to well interference, and conversely recharge flow rates may be inhibited. The potential may exist for movement of poor quality water into the storage zone from overlying or underlying aquifers through semi-confining layers during the period of ASR recovery. This mixing mechanism has been noted at three ASR wellfields.

In addition to spacing, well arrangement also affects ASR recovery efficiency in situations where mixing between stored and native water is to be minimized. To date, no ASR wellfield has been operated in such a way as to attempt to displace native water potentially trapped between ASR wells. However a few sites operate their wells in such a manner as to recharge in one or more wells and recover in one or more other wells, thereby achieving subsurface improvement in water quality. This situation has been addressed theoretically by Merritt (1985) for water storage in a brackish aquifer using different wellfield arrangements.

Where the storage zone contains brackish water, high nitrates, or some other deleterious compound, an ASR wellfield may be designed and operated to minimize mixing through radial recharge and recovery of the stored water. Recharge would commence in the center of the well-field and proceed outward, adding wells as the stored water front displaces native water past these wells. During recovery, the opposite procedure would be followed. Central wells may be designed to recharge and recover at equal and also higher rates than peripheral wells in order to stabilize wellfield operational flow rates during recovery.

Where a radial wellfield arrangement is incompatible with available site constraints or with local geology, a linear arrangement incorporating some of the same design considerations may be appropriate. For example, a central row of higher yield ASR wells could be paralleled with two adjacent rows of lower yielding ASR wells. Initial recharge and late recovery would occur in the central wells while other ASR operations would occur in all wells. This is shown in Figure 2.3.

ADVECTIVE MIXING

ASR wellfields are subject to lateral or "advective" movement of the stored water away from the well at a rate that is usually very slow, depending upon the regional gradient and the aquifer hydraulic characteristics. The lateral distance that the stored water moves between

112
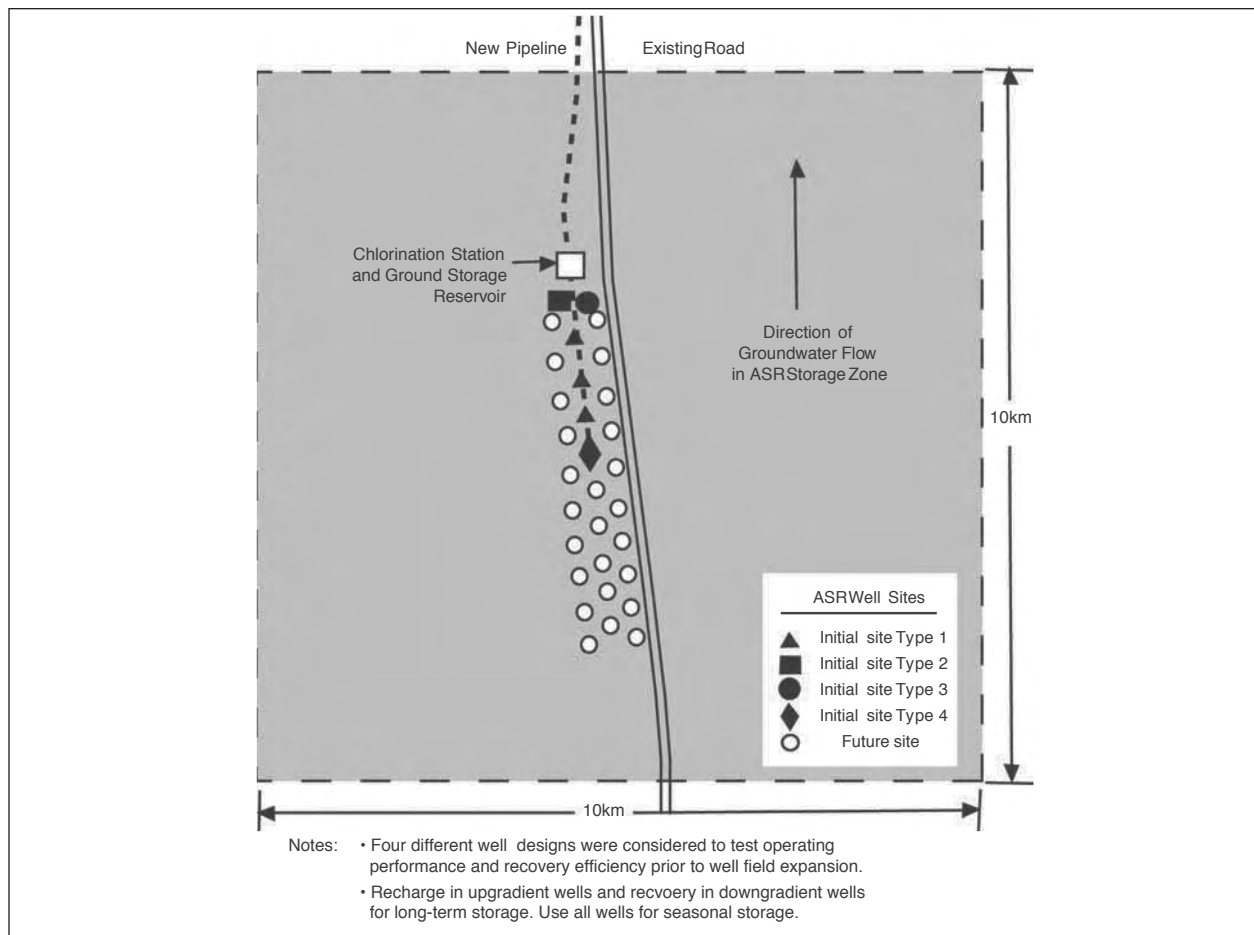
Figure 2.3 Strategic water reserve: proposed well layout, Kuwait

recharge and recovery is usually insignificant when compared with the radius of the stored water bubble during a typical recharge and recovery cycle. It is not unusual for the cyclic volume stored to occupy a theoretical radius of a few hundred meters around the ASR well, whereas the lateral movement of the storage bubble typically may displace this volume at the rate of only a few meters per year. Consequently, the loss in recovery efficiency is slight and therefore difficult to detect.

Some ASR wellfields may potentially store water in aquifers for which the background lateral rate of movement is significant relative to the radius of the stored water during a typical ASR cycle. For example, the cycle may entail water storage for several years to bridge drought/flood periods or to meet emergency needs. Alternatively, the storage zone may be an unconfined aquifer, which typically has a greater rate of groundwater movement than a confined aquifer due to steeper hydraulic gradients, local recharge, and lateral movement toward drains such as ditches, lakes or rivers. In these situations, improved recovery efficiency should be possible by elongating the wellfield design in the direction of expected regional groundwater flow, providing for a greater portion of recharge in upgradient wells and a greater portion of recovery in downgradient wells.

113

[Figure 2.3](#) shows an example of this kind of situation. It is a conceptual layout of an ASR wellfield to store drinking water in a brackish, confined limestone aquifer in Kuwait, designed to help meet seasonal peak demands during summer months and also to provide a strategic water reserve for emergency purposes. The regional gradient would not be a significant factor affecting recovery efficiency for annual ASR cycles; however, that portion of the potable supply in long-term storage to meet emergency needs would be subject to lateral or advective losses. Hence, the wellfield is arranged in a linear fashion in the direction of regional groundwater flow. This well-field has not been constructed, however an alternate ASR wellfield project to store reclaimed water for agricultural irrigation seasonal use is planned.

ALTERNATIVE ASR WELLFIELD CONFIGURATIONS

In the United States, the ASR regulatory framework provides for storage of water underground but does not provide any allowance for natural treatment of the water during storage. In the Netherlands, conventional practice for several decades has been to rely upon aquifers for treatment of water to provide disinfection and other improvements in water quality, however no provision has been made for storage of water in the same wells. This dichotomy points the way to an adaptation of ASR technology to provide both treatment and storage. To date this has not been applied anywhere, however it is anticipated that this approach will soon be implemented, possibly first near Adelaide, Australia.

A pair of ASR wells, or possibly a linear array of well pairs, would be constructed. For each pair of wells, recharge would occur in one well while recovery would occur in the other well. Well spacing would be sufficient to provide whatever natural treatment is needed, probably in the range of 50m to 100m. Recharge and recovery operations would be conducted in such a manner that both wells are enveloped by the stored water during recharge periods, providing a seasonal or emergency supply of water to meet ASR objectives. Each well in a well pair would be fully equipped for both recharge and recovery, providing operating flexibility to accommodate variability in recharge and recovery flow rates and also backflushing capability, however recharge would occur preferentially in one well and recovery in the other well.

This concept seems particularly applicable for areas of the world where a great need exists for seasonal water storage, however the cost of providing full conventional treatment of season-ally available storm water to meet all drinking water standards prior to ASR recharge is prohibitive. Pretreatment of the recharge water to reduce turbidity and suspended solids to acceptable levels, combined with periodic backflushing of the ASR well, followed by natural treatment during subsurface storage and flow to the recovery well, may be sufficient to ensure a reliable supply of seasonally available water of acceptable quality.

A logical extension of this approach is to combine storage, treatment and conveyance during wellfield design. This is further removed from the ASR concept but is still a viable water management approach in some areas, particularly where ambient groundwater quality in the aquifer is fresh everywhere. Water is injected into a well when available and travels through the aquifer to one or more recovery wells in the same aquifer. Distance to the recovery wells is such that the associated travel time may be several years, or even hundreds of years. However, short term needs are met and the overriding commitment to recharge the aquifer is achieved while avoiding the need for construction of expensive pipeline and pumping facilities. This is the practice in El Paso, Texas, where highly treated reclaimed water is recharged into eleven ASR wells in the Hueco Bolson aquifer while recovery by the City's water supply wells occurs sufficient distance

downgradient so that travel time exceeds one year and adequate natural treatment is provided. This is common practice also in Colorado and Arizona. A common denominator for these projects is that the recharge wells require backflushing periodically in order to maintain their recharge capacity. Equipping these as ASR wells therefore ensures sustainable operations.

STACKING

Where more than one potential storage zone is available, ASR wellfields may utilize the concept of "stacking." Instead of adding more wells in a single storage zone, with associated pumps, pipelines, powerlines, access roads and supporting facilities distributed over a wide area, wells may be added in two or more storage zones at the same site. This has the effect of concentrating ASR wellfield facilities into a small area, reducing construction and operation costs. Wells are often the least expensive part of an ASR wellfield. The wellhead and piping facilities are often more expensive. Where these can be reduced, the economic savings can be substantial.

An excellent example of this is at Peace River, Florida, where ASR storage currently occurs in the Tampa Formation at a depth of 400 to 500 feet (122 to 152 m), and also in the Suwannee Formation at depths of 700 to 900 feet (213 to 274 m). Both of these formations yield approximately 1 MGD (3.8 ML/d) to a well. An underlying aquifer at 1,100 to 1,300 feet (335–396 m), the Avon Park Formation, holds promise as a third potential storage zone with potential individual well yields exceeding 3 MGD (11 ML/d) recovery capacity. A test well has been constructed in that zone but testing has not yet been completed. Other similar opportunities for ASR well stacking are common.

OBSERVATION WELLS

Observation well considerations for ASR wellfields are more complex than for conventional production wellfields. First, it is advisable to monitor the regional and local gradient in the potentiometric surface for the storage zone so that the direction and rate of movement of the stored water can be estimated. Water levels will vary during recharge and recovery periods, so at least monthly measurements over a period of a year will indicate typical local mounding and direction of movement. Several observation wells may be needed if the regional gradient is not already known. In conjunction with pump test data, the potentiometric surface gradient will then provide a basis for estimating the rate of movement. Typically annual movement of the stored water bubble at an ASR well is not a significant operating factor, particularly in confined aquifers, however it is wise to address this issue during wellfield design and operation.

Observation wells may be needed in overlying and/or underlying aquifers to monitor water level or pressure response to ASR operations. This is particularly important where confining characteristics of the storage zone will determine the viability of the ASR program. Upconing movement of brackish water from beneath a storage zone due to extended recovery and/or large drawdowns can be monitored using a "lower zone monitor well," providing early warning of when a breakthrough of saline water may be expected. Impacts of ASR operations upon the water table and any associated wetlands can be monitored using a water table monitor well.

Storage zone monitor wells are provided at many sites, addressing questions regarding hydraulic response of the aquifer to ASR operations; lateral movement of the stored water bubble; changes in water quality with distance from the ASR well; aquifer dispersivity; well plugging; well efficiency and other issues. The first storage zone monitor well is frequently utilized as an

115

exploratory well, gathering extensive data during construction and testing with which to guide final design of the ASR wells and remaining observation wells.

At Beaverton, Oregon, a detailed wellhead protection plan was implemented, including installation of several observation wells at the 10-year time-of-travel boundary, as an early warning system to prevent contamination of the stored water in the ASR wells. A similar wellhead protection plan protects the Miami-Dade County, Florida, West Wellfield, which includes not only Surficial Aquifer public supply wells but also 15 MGD of recovery capacity from three ASR wells.

WELL OPERATION RECOVERY EFFICIENCY

Recovery efficiency usually has little significance where both stored water and native groundwater are potable. In such situations, the main concerns are usually aquifer plugging and redevelopment frequency. However, to the extent that the difference in water quality between stored and native water is significant so that mixing has to be controlled, recovery efficiency can become an increasingly important factor in the assessment of ASR feasibility. Recovery efficiency can be defined as:

> The percentage of the water volume stored in an operating cycle that is subsequently recovered in the same cycle while meeting a target water quality criterion in the recovered water.

If 1 Mm^3 (264 MG) of drinking water is stored in a brackish aquifer and subsequently 0.8 Mm^3 (211 MG) is recovered before the total dissolved solids (TDS) concentration of the recovered water exceeds a target criterion of 500 mg/L, then the recovery efficiency for that ASR cycle is 80%.

A key element of this definition is that it is based upon volumes stored and recovered. It may be of theoretical interest to some individuals to evaluate recovery efficiency based upon percentage recovery of a tracer in the recharge water. Sometimes referred to as "counting the molecules," this approach will always lead to a lower estimate for recovery efficiency since it eliminates any allowance for mixing between stored and native water. Such mixing can occur without any adverse effect upon use of the recovered water, so long as the degree of mixing is within the limitations of the water quality criteria for the recovered water. However, most people interested in ASR are less concerned about whether the same molecules are recovered that were injected, but are more interested in knowing the volume of water recovered that is useful for their intended purpose. An illustration of this difference is as follows:

> Assume for the example above that the average recharge TDS concentration is 200 mg/L; background TDS concentration in the aquifer is 1,000 mg/L; the drinking water standard for TDS is 500 mgL; and that during recovery, TDS concentration increases as shown in Figure 2.4, reaching the target criterion of 500 mg/L TDS at 80% recovery.
>
> The recovery efficiency is 80%. However, at the beginning of recovery, the water is 100% recharge water, while at the end of recovery the water is a blend of 62.5% recharge water and 37.5% native water. Integrating beneath the recovery water quality curve suggests that about 70%, or 185 MG, of the actual stored water

116
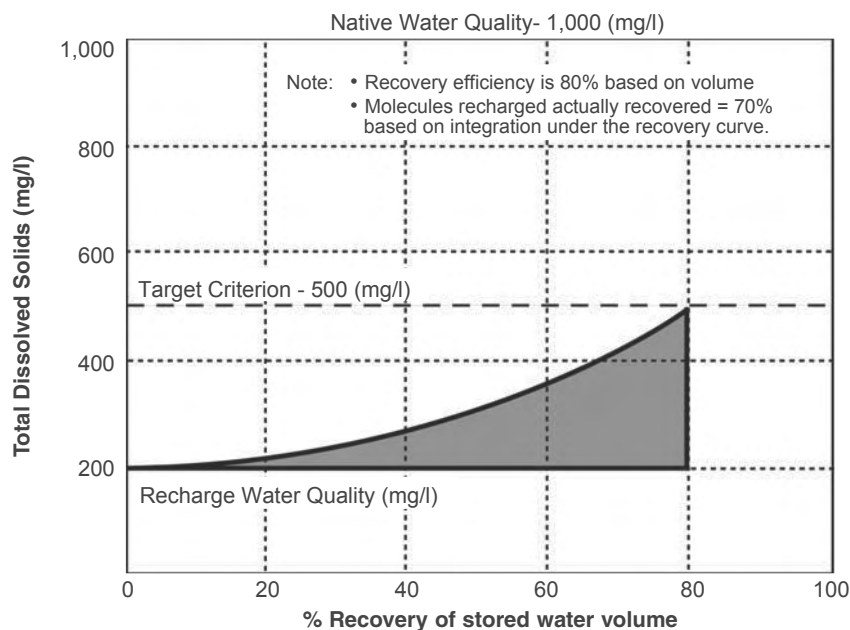
Figure 2.4 ASR recovery efficiency example

was recovered during this cycle, while the balance (79 MG) was from native water in the aquifer.

In practice, the difference in analytical approaches is sometimes more significant than this example would suggest. By suggesting a lower percentage recovery, the second approach illustrated in the example can cause confusion among non-technical decision-makers trying to understand and evaluate the results from an ASR test program. The confusion can easily contribute to some loss of confidence in the program. It is much simpler to follow the recommended definition of recovery efficiency consistently, while being aware that individuals with a theoretical rather than an operational interest may occasionally ask valid questions regarding recovery efficiency calculated as performed in the example.

A second key element of the definition of recovery efficiency is that the target water quality criteria can easily vary from site to site, depending upon hydraulic, regulatory and other factors. Most ASR sites are located at water treatment plants or at locations in the water transmission or distribution system where blending can occur between recovered water and water flowing through the plant or distribution piping. So long as the water quality of the blend meets applicable drinking water standards, regulatory criteria are met. Consequently, it is usually not necessary to terminate recovery when drinking water standards are reached. Recovery can continue until such higher concentration is reached that the blend going to the consumer approaches but does not exceed applicable standards. Obviously, the target water quality criteria will depend upon a number of factors such as the available blend with water from other sources during recovery periods, water quality for these other sources, and local regulatory constraints.

For the situation where the ASR well is located within the distribution system and consumers may receive ASR recovered water directly without any blending, then drinking water

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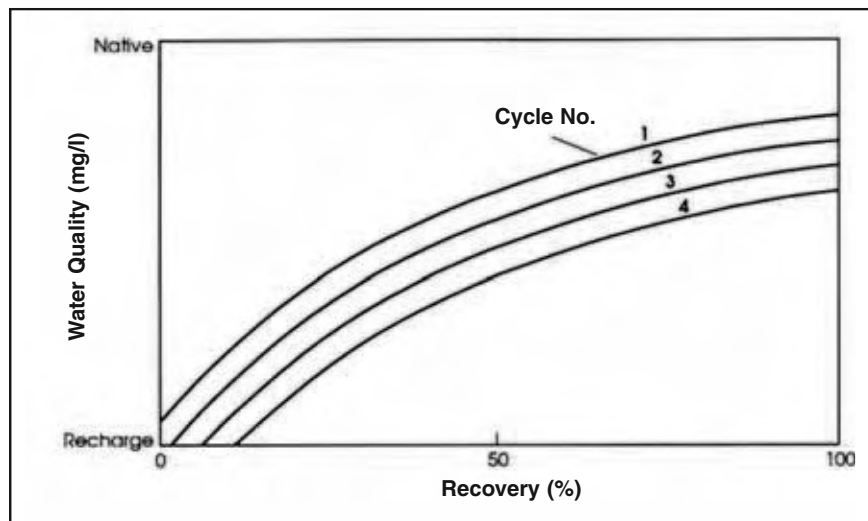
Figure 2.5 Water quality improvement in successive cycles

standards will govern the target water quality criterion. This is uncommon, based upon experience to date.

WATER QUALITY IMPROVEMENT WITH SUCCESSIVE CYCLES

Recovery efficiency tends to improve with successive cycles when the same volume of water is stored in each cycle. This is because the residual water not recovered in one cycle becomes a transition or buffer zone of marginal quality surrounding the stored water in the next cycle. This is illustrated in Figure 2.5, which is based upon data from several operational sites. At each of these sites native water quality is worse than recharge water quality.

Building the buffer zone around each ASR well is usually completed over a series of cycles, typically about three to six, at the end of which the ultimate recovery efficiency for the site is achieved. However, it can also be completed at one time by storing an initial large volume of water in the well immediately after construction, and then proceeding with the expectation of achieving the ultimate recovery efficiency of water stored from that point on. This is closely analagous to filling a reservoir following dam construction, before using the reservoir for water supply, recreational or other purposes.

The financial investment in stored water that is required to achieve the ultimate recovery efficiency at a site is usually quite small relative to the cost of the ASR facilities. This investment is made with water produced during off-peak months and therefore has relatively low marginal costs, reflecting only electrical power, chemicals and residuals disposal. The investment may be made over a period of several years through successive full-scale cycles in which increasing volumes of water are recovered each year. Alternatively, it may be made up front during several weeks or months of continuous recharge and no recovery. The value of the buffer zone water invested is usually quite small relative to the savings achieved by proceeding with an ASR solution to water supply needs.

118

Table 2.5
ASR recovery efficiency in brackish aquifers

Site	Native Water TDS (mg/L)	Recovery Efficiency (%) (a)
Peace River, Florida	700–920	100
Cocoa, Florida	1,000–2,000	100
Port Malabar, Florida	1,320	100
Boynton Beach, Florida	3,910	98.6
Marathon, Florida	37,200	40–75 (b)
Lake Okeechobee, Florida	7,000	38 (c)

NOTE: (a) Ultimate recovery efficiency after initial formation of underground reservoir.
(b) Range reflects duration of trickle flow of about 50 gpm to offset losses due to density stratification.
(c) Incomplete development after four small cycles.

The ultimate recovery efficiency attainable at any site has to be determined through testing and operations. At most ASR sites, 100% recovery efficiency is attainable; however, the number of cycles of operation to achieve this level may vary, as may the volume of buffer zone water invested. Where the buffer zone is formed initially at one time, the number of cycles required to achieve full recovery efficiency is greatly reduced.

Where successive cycles are implemented instead of initial formation of the buffer zone, but 100% efficiency is not attained after several cycles, several factors may contribute to this result:

- inappropriate ASR well or wellfield design or operation
- testing at too small a scale for the storage zone
- insufficient number of cycles to develop the storage zone
- increasing volumes on successive cycles
- density stratification in highly saline aquifers
- high transmissivity of storage zone, particularly with more brackish or poorer water quality aquifers
- advective loss of stored water due to regional hydraulic gradient in the storage zone
- regulatory constraint designed to achieve aquifer recharge by requiring that a certain percentage of the water remains underground

Table 2.5 presents selected results for improvement in recovery efficiency with successive cycles for several ASR sites in brackish artesian aquifers in Florida. All show improvement in recovery efficiency with successive cycles; however, not all attained 100% recovery efficiency.

Those that have not include Marathon in the Florida Keys and Lake Okeechobee. Marathon utilized a storage zone containing seawater, while Lake Okeechobee utilized a very transmissive, thick storage zone containing very brackish water. Neither of these sites reached 100% recovery efficiency. Boynton Beach has reached about 99% recovery efficiency. As discussed below, recovery efficiency below 100% may still represent a wise and cost-effective water management decision.

119
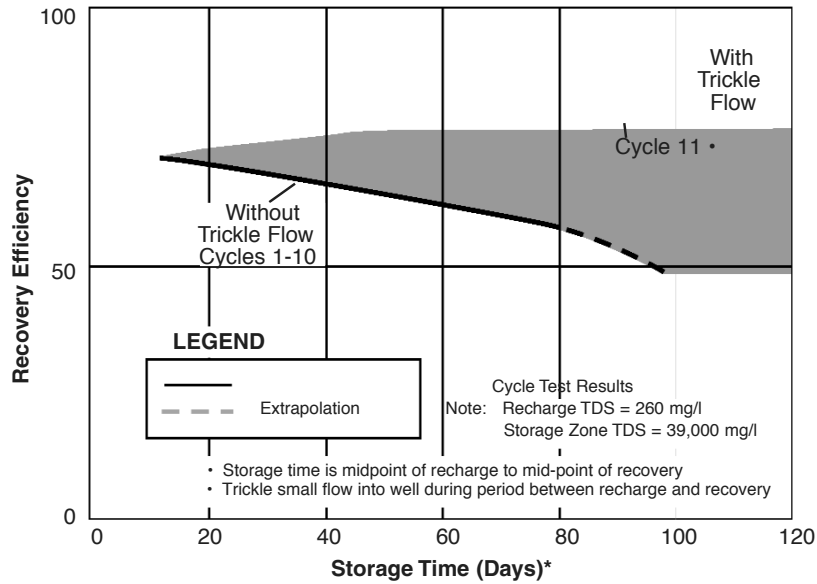
Figure 2.6 ASR recovery efficiency, Marathon, Florida

The Marathon site, currently not operational, had a 40 ft (12 m) thick storage zone in a sand aquifer containing seawater with a TDS concentration of 37,200 mg/L, causing a substantial tendency for density stratification. As shown in Figure 2.6, ultimate recovery efficiency is primarily related to storage time at this site, and secondarily related to storage volume.

Expected recovery efficiencies under long-term operating conditions at the Marathon site are in the range of 50 to 75 percent. The annual investment in water not recovered is small compared to the cost of other alternatives to supply water during emergencies that may occur, such as loss of water treatment or transmission facilities during a hurricane. The planned operating strategy at this site was to store a given volume immediately prior to the hurricane season, maintain a target recovery volume during the hurricane season by adding a trickle flow of water to offset density stratification losses, and then to recover the water during the following winter peak demand season. A seawater desalination treatment plant alternative was subsequently constructed at double the capital and operating cost.

The Lake Okeechobee ASR well has a storage zone TDS concentration of 7000 mg/L. The aquifer transmissivity is very high, about 60,000 m²/day (4.5 million G/day/ft). Furthermore, the well was designed for disposal, not recovery, and all testing to date has been at a scale too small to properly draw conclusions regarding attainable recovery efficiency. Nevertheless, any recovery efficiency greater than about 40 percent at this site represents a net gain to the water management system since evapotranspiration and seepage losses associated with surface reservoir storage and canal conveyance are at least 60 percent. Water not recovered from storage in this aquifer at ASR sites along the coast will ultimately benefit the region since the aquifer is increasingly being relied upon for brackish water supply to desalination treatment facilities, which tend to be located in coastal areas. The lost ASR water will recharge the aquifer and may eventually tend to reduce the TDS concentrations. Experience gained from this ASR test well, combined with experience from other ASR demonstration wells, is providing the basis of design for a planned ASR wellfield with

120

Table 2.6
Backflushing frequencies at selected operational ASR sites

Site	Backflushing Frequency	Lithology
Wildwood, New Jersey	Daily	Clayey sand
Gordons Corner, New Jersey	Daily	Clayey sand
Peace River, Florida	Seasonal	Limestone
Cocoa, Florida	Seasonal	Limestone
Palm Bay, Florida	Monthly	Limestone
Las Vegas, Nevada	Seasonal	Alluvium
Chesapeake, Virginia	Bimonthly	Sand
Seattle, Washington	Weekly	Glacial drift
Calleguas, California	Monthly (approx.)	Sand

330 ASR wells, each with 20 ML/d (5 MG/day) design capacity, as part of the Comprehensive Everglades Restoration Program.

Boynton Beach has a storage zone TDS concentration of 3,910 mg/L. ASR operations from 1993 to 2004 have included 21 cycles. Recovery efficiencies started at 28 percent on the first cycle, climbed to 80 percent on the seventh cycle and most recently were calculated to be at about 99 percent. The target volume in storage from 1993 to 1998 was about 60 MG, however after a severe drought this target was increased to over 200 MG so that the ASR well would provide not only seasonal storage volume but also emergency storage. A second ASR well has recently been constructed.

Two other operational Florida ASR sites shown in Table 2.6 have achieved 100 percent recovery efficiency in aquifers that have TDS concentrations in the range of 1,000 to 1,320 mg/L. A fourth site, Peace River, has just completed a fourth phase of ASR expansion to include 21 wells and is still building the expanding buffer zone volume. Operating results to date suggest that full recovery efficiency should be attained. The storage zone TDS concentrations at Peace River range from 700 to 1,000 mg/L.

When 100 percent recovery efficiency is not expected or attained, it is pertinent and usually necessary to consider the value of the water that is not recovered. This can be determined on an annual basis and compared with alternative water management measures using standard economic methods. The value of the lost water is usually quite small, representing a cost-effective water management alternative.

Where water supply is quite limited, or prices are already high due to major capital investments in treatment and transmission facilities, public reaction to the apparent loss of water during development of an ASR wellfield can be a more difficult problem to handle than the actual value of the "wasted" water. As such, it is always advisable to strive for as high a recovery efficiency as possible, using whatever tools are available to achieve this end. Careful site selection, well design and operation are major factors in achieving this goal. Other important factors include careful control of expectations of ASR program early results, particularly in higher risk situations.

ASR testing in storage zones containing brackish or poor quality water usually includes at least three cycles with the same volume stored, in order to evaluate the trend in recovery efficiency improvement. If storage volumes in successive cycles vary, different recovery efficiencies will result in each cycle and may or may not show an improvement with successive cycles. For

121

example, following a series of equal, larger volume test cycles with a smaller volume cycle may substantially increase recovery efficiency in the smaller cycle due to the relatively large buffer zone available from the earlier cycles. Conversely, a series of test cycles, each of which is larger than the one before, will tend to reduce or eliminate any increase in recovery efficiency between cycles since the buffer zone formed from the previous cycle is small relative to that required for the larger subsequent cycle. Similarly, recovery of all water stored during each cycle until background water quality is achieved will always tend to demonstrate relatively low recovery efficiencies, even if successive cycles utilize increasingly large storage volumes.

WATER QUALITY DURING THE INITIAL ASR CYCLE

The first ASR cycle at a new site provides a unique opportunity to gather useful data that can provide an early indication of ultimate recovery efficiency and ASR performance. Once the first cycle is completed and residual water in the aquifer around the well no longer reflects background water quality, then evaluation of performance in subsequent cycles is more complicated because it has to be interpreted with careful consideration of previous operations. For this reason, the first cycle should be planned carefully and implemented under conditions that are as well controlled as possible.

In theory, it is always possible to repeat the first cycle after recovering to background water quality. This would provide the opportunity to correct problems that may have arisen or to vary some of the test conditions, such as altering the volume stored to determine the effect of operating scale upon recovery efficiency on the first cycle. In practice, this is not easy to implement. There are invariably practical, operational constraints that provide a strong incentive to build the buffer zone and achieve ultimate recovery efficiency at the earliest possible date, usually in time for the next anticipated operational recovery period.

The first cycle is usually designed to confirm that wellhead facilities are operating correctly, to gather preliminary data regarding aquifer hydraulic response, to determine geochemical and biological changes, to assess the recovery water quality response due to mixing in the aquifer, and to revise the remaining test program, if appropriate. The volume is usually small relative to subsequent cycles. Typically, recharge will occur for about a week, followed immediately by recovery.

Figure 2.7 shows the initial cycle recovery water quality results for several ASR sites in brackish aquifers. Of some interest is the difference in the shape of these curves. Sites such as Marathon, Chesapeake and Port Malabar (now Palm Bay) show very little mixing with surrounding brackish water until late in the recovery portion of the cycle. These three sites utilize relatively thin, confined aquifers for ASR storage. Conversely, Lake Okeechobee shows substantial mixing at the beginning of the cycle, reflecting the relatively small volume utilized for testing, the high transmissivity of the aquifer, the substantial aquifer thickness, and the high TDS of this zone, as discussed above.

The shape of the recovery water quality curve on the first cycle is an indication of the mixing or dispersion characteristics of the aquifer in the vicinity of the ASR well. Curve shapes that are initially flat, showing little or no mixing close to the well, are encouraging signs that successive cycles are likely to form a buffer zone that will support higher ultimate recovery efficiencies. Curves that are initially steep, showing mixing close to the well, are indicative of lower ultimate recovery efficiencies. Where the storage zone is fresh or only slightly brackish, the curve shape may not be very significant. However, where the storage zone is very brackish and little
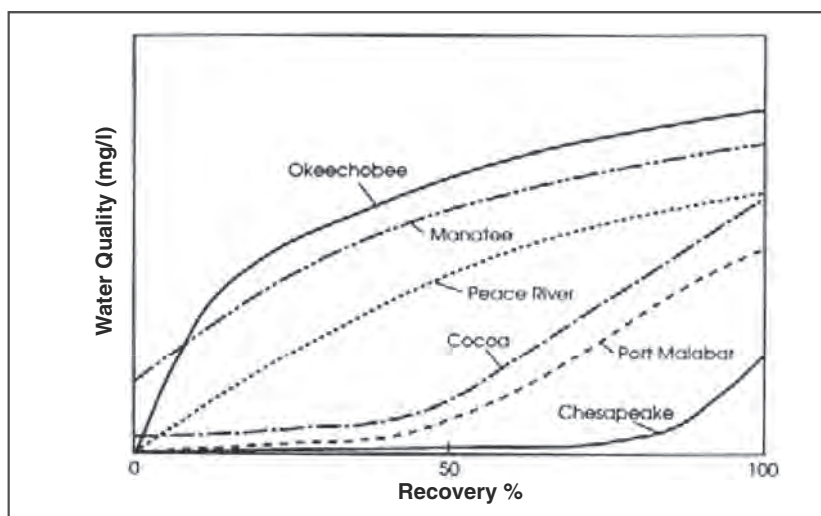
Figure 2.7 Water quality during initial cycle recovery

mixing can be tolerated in the recovered water, the curve shape needs to be reasonably flat at the beginning of recovery in order to sustain expectations for ASR storage zone development to achieve high recovery efficiency.

The shape of the recovery curve is only determined following expenditure of much time, money and effort for preliminary investigations, design, permitting and well construction. There is only so much that can then be done to improve recovery efficiency with successive, equal volume cycles once the facilities are constructed and initially tested. A preferred approach to quickly achieving ultimate recovery efficiency is to initially build a buffer zone around the well so that mixing and geochemical reactions occur at some reasonable distance away from the well at all times. Development of this "Target Storage Volume (TSV)" approach is discussed below. For many ASR sites with substantial mixing characteristics in a storage zone containing poor quality groundwater, initial recharge with a very large volume of water to form the buffer zone may be a successful strategy for achieving acceptable recovery efficiency.

Occasionally, initial test results may indicate that the well requires partial backplugging to eliminate a zone of poor water quality at the base of the storage zone or to improve lower confinement. Following such corrective action, the test program would then proceed.

Assuming that the best available zone for the intended use has been selected, and that the well is designed and constructed appropriately, the remaining variables that can be used to improve recovery efficiency are primarily operational: storage volume, recharge and recovery rates, and storage time between recharge and recovery. Results from the first cycle then can be used to adjust the planned test program so that recovery efficiency is enhanced. This may entail use of larger storage volumes, higher recharge and recovery rates, shorter storage times than perhaps originally planned, or addition of a trickle flow to the well during the storage period to compensate for losses due to density stratification. Through data collected during the test program, reasonable ranges for these operating variables can be determined to support ASR feasibility assessment and to guide subsequent planning, operations and ASR expansion.

123

TARGET STORAGE VOLUME

The conventional approach to ASR facilities development is to conduct a series of operational cycles, monitoring water levels and water quality to determine system performance and improvement with successive cycles. Having observed this consistently at numerous sites it became apparent that a more rapid and cost-effective approach to ASR development would be to initially develop the buffer zone around an ASR well so that subsequent cycle testing would start out as close to 100% recovery efficiency as possible. The Target Storage Volume (TSV) is defined as the sum of the volume of water required to form the buffer zone separating stored water from surrounding ambient groundwater, plus the volume of water required for recovery to meet the objectives of the ASR program.

This total volume of water is recharged upon completion of construction and baseline testing of well and wellhead facilities, and prior to initiation of cycle testing. If the correct volume is recharged, subsequent equal-volume ASR cycles should achieve 100% recovery efficiency, or whatever is the ultimate recovery efficiency achievable for that site. Recharge of the TSV would occur most likely during times of the year when spare capacity is available at existing water supply, treatment and transmission facilities so that the cost of the stored water is minimized, representing only the marginal cost of the water. This would include power, chemicals and residuals disposal.

Estimation of the TSV is primarily based upon experience; however, a general range is about 50 to 350 million gallons (MG) per million gallons per day (MGD) of installed recovery capacity. The unit of measurement is therefore in days. The lower end of the range would tend to be for thin, sand and sandstone aquifers containing slightly brackish water, with ASR systems designed to meet seasonal variations in demand. The higher end of the range would tend to be associated with thick, heterogeneous or karst limestone aquifers containing more brackish water, with ASR systems designed to meet seasonal variations in supply, demand and quality. In particular, where the source water supply is unreliable, the TSV will increase as needed to ensure a sustainable supply from ASR storage.

A good example of TSV development is at Delray Beach, Florida. The ASR storage zone is a confined, limestone artesian aquifer containing brackish water with an ambient TDS concentration of 4,200 mg/L. The storage zone is 57 m (184 ft) thick and recharges or recovers water at rates of about 9.5 ML/d (2.5 MGD). The objective at this ASR site is to meet seasonal variations in demand for drinking water. The source water supply is groundwater from a different aquifer, treated to meet drinking water quality standards. An initial criterion was established to continue ASR recovery until the chloride concentration in the recovered water reached 225 mg/L, equivalent to a TDS concentration of 550 mg/L. Based upon limited experience at that time, a TSV of 950 ML (250 MG) was estimated and was then recharged. Recovery efficiency on the first ASR cycle exceeded 100 percent. The TDS criterion was then reduced by the City to 450 mg/L. The TSV was increased by a further 70 MG to offset the tighter standard for recovery water quality. After five operating cycles during a period of one year, each with a recharge volume of about 190 to 230 ML (50 to 60 MG), 100 percent recovery efficiency was once again demonstrated, associated with a TSV of 320 MG. For this site, the TSV corresponds to about 130 MG/MGD of recovery capacity, or 130 days.

Another example is at Peace River, Florida, for which the estimated TSV is 350 days. The principal reason for this large volume is that the source of supply can experience periods as long as about six consecutive months when no diversions are available from the river due to regulatory

low flow restrictions. Other contributing reasons are more common, including seasonal variability in water demand in the three-county service area, variability in source water quality even during times when the river flow is available for diversion, and other local design and operational constraints at the ASR wellfield.

Several factors comprise the estimation of the TSV, including native water quality, aquifer thickness, confinement, effective porosity, transmissivity, regional hydraulic gradient, variability in source water availability and quality, and variability in recovered water demand. The TSV approach is clearly applicable for situations where mixing and dispersion are the predominant mechanisms for changes in water quality underground during storage. For situations where the natural regional gradient or the imposed hydraulic gradient in the storage zone reduces recovery efficiency, or where the storage zone contains native water so brackish that a significant density difference exists between the recharge water and the native groundwater, the TSV approach should be combined with additional wellfield design and operational measures to compensate for potential losses of stored water.

Where the storage zone contains water quality that is fresh and essentially the same as the stored water, no buffer zone is required. In this case the TSV equals the stored water volume required for recovery. Where the storage zone contains water quality that has a TDS exceeding about 5,000 mg/L, the density difference between the stored water and the surrounding ambient groundwater can reduce recovery efficiency, particularly during extended storage periods. Adding a small trickle flow of recharge water during extended storage periods can compensate for losses due to density stratification.

In some situations the marginal cost of water required for creation of the TSV is quite high so that this becomes a significant financial investment, even when conducted during offpeak periods. At one such ASR site, located at Kiawah Island, South Carolina, the solution was to obtain water from an adjacent water supply well that was utilized for golf course irrigation and produced from a different aquifer containing water with a TDS of 1,600 mg/L compared to 6,800 mg/L in the ASR well. The buffer zone was formed substantially with water from the irrigation well, and was then followed with a much smaller volume of treated drinking water.

A significant advantage of the TSV approach is that it pushes away from the ASR well the mixing zone between stored water and ambient groundwater. This is the zone where geochemical reactions are most likely to occur, particularly during early cycles. Such geochemical reactions tend to attenuate with successive cycles at the same volume, however if they can be controlled so that they do not occur next to the well bore or well screen, then recovery water quality during initial cycles will be improved.

Simulation modeling has recently been conducted by Dr. Christopher J. Brown as part of a PhD dissertation (Brown 2005) at the University of Florida, addressing prediction of recovery efficiency in ASR wells. A numerical model comprising MODFLOW and MT3DMS was developed to estimate recovery efficiency associated with storage zones with TDS concentrations below 5,000 mg/L so that density stratification would be kept to an absolute minimum. Aquifer transmissivities were assumed to be 20,000 ft^2/day. Dispersivity estimates were set at one foot and 25 feet. Ambient groundwater TDS was assumed to be 4,000 mg/L while recharge TDS was assumed to be 150 mg/L. Hydraulic gradient in the aquifer was assumed to be 0.0001 and porosity 25%. The single layer model domain was 40 miles with 5 ft grid resolution at the ASR well. The model input data is representative of aquifer conditions in southeast Florida. Model baseline scenarios included six ASR cycles, each with 5 MGD recharge for 30 days followed by 5 MGD recovery until chloride concentration reached 250 mg/L. Once this simulation run was completed,

125

an alternate approach was tested. For each scenario with a dispersivity of one foot, a buffer zone of 100 MG was added prior to starting cycle testing. For each scenario with a dispersity of 25 feet, buffer zone volumes of 100 MG and 500 MG were added prior to starting cycle testing. Results of the "baseline" and "TSV" simulation runs were then compared.

For the low dispersivity scenarios, adding the buffer zone prior to cycle testing caused 100% recovery efficiency during the initial cycle. Instead of climbing slowly from 78% to 98% over six cycles, individual cycle recovery efficiency stayed at 100% for all six cycles. For the moderate dispersivity scenarios, adding a 100 MG buffer zone before beginning cycle testing caused initial cycle recovery efficiency to increase from 31% to 62%. Subsequent cycles showed steadily increasing recovery efficiency, reaching 82% after six cycles. Adding a 500 MG buffer zone caused initial cycle recovery efficiency to increase from 31% to 100%.

The model results tend to support field experience at a limited number of ASR sites, suggesting that initial development of a TSV is effective in achieving higher initial recovery efficiency at ASR wells. Further numerical modeling is warranted to investigate the effect of scale-dependent mixing and hydrodynamic dispersion on the selection of an appropriate TSV size. Future modeling results will help further refine the required TSV size for a range of hydrogeologic characteristics and operating conditions. Until such modeling is conducted and until additional field experience is gained at a broader range of operating ASR sites, the TSV approach to ASR well development will, at most sites, reduce the cost and duration of cycle testing at most sites and expedite achievement of ultimate recovery efficiency. The TSV should be developed upon completion of well construction and baseline hydraulic and aquifer performance testing, and prior to initiating cycle testing. Through forming the TSV, cycle testing can be abbreviated, placing the well into full operation at an earlier date.

At a few sites, concern has been expressed by some that TSV formation may push end products of subsurface geochemical and microbial reactions away from the ASR well, potentially contaminating adjacent portions of the aquifer. Water quality data obtained from monitor wells generally supports the conclusion that such reaction products typically are not found more than 200 feet away from the ASR well, even though the buffer zone may extend several hundred feet. Research is needed to better understand geochemical and microbial reactions occurring close to an ASR well, within a radius of about 150 feet.

WELL PLUGGING AND REDEVELOPMENT

Artificial recharge of groundwater through a well usually results in increasing resistance to flow, or head buildup near the well, which is referred to as "plugging" or "clogging." The primary sites of plugging are the gravel pack (if present), the borehole wall, and the formation immediately surrounding the borehole wall. Increased head buildup in the well, due to plugging, changes the hydraulic characteristics of the well. Plugging during recharge can result in a decreasing rate of recharge or the need to continually increase the recharge head to maintain a constant recharge rate. Plugging that occurs during recharge and remains during recovery, otherwise known as "residual plugging," will have a negative impact on pumping. Residual plugging increases drawdown during pumping (decreased specific capacity) and thus reduces the pumping rate and/or efficiency during pumping. Residual plugging is probably aggravated by increasing recharge pressure or water mounding to excessive levels in order to maintain recharge rates.

To mitigate the effects of plugging, ASR wells are periodically redeveloped by pumping. Single-purpose injection wells are typically redeveloped by installing a vertical turbine pump, by

126

air-lift pumping (sometimes with packer systems), or by swabbing and bailing with cable tool drilling equipment. ASR wells with a permanent pump can be redeveloped at more frequent intervals (daily, weekly, monthly, seasonal), whereas single-purpose wells are typically redeveloped at intervals of one year or longer. ASR wells are more suitable for the majority of applications where annual redevelopment is insufficient to maintain recharge capacity.

The preferred method of redevelopment is periodic pumping to minimize plugging and to prevent any lasting effects of residual plugging. Such redevelopment of a well is easily managed where a permanent pump is installed and where redevelopment flows can be conveniently discharged. Difficulties with discharging the redevelopment water require consideration of longer intervals between redevelopment activities. In situations where the number of ASR wells to be used is large and conditions are not ideal, the frequency of redevelopment and the means for disposal of water produced during redevelopment can be key issues in determining project feasibility and cost.

Since the rate of plugging during recharge ultimately determines the required frequency of redevelopment, it is appropriate to investigate the factors affecting the rate of plugging. With an increased understanding of plugging mechanisms, predictive tools can be used during the planning stages of ASR programs to estimate redevelopment requirements. An increased understanding of plugging also will be useful during operations in diagnosing the magnitude and origin of plugging, and in developing operations and maintenance guidelines.

PLUGGING PROCESSES

Researchers have documented a list of processes that are primarily responsible for plugging of recharge wells (Rhebun and Schwartz 1968; Sneigocki and Brown 1970; Leenheer, Malcolm and White 1976; Huisman and Olsthoorn 1983). These processes include entrained air and gas binding, deposition of total suspended solids (TSS) from the recharge source water, biological growth, geochemical reactions, and particle rearrangement in the aquifer materials adjacent to the well. Site-specific conditions such as aquifer and groundwater characteristics, well construction, recharge facilities design, and source water quality determine the influence of these processes on well plugging.

Each plugging mechanism or process is briefly described below, followed by a discussion of its relative importance. Figure 2.8 illustrates the typical relationship between time and resistance to flow for plugging caused by suspended solids, entrained air and biological growth.

During recharge, an increased resistance to flow results in an increase in the water level in the well. Comparing a graph of water level rise due to plugging with these typical curves can be a useful tool for diagnosing the cause of the observed plugging.

Entrained Air and Gas Binding

During recharge, air bubbles may be entrained by free fall of water inside the well casing or by allowing air to enter the recharge piping where negative pressures occur. If recharge water with entrained air is allowed inside the well, there is a danger that these air bubbles will be carried downhole, through the well screen, perforations or open hole, and out into the aquifer formation. For entrained air bubbles to move downward in the well casing, the downward velocity must exceed 0.3 m/sec (1.0 ft/sec), which is the rate at which 0.1 to 10 mm bubbles rise in still water (Leenheer, Malcolm and White 1976). When the entrained air enters the formation materials, the
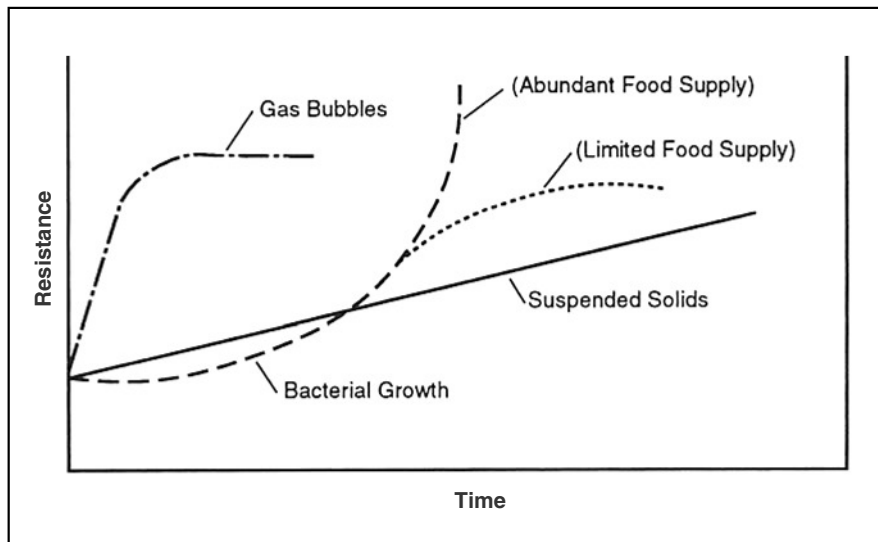
Figure 2.8 Typical clogging processes

bubbles tend to lodge in pore spaces. This increases resistance to flow, resulting in higher water levels within the well. Furthermore the associated change in the oxidation-reduction potential (ORP) of the recharge water can cause geochemical reactions and microbial activity in the storage zone, the results of which can create further plugging.

Air entrainment is characterized by a rapid increase in the resistance to flow, which levels off within hours. Air entrainment effects stabilize because the rate at which bubbles redissolve into the flowing water equalizes with the rate of bubble formation. The plot described as "gas bubbles" in Figure 2.8 illustrates a typical well response to entrained air.

Typically, the possibility of air entrainment is prevented by proper wellhead design and operation. Maintaining positive pressure in the injection tube or pump column prior to discharge below the water level in the well is the most common method of preventing entrained air. Another method is to recharge with a wellhead designed to be airtight. Even though the recharge water may cascade within the well's annular space, injection tube, or pump column, preventing air from entering the well eliminates the possibility of air entrainment.

A plugging mechanism related to air entrainment is caused by a release of dissolved gases within the aquifer formation after injection, which also causes gas binding. This results in reduced permeability. Dissolved oxygen (DO) is an indicator of the concentration of gases in solution. Generally, gas dissolution is not a concern unless DO concentrations exceed 10 mg/L. If dissolved gases are present, they may be released due to an increase in temperature or a decrease in pressure, causing a dissolution of gases contained in the recharge water. An increase in temperature is more likely in northern climates where cold, oxygenated water may be available during winter months for storage in seasonally warmer aquifers. However, a decrease in pressure is unlikely in ASR operations, particularly during recharge. On the contrary, an increase in pressure tends to occur as the water moves down the well and into the storage zone. This pressure increase tends to keep dissolved gases in solution.

Microbial activity may also release gases as a metabolic by-product, which can result in reduced permeability. Although subsurface microbial activity is increasingly recognized as being

128
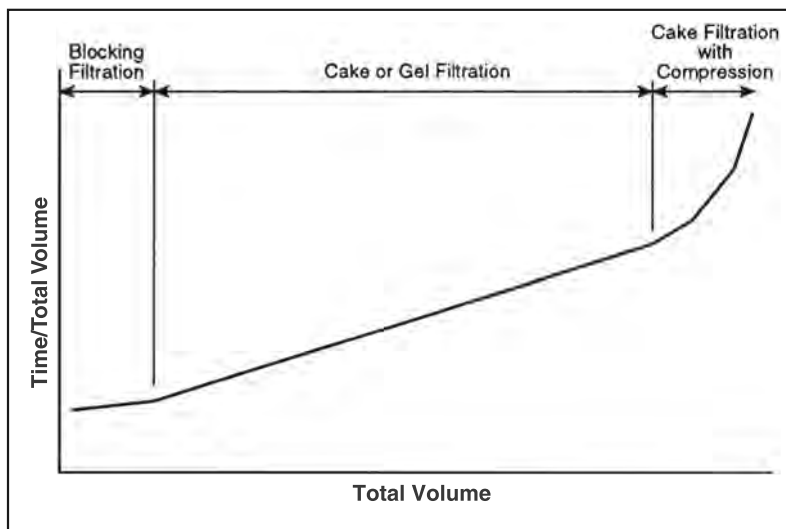
Figure 2.9 Stages of clogging on a membrane filter

prevalent in ASR operations, no evidence of plugging due to release of gases from microbial activity has been noted in ASR wells to date.

Suspended Solids

In unconsolidated formations (typically sands and gravels with minor silts and clays), suspended solids are removed from the recharge water as it flows through the gravel pack into the formation. Resistance to flow near the well increases as the filter cake accumulates due to filtration.

A theoretical analog of this process is plugging of a membrane filter, which has been described as a three-phase progression: blocking filtration, cake or gel filtration, and cake filtration with compression. A typical filtration curve is presented in Figure 2.9. Blocking filtration is characterized by particles physically blocking pore spaces in the filter medium. The duration of this process is typically short, and the magnitude of plugging is minor compared to the later stages of plugging of membrane filters. Blocking filtration may be more consequential in ASR and other recharge wells because the pores in the formation and the filter pack are larger than the pores in a membrane filter. The filter pack surrounding the well screen may trap larger particles before they reach the borehole wall, thus reducing the long-term plugging rate by acting as a coarse pre-filter. It is possible that blocking filtration in the filter pack continues while cake filtration progresses at the borehole wall.

The next stage of plugging is cake or gel filtration. Cake filtration begins when the layer of filtrate begins to thicken on the filter. The resistance is directly proportional to the thickness of the filtrate. Cake filtration in an ASR well is evidenced by a linear increase of injection head over time while maintaining a constant injection rate. This linear response conforms with the response of a membrane filter during the cake stage of filtration.

Cake filtration continues until the filtrate thickness increases enough to allow compression of the filtrate, thus initiating the final stage of plugging: cake filtration with compression. Cake filtration with compression is characterized by a sharp increase of resistance to flow, which is dependent

129

on the compressibility of the suspended solids. If this stage of plugging occurs at an ASR well, continuing injection after this point may not be practical due to the associated high plugging rate and/or resulting increased difficulty of redeveloping the well. Identifying the beginning of this stage of plugging during recharge may provide the signal for redevelopment of the well.

Suspended solids are present in the recharge water for virtually all ASR wells constructed to date. While data on turbidity is readily available for potable water sources, data on total suspended solids is not commonly available. Experience at many different ASR sites has shown the presence of an interesting range of solids in the recharge water, including sand, rust, diatoms (single cell algae), alum floc, twigs, branches, dead mice, live shrimp, construction lumber, albino slugs, a plastic doll's head and a rather mutilated empty can of spray lubricant. Accordingly, it is wise to assume that solids are probably present and take steps to quantify their occurrence and typical concentrations. This provides a basis for remedial design and operational measures. Solids typically occur in short time intervals, probably associated with pressure transients and flow reversals in adjacent portions of the water distribution system. Discrete, small volume samples are less likely to define the solids loading in the recharge water than long-term composite sampling. Similarly, samples from the bottom of a horizontal pipe are more likely to be representative than samples from the side of the pipe.

Biological Growth

Plugging that occurs due to biological growth during recharge is not well understood. Plugging mechanisms include an accumulation of impermeable slimes, development of a mat of dead cells and by-products, and the dispersion or alteration of colloidal particles in the soil-aquifer matrix. The degree of biological growth is directly related to the amount of carbon and nutrients present. Although the concentration of nutrients in the source water may be low, the process of concentrating suspended particles due to filtration near the well often provides the substrate needed to foster biological growth.

A common method of controlling biological growth during recharge is to maintain a chlorine residual of 1 to 3 mg/L in the source water. However, even with chlorination during recharge, a pause in operations for more than about two days can allow biological growth to form (Rhebun and Schwartz 1968). Continuous addition of chlorinated water at a trickle flow rate between periods of recharge and recovery is frequently practiced to maintain a chlorine residual and thereby control bacterial activity in the ASR well. The trickle flow rate can be estimated by monitoring chlorine residual in the well following the end of a recharge period to determine the number of days before the residual has dissipated. Typically, this is one to three days. The trickle flow rate is then determined so that the volume of water in the well is displaced in about half of that period. Typical trickle flow rates for disinfection purposes range from about 0.1 to 0.3 L/sec (2 to 5 gpm). Where this has been practiced, no bacterial plugging of ASR wells has been observed. Where trickle flow has not been practiced, some sites have had no problems to date with bacterial activity downhole, but some sites have had such problems.

A drawback of chlorination during recharge and storage is potential formation of disinfection byproducts (DBPs) such as trihalomethanes and haloacetic acids. However, as shown subsequently in this chapter, DBPs generally decline in concentration during aquifer storage in confined aquifers, particularly where the recharge water contains sufficient carbon and nutrients to support subsurface microbial activity.

130

Unconfirmed reports from operators of the barrier injection wells for Los Angeles County, California, indicate that in early years chlorine was added prior to injection. Later, the practice of injecting Colorado River water with only the residual chlorine remaining from the water treatment plant was adopted, resulting in satisfactory operations for over 20 years. However, since the water treatment plants have switched to chloramines (a combination of chlorine and ammonia) for disinfection to reduce DBPs, operators have observed an increase in plugging and an increased difficulty in redevelopment of the injection wells. Despite some effort, these reports have not been confirmed. Most of the operating ASR sites in the United States are currently recharging chlorammoniated water. There has been no apparent problem with plugging that could not be controlled through periodic backflushing of the wells to waste.

In a few European countries, well recharge is practiced using water with little or no residual chlorine since water treatment includes chlorination followed by dechlorination. Pretreatment to reduce total organic carbon (TOC) is sometimes practiced to remove undesirable organic constituents and also to control bacterial activity in the well. While this is effective, it is also expensive. Where undesirable organic constituents are absent in the recharge water, it may be more cost-effective to control bacterial activity in the well with chlorine rather than with TOC removal treatment processes. Natural processes occurring in the aquifer may reduce or eliminate these constituents in the ASR well during storage.

Geochemical Reactions

During recharge, geochemical reactions can occur that adversely affect aquifer permeability or cause changes in the quality of the recovered water. These chemical and physical changes are a function of:

- Recharge water quality
- Native groundwater quality
- Aquifer mineralogy
- Changes in temperature and pressure that occur during recharge and recovery

The most notable of the possible adverse geochemical reactions are precipitation of calcium carbonate (calcite), precipitation of iron and manganese oxide hydrates, and the formation, swelling, or dispersion of clay particles.

Particle Rearrangement

Repeated cycles of recharge and recovery may result in rearranging and settling of the aquifer materials in the vicinity of the well (mechanical jamming), which may lead to a decrease in pore spaces and a reduction in permeability (Olsthoorn 1982). This effect may extend to a maximum distance of several feet from the well bore. After the initial settling of particles, no further reduction in the permeability is likely to occur. Rapid plugging, which occurs during initial startup of injection, may be caused by particle rearrangement.

Reductions in permeability caused by particle rearrangement are small and are not likely to be an important mechanism in plugging. After the initial settling of formation particles occurs, plugging due to particle rearrangement is not likely to have an appreciable effect during recharge.

131

Most ASR sites have experienced a difference in recharge and recovery specific capacities, with the recharge specific capacity (also known as "specific injectivity") almost always being lower than the recovery specific capacity. Exceptions include some sites in highly transmissive limestone aquifers where little or no difference occurs. At one ASR site in Yorkshire, England, the specific injectivity exceeded the specific capacity. Surveying indicated that a 7mm ground level rise occurred close to the ASR well during recharge of a relatively shallow sandstone aquifer, which was overlain by a 60m clay layer to land surface. At the end of recharge the ground elevation recovered to the original level.

The ratio of recharge to recovery specific capacity for comparable flows and durations typically ranges from 25 to 100 percent, with 50 to 80 percent being a reasonable range for unconsolidated aquifers. The reason for this difference has prompted considerable discussion over the years. Particle rearrangement, or a "skin effect," is usually postulated as the reason however other reasons have also been proposed.

An alternate hypothesis that has been proposed (Pyne 2005) is a "balloon effect." It is easier to let the air out of a balloon than to inflate it. This would imply a hysteresis effect, or differential response of the storage zone and overlying formations to the stress imposed during recharge and the release of that stress during recovery. It would also imply that lower specific capacity ratios would be expected for more compressible formations and for deeper storage zones. Further work on this issue would be helpful, to provide an improved basis for planning and design of ASR wellhead facilities, which are frequently designed and constructed before data are available to determine the specific capacity ratio.

MEASUREMENT METHODS FOR ASR WELL PLUGGING

The drawdown in a pumping well is a function of: (1) the aquifer hydraulic parameters, (2) the design and construction of the well and pumping facilities, and (3) the pumping (discharge) rate. When water is recharged through a well, the same three items (with discharge rate modified to become recharge rate), plus the changes that occur from plugging and any changes in water density, determine the water level rise in the well at any given time. For the following discussion it is assumed that density differences are not significant.

Water level data collected at any ASR well during recharge operations can provide a basis for evaluation of plugging characteristics and comparison to similar data from other sites. Such data may be adjusted to reflect barometric variations and regional groundwater level changes. Three methods have been developed for evaluation of plugging from the adjusted data:

1. Specific time of injection,
2. Water level difference, or
3. Observed versus theoretical water level rise.

Specific Time of Injection Method

The "Specific Time of Injection" method is particularly useful when an observation well is not available since only the water level in the ASR well is used for the analyses. The theory behind this method is that for a selected recharge rate, held constant during the test, the water level rise in the well since the start of recharge is repeatable, assuming no plugging has occurred. If the water level rise is only attributable to well losses (laminar and non-laminar) and aquifer

132

response, then repetition of the same recharge rate over the same period of time should produce the same resulting water level rise. Therefore, a comparison of two specific time of injection measurements taken at the same recharge rate and time interval indicates whether plugging has occurred. Any given time interval could be chosen for this analysis, but, typically, an elapsed time of two to four hours since start of recharge is used so that the water level measurement is taken when the rate of water level change is reduced.

A drawback to the specific time of injection method is lack of control over recharge rates. Between tests, these often vary due to factors beyond the control of the operator. Rising water levels in the well can affect the hydraulics of the recharge system and thereby change recharge rates substantially. One way around this disadvantage is to conduct step injection tests over a range of flows at the beginning of cycle testing. Presumably these tests are conducted during "pre-plugging" conditions. Later, step injection tests conducted over the same time period can be compared with an interpolated value from the pre-plugging step injection tests to determine the magnitude of plugging. Interpolation of the pre-plugging data is done by fitting the data to a power curve ($y = a \times b$). The type curve is then used to obtain a calculated water level rise at a specific injection rate. In this instance, the equation for a power curve was considered more appropriate than the Theis equation because non-linear well losses are the dominant factor in water level rise for the short test period.

Difference in Water Level Rise Method

The "Difference in Water Level Rise" method to determine in situ plugging rates uses data from both the ASR well and one or more observation wells. The accuracy of this method is predicated on the assumption that the recharge rate is kept constant and that the wells are perforated or screened in the same interval. When the flow regime in the aquifer system has reached a quasi-steady state, the difference in the water levels in the ASR well and the observation wells theoretically will remain constant. An increasing difference in water levels indicates plugging.

Observed Versus Theoretical Water Level Rise Method

The water level rise observed in the ASR well is a combination of aquifer response and well losses. It is assumed that for a constant recharge rate, the well losses should remain constant, and therefore any water level rise in the well, without plugging, would be due solely to aquifer response. Therefore, using estimates of aquifer parameters, transmissivity and storativity or specific yield, the water level response in the aquifer is estimated and compared to the observed change in water level in the well. The difference between the calculated, or theoretical water level and the observed water level is presumably due to plugging. The term "theoretical water level" is used as a reminder that theoretical aquifer conditions (e.g., homogeneous, isotropic, and infinite in areal extent) are some of the assumptions when using a groundwater flow equation to calculate water level response during recharge or pumping. When using this method, it is best to calculate differences in theoretical water levels and compare them with observed changes in water levels. Avoiding the variability that can occur early is accomplished by choosing a beginning time that occurs several hours after the start of recharge. The average rate of recharge for the beginning time and ending time, along with estimates of aquifer parameters, is used to calculate a theoretical change in water level for comparison with the observed change in water level in the ASR well. For this method to be valid, recharge rates at the beginning and ending times must be the same so that

133

well losses will be similar. Varying recharge rates between measurements is not a concern and is factored into the average recharge rate used to calculate the rise at the ending time.

If the recharge rate is held constant during injection and the plugging rate is low, a graphical procedure can be used. With the graphical method, water level rise vs. time is plotted on a semi-log chart, and a straight line is drawn through the moderate time data points (i.e., greater than 2 hours but less than 24 hours). Theoretically, the water level rise would plot along a straight line, assuming no plugging is occurring and boundary conditions within the aquifer are not reached. Therefore, the variance from the straight line can be an indication of plugging.

NORMALIZATION OF PLUGGING RATES

Common factors affecting long-term plugging rates during recharge include:

1. Velocity or hydraulic loading (herein referred to as "flux") at the borehole wall, which is a function of the surface area through which the water is entering the aquifer and the rate of recharge, and
2. Viscosity of the recharge water, which is a function of temperature. The flux of water entering the aquifer could be likened to the hydraulic loading rate of filters. Higher hydraulic loadings cause faster plugging because of the greater amount of total solids load over a given time interval. Previous studies of recharge wells and filters performed in the Netherlands have demonstrated the effects of suspended solids and temperature on plugging rates.

Use of a standard flux at the borehole wall and a standard temperature to normalize recharge well plugging data allows for a more meaningful comparison of plugging rates. Normalized rates are not necessarily estimates of actual plugging rates under those conditions, but are meant to adjust the relative plugging rates of various recharge and ASR well tests for comparison purposes. The following formula (modified from Formula 3.22; Olsthoorn 1982) was used to calculate normalized plugging rates:

$$\Delta\Phi_{norm} = \Delta\Phi \left[\frac{q_s}{q}\right]^2 \left[\frac{\mu_s}{\mu}\right]$$

where $\Delta\Phi_{norm}$ = rate of plugging normalized for a recharge flow velocity (flux) of 3 ft/hour at the borehole wall over period of one year at a temperature of 20°C

$\Delta\Phi$ = rate of plugging (feet of head per year)

q_s = standard flux (loading rate or velocity) at borehole wall of 3 ft/hour

q = calculated average velocity (flux) at the borehole wall in ft/hour. This is the injection rate/infiltration surface area (over the effective saturated thickness or perforated/screened interval)

μ_s = viscosity at a standard temperature of 20°C (centipoise)

μ = viscosity at temperature of injection water (centipoise)

134

SOURCE WATER CHARACTERIZATION

Samples of recharge water, native groundwater and recovered water have to be collected and analyzed to investigate possible physical, chemical or biological factors that contribute to well plugging. Where chemical and biological factors can be eliminated from consideration, and where air binding is controlled through appropriate ASR well design, construction and operation, particulate plugging frequently remains as a significant issue requiring evaluation. Particulate plugging is common to almost all ASR sites, even when recharging treated drinking water. The important issue is the rate at which it will occur and the associated backflushing frequency required to maintain acceptable recharge capacity.

The two basic concerns with quantifying the suspended solids concentration of the recharge water are:

1. Obtaining accurate measurements for low concentrations and
2. Obtaining measurements that account for the changes in concentration that may occur during recharge. The possibility of flushing sediments contained in the pipelines is a concern since recharge rates often create high flow velocities in a direction opposite to the normal flow pattern. Periodic sampling may miss "slugs" of sediment-laden water entering the well.

Several direct and indirect measurements of suspended solids in potable water for the purposes of ASR testing have been unsuccessful. These unsuccessful test methods have included turbidity measurements, standard laboratory suspended solids measurements, and Rossum Sand Tester measurements. The measurement of turbidity, a common measurement of drinking water quality, which indirectly measures suspended solids, has been shown to have limited value for ASR purposes. The correlation between turbidity and suspended solids concentrations is poor, and the range of turbidity measurements is small. Typical measurements of turbidity for potable water range from 0.1 to 0.7 nephelometric turbidity units (NTU).

Total suspended solids (TSS) measurements conducted during routine laboratory testing of potable water samples have typically indicated non-detectable results for potable water. Since suspended solids were often the suspected clogging mechanism during injection, it was determined that the TSS detection limit of 0.4 mg/L for *Standard Methods* (APHA, AWWA, and WEF 2005) was inadequate to measure suspended solids in the recharge water. Attempts were made to increase the detection limit for the laboratory analysis of TSS by increasing the volume of filtered samples from 1 L to as much as 10 L. Laboratory experience indicated that filtering large sample volumes tended to erode the filter material, resulting in inaccurate measurements.

The Rossum Sand Tester is a standard device commonly used during well development or when a production well is suspected of producing sand to measure suspended materials in the discharge water. Data collected with Rossum Sand Testers during injection have indicated a lack of sensitivity when used with potable water. Typically, the test results show a low concentration of suspended materials in the water during the initial startup of an ASR well and non-detectable results during the remainder of the testing.

Membrane Filter Index

A method that can be used to define the plugging potential of potable water is the membrane filter index (MFI). The Silt Density Index (SDI) method is used more commonly in the United States and is based upon similar but not identical test methodology. If either test is used consistently it yields useful results.

The theory, equipment, and methodology for MFI testing was developed in the Netherlands. Originally, MFI testing was developed for measuring the potential of waters to plug membranes during reverse osmosis water treatment. Later, MFI was adapted for use on injection and ASR wells. MFI testing equipment and methods, derived from work by Schippers and Verdouw (1980), were first used by Huisman and Olsthoorn (1983) during the early 1970s. The testing procedures have subsequently been refined for ASR purposes, based upon experimentation and field experience.

The basic theory behind MFI testing is to assume that the rate at which a filter becomes plugged at a constant pressure can be used to define a "plugging index" for a specific water at a given temperature. The membrane filter tests were conducted by directing recharge water through a 0.45 micron (μm), 47-mm-diameter membrane filter at a constant measured pressure of 15 to 30 pounds per square inch (psi). A temperature measurement was made during each test. The filter operated initially at 0.2 gpm or less. A single test typically required 15 minutes to 1 hour of field time.

The membrane filter tests were used to develop an MFI for each water source. The MFI is represented by the slope of the straight portion of the plot of time/volume (t/V) vs. volume (V) on a linear scale. Because of the small amount of water and the short times used in the test, the reporting units for MFIs are seconds per liter/liter (sec/L/L).

MFIs, as determined by plotting, were normalized to standard conditions so that MFIs measured with different pressure and temperature conditions could be compared. The standard conditions used were a pressure drop of 30 psi and a temperature of 20°C. The following equation was used to normalize the measured values to standard conditions:

$$MFI_{norm} = MFI \times \frac{\mu_{20}}{\mu} \times \frac{P}{30}$$

where MFI = slope of the straight portion of the plot of individual values (sec/L/L)
μ_{20} = viscosity of water at standard temperature of 20°C (centipoise)
μ = viscosity of water at measured temperature in °C (centipoise)
P = pressure drop across filter (psi)

Bypass Filter Test

Bypass filter tests (BFTs) are conducted on the source of recharge water to measure the average concentration of suspended solids over periods of time ranging from a few hours to a week or more. The source water is directed through a 5-μm, 10 inch long, spun polyester cartridge filter at pressures ranging from 5 to 30 psi. Cartridges with smaller pore sizes (0.45μm or 1μm) are available but have higher costs and shorter life expectancy due to rapid plugging. A flowmeter,

136

similar to those used by utilities for household water use, is installed in the filter piping to measure the volume of flow through the filter at each site.

The bypass filters are used to measure the suspended solids concentration in the recharge water over extended periods of time. This is preferable to the MFI test in many ways since the composite sample is likely to detect occasional slugs of particulate material that would not be detected by a typical series of individual MFI tests. The filters are dried and weighed to the nearest 0.1 gram (g) in the laboratory. The totalizer on the flowmeter is read prior to putting the filters into service. The filters are operated during injection until the flow rate through the filter decreases to about 25 percent of the initial flow rate. When a filter is taken out of service, the flow-meter is read, and the spent filters are put in plastic bags and delivered to the laboratory for drying and weighing. The polyester filter material cannot withstand the 105°C temperature of the standard drying oven. Therefore, the filters can be placed on top of the ovens and dried for several days. The difference in filter weights and the meter readings are used to calculate the concentration of suspended solids in the recharge water. Filters utilized in tests to date were operated between 2 to 20 days with 10 days as the average service life.

WELL PLUGGING RELATIONSHIPS

If long-term plugging is assumed to be a function of suspended solids in the recharge water, the rate of plugging primarily will be a function of recharge water plugging potential and aquifer conditions (defined in terms of hydraulic conductivity). The plugging rate can be normalized for rate of recharge and water temperature by a normalizing procedure that accounts for the flux (also referred to as "velocity") of recharge water at the borehole wall (a function of injection rate, well diameter, area of perforation/screen, and effective saturated thickness) and the viscosity of the source water. The flux at the borehole wall is analogous to the hydraulic loading rates applied to filter media. The adjustment for viscosity accounts for the increment of head buildup created by recharge source waters of different temperatures.

It seems reasonable that, with enough data points from operating facilities, a family of type curves could be developed that relate normalized plugging rates to hydraulic conductivity for source waters of different suspended solids concentrations. The objective is to use these type curves during Phase 1 ASR feasibility investigations to estimate well plugging potential and probable frequency of well redevelopment required.

These type curves could also be used to determine whether an ASR well is operating within a "normal" range of plugging. Determining whether well plugging is excessive could provide a signal to investigate other causes of plugging.

PLUGGING RATE SITE INVESTIGATIONS

Data were collected during testing at nine ASR sites, including information regarding treatment and conveyance of the water prior to recharge, well construction, recharge rates, pumping rates during redevelopment, hydrogeology and aquifer parameters.

The water level data from the ASR test wells and, when available, the data from nearby observation wells, were used to estimate plugging rates. The three methods of analysis of plugging previously described were used where applicable. Due to ease of use, the most predominant method was the observed vs. theoretical water level rise method. Only a few sites had observation wells nearby, which are necessary for the water level difference method to be used. The specific

137

Note: The size of type used to label the data points is intended to be roughly representative of the magnitude of suspended solids in the source water.
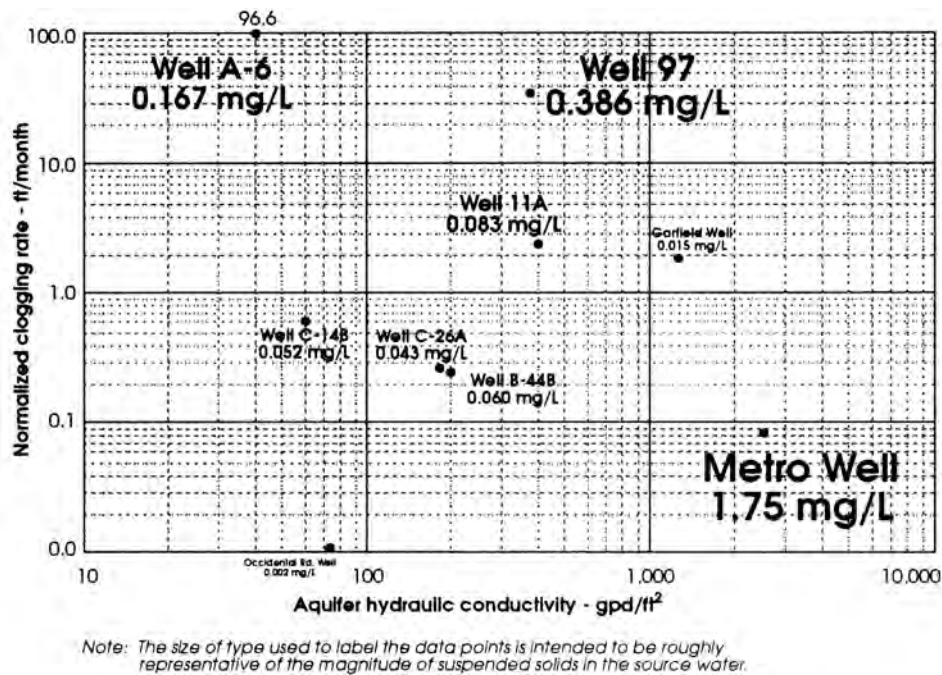
Figure 2.10 Relationship between clogging rate, hydraulic conductivity, and total suspended solids

time of injection method requires testing and data collection procedures designed specifically for this method and was therefore only performed at a few of the sites. Plugging rates varied widely, from undetectable to 220 ft/month.

Each of the source waters tested were considered potable water, yet the testing results indicate a wide range of suspended solids concentrations and associated plugging potentials that can affect ASR well performance. Where water was delivered from an existing water distribution system, the sediment loads in the water were always higher at the beginning of recharge, and sometimes sediment loads would increase for short periods of time during recharge. These data suggest that reversing the flow through existing pipelines often results in sweeping sediments contained in the pipes into the ASR wells.

The relationship between normalized plugging rates, hydraulic conductivity, and suspended solids concentration is shown in Figure 2.10. The size of type used to label the data points is intended to be roughly representative of the magnitude of suspended solids in the source water. Generally, the data points appear to follow a logical pattern, such as a comparison of Well 97 and Well 11A. These wells have similar hydraulic conductivities, but the well with the highest suspended solids concentration has the highest plugging rate. An anomalous data point is the Garfield Well, which, in comparison with the other data, should have a plugging rate that is considerably lower.

The results of this testing indicate that the relationships between ASR well plugging, source water quality and aquifer permeability generally follow intuitive reasoning. The instances where the ASR well performance does not follow the pattern of other wells is possibly due to inability to accurately measure the controlling factors or due to other factors that have not been identified or adequately accounted for. The data from the Occidental Well is possibly the most

138

significant data presented in this study because it demonstrates conclusively that for low aquifer permeability and low suspended solids content in the source water, plugging does not occur. Well A6 is another important data point since it demonstrates that for low aquifer permeability and moderate suspended solids content, the plugging rates are high. The Metro Well demonstrates that low plugging rates can occur with high suspended solids if the aquifer is highly permeable.

Data from additional ASR well sites would further define the relationships presented here. However, this analysis provides a reasonable approach for estimating plugging rates at new well sites prior to well construction and testing, based upon literature values for aquifer parameters, assumed well design and field measurements of recharge water characteristics. Estimated plugging rates, in turn, can provide a basis for well selection, design, and pre-treatment to achieve acceptable backflushing and redevelopment frequency and satisfactory operational performance of recharge facilities.

REDEVELOPMENT

Despite the calculations and estimates discussed above, the frequency and method of redevelopment pumping in an ASR well ultimately has to be determined based upon initial testing and operating experience at each site. One of the three methods described above is applied to determine the plugging rate during initial ASR test cycles, following which a judgement is made as to how frequently to redevelop the well in order to maintain recharge rates and also to avoid residual plugging.

It is pertinent that many dedicated injection wells have been constructed in the United States, either to dispose of wastewater or stormwater, to recharge aquifers or to achieve other goals. Unless transmissivities are exceedingly high, these wells tend to plug. They have no ability to purge particulates from the wellbore prior to starting recharge, thereby forcing sand, silt, bacteria, chemical flocs and other particulates into the screen and formation each time that recharge commences. Several such wells at one site lost 10 to 20% efficiency each year, rendering them unusable within 3 to 5 years. At another site, four wells comprising an injection wellfield sealed shut within 20 minutes of starting initial operations, primarily due to particulates and air in the recharge water. By providing a pump in the recharge well, ASR wells overcome this fundamental weakness of dedicated injection wells, providing sustained operating performance through periodic backflushing and also through recovery operations.

A useful starting point is to avoid recharging at a rate or for a duration that would cause the water level rise during recharge to exceed the available water level decline in the ASR well during pumping. Where the recharge specific capacity is estimated at half of the recovery specific capacity, the recharge rate initially could be set at half the recovery rate, or at a slightly lower rate with anticipated less-frequent backflushing. The actual plugging rate would then be monitored to compare with the expected value. The well would then be pumped to waste for a few minutes or hours to purge solids from the well. Assuming that recharge and recovery specific capacity are restored, the recharge rate or duration could be extended in small increments in later cycles, each of which would show greater plugging. So long as redevelopment pumping is able to restore specific capacity, the incremental increases in rate or duration would continue until either the desired recharge rate is achieved, the duration extends to a full recharge season, or signs of residual plugging become evident, such as inability to easily restore recharge specific capacity. The ideal situation is one in which the plugging rate is sufficiently slow that redevelopment only

139

needs to occur at the beginning of scheduled recovery, however this typically only occurs in highly transmissive storage zones such as the karst limestone ASR wells in Florida.

Redevelopment pumping or backflushing usually involves pumping the ASR well with the permanent pump that is also used for recovery. Water is pumped to waste for anywhere from 10 minutes to two hours. Assuming recharge with treated drinking water, the duration depends primarily upon the materials of construction. Surging the well by alternately turning the pump on and then shutting it off for two or three cycles in a period of about three hours or less is practiced at some sites. This is usually sufficient to restore specific capacity during recharge. However, care should be exercised to avoid damaging the motor on submersible pumps or the shaft on all pumps by restarting it too soon after shutdown.

A common criterion for initiating backflushing is to monitor specific injectivity. For Orange County Water District, California, backflushing is performed when specific injectivity declines by 50%. One hundred well volumes are then pumped during a few hours of pumping, without surging. Calleguas Municipal Water District, California, monitors specific injection and water level rise, backflushing by pumping each ASR well for 15 minutes after 720 hours of recharge. For continuous recharge this would be about one month. Centennial Water & Sanitation District, Highlands Ranch, Colorado, backflushes approximately monthly during recharge periods, timing the events to avoid power company demand charges.

For recently-constructed injection wells in the Orange County, California, salinity barrier program, air lift pumping is utilized for well redevelopment, alternately pressurizing the well casing with air and then releasing the pressure and starting air lift pumping to purge particulates from the wells. This process which was first proposed in the Netherlands and applied in Orange County is also known as "juttering."

The frequency of redevelopment pumping varies substantially among ASR sites. Table 2.6 lists a number of operational ASR sites and the typical redevelopment frequency. Information is also included regarding the lithology at each of the sites to aid in comparison of their operating experiences.

It is usually desirable to pump the ASR wells either to waste or to retreatment during back-flushing. Pumping to waste usually provides the opportunity to pump at a high rate since the discharge head on the pump is substantially reduced or eliminated. This is desirable as it helps to purge solids from the well. At Las Vegas Valley Water District, Nevada, an orifice plate is provided in the discharge to waste pipeline to keep the pump on its pump curve. This prevents excessive well production that could mobilize sand and potentially damage the formation. Sand production and duration at the beginning of backflushing is monitored to allow comparison with data from previous backflushing events.

At a few sites, regulatory restrictions on the disposal of water during backflushing operations are sufficiently rigorous that special containment and treatment provisions are required. This is the case for the salinity intrusion barrier injection wells in southern California. Where this is the case, or may be reasonably expected in the future, greater care is needed during design of well and wellhead facilities to minimize the volume of solids entering the well and thereby reduce the frequency of backflushing as well as to improve the quality of the backflush water.

For unconsolidated aquifers, experience suggests that recharge rates tend to approach an equilibrium level that is lower than the initial recharge rate at the beginning of testing, but can be sustained by periodic backflushing. Some early loss of initial recharge capacity occurs at many such sites while still maintaining long-term rates at a useful level. For consolidated aquifers, such loss in capacity is less apparent.

140
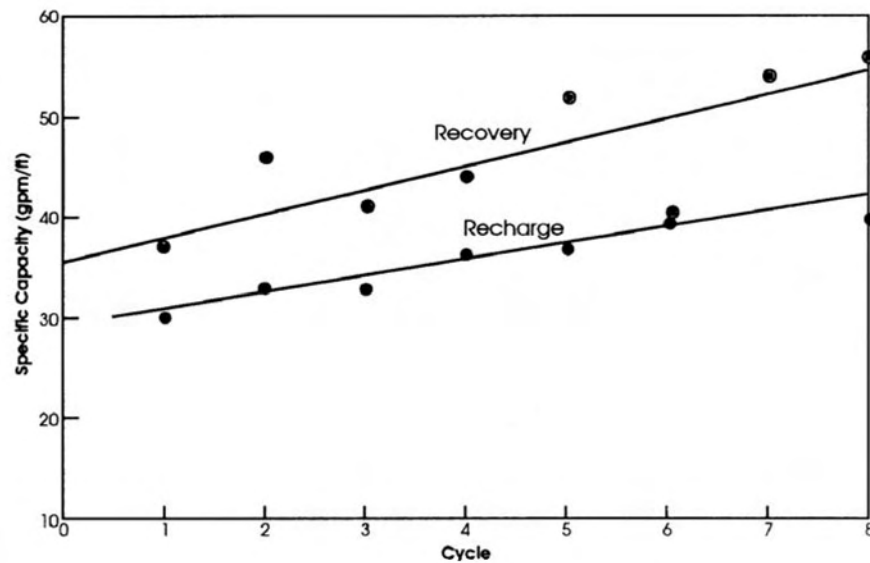
Figure 2.11 ASR well increase in specific capacity, Manatee County, Florida

It is probably wise to assume that ASR wells will need full redevelopment about every five years, including pulling and setting the pump, cleaning, chemical treatment, acidization, disinfection and possibly other methods to restore its condition. This may not be required at some sites, particularly those in consolidated aquifers; however, in the absence of site-specific evidence to the contrary, the need for redevelopment every few years should be assumed for budgetary and planning purposes. As discussed further in the next section, routine well rehabilitation can prolong the useful life of a well and maintain its recharge and recovery capacity.

The ASR system at Manatee County, Florida, was initially recharged with water that was diverted from the water treatment plant prior to final pH stabilization. The recharge water was slightly aggressive but rapidly stabilized when it came into contact with the limestone in the storage zone. Figure 2.11 shows the increase in specific capacity that occurred at this site during the first few cycles of testing. Calculations indicate that the volume of calcium carbonate in the storage zone that was dissolved during this process was very small, and not significant to long-term well operations. This eliminated the need for periodic redevelopment of the ASR wells at this site and saved the county the cost associated with stabilizing the water in the treatment plant. During pH stabilization in the aquifer, the TDS concentration of the recovered water showed an increase of about 25 mg/L. This was evident at the beginning of each cycle, even when the storage periodbetween recharge and recovery was about an hour or less. Subsequently the source of water for recharge was changed to stabilized drinking water.

A similar approach has been considered, but not implemented, for storage of aggressive waters produced from desalination plants in the Arabian Gulf. These plants typically are located over brackish limestone aquifers, some of which would be suited to seasonal, long-term and emergency storage of drinking water while similarly achieving savings in water treatment costs through pH stabilization in the aquifer.

141

To overcome plugging, a few ASR wellfields in southwest Florida add about 0.5 mg/L of carbon dioxide to the recharge water, forming carbonic acid. This weak acid slowly dissolves the limestone in the storage aquifer, counteracting any reduction in specific injectivity due to plugging.

Where redevelopment pumping is conducted with sand-producing wells, it is important that adequate backflushing times are maintained. Premature shutdown during pumping to waste and transition to recharge may cause well or pump damage by sand-locking of the pump bowls as the sand in the water column settles out or is otherwise trapped in the pump bowls. Periodical checking of sand production amount and duration is therefore important during backflushing operations at ASR wells.

WELLHEAD FILTRATION

If the rate of ASR well plugging, or the expected frequency of required backflushing, is perceived as a potential operating problem, a desirable solution is to keep the solids out of the well in the first place. As discussed previously, this involves at least purging the recharge piping to waste prior to initiating recharge and may also involve wellhead filtration.

One solution that has been implemented at an ASR site in New Jersey is to incorporate a short length of large-diameter (1,500 mm, 60-inch) pipeline into the recharge piping at the wellhead in order to reduce flow velocity and thereby settle out any solids. The primary purpose of this large pipe is to provide detention time for chlorination of recovered flows before they enter the distribution system; however, it is anticipated that it will also serve to settle out any solids in the recharge water. A blowoff valve is provided to periodically remove accumulated solids.

Gordons Corner, New Jersey, has operated an ASR system since 1971 that incorporates wellhead gravity sand filtration in order to minimize entry of solids into the ASR wells. It is anticipated that other ASR systems recharging treated drinking water may, in some cases, benefit from providing wellhead filtration. Solids in distribution systems often include sand, rust, alum floc or other constituents capable of plugging an ASR well.

As an ASR operating practice, wellhead filtration is not common. Only five sites (Salt Lake County, Utah; Salt River Project, Arizona; Gordons Corner, New Jersey, Marco Lakes, Florida and Calleguas Municipal Water District, California) are known to be providing these facilities. The Utah test site utilizes a pressure sand filter. Calleguas installed Y-strainers to strain out large pieces of debris from large diameter, cement lined transmission mains and tunnels. Such debris had previously caused ASR well plugging. The Arizona test site has utilized a drum filter. The New Jersey operational site utilizes a gravity sand filter, and the Florida site utilizes pressure sand filters. Some new sites recharging treated drinking water into aquifers prone to plugging are expected to include wellhead filtration facilities. In addition, future sites using ASR technology to store surface water containing low levels of suspended solids are expected to incorporate either wellhead or riverbank filtration as a basis of design.

The technology for wellhead filtration is widely available, as developed for the water utility, mining, and agricultural sectors. Costs of these systems tend to be highest for the water utility applications and lowest for the agricultural applications. However, the particle sizes removed tend to reflect the system costs. The optimum tradeoff between investment in wellhead filtration facilities and system operating costs remains to be determined. For proposed sites where redevelopment pumping is a problem due to cost, potential electric motor damage, water disposal, or permitting difficulties, the investment in wellhead filtration facilities may be advisable and

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should be considered during design. A reasonable solution for many sites will be to provide space in the wellhead design to incorporate wellhead filtration at a later date, if required.

Complete sand media filtration systems for agricultural applications are readily available. Each 300 gpm modular unit has about a 6 ft × 6 ft imprint and stands about 6.5 ft high. Multiple units would be manifolded to achieve the desired filtration flow rate. Such systems are used widely to prevent clogging of spray nozzles in microjet irrigation systems. An alternative approach using a ring filter capable of treating flows of 100 gpm is also available. Horizontal collector well technology also achieves significant reduction in the solids content of surface water as a result of sand filtration. Through patented trenching and backfilling methods, a slotted pipe-line drains water from the water table and any adjacent surface water sources through a sand formation or backfill, thereby improving water quality prior to recharge into an ASR well.

For municipal applications, a multi-media pressure sand filter can remove particles down to 10 microns. A system capable of filtering 0.5 MG/day would include two 48-inch diameter tanks, each 60-inch tall. Filtering 1 MG/day (3.8 ML/day) would require four such tanks, or three tanks each 54 inches in diameter. Filtration down to 5 microns is also feasible, using crushed glass filter media.

An alternative approach is to use disposable filter cartridges within an inline pressure filter. Different cartridges are capable of providing filtration from 15 down to 2 microns. Each cartridge is capable of filtering about 50 gpm, requiring multiple cartridges to achieve flow rates of interest to ASR operations. Vessels capable of handling 18 cartridges are readily available. The initial pressure drop across the 2, 5, 10, and 15 micron cartridges is about 0.9, 0.5, 0.4, and 0.3 psi, respectively. Cartridges have to be replaced when the pressure drop reaches 35 psi. The cartridges are reported to be able to handle 21 to 24 pounds of suspended material before changeout is required. These data are reported from tests using silica dust at 40 gpm. Assuming 5 mg/L suspended solids across a cartridge at 50 gpm, the cartridge would function for 8 to 9 days. If the loading was 2 mg/L, the cartridges would require changing every 21 to 24 days. This assumes that all of the suspended solids are trapped by the filter. In reality, the particles trapped will be a func-tion of the size distribution of the particles and the size of the filter pore spaces. The cartridges may last longer than the numbers presented; however, it is not possible to estimate the duration without first investigating the water to be filtered. The manufacturer provides an analysis of particle size and estimated cartridge life upon request.

Membrane processes also may be used for wellhead pre-treatment. In particular, microfil-tration systems can provide a higher level of treatment but at somewhat higher cost. Operating pressures for these units range from 25 to 40 psig, while the pressure differential across the membrane varies from 2 psig for a clean membrane to 15 psig when the membrane is fouled. Particle sizes are reduced to below 0.2 microns with this process. The backwash volume is about 2 to 7% of the feedwater volume. The membranes are chemically cleaned when pressure differentials exceed about 15 psig, using caustic-based solutions at pH values above 12. These units retain protozoan cysts such as *Giardia* and *Cryptosporidium*, nearly all bacteria of health concern, and turbidity. They also provide between 2- and 4-log removal of viruses.

Considering the range of alternatives presented above, it appears that sand filters, ring filters, drum filters, and horizontal wells can filter recharge water to small particle sizes generally suitable for agricultural applications that would not plug irrigation systems. In some cases, these may also be suitable for ASR wellhead filtration, particularly with storage zones that have high transmissivity. Where aquifers have lower transmissivity, other filtration systems are available that can reduce particle sizes down to between 2 and 10 μm using multi-media pressure filters or

143

cartridge filters. Microfiltration systems using membrane filter technology can remove particles down to 0.2 μm. Selection of the appropriate technology to meet technical and regulatory requirements for ASR operation has yet to be clearly defined. Most likely a combination of these technologies, probably in series, will be appropriate for many ASR projects.

Wellhead filtration is one of the aspects of ASR technology that is evolving. Experience is demonstrating the need to keep solids out of the ASR wells. While a higher quality of recharge water will tend to improve overall performance of an ASR system and also expedite regulatory approval, the most cost-effective tradeoff between the investment in wellhead filtration and ASR performance has yet to be established for municipal and agricultural applications.

An important difference between ASR applications and other applications of advanced filtration technology is that ASR wells are provided with a backup capability to remove solids from the well that pass through the filter. This is the periodic backflushing operation. This would not be the case for applications preceding reverse osmosis membrane treatment plants, for instance. Experience at several sites will be required to estimate the most cost-effective combination of wellhead filtration and backflushing frequency for aquifers with different hydraulic characteristics.

WELL MAINTENANCE AND REHABILITATION

Like most wells, ASR screened wells in unconsolidated aquifers can experience loss of specific injectivity or specific capacity as a normal process of aging. Two principal causes of this loss are as follows:

- Mineral and biological deposition on the surfaces of the well screen, filter pack and the aquifer
- Air entrainment and movement of fines resulting in pore water blockage and mechanical blockage.

Mineral deposition on the surfaces in wells is a combination of bacteria growing in biofilms, minerals that the bacteria assimilate from the water as it passes over these surfaces, or minerals that precipitate due to pressure reduction around the well during pumping. Within the biologically accumulated material, fines from the formation (clay, silt, and fine sand) can also become trapped.

Operation of ASR wells, as compared with normal production wells, often but not always results in an increased rate of subsurface microbial activity, oxidation or deposition of minerals. Some of the potential for mineral oxidation and deposition depends upon the water chemistry of the recharge water and that of the native aquifer water. In many cases the recharge water for an ASR well contains dissolved organic carbon, nitrogen, phosphorus and other trace minerals at concentrations that exceed that in the aquifer and therefore stimulate subsurface microbial activity to a greater extent than for a normal production well. Most of the plugging that results in loss of specific injectivity occurs very close to the well. At greater distances the pore volume of the aquifer is significantly larger and some deposition on the surfaces does not result in hydraulic losses. It is close to the well, particularly within approximately the first two feet, for which deposition on the surfaces results in hydraulic losses.

To date, after more than 20 years of operating experience, there is no indication that ASR wells experience a loss of capacity more rapidly or more slowly than water supply wells that are

144

only pumped. Any increased rate of biofouling and associated mineral deposition on the well screen and aquifer surfaces would probably be due to the presence of organic sources in the injection water resulting in enhanced biological growth and possible higher concentrations of dissolved minerals. The higher concentrations of dissolved minerals would probably be due to reduced conditions in the aquifer created by the enhanced rate of biological growth and the resultant consumption of the oxygen in the recharge water. The bacteria can then utilize other terminal electron acceptors and reduce Fe^{+3} (precipitated) to Fe^{+2} (dissolved) and Mn^{+4} (precipitated) to Mn^{+2} (dissolved).

The dissolved minerals in the water are more significant than the accelerated growth rate of bacteria as a cause of lost well capacity. Deposits in wells and aquifers that plug the pore volume are normally composed of both mineral and biological components (Kirk et al. 2004; Timmer and Stuyfzand 1998). The mineral content of the deposit is normally the higher percentage of the deposit (80% on average) and the organic component normally is the smaller percentage of the deposit (20% on average).

During operation of ASR wells there may be less mineral deposition than if the same well is used as a normal water production well. This will depend primarily upon the difference between the recharge water quality and the ambient aquifer water quality. If the recharge water has less mineral content than the aquifer water, which is usually the case, there will be less deposition in the pore volume of the aquifer near the well and within the stored water bubble. In some cases acidification or oxidation of the aquifer may occur, releasing and flushing minerals from around the well to a greater distance in the surrounding aquifer. This would tend to offset plugging due to enhanced microbial activity close to the well screen. During ASR recovery, minerals deposited in the aquifer away from the well appear to remain in the aquifer as long as the buffer zone is maintained around the well.

It is commonly believed that iron bacteria, which cause significant biofouling problems, are introduced into wells during construction or as a result of subsequent downhole activities. That is probably not the case since iron bacteria are naturally present in aquifers in abundance. Iron-related bacterial plugging problems are due to indigenous bacteria that proliferate within an altered subsurface environment. It is these environmental conditions that will determine whether or not a well experiences an "iron bacterial" problem. Many different types of bacteria are present underground which could cause the typical depositions of iron that are commonly recognized as iron bacterial problems or plugging. The much more significant factor for determining whether or not a well will experience an iron bacterial problem is the availability of dissolved minerals, of which iron is the most common.

The typical operational procedure regarding well maintenance is to wait until the well has experienced a significant problem before performing some type of rehabilitation treatment. Often the amount of deposited material can be very extensive and complete removal of the deposits can be difficult. ASR wells are periodically (daily, weekly, monthly, seasonally) pumped in order to backflush the well. During this backflushing event the well is switched from recharge mode to production mode and the change in flow direction is typically effective at removing the softer material. Backflushing can therefore extend the time between more aggressive rehabilitation treatments. This periodic backflushing may not be effective at removing all of the attached material which will continue to build up and reduce the pore volume.

Rehabilitation treatments are often performed with the pump in place to reduce the cost of rehabilitation treatments. Treating a well with the pump in place creates a significant limitation for removing material from the well and the surrounding aquifer. Consequently, the bottom parts

of many wells are not effectively cleaned due to the lack of effective energy needed to get deposits detached from surfaces and effectively "flush" them from the well.

In order to remove deposits from surfaces, it is necessary to deliver adequate energy into the well, filter pack and surrounding aquifer. The energy needs to be able to dissolve, disrupt or detach the deposits from the surfaces of the sand, gravel and well screen in addition to removing them effectively during the development phase. The effective removal of the detached sediments is often difficult from the bottom zones of the well screen since often there is less flow entering the well screen near the bottom. This may be due to several reasons, including filling of the bottom of the well screen with sediments, substantial head differential from the top to the bottom of the screen during pumping of the well, or reduced initial hydraulic conductivity of unconsolidated material in this screen section. In order to get removal of detached sediments requires keeping the sediments in suspension, which can be accomplished by agitation of the flow while pumping the sediments from the well.

Longevity of Rehabilitation Treatments

During rehabilitation it is difficult to remove 100 percent of the deposited material which is in the well and surrounding formation (Mansuy and Layne Geosciences 1999). Re-growth occurs more quickly with the organic nutrients left in the pores. Increasing the time frame between treatments requires effective removal of the deposits. Periodic backflushing of the well is important but probably does not remove all of the accumulated deposits surrounding the well screen. Rehabilitation treatments that are more effective at removal of deposits will increase the well's performance and the length of time before the next major rehabilitation will have to be performed.

Specific capacity or specific injectivity starts to decline due to plugging when the flow through the pores in the sand and gravel sediments surrounding the well, and through the well screen, changes from laminar to turbulent, thereby causing increased friction losses. This may require several years to become apparent, during which deposition is steadily reducing the pore space in the aquifer close to the well. Wells are typically not operated at their maximum production capacity initially, so the effect of this plugging and mineral deposition is not immediately apparent.

A well may experience several years of service before any loss in specific capacity is noticed. Figure 2.12 represents a common deterioration of well performance without effective maintenance. The declining specific capacity each time a well rehabilitation treatment is undertaken is a result of incomplete removal of plugging deposits and less pore volume existing in the well screen, filter pack and surrounding aquifer.

Well plugging mechanisms are complicated further by changes in the producing or recharging intervals of a well screen with time. During ASR recharge the most productive sections of the well screen will typically accept the greatest initial portion of the flow, and will therefore experience the greatest potential for plugging. As plugging develops, other sections of the screen accept a relatively greater proportion of the flow. Without backflushing and well maintenance, recharge will tend to occur primarily in the least plugged section of the screen, which will tend to change with time. As the pore volume is lost due to bacterial plugging, mineral deposition, air entrainment and mechanical blocking, flow shifts from laminar to turbulent, friction losses increase near the well screen, and specific capacity or specific injectivity falls. Friction losses vary as the square of the pore velocity. Small initial losses in pore volume have little effect upon well

146
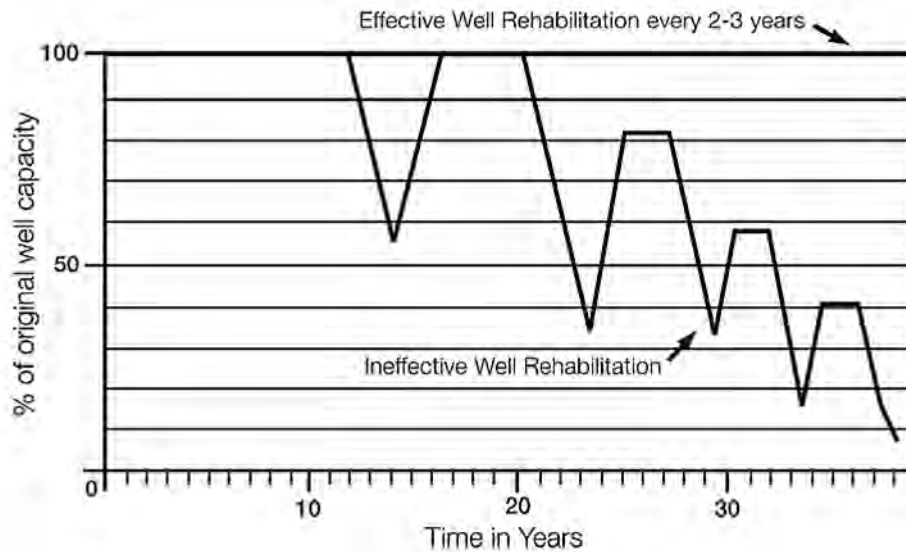
Figure 2.12 Typical loss in specific capacity of a screened well

capacity. As the pore volume declines around the well screen, the pore velocity increases proportionally but the friction losses increase exponentially. When ASR recovery or backflushing occurs, most but probably not all of this deposition is reversed.

If the well is then rehabilitated, specific capacity may be restored to the original value without removing all of the deposited material that is in the well. However if all of the deposited material is not removed, it does not take as long for the well to begin to plug up again. Once the well is back in operation, material is again being laid down over the surviving plugging material and the well's capacity will decrease again more quickly than if all of that material had been removed. The simplest way to look at this concept is that the rate of deposition (mineral, biological) on the surfaces may not change from the very beginning of putting a well in service but many years can pass prior to turbulent flow losses occurring in the pore spaces of the porous media.

PREVENTIVE MAINTENANCE OF ASR WELLS

Whether for ASR wells or for normal production wells, preventive maintenance programs can be effective in reducing well problems and can help to maintain the well's production capacity. It is common practice to operate wells until they experience a significant loss of specific capacity before rehabilitation efforts are performed. Typical well rehabilitation periods are from five to ten years, depending upon the type of aquifer lithology, the quality of the recharge water and also of the native groundwater. At this point it can be difficult, if not impossible, to restore the capacity to its original condition because of the amount of plugging deposits in the pore volume of the aquifer and the perforations of the well screen. Preventive maintenance treatments offer the advantage of removing deposited material at the early stages of plugging.

The periodic removal of the deposits at the early stages of their formation may also help to slow the rate of corrosion experienced in wells. Much of the corrosion in wells is caused by "tuberculation" resulting from a symbiotic growth of iron-related and slime- forming bacteria with the sulfate-reducing bacteria. The sulfate-reducing bacteria grow in the anaerobic zone

147

created by the aerobic slime-forming bacteria. Most of the corrosion occurs under the tubercles and therefore is commonly referred to as "under deposit" corrosion. Since the deposit is often necessary for the corrosive environment to occur, by periodically and effectively removing the deposits it is possible to prevent much of the "under deposit" corrosion.

In the past, a 15% to 25% loss in specific capacity was the recommended guideline to commence rehabilitation. In many cases this may already be too late to create sufficient disruptive action with the pump in place because significant deposition can occur prior to recognizing a loss of specific capacity or specific injectivity. It would be better to use a time schedule (e.g., rehabilitation once a year) than to rely on loss in specific capacity, and it is clearly better to rehabilitate a well more frequently rather than less. Using the scheduled approach to maintenance, the time frame between maintenance procedures could be determined geographically from experience in an area, well field or well. For ASR wells, preventive maintenance needs to be performed as soon as there are any losses evident in specific capacity from one cycle to the next, but would be more valuable if performed prior to losing any specific injectivity in successive cycles. The tradeoff between increase in water level or pressure during ASR well recharge, available drawdown during recovery or backflushing, frequency of backflushing and frequency of rehabilitation will need to be determined for each site based upon experience.

SHOCK CHLORINATION AND CHEMICAL TREATMENTS

Shock chlorination, using doses greater than 1,000 mg/L as a preventive maintenance or rehabilitation procedure, has been extensively used but is too limited in its ability to remove deposited material. The effectiveness of shock chlorination is lost once the microbes have developed their natural protective mechanisms, which are the biofilms with associated mineral deposits. Once these protective mechanisms are formed, disinfectants and possibly other chemicals are often not able to eliminate the biofilms and achieve their removal from the surfaces.

Chemical treatments are also used but are often not capable of delivering the necessary energy to remove all the deposited material and return the surfaces of the well screen and aquifer materials to their original condition. Less pore volume remains open each time a rehabilitation or maintenance treatment is performed and therefore the timeframe between treatments becomes ever shorter.

It is often necessary to remove the pump during well rehabilitation in order to deliver sufficient energy into the well and the surrounding aquifer. Wire brushing of the screen followed by acidization or other chemical treatments may be conducted, in conjunction with surging, jetting or other well development methods to purge accumulated debris from the well. Once these procedures are completed, the well is disinfected with chlorine. The process is repeated until rehabilitation is deemed complete.

Even with the best methods available for preventive maintenance or full rehabilitation it can still be difficult to get good removal of material, in large part because of the inability to get enough energy into the well screen and the surrounding aquifer. Chemical treatment approaches (acids and acid blends) will often fail to contact the deposits that need removal since the flow paths in the aquifer are plugged. The choice of chemical, even though important, is not as important as the method of application.

A new approach has been developed to overcome this problem with the pump in place. This concept relies on the permanent, strategic placement of energy injection equipment at various points in the well using a small diameter pipe that is placed to the bottom of the well

148

screen during pump installation. This system is designed to allow for the delivery of chemical energy, mechanical energy, or combination energies. The idea is to keep surfaces clean in a well and the surrounding aquifer instead of waiting for the surfaces and the pore volume to become partially or completely fouled and encrusted. Carbon dioxide and/or chemicals are injected at various depths into the well screen area without removing the pump. This periodic cleaning of the surfaces can be performed on a scheduled interval, determined by the historic rate of fouling and the current data readings so that the surfaces can be kept clean. The cost of the periodic cleanings is significantly less than the cost of a major well rehabilitation and the cleaning can often be performed with the use of carbon dioxide alone. This offers the advantage of not having to neutralize or dispose of spent chemicals. Once the material has been detached from the surfaces it needs to be removed from the bottom part of the well and the surrounding formation. This can best be achieved with the simultaneous pumping and occasional agitation of the sediments and deposits. This will allow more complete cleaning of surfaces and maintenance of the original pore volume, extending the timeframe between rehabilitation efforts.

The application of gaseous or liquid carbon dioxide removes biological slimes and also removes mineral scale. The bulk of the activity is due to phase changes and the release of energy. This energy results in the detachment, dissolution and removal of sediments and encrustation from the surfaces within the well screen and the surrounding aquifer.

CHAPTER 3
LESSONS LEARNED FROM CASE STUDIES

BACKGROUND

The science and technology of sustainable underground storage, presented in Chapters 1 and 2, addresses not only the theory and best practice of surface recharge and well recharge facilities, but also has been tested and field verified through operating experience at several sites. As part of this AwwaRF project, operating facilities were visited throughout the United States, and also overseas, to interview operations staff and learn from their experiences. The "lessons learned" have been distilled from the site visits and are summarized in this Chapter. A few failed sites were also evaluated through telephone interviews but, with one exception, were not visited. Table 3.1 lists the sites that were visited or otherwise evaluated.

For each of these sites a 10- to 20-page case study was prepared, including text and graphics. Case studies are included on a CD at the end of this report, providing a valuable source of information.

Sites were selected to achieve a reasonable geographic and technical diversity of operational conditions. Sites with several years of operating experience were selected so that issues relating to sustainability could be fairly evaluated. About half of the sites practice surface recharge while the remainder practice well recharge. Some sites practice both types of recharge.

During the site visits, each agency identified the issues and resolutions that improved the performance and sustainability of their individual recharge system. The key lessons learned were then reduced to generic guidelines. Certain lessons learned are repeated for various agencies if that lesson was critical to the success of each facility. Simple lessons that are common practice today are noted for the agencies that struggled to identify these issues over the last 20 years. Sites are discussed in the order of their presentation in Table 3.1.

JOINT DIRECT INJECTION AND INFILTRATION BASIN FACILITIES

As part of a comprehensive water management strategy, some agencies are using both infiltration and direct injection or ASR practices to recharge the groundwater system for a variety of reasons. The three representative examples of this application are noted below.

Equus Beds Groundwater Recharge Demonstration Project—Wichita, Kansas

Located in Wichita, Kansas, the project was initiated in 1997 to examine artificial recharge techniques and their effects on water quality in the Equus Beds Aquifer. At two facility locations, recharge was tested through a large diameter production type well, five infiltration basins, one trench, four galley vadose zone wells, and 10 small diameter passive siphon wells. Two source water types were used for testing. The demonstration project served as an effective tool to establish the technological components of a full scale underground storage project, and also provided essential information to regulatory agencies and the public. The key lessons are as noted.

151

Table 3.1
Site visit locations

Both Well Recharge and Surface Recharge	
Equus Beds GW Mgmt District No. 2, McPherson County, KS	Surface water recharge through basins and wells
Orange County Water District, Southern California	Injection wells, surface/reclaimed water through basins
Well Recharge	
Scottsdale Water Campus, Scottsdale, AZ	Vadose zone wells
Calleguas MWD, Thousand Oaks CA	ASR wells
Highlands Ranch ASR Program, Centennial WSD, CO	ASR wells
Peace River/Manasota RWSA, FL	ASR wells
Las Vegas Valley WD, NV	ASR wells, injection wells
Beaverton, OR	ASR wells
San Antonio Water System, TX	ASR wells
Surface Recharge	
Nassau County DPW Recharge Basins, Nassau County, New York	Surface water recharge through basins
Central Avra Valley Storage/Recovery Project, Tucson, AZ	Recharge of CAP recharge water through basins
C-111 infiltration basins, Homestead, FL	Surface water recharge through basins
Fort Dix, NJ	12 recharge basins for reclaimed water
Edwards Underground W. District, San Antonio, TX	Recharge basins for surface water
Failed SUS Sites	
Urrbrae Wetland, South Australia	ASR
NW Hillsborough, FL	ASR
East Meadow/Cedar Creek, NY	Surface recharge[*]/ injection well[†]
Bay Park, NY	Surface recharge[*]/ injection well[†]

[*] Succeeded.
[†] Failed.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
    - Provide seasonal storage of excess runoff from river.
    - Restore groundwater levels in the aquifer.
    - Expand the usability of local well fields.
    - Prevent the migration of nearby chlorides and saltwater plumes towards the well fields.
    - Conserve water in the local reservoir

152

- The source water is raw river water, either bank filtered and pumped from a river-side well, or treated to reduce turbidity, organics, and herbicides.
- The sustainability of the source is sporadic depending on excess storm flow events.

Feasibility, Design, and Expansion

- The side walls of the infiltration basin were lined and ripraped to prevent topsoil erosion during heavy rainfall.
- The installation of access ramps into the basin allowed easy access for the equipment usedto clean the sediment and debris.
- A clay layer located below an infiltration basin can significantly reduce the recharge rate, and communication to adjoining aquifer layers, to less than 1 to 2 feet per day.
- Recharge trenches started operating at extremely high recharge rates of 60–75 feet per day with difficulty wetting the entire trench area. The trenches clogged rapidly through the precipitation of iron oxides.
- The installation of small diameter passive siphon wells that penetrated the clay layer beneath the recharge basins significantly increased recharge rates (400 percent).
- A modified recharge trench method using 12-foot diameter columns was used, simulating vadose zone galley type wells. Recharge rates varying from 5 to 30 feet per day were achieved.

Operation and Maintenance

- If powdered activated carbon (PAC) is used to remove organic compounds during pretreatment, it must be removed prior to recharge to prevent clogging.
- Lower recharge rates yielded longer recharge times and higher recharge volumes than higher recharge rates.
- Recharging treated high turbidity raw water made for an easier operation as compared to recharging low turbidity raw water from a bank filtration well.

Orange County Water District Infiltration Basin Project—Orange County, California

Located south of Los Angeles in Southern California, the project recharges raw (untreated) surface water from the Santa Ana River into a gravel, sand, silt and clay aquifer of potable quality. The county agency does not provide service connections, but as a groundwater basin manager, replenishes the groundwater system for nearly 500 permitted municipal, industrial, and agricultural wells located in 34 cities. Recharge is through 1200 acres of infiltration basins capable of percolating over 350,000 acre feet per year (AFY). Over 40 years of recharge operation has allowed the agency to raise the permitted basin draft from 150,000 AFY to over 500,000 AFY. No recharge water is recovered by the agency. The key lessons are as noted.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  - Replenish groundwater basin for managed pumping by others.
  - Overfill the groundwater basin to increase yearly pumping permits.

- Capture all surface flows before they are lost to the Pacific Ocean.
- The source water is urban storm water filled with micro-organisms, silt, fish detritus, and trash. It also includes imported raw Colorado River water and reclaimed water.
- The sustainability of the source is good. The agency has secured all water rights to the river, allowing sole right to storm flows and effluent discharged upstream.
- It was progressive for a county to take physical control of a groundwater basin. This was not just regulatory oversight, but applied management of the basin through managd aquifer recharge programs. The recharge program was increased in scale to balance the increase in permits for groundwater pumping. The basin draft is balanced.
- The water utility does not deliver the water through pipes to the customers, it places the water in an underground reservoir for the user to recover at a location of their choosing through their own reservoir connection—a well.
- It was progressive for the agency to secure water rights on all Santa Ana River flows to allow capture of all flow before it is lost to the ocean.
- It was progressive for the agency to put non-native surface flow to beneficial use, considering that most of the source water is treated effluent released by upstream communities.

Feasibility, Design, and Expansion

- Analyze drill logs and map the confined and un-confined sections of the basin. This will allow definition of the target area for infiltration recharge facilities. Land acquisition should include the largest flood plain area possible around creeks and rivers; adjacent and tributary creek beds; recreation lake opportunities; flood control detention basins, and old quarry pits.
- The use of an in-channel capture basin just above the receiving hydrologic basin offers some retention of water and controlled releases for recharge.
- Separating the river bed flood plain into two channels allows controlled release flow down one half of the river bed, while the other half is being groomed for high percolation rate recharge.
- The natural, or primary, channel "armors" itself through seasonal erosion of silt and sand, leaving rocks and gravel behind. Silt clogs the areas between the rocks resulting in low percolation rates. This type of riverbed is difficult to clean. All the infiltration basins are groomed to remove the rocks, leaving a sand bed. The highest percolation rates will be achieved in the sand.
- In a surface recharge basin design, the linking of the basins in a chain allows for progressively cleaner water moving to the final basin. This results in the highest overall system recharge performance. Diversion channels or pipes should be installed for each basin to allow selective removal of basins for maintenance without disturbing the chain-filtration system.
- Basins should not be designed as dead end sinks. A flow must be maintained through the Ponds at all times to keep the water "alive." This may require outflow from the final facility to a discharge sink, an open dry creek bed stretch, or return to the native river.
- Deep basins can be co-used for public recreation use. This has drawbacks: the basin level must be maintained for boating; the fish detritus and other bacteriological

154

organisms supporting a healthy fish environment increase the basin clogging rate; and draining the basin for maintenance requires costly fish kills, removal, and re-stocking.

- Flood control basins should be planned for conjunctive use as infiltration basins. In this way precious land in creek paths could be used for recharge facilities most of the year. During the storm season, the recharge ponds can be drained to allow increased storm capture.
- The horizontal recharge rate in pit walls is low. Re-structuring pit walls to a gentle slope will increase the surface area for downward infiltration and therefore increase subsequent percolation rates for the basin.
- Historic deep pits used for recharge make good de-silting basins. The volume and low flow allow residence time to settle finer material.

Operation and Maintenance

- Installation of an inflatable dam in the main river channel supports the operation by impounding the river to increase surface area contact and in-channel recharge rates; diverts water to a side channel and basin system; and provides the driving head for the side channel and basin system recharge operations. The dam can allow overflow down the river bed, or can be deflated during flood events to prevent the trapping of sediment.
- The development of automated trash racks to remove "urban" trash from the intake grates maintains a consistent operation flow. Trash clogging the intake grates will reduce the head on the recharge system, forcing flow over the dam. The chain and spike system routinely pulls the vegetation, plastic bottles, and plastic bags off the grate and deposits it in a drying area.
- The initial diversion channel is a critical component in a chain of connecting basins. Basin diversions are available to allow individual basin rehabilitation, but the main diversion channels cannot be bypassed. Two separate structures should be planned at each diversion to allow maintenance of this piece of the system without having to take the entire system off-line.
- The agency has been progressive in designing automated basin cleaning vehicles. These machines clean the basin floor while the facility is maintained in service.
- The use of "T" diversion levies provides the highest percolation rates. The tortuous path directs the shallow flow across the entire surface area. The increased velocity prevents deposition of silt. This results in higher percolation rates with lower clogging rates per unit of operation time.
- Bank filtration to subsurface collector pipes may provide an alternate solution to de-silt the water to a higher quality than through the use of settling basins. The high quality water could then be piped to off-channel basins. The superior quality water would reduce the annual clogging and cleaning frequency of the basin.
- Shallow basins have a better percolation rate than deeper basins. Water quality and application best dictates the basin depth. Highly polished water could be recharged in deep basins due to low clogging potential. Water with higher silt concentrations achieves greater infiltration rates in shallow basins due to a reduction of the hydrostatic head on the basin floor and resulting compaction of clay platelets. The reduced detention time in the shallow basins also reduces the opportunity for algae

155

formation, which can precipitate calcium carbonate deposition and therefore inhibit recharge rates.

- The system should be designed to allow emergency closure of the main river intakes at such times as the water quality is unacceptable. During storm events, the sediment load in the water may not settle in the conventional desilting system, resulting in clogging of connected recharge basins. These flows must be blocked, either by water quality sensors and automated diversion gate control; 24-hour operator observation and control; or a midnight call that a storm is developing and maybe someone should close the gates (current system).

- Designing the basins to operate by river stage and gravity feed may result in operational limitations. In particular it is only possible to use facility locations that are down stream and generally in the same valley. To divert excess flows to other tributaries, off-channel basins, or upstream locations, pumping plants and pipelines would be necessary and should be considered in the master plan.

- Full basin drying, rehabilitation, and re-shaping takes about a month. The first few inches of sand are removed with heavy equipment. New sand is trucked in to replace the lost material and maintain the depth of the facility.

- The material removed from scraping and dredging is washed to recover the native sand. The silt is discarded and the sand is recycled back into the basin restructuring program. Loss of material over decades of operation would otherwise result in deeper basins and the need for purchase of new fill material to maintain elevations.

- For insect prevention, deeper water that is moving has lower potential for larvae development. Shallow water with slow movement increases the fly and mosquito larvae potential.

- Managed aquifer recharge to the regional basin must be spread out. Placing all the facilities in an isolated portion of the basin may result in water stuck in slow movement in the aquifer, not fully balancing the effects of pumping. Recharge facilities may need to be in close hydraulic connection with the major pumping centers, if possible. This may require piping the source water considerable distances for infiltration or direct injection at the location of need in the basin.

Orange County Water District Sea Water Intrusion Barrier Direct Injection Project—Fountain Valley, California

Located in Fountain Valley, Southern California, approximately one mile east of the Pacific Ocean coastline, the project recharges highly treated effluent and potable water into a gravel, sand, silt, and clay aquifer to prevent the intrusion of sea water into the groundwater basin. In operation for over 40 years, the barrier is upheld by the hydro-static pressure applied from a string of 26 facilities containing nested wells screened in 3 to 4 aquifer units. The yearly recharge volume is in proportion to the basin draft, with the current recharge capacity averaging 12 mgd. Expansions to be completed in 2007 will allow 30 mgd delivery to the barrier wells. This program not only protects the basin, it contributes approximately 13,000 AFY of usable recharge to the basin that can be permitted for pumping. The key lessons learned are as noted.

156

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  - Protect the groundwater basin from the intrusion of high chloride sea water during heavy pumping.
  - Provide a source of water to support, or feed the reverse gradient into the basin. This becomes managed aquifer recharged water.
  - Provide beneficial reuse of the in-county effluent through advanced treatment processes.
  - The recharge program is driven by a mandate from the state.
- The source water is highly treated effluent using microfiltration (MF), reverse osmosis (RO), and ultraviolet (UV) disinfection processes. The new plant will include advanced oxidation processes (AOP).
- The sustainability of the program is very good. Growth will be matched by the availability of more effluent to sustain the barrier program.
- Water placed in a sea water intrusion barrier system replenishes the aquifer, providing recharge water available for beneficial use within the basin.

Feasibility, Design, and Expansion

- Minor changes in conventional sewage effluent treatment to include RO, UV, MF, and AOP (no Cl_2) can render approximately 85% of the water useful as purified for direct injection, as approved by the Regional Water Quality Board. Approximately 70 mgd of effluent will be returned to beneficial use with the completion of the new treatment plant.
- A groundwater barrier can be successfully developed and maintained for over 40 years to hold back a non-potable source (the Pacific Ocean) from a potable basin. With monitoring and controlled injection, the barrier can be sustained with groundwater production increasing over 125%, so long as the basin is replenished each year through natural and managed practices (no adverse water level decline rates per year).
- The Talbert Gap barrier can be maintained using approximately 8–10% of the annual basin pumping. The current application of 4% annual pumping is not adequately holding back the high chloride migration.
- Cluster well construction in a single borehole was not the most successful path to take. Though grout was placed between aquifer zones, pipes reaching to deeper zones corroded in their passage through adjacent zones. This resulted in zonal leakage. Special removable insert tubes had to be designed to sleeve between the target screen and the surface sanitary seal, providing a leak-tight seal.
- Cluster well construction used stainless steel casing and screen, although mild steel was substituted for the collars and centralizers. The collar inspection windows were not welded closed. Corrosion, galvanic and possibly other types, focused on these inspection windows, forming holes through the stainless steel casing and thereby creating leaks.
- Cluster well use of 6-inch casing proved too small for the repair of the holes with a swedge. The casing is too small for development pumps capable of providing the necessary yield.

157

- Installation of a single well per borehole is the approach with the highest success and lifespan.
- Well casing is increased to 12-inch, allowing adequate room for recovery pumping equipment. Construction is all stainless steel, including collars, centralizers, sounding tubes, and gravel feed tubes.
- Improved well equipment includes borehole flow control valves, back-flush recovery pumps, and automated flow control based on installed electronic water level probes.
- Target zone wells are individually controlled, allowing selected zone injection to produce an isolated, desired aquifer response.

Operation and Maintenance

- Backflushing is performed when specific injectivity drops 50%. The backflush typically requires removal of 100 well volumes over a few hours of pumping. Backflushing utilizes constant pumping without surging.
- Low permeability barriers (mesas) are still transmissive. They will slowly leak. It is necessary to account for this low-permeability leakance in the original design. OCWD now has to develop a catch-up plan since leakage has occurred into the mesas.
- Laboratory modeling of a "Model Injection Well" enabled refinement of chemical and bacteriological interaction with the wells. The program resulted in eliminating the use of the iron-rich Deep Aquifer water as a recharge water source for the barrier injection wellfield.
- Well plugging issues and specific common well bacteria problems have been fully identified through sampling and laboratory testing. This enabled a specific chemical rehabilitation program to be developed that works for the barrier wells.
- Specialized well rehabilitation equipment has been incorporated into a customized truck allowing daily work at well sites located in traffic corridors. The work can proceed during low traffic hours and easily demobilize during high traffic use hours.

DIRECT INJECTION WELL FACILITIES

Due to hydrogeological, environmental, or political constraints, recharge water may have to be placed directly into the aquifer through wells. This technology is slightly more complex than surface recharge and typically involves considerably more supporting equipment. Examples of this well recharge technology are noted below.

Scottsdale Water Campus—Scottsdale, Arizona

The City of Scottsdale's Water Campus is a multi-use surface water and wastewater treatment facility near Phoenix, Arizona, utilizing underground storage as a component of the operation since 1999. The complex includes a 54 MGD surface water treatment plant, a 12 MGD water reclamation plant, a 10 mad advanced water treatment plant using micro filtration and reverse osmosis, 27 vadose zone injection wells, and 28 vadose zone emergency plant overflow injection wells. Treated effluent not used by golf courses is further treated with advanced methods and stabilized prior to injection into the wells. The wells penetrate 180 feet into the unconsolidated alluvial fill with the water table at a depth in excess of 600 feet bls. Individual well recharge rates

158

average 300 gpm with a yearly target capacity of 15,000 AF for the entire wellfield. This vadose zone wellfield is believed to be the largest and most successful in the United States for storage of reclaimed water. The key lessons are as noted.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  - Disposal of highly treated wastewater.
  - Provide aquifer recharge credits for removal elsewhere in the basin.
  - Raise aquifer water levels upgradient of the existing production wellfield.
  - Bank unused Colorado River allotment during periods of surplus.
- The source water for the primary vadose zone wells is ultra-pure and highly polished. Tertiary treated wastewater is further treated through a micro filtration plant and reverse osmosis train. The water is then degasified to remove carbon dioxide, and stabilized with lime to reach a non-corrosive, non-precipitating pH. The water is delivered into the wells through controlled orifices to maintain a backpressure and no air entrainment.
- The source water for the emergency plant overflow wells is the out put from the reclamation plant, or tertiary treated wastewater. To offset the impacts of this relatively poor water quality recharge, the emergency wells are designed for easy wellbore filter pack removal, cleaning, and re-insertion.
- The sustainability of the source is very good, with growth spurring additional waste water supplies in the future. Having the surface water plant for treating imported Colorado River water on the same property offers an additional source of recharge supply. The plant is designed with a leg in the treatment train specifically to treat surface water for vadose well recharge.

Feasibility, Design, and Expansion

- Nearly 5 years of pilot studies on filtration methods, vadose "dry-well" design and operation led to the final design of the Water Campus recharge component.
- To ensure percolation the recharge water needed to be placed below the surface layers and soils.
- Recharge to the aquifer is monitored by three water table monitor wells located in the three potential flow directions.
- Two injection well designs are used, based upon the source water quality. The wells using polished water are cased with PVC whereas the emergency plant overflow wells receiving higher particulate matter are constructed with socks filled with gravel pack. The socks are removed for cleaning and re-inserted into the wells.
- In the 28 emergency wells, the injection tube extends 20 feet into the gravel with vent pipes plumbed to within 40 feet of the 180 foot base.
- In the primary vadose zone wells, the injection tubes extend into the well through the center of the 10 inch casing to near the 180 foot base. The tubes are fitted with designed orifices or automated borehole flow control valves to maintain a positive back-pressure under operation.

159

- Flow testing and design of the emergency wells resulted in average recharge rates of 300 gpm with maximum mounds rising to 70 feet below surface. Flow testing and design of the primary wells provided average recharge rates up to 450 gpm with water level rises to within 70–80 feet of surface.
- The level of pretreatment for the primary vadose zone wells is substantial, however with over 7 years of continuous operation this recharge approach appears to be sustainable.

Operation and Maintenance

- During peak wet weather conditions when the wastewater plant is overloaded, the effluent is diverted to the emergency wells to reduce the loading on the advanced wastewater treatment plant.
- The emergency wells operate in clusters of 9 at 300 gpm each. Other clusters are rotated into use as necessary.
- Fixed orifice plates required frequent changing as water mounds develop. To avoid rig and labor costs for removing and re-installing the recharge tubing, wire line techniques were developed to set-in a smaller orifice plate.
- Operation of the vadose zone wells indicated a performance impact of wells operating side by side. Since only a fraction of the wells are used at any one time, the optimum operation is to use every other well at a minimum.

Calleguas Municipal Water District ASR Project—Calleguas, California

Located north of Los Angeles, California, in Thousand Oaks, the project recharges filtered surface water into a gravel, sand, silt, and clay fill aquifer of potable quality. As a wholesale water distributor, Calleguas, in partnership with the Metropolitan Water District of Southern California, uses 18 wells with an injection capacity near 48 mgd to store water underground as part of the normal storage and distribution system infra-structure. Construction of most of these facilities was completed and placed into operation during 2004. Approximately 16,000 AF has been recharged over the 13 years since the initial ASR test well was tested. The key lessons learned are as noted.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  - Provide a dry season supply to supplement existing surface reservoirs.
  - Enable full year use of the transmission capacity available in state canals and pipelines using underground storage to accept the water when surface facilities are full.
  - Increase total seasonal storage capacity.
  - Emergency supply in service area in the event the transmission line or delivery tunnel is damaged by an earthquake.
- The source water is mostly in-state surface water that is imported from Central California. The water is filtered, recharged into the aquifer, recovered, and placed back

160

into the transmission line delivering water to surface reservoirs and water agency treatment plants.

- The sustainability of the facility is good in that the technology is being used as an integral part of the regional water storage and delivery system.
- Maintenance costs are high due to well clogging, but still outweigh the surface storage choices. Recovered water costs range from $0.51 to $0.72/1,000 gallons ($166 to $235/AF).

Feasibility, Design, and Expansion

- A design feasibility study included hydrologic investigations, geochemical studies, water availability modeling, and field testing on an existing well.
- Computer modeling of the groundwater system and surface piping system were done to ensure harmony when recovery is required. Under times of need, water cannot be recovered to a pipeline that is already at maximum capacity.
- It is important to inform other groundwater users in the area of your intentions and potential impacts. Work out compacts and agreements up front to ensure that you retain the public trust.
- Demonstration testing was conducted on an existing well. The performance declined rapidly on the old well, most likely due to debris plugging, very poor recharge water quality, and temperature difference. Recharge efficiency recovered with pumping.
- New recharge wells were cased at 12 to 20 inches in diameter with stainless steel casing and screen. Capacity is 0.5 to 2.0 mgd. The casings have been sized to accommodate the necessary recovery pumps.
- It is important to install a backup sounding tube to allow manual water level measurements as a validation check for downhole pressure transducers
- Recharge is through the pump bowl with wide open system valves and no throttling of pressure. Sustaining pressure is provided by a local storage tank.
- An on-site storage tank provides a single point of recovery for all the wells, allowing disinfection of recovered flows and any other treatment process that may be required. Additional benefits include release of any air pumped from the wells, hydraulic equalization, and control of well operations.
- For water-lubricated pumps, the water flow should be maintained at all times during recharge to prevent the formation of mineral deposits on the shaft and guides.
- It is a good idea to meter lubrication water flows as they may receive recharge credits.
- To distribute the aquifer hydraulic load, three ASR wellfields are planned along the valley axis to meet full build-out storage capacity of 300,000 AF.

Operation and Maintenance

- Recharge startup includes pumping the well to waste to clear any debris and stale water, and then diverting the flow to the storage tank. The production is stopped with all valves full open, resulting in a siphon back down the well.
- Siphon startup requires all vent ports to be sealed until the system slowly develops a positive pressure from the supporting storage tank.

161

- All controls and valves are automated, including the casing and column pipe air vacuum/air release valves.
- Back flushing the wells is done frequently to maintain recharge performance. Transition between recharge, flush, and recharge is automated.
- Well waste lines are equipped with an orifice plate to limit the discharge and hold back the proper pressure, simulating normal pump operation.
- A SCADA scheduler program selects the best choice recharge or production well to meet the pre-programmed parameters of efficiency and a balanced wellfield operation.
- Well performance is measured by specific injection capacity and water level rise.
- Pipeline debris from the regional pipelines and tunnels is filtered at the wellhead with an inline strainer to prevent damage to the pump and open, water lube shafts.
- Recovered water quality has increased iron, manganese, and radon levels, though not a concern at this point. The area and location for a future treatment plant should be considered in the beginning.
- Recovered water THM concentrations reduced during storage.
- Power generation equipment was placed in a few wells, though the benefits were not worth the costs and the program has been suspended.

Highlands Ranch ASR Project—South Denver, Colorado

Located in the South Denver metropolitan area, Colorado, the Centennial Water and Sanitation District operates the Highlands Ranch project, recharging potable system water into a deep bedrock sandstone and siltstone aquifer. The water table is 800–900 feet with aquifer sections exposed at 900–2,000 feet. The system captures surplus storm flows in surface reservoirs. During periods of low system demand, the excess treatment plant capacity is routed to the recharge wells. Highlands Ranch has 53 wells for an 11 mgd production capacity and uses 23 wells for recharge for a 5–6 mgd recharge capacity. The key lessons are as noted.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  - Provide a dry season supply to supplement existing surface reservoirs.
  - Increase total seasonal storage capacity for annual delivery.
- The source water is treated potable water from the distribution system containing a chlorine residual.
- The sustainability of the storage system is good in that it has the capability to capture sporadic storm flows that may not be available every year. Surplus storm flows are captured in surface reservoir(s) and distributed for ASR recharge.
- Wells and ASR were part of the master plan during development of the original distribution system network between 1982 and 1992. The system was sized and designed to support full build-out water demands of the master plan community.

Feasibility, Design, and Expansion

- ASR water is delivered through normal surface water conveyance pipelines to local storage reservoirs, and then reverse-flowed to the wells.

162

- The community is located on the upstream side of a major urban area, providing an optimum location for first capture of storm surplus on the river system.
- All water banked can stay any length of time in the aquifer and is fully removable at any time or rate. Banked water is used to make up for drought shortfalls.
- The first 26 wells were drilled to secure water rights, though were not constructed to accomodate ASR. The wells were constructed under-sized and were built with mild steel screen with no gravel pack. Due to sanding issues, most of the wells were replaced within 10 years.
- Replacement wells were constructed up to 18-inches in diameter with stainless steel wire wrap screen and a gravel pack envelope. Mild steel is still used for the blank sections.
- Wells are completed to very deep aquifer sections exposed at 900 ft to 2,000 feet below surface. Water table is at 800–900 feet. Submersible pumps are used with settings around 1,350 feet.
- A borehole flow control valve was first developed for this site in the early 1990s. The valve allows for a controlled flow out through the side of the pump column into the annulus between the pump column and the well casing annulus, without having to pass through the pump bowls. The valve can be used to throttle and hold specific flow rates.
- The recharge borehole control valve is placed just above the pump, 300–400 feet below the water table. This reduces the hang weight of column sections and torque potential by the pump on the valve.
- The borehole control valve is controlled by a nitrogen gas actuating system. The pressure is monitored and alarmed in the SCADA control system to ensure that enough cylinder pressure is available to operate the valve.
- Of 23 equipped ASR wells, two wells are equipped for full automation of the borehole flow control valve through program logic control. Control includes open, close, and automated adjustment of the valve to maintain the set flow rate. Future improvements include equipping more sites this way.
- All wells are equipped with wellhead air-vacuum valves, electromagnetic flow meters, and a globe check valve. The globe check valve is piloted for recharge on and off control.

Operation and Maintenance

- In preparation of recharge startup, each well is pumped to waste to purge stale water from the well casing and any loose sand.
- The initial recharge startup of the wells includes displacing all air in the pump column with water prior to starting injection.
- The borehole control valve is opened manually to the desired operation rate or water level. The above-ground globe valve controls the start and stop of recharge. Under automated stops, the air-vacuum valve fills the column with air. Automated restarts can force the column of air into the wellbore and aquifer, causing well clogging due to air binding.
- During recharge operation, the wells are back-flushed once a month to clear any clogging and improve operation efficiency. Purging is completed at the beginning and

163

end of every other month to avoid power company demand use charges in the center month.

- Recharge water is recovered directly to the distribution system with no treatment other than chlorination.
- The deepest aquifer has poor quality water that is pumped up to the surface stream system and reservoirs for blending. This provides surface credits. The blended surface water is removed downstream for recharge in the shallower aquifer units.
- Recharged water does not have to be removed at the point of injection, rather it can be removed at any location in the same aquifer body. This reduces pipelines and distribution system improvements.
- Performance evaluation of the recharge wells is based upon increased changes in water level or injection rate.
- Continued modeling is employed to integrate recharge technology and community planning and thereby meet increasing water demands associated with growth.
- The hydraulic impact of the Highlands Ranch Recharge project is very small on the vast aquifer system.

Peace River ASR Project—Desoto County, Florida

The Peace River Project is located in Desoto County, southwest Florida, and has been recharging chlorinated, treated potable water into a deep limestone bedrock aquifer for 19 years. The ASR project is administered by the Peace River/Manasota Regional Water Supply Authority to supplement Peace River surface water diversions during times of low flow. Over the last 20 years, the facility has been expanded several times and now includes an 85 acre open reservoir, 24 mgd water treatment plant, and 21 ASR wells used for potable water storage. The ASR system has successfully stored approximately 33,150 AF and recovered approximately 25,938 AF. Well recovery capacities range from 0.5 to 3.0 mgd. Recovered water from the new wells is currently blended with raw water and re-treated for potable delivery. Recovered water from the older wells does not require retreatment other than disinfection.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  - To provide long-term and seasonal water supply in conjunction with the Peace River surface diversions.
  - Bank up to six months of water supply to supplement Peace River diversion reductions during low-flow or drought periods.
  - Capture Peace River surplus flows before it is lost to the ocean.
- The source water is treated water from the potable distribution system. The origin of the water is surface flows from the Peace River.
- The source quality is variable depending upon the flow characteristics of the river with TDS values ranging from 160 to 596 mg/L, chlorides ranging from 30 to 162 mg/L, and sulfate ranging from 32 to 175 mg/L. The source water also contains moderate levels of metals such as calcium and iron. The treated water has low color and turbidity with values estimated at 0.02 ntu.

164

- The groundwater quality within the storage zone is slightly brackish with TDS values ranging from 650 to 800 mg/L, chloride ranging from 151 to 206, sulfate ranging from 216 to 232 mg/L, and low levels of metals including arsenic (7 µg/L), silver (8 µg/L), and calcium (75 mg/L).
- The sustainability of the program is good. More source water is lost to tide than can be captured. Low river flow events exceeding 100 days are frequent. Droughts exceeding 180 days duration have been experienced recently, making the ASR program an integral part of the storage and delivery system.

Feasibility, Design, and Expansion

- It is necessary to complete a well inventory of all local well users during the initial well site selection exercise to aid with selection of an ASR storage zone.
- The Peace River facility has been designed and constructed in phases, for efficient use of limited capital, adding a few new wells each year. This has resulted in a variety of types and sizes of pump, flow meters, and valves due to the ad hoc nature of the well field expansion over the years.
- Facility equipment should use standardized well equipment, casing size and piping instrumentation in order to improve reliability and minimize down-time.

Operation and Maintenance

- The pilot testing was very successful and indicated that the potential recovery could approach 100%. Subsequent site operations have routinely achieved 100% recovery efficiency using a blend of stored ASR water and reservoir water.
- The system must be capable of shutting down during poor water quality events in the source river such as extensive algae blooms that result in difficult water treatment and taste and odor problems.
- Both the water treatment system and the ASR system are highly automated and controlled by a state-of-the-art SCADA system.
- Well clogging has not been a major problem at the Peace River project. Minor specific capacity reductions are restored during annual ASR recovery operations.
- Recovered water data in new wells revealed serious water-rock reactions that are suspected of releasing arsenic during ASR storage. The recovered water from new wells is re-treated in the onsite water treatment plant at a significantly higher cost. Recovered water from wells that have been in operation for a few years, or that have otherwise established an adequate buffer zone, do not experience elevated arsenic levels.
- The recovered water from the new wells is blended with raw reservoir water to reduce the incoming arsenic concentration. This strategy has been successful and has allowed the ASR system to continue to operate without jeopardizing public safety.
- Lateral displacement of the stored water bubbles around each well may be caused by storage or recovery operations at adjacent wells. The wellfield must be operated as a system, not as individual wells. Well spacing may be too close.
- Possible upward movement of brackish water from beneath the storage zone during extended recovery periods may occur. This would be due to substantial drawdowns at

165

individual wells caused in part by interference effects due to the close well spacing for the new wells.

- The use of CO_2 has proven to be an effective well rehabilitation tool without the safety and labor issues associated with strong mineral acids. The CO_2 can be bubbled into recharge water and did not result in any regulatory excursions or need to purge and dispose of the well treatment water as was required using acid.
- Well development treatments using acid or carbon dioxide may be exacerbating the arsenic situation by exposing more of the carbonate section and thereby releasing additional arsenic into the aquifer.

Las Vegas Valley Water District ASR Project—Las Vegas Valley, South Nevada

The Las Vegas Valley, located in Southern Nevada, is utilized for one of the nation's largest groundwater storage projects. The program recharges potable water from the distribution system through mostly older production wells to bank the water in a deep alluvial valley consisting of moderately consolidated gravels, sands, silts, and clay sized material. The system is managed by a basin wide Water Authority to ensure benefit to all communities. The agency utilizes up to 67 wells for a maximum injection capacity of 100 mgd. Another 41 production wells are used with the ASR wells for a recovery capacity of over 157 mgd. The program has been in full scale operation for 18 years and has banked over 350,000 AF. The key lessons learned are as noted.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  – Capture full allotted surface water flows before they are lost to lower river users. Excess supplies above public demand to be banked underground.
  – Develop underground water reservoir to use as an interim supply to support growth until imported water supply projects are completed.
  – To provide interim supplies during drought restrictions from a protected under ground water reservoir.
  – Provide an emergency supply of water to the communities in the event the Colorado River control dams are breached, leaving surface water intakes hundreds of feet above the native river level.
  – Controlled put-and-take program recharging water at golf course locations during the winter for recovery during summer, effectively removing this heavy water user from the summer system operation.
- The source water is chlorinated potable water. The water origin is surface water from the Colorado River.
- The sustainability of the program is very good in many aspects. The program of capturing unused water rights has a finite lifespan. The ability to capture surplus imported flows during periods of excess is very good. The facilities can be flexed into operation to capture and bank over 10 AF in less than one month, allowing surpluses called at the end of the water year to be captured.
- Some communities did not have aquifers under their city that could be used for banking, but they saw the merit of capturing the surplus flows and wanted to

participate in the program. These communities joined a consortium allowing them to pay into a groundwater banking program conducted elsewhere and own some percentage of the product.

- As elected by the voters, an oversight water authority was formed to manage the county groundwater, surface water, and import water supplies for all entities, ensuring that water supplies are fairly distributed to all communities, big or small.
- Every groundwater user pays some percentage of the groundwater banking program costs, based upon their current surface and/or groundwater use. This includes municipal, industrial, agricultural, and domestic groundwater users. The authority is provided the ability to levy fines and penalties.
- The water authority pays the utility capable of performing underground storage to recharge, store, manage, and recover water. The authority is paying utilities in their own state and two adjoining states to conduct recharge programs.

Feasibility, Design, and Expansion

- A ground water recharge program could be successfully implemented using the existing, older wells pumping to reservoirs for sand removal. Recharge could be through the existing vertical line shaft pump bowls and column pipe at normal system pressures. The resulting injection rates could be 50% to 75% of the normal production rate for the well.
- Wells drilled and constructed by the reverse circulation rotary drilling method, gravel packed, and constructed for normal production provide the most efficient recharge and recovery wells. These wells generally can be developed to operate at higher rates than wells constructed by other techniques.
- To allow simultaneous recovery of existing water rights and banked water under conditions of high demand or emergency, the agency installed additional production wells enabling nearly twice the normal recovery capacity.
- The well facilities were modified to improve the delivery volume and pressure. Distribution system piping for the wells, normally plumbed to low pressure tanks, were also connected to high pressure mains.
- After spending a few years rotating flow meters and dropping pump laterals for recharge, the sites were equipped for rapid transition from production to recharge. The pump shaft laterals were equipped with ratchets to prevent rotation. Bi-directional flow measuring electromagnetic meters were installed along with a bypass "U" pipe around the system check valve. The bypass pipe was equipped with an automated globe valve enabling SCADA start and stop control. The globe valve is piloted for pressure reduction, allowing the applied recharge pressure to be reduced to control the performance of the well. The casing water level access ports were equipped with air transfer vents.
- Air release valves were moved as close to the wellhead as possible. Flanges and gaskets were placed on the well casing tops to enable sealing the wellbore for full casing pressure injection. Pipeline purge ports were installed and discharge paths established.
- Recharge through submersible pump bowls was tested with poor success. Four out of five units exhibited repeated pump operation problems and motor failures. The pump

167

motors did not generate stray electric currents during rotation since well pump motors are not typically equipped with "excitation circuits" required to turn the motor into a generator.

- During field aquifer testing, a wide variety of rates could be achieved with a borehole flow control valve, allowing development of an injection performance curve around, or near the actual design parameters.
- Detailed satellite survey work has indicated that ground subsidence has stabilized in response to recharge.
- In a valley wide hydraulic test, 52 wells were used to inject 72 million gallons over 24 hours to determine the impact to the shallow, principal, and deep aquifer systems within a 30 mile radius. The test results became the basis for managing a mature recharge program, placing volumetric limits on specific geologic locations of the well field. The water is now logically proportioned to areas to achieve the desired hydraulic impact.

Operation and Maintenance

- The surface water treatment plant is run at peak capacity year-round with the water either being diverted to the community for consumption or recharged down wells and into the water bank.
- The location in the aquifer and the target wells for recharge are determined ahead of time by modeling and evaluation. This ensures that the volume placed produces a managed response in the aquifer.
- In the event of main-breaks or large fires, sections, or all of the recharge wells can be stopped within 10 minutes by the command center to preserve system capacity and pressure.
- During recharge, a balancing act is maintained to run booster pump stations lifting water to recharge sites in higher elevation zones and not lowering the pressure in any section of the grid. A sophisticated SCADA pump control routine is incorporated to assist operators in this water shuffle through nearly 400 booster pumps and close to 40 major reservoirs.
- The dedicated injection wells have no ability to pump or purge the wellbore prior to starting, forcing residual sand, silt and bacteria into the screen and formation each year the well is placed into operation. This results in a 10–20% loss in efficiency each year, rendering the wells unusable within 3–5 years.
- Stagnant water is purged from the pipelines until observed clear (<5 ntu). Just prior to startup, the wellhead air vacuum, air release valve is closed to prevent the intake of air. The static water level is measured. Once a positive pressure is established on the injection line, the air valve is then opened to vent the casing during shutdown.
- One day after startup, the flow, pressure, and level is checked for performance analysis. The well flow is then adjusted to a higher or lower rate as necessary to maintain performance until the end of the designated cycle.
- Water level is allowed to climb to within 25 feet of surface before the well is stopped and taken out of service for the season. The wells are not back-flushed at any time during the operating season.

- During recovery startup, the static water level is taken just prior to startup for performance evaluation and on-site water level probe calibration. The waste flow is throttled, by inline orifices, to maintain the flow in the design range, preventing over-pumping at low pressure and subsequent formation damage. The intensity and duration of the sand discharge is used to monitor for damage caused by recharge through comparisons to previous startups (performance parameter).
- At startup, the recovery wells are maintained continuously online for a minimum period of 24 hours to get startup pumping performance data. The flow, psi, levels, and electrical usage are manually recorded.
- During non-use periods, the wells sit idle without any trickle chlorine feed or periodic flushing programs to abate bacterial growth. Growth, turbidity, and debris that develop during this stand-by period are not removed from the wells prior to starting recharge.
- In a mature, managed banking program, the water level rises must be correlated to the well pump performance curves to ensure over-pumping does not occur.
- Tracking well bore performance in an aquifer gaining saturated thickness each year is difficult. This causes the saturated thickness and specific capacity to increase, masking any permanent damage induced by the recharge cycle. Recharge well performance became the leading indicator for well clogging as compared to pumping performance.
- In a valley wide coordinated effort, all pumping and recharge wells are shut off and the aquifer allowed to stabilize for a spring and fall "static" water level evaluation program. This provides high quality correlation points for assessment of the hydraulic changes in the individual aquifers. This data is then used for modeling to evaluate aquifer impacts and determine optimum recovery and recharge locations for the next year.
- A multi-port research well was installed within 150 feet of an existing high capacity ASR well. The well was ported at 10 intervals over the approximate 500 foot saturated zone. Tests were conducted on head and chemistry differences over time as the recharge water quality bubble expanded and was deflated.

City of Beaverton ASR Project—Beaverton, Oregon

Located in south, central Oregon, the project recharges potable water into a leaky, semi-confined basalt bedrock aquifer of similar water quality. Using two wells with a capacity near 2 mgd, the winter and spring storm flows are captured in the surface reservoirs and routed to the wells for underground storage. Recharged water is used to supplement peak summer demands. The system is expanding to over a 7 mgd recharge capacity with the addition of two more wells in the next few years. The key lessons learned are as noted.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  - Increase winter storage to meet summer peak demand and future growth.
  - Supplemental supply during extreme weather or drought conditions.
  - Backup water supply in the event of contamination of the surface water.
  - Local town supply in the event of a major transmission line failure.

169

– Avoid transmission pipeline upgrades through off-season transfers to town. The source water for recharge is taken from the local community distribution system during periods the public demand is below the plant capacity.
- The sustainability of the storage system is good since it has the capability to capture sporadic storm flows that may not be available every year. This requires a combination of surface detention reservoirs and direct recharge wells.
- Aquifer storage costs, at $1.26/1,000 gallons ($411/AF), are considerably lower than surface storage or water importation options.

Feasibility, Design, and Expansion

- A leaky, semi-confined basalt aquifer is being successfully used for recharge and storage. The aquifer has approximately 1,000 feet of saturation with the water table around 200 feet at the recharge facility.
- A design feasibility study should include evaluation of existing aquifer users, hydraulic impact studies, and modeling with valid field data.
- Computer aquifer modeling should include information from aquifer tests, water level data, geo-chemical data, and laboratory bench testing. For the basalt aquifer, the model should define the potential impact of multiple faults on water migration.
- Demonstration testing was conducted on an existing full scale well to evaluate the needs for a larger program.
- Old production wells were not properly constructed for injection, resulting in performance losses and special operation steps. New ASR wells would be constructed for additional needs.
- Lining the open sections of old wells can reduce turbulence during injection, effectively reducing backpressure, head losses, and turbidity.
- ASR well spacing can be fairly close together as long as interference during injection and recovery are taken into account during design.
- Borehole diameter must be large enough to accommodate the pump, downhole control valve, control lines, transducer sounding tube, and water level sounding tube. Squeezing too much in the borehole can create problems.
- Borehole alignment testing with a simulated pump assembly canister can be useful to validate well acceptance and determine maximum equipment size for the testing and design.
- New ASR wells were constructed with stainless steel casing and wire wrap screen. No zones are completed as open hole.

Operation and Maintenance

- Recharge startup through the pump pushes the standing 200 foot column of air into the wellbore and aquifer, reducing performance due to aquifer air binding.
- Using a down-hole flow control valve and a pump foot valve, the column can be filled with water, recharge started, recharge set to any desired rate, and recharge stopped leaving the column full of water and under pressure.
- Back flushing the wells is critical to managing head buildup. Back flushing is completed at least every 3 to 4 weeks for each ASR well.

170

- Well performance is measured by declines in specific injection capacity.
- A turbidity meter should be installed to monitor for potential "dirty source water" sporadic events. The system should be designed to shut down automatically in the event of a turbidity spike to minimize impact to the ASR wells.
- Recovery limited to 95% of injection by state. The 5% is to cover potential basin losses through faults from the increased basin head induced by recharge.
- Recovered percentage is 85% of the 1,629 million gallons banked over 6 years operation. The remaining 15% is not spreading out over the basin, resulting in a localized mound that impacts local recharge performance negatively.
- Recovered water from the basalt aquifer experienced small increases in radon, iron, manganese, and THM. The HAA disappeared altogether. All increases are well below drinking water standards.
- Recovered water has an elevated dissolved oxygen level for up to the first day of recovery pumping. This is probably water from close to the wellbore that has had the shortest opportunity for microbial activity whereas water recovered in later days would have had greater opportunity for microbial activity, and consequent reduction of dissolved oxygen.
- A monitoring well network is important to track the dynamic pressure response of the aquifer resulting from ASR operation. This data should be modeled for selection of sites.
- The monitoring program may include routine observation of local wells and groundwater seeps prior to, during, and after recharge operations.
- A detailed wellhead protection plan was implemented including the installation of several monitoring wells at the 10-year time-of-travel boundary as an early warning system.
- Monitoring of existing contaminated sites is recommended to ensure the ASR project is not impacting remediation operations.

San Antonio Water System ASR Project—San Antonio, Texas

Located 30 miles south of San Antonio, Texas, the project recharges potable water into a poor water quality aquifer at the Twin Oakes Facility. Edwards Aquifer groundwater from an unconfined limestone aquifer is treated and piped to the Carrizo sandstone confined aquifer for storage. Using 16 wells with a recharge capacity of over 25 mgd, surpluses in one groundwater system are stored in another groundwater system. Recharged water is recovered to supplement other water supplies and thereby meet peak summer demands. The facility is equipped with a 30-mgd treatment plant and pumping system to restore the blends of recovered water to distribution system quality and return the water to San Antonio through the same pipeline. The key lessons learned are as noted.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  – Provide a dry season supply to supplement existing resources under reduced allocations.
  – Increase total seasonal storage capacity to meet peak day demands.

171

– The ASR program is driven by a mandate from the state.
- The source water is treated potable water from the distribution system containing a chlorine residual.
- The sustainability of the storage system is good in that it has the capability to capture sporadic high water elevation events in the regional groundwater system and place the stored water in a separate confined underground location.
- The recharge water is groundwater from a distant, regional (Edwards) aquifer system. In times of high groundwater levels, surpluses are called on the aquifer, allowing the withdrawal of water for recharge. When the level drops, agencies are limited to their allocation. In drought years, the level may drop below the critical elevation, requiring reductions in normal allotments. The source is sporadic, though sustainable. Recovery will occur to supplement aquifer allotments reduced by drought.

Feasibility, Design, and Expansion

- The below ground storage location does not need to be in the local community, or even within the local municipal supply aquifer. The location could be in some other basin, valley, or hydrologic system connected through pump stations and pipes, so long as the associated water conveyance cost is deemed acceptable.
- A separate aquifer 30 miles to the south was selected for ASR storage, with native water pH of 5 and high in iron and manganese. Thus, it is an aquifer with dissimilar water quality. The storage aquifer is used for local residential water and agricultural use.
- It is important to purchase sufficient land initially to encompass the current and projected build-out needs. The agency purchased about 7,000 acres, providing 14,000 AFY water rights from the native aquifer.
- SAWS developed an agreement with local water agencies representing domestic and agricultural interests to take full responsibility to fix, repair, or deepen the local community wells should such damage be caused by the operation of the ASR facility. This would ensure that the "big city" agency would not pump their small community dry.
- Any agreement with local water agencies should also account for water ownership and benefits of the ASR program to the aquifer. Texas state law does not prevent local groundwater producers from withdrawing water from the fresh water bubble surrounding ASR wells. So facility design must account, in modeling, for the maximum concentrated pumping effort by others that may occur on the facility border before the fresh water bubble is drawn into their production wells.
- Groundwater barriers using several ASR wells on their critical border can be formed with the intent of injecting native water. This will feed the local over-pumping with native water, and force the fresh water bubble back to the recovery wellfield.
- The buffer zone concept of managing the ASR bubble is not currently planned. However to date 25,000 AF have been recharged and no water has been recovered. Full recovery of 100% of the stored water is planned. As a result it will be necessary to retreat the recovered water for iron and manganese removal. If a buffer zone were to be developed around each well, the need for retreatment would be reduced or more likely eliminated.

- A treatment plant has been built to ensure water developed from wells meets the water quality standards for the distribution system, avoiding distribution system problems and corrosion. The facility allows altered recharge water, mixed recharge and native water, and (in times of great need) native water to be pumped and treated to meet the system quality needs.
- Almost all of the ASR wells were drilled with flooded reverse circulation rotary methods for ease of development and high efficiency. The first facility well was drilled mud rotary resulting in a low efficiency well with persisting performance issues.
- Blank casing is mild steel with epoxy coating. After three years of operation, there are no known corrosion issues. Pump columns are mild steel with no special protection measures for the epoxy coating.
- The wells are equipped with all the typical manual and electronic devices. Gauges and transducers monitor the wellhead and system pressure. Bi-directional electromagnetic flow meters are used. The boreholes are equipped with water level transducers. Air/vacuum relief valves are not automated.. Almost all of the valves have position indicators. All of this information, along with motor performance data, is transmitted back to the command center and alarmed for operation.
- Borehole flow control valves are provided for half of the wells (8). They are used to provide recharge flow exceeding that which would pass through the pump bowls. The valves are operated manually to a set orifice size when used.
- The well site piping configuration includes an above ground manifold with four ports controlled by pressure operated globe valves and butterfly valves. The ports are for recovery pumping, recharge supply, pipeline pre-startup purging to waste, and pressure blow-off. The well collector line also contains a waste discharge line and control valve.
- The wells are equipped with dual-discharge heads consisting of two nested column pipes. The outer column pipe does not extend below the base of the discharge head. This second large diameter inlet pipe is plumbed to the well header line and isolated with a manual valve. When water level reaches land surface and the well is placed under full pressure recharge, the bypass line can be opened to eliminate the head losses of the pump column and flow restriction (valve, pump bowls, etc.).
- Large "Y" strainers have been placed on the well recharge lines to prevent the introduction of anything larger than 1/8-inch into the pump bowls.

Operation and Maintenance

- The wells are designed to recharge at a rate lower than the production rate to allow pumping purge forces to be greater than injection forces. This provides some comfort of being able to back-flush the well with the existing equipment and not having to rehabilitate the well with a higher capacity development pump.
- The system is fully monitored and controlled by SCADA. The system operates by a "scheduler program" placing the most efficient unit or well in a specific location, brought online first and the least needed unit is brought online last.
- All wells are equipped with recovery pumps. The pumps are operated twice a month for preventive maintenance, vibration, and performance checks to ensure operation when needed. This purge cycle helps back-flush the wells and maintain efficiency.

- A heavy reliance on instrumentation and automated operation must be supported by real-time data collection of field parameters. Quarterly electronic checks performed by the agency do not catch mechanical drift or defects. Water level transducers must be accompanied by a physical level measurement, as well as pressure and flow.
- Common operation parameters such as the time required for pumping to waste at the beginning of recovery ("waste time") change with operation of ASR wells. Waste discharge quality and duration must be frequently checked (sand and turbidity) to ensure proper flush times to remove the harmful debris. Premature shut-down during wasting and transition to recharge could cause well or pump damage due to sand-locking of the pump bowls. This may result in the need for checking and possibly changing automated waste timers on a routine basis.

INFILTRATION BASIN FACILITIES

Water from a variety of sources is infiltrated into unconsolidated and bedrock aquifers containing water tables within tens of feet to hundreds of feet below the surface. Although soaking water down through soils by gravity sounds simple, the practice is actually a complex balance of design, operation, and maintenance techniques to sustain operation at such facilities. Examples of this recharge technology are noted below.

Stormwater Recharge Basins—Nassau County, New York

Approximately 1000 stormwater recharge basins have been constructed in Nassau County, Long Island, New York since the 1930s. Initially these were constructed to resolve roadway flooding problems, however as urbanization occurred, their principal role changed to aquifer recharge. Current water table levels are approximately 5 ft higher than pre-development estimated water table levels. Recharge basins are typically open, unlined pits ranging in size from 0.1 to 30 acres. Most are about 1.5 acres with depths ranging from 10 to 40 ft.

Objective, Source, and Sustainability

- The primary objective is aquifer recharge. An important secondary objective is to retain stormwater runoff, thereby reducing flood peaks in downstream areas
- Recharge basins in coastal areas where surface elevations are low and water table levels are high, may contribute to flooding under heavy rainfall conditions such as hurricanes.
- The water source is stormwater runoff, conveyed to the basins through storm sewers.
- The system is clearly sustainable, having had a substantial beneficial impact upon water resources management on Long Island for several decades.

Feasibility, Design, and Expansion

- A standard approach has been developed, including area reconnaissance, site selection, field testing to evaluate soil properties. This is followed by development of design criteria for the basin and then by construction.
- Minimum depth of 10 ft between bottom of the basin and the water table.

174

- The required storage volume is calculated from an empirical equation that has been developed during the past 50 years, assuming no outlet from the basin from 5 inches of rainfall occurring once every 10 years, and the tributary area of residential, business and shopping center land uses.
- Basin sidewall slopes range from 2:1 to 1:2 and are stabilized to prevent erosion and sloughing, usually with a vegetated cover.
- Inlets are designed to prevent scouring, using rip-rap or baffles.
- Many recharge basins are multi-level, with an inlet area for collection of debris and fine-grained materials and an auxiliary infiltration area to allow for increased infiltration.
- In some cases a 10 ft diameter precast concrete cylinder, filled with coarse sand and gravel, is installed to penetrate through shallow restricting layers.

Operation and Maintenance

- Median vertical hydraulic conductivity was 1.63 ft/ hour at 72°F. Median infiltration rate was 1.83 ft/hour, within a range of 0.13 to 5.63 ft/hour.
- At two recharge basins studied, mounding of 0.5 ft occurred in response to 1 inch of rainfall, and 2 ft in response to 2 inches of rainfall. Mound dissipation occurred in less than 14 days.
- Clogging is due to four main factors: permeability of the basin materials; land use within the basin's drainage area; age of the recharge basin, and intersection of the basin floor with the water table. A clogged basin is defined as one that retains water for more than five days after one inch of rain.
- Regarding the permeability of basin materials, surficial geology had the largest effect upon infiltration rates and the percentage of clogging basins. Areas with perched water had the second largest effect, followed by the grain size distribution of the soils.
- Commercial and industrial contributory areas contributed to higher clogging rates due to the inflow of asphalt, grease, oil, tar and rubber particles from parking areas.
- Over 50% of the basins constructed prior to 1950 have clogged, compared to 17% for those constructed since that time, suggesting that efficiency has decreased with age, possibly due to the accumulation of fine-grained sediment and microbial activity.

Central Avra Valley Storage and Recovery Project—Tucson, Arizona

The city of Tucson in Arizona has been employing managed aquifer recharge since 1992 through wells and infiltration basins using treated potable water (wells), surface river water, and treated secondary effluent. The recharge facility in the Central Avra Valley was placed into operation in 2001 and represents a stand-alone, state of the art application for infiltration basin and recovery facilities. The complex consists of 11 recharge basins covering 330 acres, 27 recovery wells, a 54-MGD booster station, a 10 million-gallon reservoir, and approximately 25 miles of pipelines. The depth to static water level in the aquifer is about 350 feet bls. Infiltration rates for the basins range from less than one to several feet per day with yearly recharge and recovery volumes reaching 60,000 AF. The key lessons are as noted.

175

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  - Provide soil aquifer treatment of surface water imports.
  - Put surface water imports to beneficial use through indirect methods.
  - Restore groundwater levels in the aquifer.
- The source water is raw Colorado River water delivered through hundreds of miles of open cannels.
- The sustainability of the source is excellent with only 50% of the supply currently being utilized.

Feasibility, Design, and Expansion

- Three years of pilot operations were conducted in three 20 acre basins, recharging 47,000 acre-feet of surface water at an average rate of 1,100 acre-feet per month.
- Each basin is a shallow rectangular box with shallow angle sides, concrete water entry aprons, and staff gauges. Dimensions are 660 feet wide by 1200 feet or greater in length. Access ramps are provided for heavy equipment for scraping and rehabilitation.
- A pump station at the canal delivers water to the basins at 13,000 gallons per minute (gpm) through a 36-inch line.
- Hydraulic responses at the project site are monitored in 17 ground water wells and 32 vadose zone viscometers. The piezometers are completed on top of clay or fine-grained layers that were potential perching zones for recharged water. Each basin has a similar array of monitoring points and the same general patterns of response have been noted for each site.
- Pilot recharge operations and public outreach programs are contributing to the success of the recharge facility. A primary goal was to deliver water that has an acceptable quality to customers which means it should taste good and should have a corrosion rate that is equal to or less than occurs with ambient groundwater.
- The experience gained from the operation of the pilot program has been integrated into the design of the full-scale facility.

Operation and Maintenance

- At the beginning of recharge activities, the depth to ground water was about 370 feet bls. The current depth to water under the central basin is approximately 300 feet bls.
- The regular drying of the recharge basin allows cracking of the fine surface sediments and opens up potentially cemented surfaces. Normal operational wet cycle lengths have varied between 5 and 15 days, with interceding dry cycles of 3 to 14 days.
- At an intermediate depth of 100–140 feet bls, the perched water resulting from successive wet cycles tended to coalesce in a more sustained presence of perched water. The greatest lateral extent of perched water on the finer-grained layer was observed to extend 1,300 feet from the basin.

- Over time, the vadose zone's ability to transmit recharged water downward appears to improve; perched zones at the intermediate and deeper depths become less pervasive even as recharge activities continue.
- Iron release studies showed that pH was the single most important factor in controlling corrosion in galvanized pipelines. Recharge water was treated to a pH of 8.5, disinfected with chloramines and had polyphosphate added at a concentration of 3–5 mg/L. This yielded water recovered from the aquifer with an acceptably low median iron concentration of 0.7 mg/L.
- The average infiltration rate at the pilot recharge project is 1.2 feet per day. At a hydraulic loading rate of about 220 feet per year, recharge water is transmitted to the water table at 370 feet bls resulting in a mound that is about 56 feet high.
- As the recharge water front advances through the vadose zone, dissolved salts in the soil moisture are flushed to the water table. The salts are diluted as the recharge front reaches the water table. Groundwater quality showed a sharp increase in TDS concentrations as salts from the vadose zone were flushed to the water table. As recharge water reached the water table, the sulfate concentrations increased to 240 mg/L.

Canal-111 Infiltration Basin Project—Dade County, Florida

The C-111 Project is located in Dade County, Florida, and has been recharging untreated surface water into a limestone bedrock aquifer for 5 years. In seeking more ecologically-sensitive ways of managing the surface water diversions and shallow aquifer hydrology of the Everglades National Park (ENP), the U.S. Army Corps of Engineers is infiltrating this water directly into the shallow aquifer to provide managed aquifer recharge and create a ground water mound on the ENP eastern boundary to prevent the loss of shallow groundwater to the urbanized areas. The project currently utilizes 1490 acres of infiltration area with a percolation rate slightly greater than 1 cfs/acre. The agency plans to expand this program to include facilities along most of the length of the C-111 canal to capture storm water events.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  – Provide ecological restoration for the ENP.
  – Create a groundwater barrier to control groundwater loss to urban areas.
  – Increase the groundwater supply for urban development.
  – Provide flood control protection.
  – Reduce the volume of poor water quality discharged at the ocean outlet.
- The source water is untreated surface water from the C-111 canal. The origin is mostly frequent summer precipitation and surface run-off.
- The source quality is variable but generally contains concentrations of total dissolved solids less than 500 mg/L, and low nutrients with less than 20 ppb of total phosphorus.
- The sustainability of the program is very good. Current losses to tide are creating problems in the bay necessitating an expansion of this SUS program.

Feasibility, Design, and Expansion

- The spatial extent of fine-grained mudstone stringers that effectively retard infiltration must be defined in the project area. Continuous coring should be used to define the physical attributes of the impermeable layers.
- The selection of potential basin locations should include consideration of threatened or endangered species impacts on operation, leading to an under-utilized infiltration system.
- Site selection should consider land use of the proposed basin areas to avoid the presence of contaminated soils, delays during construction, and additional costs.
- The hydraulic mounding impact will ultimately reduce easterly seepage away from the ENP, and through long-term operation of the infiltration basins, annual water losses from the ENP can be minimized.
- A continuous series of infiltration basins will provide a buffer between the ENP and more urbanized areas to the east.
- Feasibility studies include many soil borings, monitoring wells, excavated pits and standard double-ring infiltrometer tests to define the aquifer and impermeable layers.
- During the small-scale infiltration tests, the water table elevation did not rise to the bottom of the test apparatus, thus allowing more vertical infiltration and less lateral infiltration.
- Percolation tests must be of a size to be representative. Standard small-diameter tests will tend to give skewed results because of the heterogeneous nature of the limestone material. For the variable bedrock, test cells of 10 feet by 10 feet were used.
- Basins are constructed with 6 foot levees, 4 foot of operating capacity and a concrete spillway.
- System testing at full-scale is recommended as a best practice for infiltration basin operational design. Full-scale testing allows the designer or owner to develop a final operating manual for the project.

Operation and Maintenance

- Canal water is pumped to the infiltrations basins for percolation into the subsurface soils and bedrock. Pipes and canals provide a gravity-fed connection between basins.
- Three basins are designed for infiltration only, the fourth is an 800+ acre channel used for joint infiltration and conveyance of surface overflow into the ENP.
- In 2004, over 142,000 AFY was recharged through the 4 basins, indicating a highly successful project capable of recharging vast quantities of storm water into the aquifer.
- The project does not employ pre-filtration or pre-settling basins prior to delivery to the infiltration basin. Basin clogging has been minimal with no cited cleaning routines.
- The depth to the water table for any infiltration basin is very important. Even though the formation was permeable, infiltration capacity was constrained by the high water table, typically within a few feet of the land surface.

178

Fort Dix Land Application Site Infiltration Basin Project—Fort Dix, New Jersey

The project is located on the Fort Dix Army Base and Maguire Air Force Base in Fort Dix, New Jersey. The facility has been percolating treated effluent into the shallow aquifer system for 22 years. The program utilizes 12 four acre basins to recharge between 3 to 12 mgd. The agency does not recover the water.

Objective, Source, and Sustainability

- Groundwater recharge is being used for disposal of treated wastewater.
- The source water is tertiary treated chlorinated wastewater.
- The source quality meets all New Jersey Department of Environmental Protection Discharge to Groundwater requirements.
- The sustainability of the program appears satisfactory, provided the source supply (military base) is sustained and the water table does not mound in the aquifer, decreasing infiltration ability.

Feasibility, Design, and Expansion

- Basins are constructed in pairs with individual sluice gates per basin to allow separate operation.
- The distribution piping allows selection of any combination of basins for recharge.
- "Basin-field" development required each basin to be at a lower elevation resulting in the last basin being too close to the water table to be an effective infiltration basin.
- The basin bank walls should be stabilized to prevent sloughing. The use of low angle bank walls allows access for maintenance and grass mowers.
- The entry and exit flumes for the basins should be thoroughly stabilized with large rip-rap boulders and concrete aprons.

Operation and Maintenance

- The basins are operated with 2–4 days wetting and 4–7 days drying to maintain the best overall infiltration performance.
- Basin water levels are maintained at less than 24 inches for best performance.
- Native material used for bank walls erodes and sloughs into the basin floor. This increases bank maintenance and basin cleaning needs.
- Bank stabilization with mixed grasses did not fully stabilize the sandy bank material. Plastic slope stabilizer mesh became tangled in mowing machines.
- Dislodged rocks from the basin entry flume can damage scarifying equipment and delay basin rehabilitation efforts.
- Minor seepage through the embankment to adjacent basins occurs due to the lack of clay cores.
- Algae and vegetation growth presented visual issues in one basin, but had little effect on basin performance.
- The tertiary treated wastewater recharge program has not negatively impacted the groundwater quality.

Edwards Aquifer Authority Recharge Program—San Antonio, Texas

Since 1974, the Edwards Aquifer Authority has been recharging the Edwards limestone aquifer in support of delivering potable water supplies to San Antonio, Texas. The program utilizes concrete dams on four intermittent creeks to impound storm runoff for infiltration. The basins are strategically placed to infiltrate the water into a narrow band where the Edwards Aquifer formation is exposed at the land surface, with unconfined conditions. Over the operational period, over 150,000 AF has been recharged with high years averaging 14,000 to 21,000 AF and low years at 200 to 900 AF. The median is 4,900 AF per year benefit to the aquifer above natural recharge. The key lessons are as noted.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  - Recharge the aquifer to allow additional pumping during peak periods.
  - Increase the capture of surface storm flows and store this water in a reservoir below ground to best use the storage availability.
  - Preserve aquifer levels above critical set points to avoid groundwater pumping restrictions during low flow periods.
  - Provide increased aquifer pressure and sustainable flows to critical springs and habitats.
- The source water is precipitation runoff in intermittent Seco, Parker, Verde, and San Geronimo Creeks.
- The sustainability of the source is sporadic depending on the frequency of rain events. Over 30 years of operation, the program has captured over 150,000 AF. Numerous other creek beds cross the un-confined section of the aquifer allowing considerable potential to increase recharge.

Feasibility, Design, and Expansion

- The recharge program is in response to a state water board requirement to develop water programs for the communities.
- Natural recharge zones or un-confined portions of the primary aquifer were identified as potential managed aquifer recharge sites.
- Infiltration basins are located on top of the primary aquifer un-confined outcrops, providing recharge directly to the aquifer portion.
- Extensive modeling of potential additional infiltration basins provided information on potential losses to major springs from over-filling portions of the aquifer.
- Additional infiltration sites have been modeled to provide the best retainable and recoverable potentials.
- Expansion programs will include installing retention basins upstream of the recharge zone for additional capture and slow release into the recharge zone.
- Future water exchanges and importations will be considered for recharge into the Edwards Aquifer through strategically placed infiltration basins.

180

Operation and Maintenance

- Passive infiltration basins on intermittent creeks provide a good, low maintenance recharge option. Natural drying periods and intense storm events rehabilitate the basins to allowed sustained infiltration.
- Basin levels are measured with staff gauges and recorded on strip charts.

FAILED SUS SITES

Urrbrae Wetland ASR Project—Adelaide, South Australia

In 1999 an ASR trial was initiated at the Urrbrae Wetland site in metropolitan Adelaide, South Australia to test the viability of injecting wetland-treated urban storm water into an uncon-solidated siliceous aquifer so that the recovered water could be used for landscape irrigation of adjacent school grounds. The trial failed due to irreversible clogging of the ASR well. The ASR well was completed to nearly 300 feet with 8-inch PVC blank casing and 6-inch stainless steel wire wrap screen. After 6 weeks of recharge operation starting at 35 gpm, the well clogging grad-ually reduced the operation to nothing. Repeated back flushing events and development tech-niques could not revive the well efficiency. The key lessons are as noted.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  - Capture urban storm runoff and store for beneficial use, reducing the flooding hazard.
  - Use banked water for school turf irrigation and help sustain critical pool levels in the wetland during dry months.
  - Use the recharge pre-filtration ponds to develop an educational wetland habitat.
- The source water is urban storm run-off containing abundant leaf matter, organics, nutrients, some suspended solids and on occasion, motor oil. The large contribution of organic matter results in elevated TOC concentrations causing periodic oxygen depletion within the wetland. Inorganic fines and colloidal matter are generally a second-order phenomenon, except during periods of building construction within the catchment. The water is pre-filtered through two lined settling ponds prior to being pumped through canister type sand filters to a storage tank. The water is then gravity fed through a 1-inch PVC tube extending to below the water table.
- The sustainability of the source is sporadic depending on excess storm flow events.

Feasibility, Design, and Expansion

- The wetland was built in 1996 to mitigate local flooding and was engineered to handle peak storm water flows associated with a 1-in-5-year storm event. The major features of the wetland include the main lagoon and rubber-lined holding pond, providing primary and secondary settling prior to conveyance to the recharge well.
- The wetland water is significantly fresher than the marginally brackish ambient groundwater (by a factor of six in terms of the chloride concentration).

- The source water contains sufficient particulate matter to reduce pore-space of the media close to the well screens and sufficient organic matter and other key nutrients to promote biofilm production.
- From a well clogging perspective, limestone aquifers are the more tolerant of poorer source water quality due to the offsetting effect of matrix dissolution. Unconsolidated fine-grained aquifers, as used in this project, present challenges to maintaining adequate rates of injection in ASR wells and to designing around the clogging issues.
- Well drilling was with mud rotary methods. Development included extensive air-lifting and backwashing to dislodge residual drilling muds and develop a natural gravel pack.

Operation and Maintenance

- Recharge water was pumped through 1 mm sand filters prior to injection. These filters did not remove the suspended material, colloidal material and elevated TOC. Many of the particulates fractionated by abrasion with the sand, enabling the smaller components to pass. The sand filters back-washed automatically every 2 hours for 5 minutes.
- Storm water sources produce significant influx of leaf and other organic debris throughout the year. The large contribution of organic matter results in elevated TOC concentrations causing periodic oxygen depletion within the wetland. Inorganic fines and colloidal matter are generally a second-order phenomenon, except during periods of building construction within the catchment.
- There are inherent temporal variations in the quality of water in the wetland due to storm-water runoff and algal growth in the shallow, nutrient-rich water.
- Periodic backwashing of the well upon subsequent installation of a recovery system failed to stop the decline in injection rates.
- After modifying the headworks by installing an injection line, air entrainment in the injected water was eliminated.
- Rapid clogging occurred despite pre-treatment of the injectant by rapid sand filtration. During this period the particulate matter, which was abundant in storm water, was subsequently found not to be substantially reduced by the rapid sand filter.
- The potential causes of clogging included: suspended solids or hydrocarbons entering the well; biofilm production on the well screens and surrounding natural gravel pack; and remobilization of drilling muds or fines from the aquifer. Chemical precipitation and gas binding by entrained or evolved gases from the injectant were eliminated.
- The first approach involved intermittent backwashing over a 5-day period. Recurrent pumping and the demonstrated removal of at least some of the clogging agents from around the well failed to produce a measurable improvement in well efficiency.
- A slug of chlorine solution was introduced into the well to oxidize the organics prior to further backwashing. Injected into the well over a 2 day period, the chlorinated slug remained within the gravel pack of the aquifer for a further 6 days before it was pumped out over a 4 day period. Once again, pump testing revealed little or no improvement in the specific capacity of the well.

182

- A downhole video camera survey revealed heterogeneous discolorations on the stainless steel well-screens, symptomatic of persistent fouling of the screens, presumably the result of excessive microbial activity.
- Well rehabilitation encountered excessive sand flowing into the well. This was found to be the result of a damaged rubber seal (the so called "J-latch") set between the narrower 6-inch telescopic screen assembly and the wider 8-inch casing. Evidence derived from flow metering revealed this had significantly contributed to the sand entry into the well as well as over 50% of the flow contribution.
- The flowmeter data revealed that the in-filling of the lowest screened interval had not been the cause of the substantial decline in injection rate.
- Experience has shown that higher levels of pre-treatment than was provided to the recharge water at Urrbrae is required to avoid excessive well clogging problems.
- This case convincingly illustrates the point that water quality criteria cannot be considered in isolation, but must also consider the nature of the receiving formation.
- The attempt to recharge passively-treated urban storm water via a multi-completion ASR well that targeted two confined, unconsolidated siliceous aquifers, resulted in a significant decline in injection rates and the cessation of operations within the first year.
- Three different attempts were made to restore the clogged ASR well. They included: repetitive backwashing; injection of chlorine disinfectant and backwashing; and bailing/surging to recover sand that had in-filled the lowest screened interval. These techniques proved to be ineffective in restoring the performance of the well.

A contributing problem was premature injection of water before a pump was installed to allow redevelopment. This probably resulted in filter cake compression and made subsequent redevelopment much more difficult. Infilling of the well with sand was another major problem.

Northwest Hillsborough County Reclaimed Water ASR Project—Hillsborough County, Florida

The Northwest Hillsborough County ASR project is located near the City of Tampa in southwest Florida, and has recharged tertiary treated reclaimed water into a deep limestone and dolomite aquifer with unfavorable results. The ASR project is administered by Hillsborough County Water Department and the Southwest Florida Water Management District and consists of one ASR well completed into a brackish aquifer with a recharge flow rate between 600 to 1,300 gpm. Only 60% of the available treated waste water is used for irrigation, allowing storage of the remaining water for recovery during high demand periods. Upconing from a lower brackish aquifer during recovery resulted in the invasion of poor water, effectively reducing the recovery efficiency to approximately 25%. Bedrock chemical reactions may have contributed to the increase in metals and arsenic above ambient background in the recovered water.

Objective, Source, and Sustainability

- Groundwater recharge is being considered for the following reasons:
  – Use a higher percentage of available effluent as irrigation supply water during drought periods.

183

- Increase agricultural use commitments with recovered water.
  - Provide a 3-month sustainable supply for over 8,000 irrigation users during seasonal dry periods.
  - Reduce discharges of treated wastewater to Tampa Bay.
- The source quality is good. The average TDS, chloride, and sulfate values were 620 mg/L, 160 mg/L, and 71 mg/L, respectively, with low metal concentrations in the recharge water including 15 mg/L of potassium, 8.8 mg/L of magnesium, 0.8 µg/L of arsenic, and 50 µg/L of iron.
- The sustainability of the program is poor until new technology is employed at the facility. The water is available, but the aquifer conditions are leaky.

Feasibility, Design, and Expansion

- Incomplete characterization of the hydrogeologic system can lead to incorrect assumptions regarding the confinement above and below the ASR storage zone.
- Inadequate lower confinement in the ASR storage zone in brackish aquifers will undoubtedly result in poorer recovery efficiencies.
- The criterion for sizing the facilities was based upon historical demand records for the reclaimed water program.
- Pilot testing and cycle testing with 100% recovery was not very successful and indicated that the potential recovery efficiency would be around 25 to 28%.
- Additional cycles may improve the recovery efficiency, especially if a larger freshwater buffer was left in place, however upconing was evident within a few hours of the beginning of recovery. With inadequate lower confinement and a brackish aquifer, recovery volume with acceptable quality will tend to be similar regardless of the amount of previously stored water.
- Brackish native groundwater upconed during recovery, increasing TDS from 650 mg/L to 3,250 mg/L; chloride from 171 mg/L to 1,616 mg/L; sulfate from 81 mg/L to 283 mg/L; and sodium from 123 mg/L to 869 mg/L. These concentrations are well above those measured in the recharge water or ASR storage zone, suggesting deeper saline sources.
- Recovered water arsenic increased from 6 µg/l to 23 µg/l, and magnesium increased from 9 mg/L to 31 mg/L, well above concentrations measured in the source water or ambient groundwater, indicating some type of water-rock reaction.

Operation and Maintenance

- During recharge, gradual plugging was noted by increases in wellhead pressure.
- Apparent plugging may be due to a combination of hydraulic effects, suspended solids mass loading, declining water density, and possibly bacterial growth.
- Well clogging has been reversed through periodic well recovery or back-flushing.

A monitoring well was not installed below the ASR storage zone. Such a well would probably have shown that the formations were in direct communication.

East Meadow Reclaimed Water Recharge Project—Long Island, New York

The East Meadow Project has been recharging chlorinated, treated wastewater since the 1930s into the shallow groundwater system. The project is administered by the Nassau County Department of Public Works and uses eleven recharge basins and five shallow injection wells. This artificial recharge program returns an estimated average of 60 mgd annually to replenish the over-pumped shallow aquifer system. The agency does not recover the water. The surface recharge facilities are highly successful however the wells have not been successful.

Objective, Source, and Sustainability

- Groundwater recharge is being used for the following reasons.
  - To balance groundwater removal and maintain water levels.
  - To improve stream flows as a result of higher water tables.
  - To capture treated wastewater for beneficial use that would have been otherwise discharged directly to the Atlantic Ocean.
- The source water is treated effluent with varying degrees of turbidity. The water contains low concentrations of inorganics, metals, and volatile organic compounds. In most cases, the water recharged to the aquifer was of equal or betterquality than the native groundwater.
- The sustainability of the program is good. Historic leach fields provided a return to the aquifer. Current sewer-collection piping, treatment plants, and ocean out-falls eliminated this recharge, causing water level declines. The program returns the water back to the original aquifer source to maintain a balance and allow further beneficial use.

Feasibility, Design, and Expansion

- Recharge is evaluated with a variety of methods and tools to determine if tertiary-treated wastewater could be recharged without affecting the physical and chemical characteristics of the aquifer, including:
  - Instrumented manholes were installed in recharge basins to provide detailed monitoring of the physical and chemical condition of the water as it percolates through the unsaturated zone.
  - Four inclined lysimeters were installed through the wall of each manhole below the basin floor to capture water samples.
  - Fourteen horizontal water manometers were installed through the walls of each manhole at varying depths to measure soil-moisture tension in the unsaturated zone.
  - Soil gas samplers were installed at varying depths to measure the oxygen content of the soil gas during recharge.
  - Soil temperature measurements were measured by thermocouples at selected depths.
  - Eleven galvanized steel neutron-access tubes extending 45 feet deep were installed at selected distances in and near the basins to measure soil moisture.
- The agency has evaluated both infiltration basins and wells for recharge.

185

- Three types of wells were constructed; one well was installed with a gravel packed screen, one has a natural pack screen, and one has both a gravel pack screen and a redevelopment system. The redevelopment system includes the installation of an eductor pipe and an air-line through which compressed air was injected to provide air lift pumping during well development. Well construction methods did not have an affect on the rate of well clogging during injection tests.
- The recharge system is supplied from a plant 6.25 miles away. Reclaimed water is pumped to an on-site storage tank for gravity feed to the eleven terminal basins and booster pump station to deliver water into the wells.
- The facility is tightly contained with ten 3,000 to 5000 ft^2 shallow basins, and one deep basin. Each basin is a dead end with no inter-basin flow.
- The facility is automated with an onsite process control building, recharge pumps, a laboratory for water quality testing, and a control room containing instrumentation that activates and monitors flow control systems, water levels, flow rates, water quality sampling, and meteorological conditions.
- Baseline ambient groundwater quality beneath the site was characterized by collecting over 200 groundwater samples from 47 observation wells.
- Forty-seven 6-inch diameter monitoring wells were installed in well clusters at 23 locations: 23 screened between 45 and 55 feet; 16 screened between 95 to 100 feet; and 8 screened between 195 to 200 feet. This allowed monitoring within and below the target aquifer.

Operation and Maintenance

- An inline water analyzer, which allows continuous monitoring of total chlorine residual, turbidity, temperature, specific conductance, dissolved oxygen, and pH was installed and designed to stop the injection process if turbidity in the source water becomes excessive. The mean daily concentration of suspended solids was 2 mg/L for the recharge basins and 1 mg/L for the injection wells.
- The recharge basins were operated two ways: the constant rate mode involved pumping a constant rate of water into each recharge basin and allowing the height of water in each basin to potentially vary; and the constant head mode involved maintaining a constant height of water in each basin with a potentially varied flow rate. The constant head mode was the preferred operation for ease of performance monitoring and clogging detection.
- Various recharge basin maintenance techniques significantly increased the infiltration rates, including scarification, stripping, alternating wet and dry periods, and tilling. Allowing natural vegetation to grow on the basin floors was believed to increase the infiltration rates by creating shallow root channels.
- Observations during the recharge basin tests show that the most significant factor limiting the rate of infiltration through the basin floor was the quality of the reclaimed water. Infiltration rates were reduced by episodic periods of high turbidity water, ultimately reducing infiltration rates from 2.6 to 0.5 feet per hour.
- After recharging for long periods greater than 1.5 months, ponding increased as a result of clogging mainly from biological growth. High infiltration rates could be maintained if recharge durations were shorter and the floors of the basins were scarified.

186

- Neutron geophysical logging during infiltration tests showed the wetting front moved 6 to 12 feet per hour in the unsaturated zone and was essentially vertical.
- Lysimeter water samples below the basin during recharge indicated that the aquifer matrix did not affect the water quality for metals and low-molecular weight hydrocarbons, however a reduction in phosphorous was noted until the adsorption capacity of the soil was exceeded. An increase in iron and manganese was noted in some locations.
- Well clogging was problematic during the injection tests and was primarily due to physical and chemical clogging associated with turbidity and precipitation of iron. Backflushing of the wells during the injection cycles and pH adjustment to control the dissolution of iron was not conducted during the injection tests.
- Recharge basins were a better way to recharge the aquifer with tertiary-treated waste water because the basin floors were more accessible and easier to rehabilitate than the inaccessible well screens.
- Injected tertiary-treated wastewater did not negatively impact the native water quality. In fact, it improved the native groundwater from the degraded native groundwater that was impacted by the cesspools, septic tanks, and industrial point source contamination.

Artificial Recharge of Treated Wastewater—Bay Park, New York

Located at the Bay Park Wastewater Treatment Plant, Nassau County, New York, this aquifer recharge demonstration project was conducted by the U.S. Geological Survey between 1968 to 1973. Results are of considerable value to the purpose of this AwwaRF project, even though operation of the recharge program was not continued after completion of the demonstration program. Nineteen injection tests were conducted, using treated drinking water and also high quality reclaimed wastewater. Additional treatment that was conducted for various tests included degasification, pH adjustment and dechlorination. Several well redevelopment and rehabilitation techniques were tried, including pumping, surge blocks, swabbing, chemical treatments using hydrochloric acid and bactericides, and also just leaving the well idle so that natural biodegradation of organic materials could occur. Of particular significance, the demonstration program included measurements of water level and water quality at 18 observation wells ranging from 20 to 200 feet from the injection well, plus one monitoring well within the gravel pack that was used for measuring water levels and also for adding a sand-filled probe for evaluation of microbial activity in the gravel pack. Lateral hydraulic conductivity of the injection zone (418 to 480 ft) averages 126 ft/day.

Objective, Source, and Sustainability

- Groundwater recharge was demonstrated in order to recharge the Magothy aquifer, which had been depleted due to diversion and treatment of stormwater and wastewater flows, reducing historic aquifer contamination but also reducing natural aquifer recharge and thereby causing salt water intrusion
- The water source was initially treated drinking water, to establish baseline hydraulic response of the injection well and the aquifer. It was then changed to reclaimed waste water, treated with an activated sludge process followed by dual media filtration,

187

activated carbon filtration and chlorination. For different test cycles, additional treatment processes included degasification, pH adjustment and dechlorination.

- Sustainability was uncertain. Specific injectivity declined during each cycle and was partially restored through a variety of redevelopment approaches.

Feasibility, Design, and Expansion

- Materials of construction included an 18-inch fiberglass well casing with a 16-inch stainless steel well screen, thereby controlling downhole corrosion and associated clogging.
- A vacuum-operated degasifier was utilized on two of the test cycles, to reduce the concentration of entrained air or dissolved gases. This had no discernible effect upon well clogging, probably reflecting the introduction of water to the well through a dedicated injection tube extending to below the static water level in the well, and also because the recharge water temperature was about equal to or greater than the ambient groundwater temperature.
- Recharge at the land surface into the well casing caused disruptive vibrations in the downhole piping, and was discontinued after one test. No recharge was attempted down the pump column.

Operation and Maintenance

- Clogging was due primarily to suspended solids in the recharge water, particularly where they exceeded 1 mg/L. Suspended solids tended to occur in high turbidity events rather than uniformly during recharge. A relationship was developed between head buildup, in feet per million gallons of recharge water, and the suspended solids loading in pounds per million gallons.
- Most clogging occurred at the interface between the gravel pack and the aquifer. However for one test where the reclaimed water was not chlorinated, clogging was evident in the gravel pack.
- Microbial clogging was not significant so long as the residual chlorine concentration during recharge was approximately 2 mg/L or higher.
- Some chemical precipitation, including iron, aluminum and phosphorous compounds, may have accumulated in the vicinity of the well, contributing to clogging in most of the tests.
- Flowmeter logs suggested that roughly 60% of the yield during pumping was contributed from the lower half of the screened interval, however during recharge 60% of the flow entered the top half of the well screen. In general, flowmeter logs were not very useful in determining whether any zones clogged preferentially.
- Redevelopment of the well by pumping and surging was effective in partly restoring the specific capacity lost during an injection test. This was enhanced if the well was rested for several weeks after injection, reflecting biodegradation. Acidization was effective in further restoring lost capacity. Minor improvement occurred with the addition of sodium hypochlorite to the well. Addition of a bactericide to the well was not effective.

CHAPTER 4
SUSTAINABLE UNDERGROUND STORAGE: A STRATEGY FOR IMPROVING RELIABILITY AND REDUCING VULNERABILITY OF WATER SYSTEMS

In recent years considerable attention has been focused upon improving the reliability and sustainability of water utility systems. In addition to the natural variability in drought-flood cycles, many of these water utilities are faced with increased demands for water; limited supplies; greater constraints upon their operations due to environmental and other regulatory restrictions; tightening water quality standards; contamination of water sources; growing concerns regarding the potential effect of global warming upon drought-flood cycles; potential loss of water supplies due to natural emergencies, hostilities and terrorism, and many other significant challenges. The ability to safely store large volumes of drinking water deep underground has emerged as a cost-effective and viable strategy for addressing these concerns in the United States and many other parts of the world. If the operation of major facilities is disrupted but drinking water can be quickly recovered from storage, disinfected and provided to meet distribution system demands, water system reliability is greatly enhanced. Such successful experiences are becoming more common as a growing number of water utilities integrate Sustainable Underground Storage (SUS) into their normal operations, reducing risks of operational disruption and providing a few days to several months of drinking water storage in reserve.

In this report the linkage between SUS systems and the goal of improving the reliability and reducing the vulnerability of water utility systems to a variety of natural and man-made disruptive events is addressed. The town of Gilbert, Arizona was selected as a representative water system where SUS facilities are operational, a risk analysis has already been completed and risk values are known. In Gilbert the water stored is treated waste water that is used for water recharge, however for the purposes of demonstration it is assumed that the water is potable so that SUS can be evaluated as an operational strategy for reducing system vulnerability and risk. With this information the viability of SUS as a risk reduction strategy for water systems can be determined.

INTRODUCTION

The analysis reported here concluded that an increase in reliability and a corresponding reduction in vulnerability is established (and measured) by employing SUS facilities. This conclusion is based on the results of the risk/vulnerability analysis model (employed by various municipal water systems in the U.S.) used in this report. Furthermore, it is recommended that SUS be considered in any comprehensive security plan for water systems as part of their vulnerability reduction strategy and be given appropriate consideration when implementing physical and policy related security practices.

Table 4.1 presents the final vulnerability measures as aggregated risk sums (defined in the report) over three phases of analysis, where: Phase I analyses assume no security practices are currently in place; Phase II analyses are based on current security practices; Phase III analyses are based on implementing a list of recommended security countermeasures.

It is seen in the table that by employing the two Gilbert water system SUS facilities as a vulnerability reduction strategy, the average weighted risk sums are reduced by around 8% for

189

Table 4.1
Weighted average risk sums (with and without SUS risk reduction strategy)

	Phase I	Phase II	Phase III
Weighted risk sums, no SUS	1.640	0.773	0.196
SUS considered	N/A	0.644	0.133
% Reduction without SUS	N/A	52.8%	74.6%
% Reduction with SUS	N/A	60.7%	79.3%

N/A = not applicable.

Phase II over the non SUS incorporated water system, and once the recommended security measures (described in a subsequent section "Recommended Security Practices for SUS Facilities") are implemented, an additional 5% reduction for Phase III is noted.

It is important to note that the percent reduction in the average weighted risk sums obtained by considering SUS facilities as a method for reducing risk are not absolute measures. In other words, a 5% reduction in the average weighted sum does not mean that risk/vulnerability has been reduced by 5%. These are relative measures and should be treated in that fashion. What it does mean is that some security practices may also show the same level of reduction. As a result, if one is considering a security plan for a water system, one can compare a variety of physical and policy based security practices using the model discussed in this report, to SUS facilities as an adjunct to those practices. In so doing, additional vulnerability reduction could be obtained, allowing for a greater sense of overall security for the entire system.

OVERVIEW OF RISK ASSESSMENT PROCESS FOR WATER AND WASTEWATER OPERATIONS

A comprehensive vulnerability analysis was conducted for the Town of Gilbert's Water and Wastewater System. That effort consisted of the following:

1. Conducting a Risk Analysis that addressed the following items for all the water and wastewater system facilities:
   – Determination of the program objectives
   – Identification of the system's most critical assets
   – Determination of potential malevolent acts (undesirable events)
   – Assessment of the likelihood of undesirable events
   – Systematic site characterization
   – Assessment of consequences associated with undesirable events
   – Definition of the risks
   – Calculation of the risk/vulnerability for each facility
   – Identification of system vulnerabilities and plans to reduce the risk

   The risks associated with each facility were evaluated over three Phases:
   – Phase I – assumes no security countermeasures are in place (base-line)
   – Phase II – risks based on current security practices
   – Phase III – risks based on recommended security countermeasures

190

2. Conducting site visits (security audit) that examined each site for an intruder's ability to perform a malevolent act where the ability to perform such an act is based on the following:
   – Access control
   – Intrusion detection and delay
   – Response and mitigation policies and procedures
3. Making preliminary security strategy recommendations for purposes of generating values for the vulnerability analysis. This entailed evaluating the vulnerability based on current security practices and procedures and evaluating the change in vulnerability based on implementing the suggested countermeasures.
4. Preparing a basic Emergency Response Plan (ERP) that reflects the results of the risk analysis and the recommended countermeasure strategies. The ERP also reflects city-wide plans related to emergency response and includes basic information needed in the event of either a declared or undeclared state of emergency.

Risk Assessment Methodology

The primary purpose for conducting a Risk Analysis Workshop is to establish the priorities and importance for implementing water and wastewater facility security practices and strategies. In addition, workshops aid in understanding the complex relationships between various raw water, wastewater, treatment, pumping, storage, distribution, discharge and other functions by facility components and operations relative to security practices. Workshops also provide the necessary data to measure the relative vulnerability of each facility to a disruptive event (malevolent act).

The workshop model used for the Town of Gilbert's Water System was based, in part, on the Sandia National Laboratories methodology as presented to Carollo Engineers. The Sandia model was adapted to take into account the specific needs of the Gilbert Water and Wastewater Systems, and was further refined by the experience of the author of this report and his colleagues in conducting workshops of this nature.

Issues of criticality, which is the ability to perform all the critical operations fully and safely, and system vulnerability (risk of suffering disruptive events) were the main considerations throughout the process.

Criteria Selection

The term "facility" used in this context will also mean a facility category that establishes similarities of operational performance. For instance, a group of well sites with similar operations on site will be categorized as a single unique facility for the purposes of the workshop. The first activity consisted of selecting the criteria by which each facility, or facility category, was ranked in order of importance to every other facility. For the water system workshop the criteria selected were:

- The facility's capacity to treat, store or pump water.
- The size of the area, or physical extent of the region served by the facility.
- The amount of critical services (i.e., hospitals, fire dept., utilities, etc.) that are served.
- The quality of the water going through or stored at the facility.

191

Ranking Facilities

Based on the criteria selected above, each water facility was ranked in a pair-wise fashion in order to establish each facility's importance as compared to the other facilities. This, in turn, partly established the priority in which a facility is addressed for implementing physical and operational security programs.

The computer model automatically rank ordered each facility based on the input of the participants' comparisons and their knowledge of all the facilities. These rankings were used in subsequent analyses.

For the wastewater system, all criteria were treated equally for the ranking process, and each wastewater facility or facility category was ranked relative to all of the other facilities. These rankings were also used in subsequent analyses.

Threat Assessment—Incident Analysis

After completion of the facility ranking process, the water system participants were asked to examine the threats associated with such acts as assaults, sabotage, terrorism, vandalism, etc., that could occur at each site. This aided the participants in understanding what threats each site is vulnerable to, and the likelihood that such an incident would occur there. The model summarized this information along with the facilities rankings, and the participants then determined the relative probability of how these incidents impact the following critical system operations:

- The ability to collect raw water prior to treatment
- The ability to treat the collected water
- The ability to store water prior to or after treatment
- The ability to distribute treated water
- The public's reaction to the incident (ability to sustain the public's confidence)

The participants were able to modify, add or remove any of the above critical operations from the analysis.

For the wastewater facilities, after completion of the facility ranking process, the participants were asked to consider all possible threats that could occur at each site. This information along with the facilities rankings allowed the participants to determine the relative probability of how such incidents impact on the following critical system operations:

- The ability to collect wastewater prior to treatment.
- The ability to treat the collected wastewater.
- The ability to store (or hold) wastewater prior to or after treatment.
- The ability to discharge the treated wastewater.
- The public's reaction to the incident (ability to sustain public confidence.)

Fault Tree Analysis

Upon completion of the incident threat analysis, a critical operations fault tree template was presented for the Gilbert Water Treatment Plant (WTP) in the water system workshop, and for

192

the Gilbert Wastewater Treatment Plant (WWTP) in the wastewater workshop. The Fault Tree analysis helped clarify which facility components are involved and at what level the destruction or impairment of these components can prevent or impede the operations. The fault tree analysis was also helpful in understanding a facility's overall operational characteristics, and how modest events can have huge impacts on overall system performance. The fault tree was used primarily in developing security countermeasure programs for the WTP and the WWTP.

Criticality Analysis

With the fault tree presentation completed, a criticality analysis based on the water system's ability to continuously operate safely was conducted. Participants were asked to decide a relative criticality rating based, in part, on previous analyses that weighed the importance of the following:

- The amount of time service would be disrupted or out.
- The income loss to the department during an outage.
- The number of customers affected by the outage.

This part of the analysis aided subsequent water system analyses that establish the consequences of a disruptive event, and ultimately the risk associated with such events at each facility. For the wastewater system this part of the analysis was subsumed in the consequence analysis.

Consequence Analysis

The model presented the data collected from the facility ranking, incident threat and criticality analyses for each water system facility. Using the data, participants were asked to decide the level of consequences (high, medium or low) that the community would experience if a disruptive event were to occur at the facility in terms of:

- Economic loss to the service area.
- Monetary loss to the water department.
- Public health risks (illness or death.)

For the wastewater system workshop participants determined the level of consequences based on their facility rankings and incident threat analyses, also in terms of the above factors. Again, the participants determined if these factors were appropriate for the system and could change them if necessary.

Risk Analysis

The model summarized the previous incident and consequence analyses with low, medium and high levels of likelihood replaced by numerical values between 0 and 1 as measures of relative probability. In the initial, or Phase I portion of the workshop, it was assumed that there are no security practices in place at any facility in order to establish base line data. Numerical risk values for this analysis were calculated by:

193

Table 4.2
Ranking criteria

	Facility Capacity	Area Impacted	Critical Services	Water Quality	Sum
Facility capacity		3	4	2	9
Area impacted	3		5	3	11
Critical services	2	1		1	4
Water quality	4	3	5		12

With respect to the criterion, the importance of one item to the other is:
5 = much greater
4 = greater than
3 = the same as
2 = lower than
1 = much lower

$$R = PA \times (1 - P_E) \times C$$

where *PA* is the relative probability of the undesired event, $(1 - P_E)$ is the relative probability of the event's success (vulnerability) based on security countermeasures, and *C* is the relative measure of the seriousness of the consequences associated with the events. Values for *PA* and *C* were calculated from the earlier incident and consequence analyses, and $(1 - P_E)$ was set equal to 1.0 (no security countermeasures in place, or 100% vulnerable) for the Phase I, or base-line data.

Results of a Water System Risk Analysis Workshop

The graphic tables that follow, summarize a possible example data set of the attendees' input for the analysis. Because of confidentiality agreements the data shown is not the actual data collected and is shown here for example purposes only. Table 4.2 gives the results (sums and ranking based on pair-wise comparisons) of the criteria that were used to rank the relative importance of each facility to all the others. A five point scale is used for both this table and the subsequent ranking tables. The values are then summed over each row to determine the final level of importance for each criteria and the weight for each facility.

It is seen from Table 4.2 that "Water Quality" was ranked as the most important criterion (sum equaled 12), followed by "Area Impacted" then "Facility Capacity," and finally "Critical Services" ranked the least important criteria with a sum of 4. Since the total number of individual facilities would be too large to efficiently conduct the water system workshop, the facilities were grouped into six categories and two unique sites for this example.

The next graphic table, Table 4.3 summarizes the rankings using the criterion "Facility Capacity." In an actual workshop three other tables using the other three criteria would also be completed. Table 4.3 presents a weighted sum (i.e., the row sum multiplied by the criteria sum) for each facility compared over the listed criteria. The example facilities shown in the following tables are for demonstration purposes only, and the actual facilities (referred to as critical assets) are shown in "Vulnerability Analysis—SUS Reduction Strategy." The weighted data, ranked by each criterion, are summarized in Table 4.4

194

Table 4.3
Pair-wise sums*

	CTP	AW	BW	AB	BB	AS	BS	#8B	Sum	Weighted
Water treatment plant		5	5	4	4	3	5	5	31	279
A-Wells	1		5	5	5	2	3	5	26	234
B-Wells	1	1		2	3	2	2	2	13	117
A-Boosters	2	1	4		3	2	2	5	19	171
B-Boosters	2	1	3	3		2	3	5	19	171
A-Storage	3	4	4	4	4		4	5	28	252
B-Storage	1	3	4	4	3	2		5	22	198
Well #8 Booster	1	1	4	1	1	1	1		10	90

*Weighted by criterion, facility capacity.

Table 4.4
Summary of pair rankings

	Facility Capacity	Area Impacted	Critical Services	Water Quality	Sum	Rank
Water treatment plant	279	385	132	360	1156	1
A-Wells	234	286	84	240	844	2
A-Storage	252	187	96	300	835	2
B-Storage	198	176	80	276	730	4
A-Boosters	171	231	64	204	670	5
B-Wells	117	231	88	228	664	6
B-Boosters	171	198	64	192	625	7
Well #8 Booster	90	154	64	216	524	8

It is seen from this graphic, Table 4.4, that the Gilbert WTP was ranked highest, followed by the A Water Production Facilities (well sites), then the A Storage sites, and so on. The Well #8 booster is ranked at the bottom.

The workshop attendees would then evaluate the likelihood of occurrence for the disruptive events (see list at the end of this chapter) in Table 4.5. In this table value of 3 represented a high likelihood of occurrence (event happened more than once), 2 represented a moderate likelihood of occurrence (happened once before), and 1 means a low likelihood of occurrence (has not happened but could). A blank space (a value of zero) means the event cannot happen at that asset.

This process led to the development of the following relative probabilities, P_A, Table 4.6 for the occurrence of disruptive events that were used for input into the final risk equation for calculating the risk values. In this table L means a low relative probability of a disruptive event impeding or preventing the critical operation at that facility. An M means a moderate probability, and an H means a high probability (N/A means that some operations are not applicable for that facility).

Table 4.5
Likelihood of disruptive events

	Incidents, Purposeful				
	Assault	Civil Unrest	Computer Hacking	Cross-connection Contamination	Extortion
Water treatment plant	3	2	2	3	2
A-Wells		1	3	2	
B-Wells			2	2	
A-Boosters	3	2	1	1	2
B-Boosters			2	1	
A-Storage		1	2		2
B-Storage			2		2
Well #8 Booster	3	2	1	1	2

	Incidents, Purposeful			
	Sabotage	Terrorism	Theft	Vandalism
Water treatment plant	2	2	3	3
A-Wells	1	1	1	3
B-Wells	1	1	1	3
A-Boosters	1	2		3
B-Boosters	1	1		3
A-Storage	2	2		2
B-Storage	2	2		3
Well #8 Booster	1	2		2

	Incidents, Non-purposeful				
	Accidents	Human Error, Operations	Natural Disaster	System Aging	System Malfunction
Water treatment plant	2	1	2	1	2
A-Wells	2	1	1	1	
B-Wells	2	1	1	1	
A-Boosters	1	1	1	1	2
B-Boosters	1	1	1	1	2
A-Storage			2	1	
B-Storage			1	1	
Well #8 Booster	1	1	1	1	2

The criticality of each facility was estimated based on outage time, cost of outage and percent of the service area affected. In Table 4.7, 3 represented a high degree of impact, 2 represented a moderate degree of impact, and 1 means a low degree of impact as noted in the guidelines in the table.

Using information from all the previous analyses (ranking, incident and criticality), the following consequence table, Table 4.8 was generated. In this table, an L means a low relative measure of serious economic, financial and health impacts on the community, an M means a moderate level of serious impacts, and an H means a high level of serious impacts (N/A, as before, means that some operations are not applicable for that facility).

Table 4.6
Relative probabilities, P_A

	For Reference		Prevent or Impede, Intake	Prevenet or Impede, Treatment	Prevent or Impede, Storage	Prevent or Impede, Distribution	Nagative Public Reaction
	Event Sum	Facility Rank					
Water treatment plant	30	1	H	H	H	H	H
A-Wells	17	2	N/A	N/A	M	M	M
B-Wells	15	6	N/A	N/A	N/A	M	M
A-Boosters	21	5	N/A	N/A	H	M	M
B-Boosters	14	7	N/A	N/A	N/A	L	L
A-Storage	14	3	N/A	N/A	M	M	M
B-Storage	13	4	N/A	N/A	M	L	L
Well #8 Booster	20	8	N/A	N/A	H	M	M

Table 4.7
Facility criticality rating

	Service Outage	Cost of Outage	Customers Affected	Criticality	
Water treatment plant	2	3	3		
A-Wells	2	2	1	5	
B-Wells	1	2	1	4	
A-Boosters	2	2	3		
B-Boosters	2	1	1	4	
A-Storage	1	1	1	3	High Criticality
B-Storage	1	1	1	3	Medium Criticality
Well #8 booster	2	2	2	6	Low Criticality

Based on Facility Characteristics and Fault Tree information, estimate the outage time criticality for each facility using:		Enter the Cell Value
High	Service outage time is measured in months	3
Medium	Service outage time is measured in weeks	2
Low	Service outage time is measured in days	1

Based on Facility Characteristics and Fault Tree information, estimate the loss of income to the department from outage for each facility using:		Enter the Cell Value
High	Dollar loss is greater than $1,000,000	3
Medium	Dollar loss is greater than $500,000 but less than $1,000,000	2
Low	Dollar loss is less than $500,000	1

Based on Facility Characteristics and Fault Tree information, estimate the number of customers affected for each facility using:		Enter the Cell Value
High	Customers affected is greater than 10%	3
Medium	Customers affected is greater than 1% but less than 10%	2
Low	Customers affected is less than 1%	1

197

Table 4.8
Consequence analysis

	Impede, Lose or Negatively Impact the Below Operations					Facility Rank	Criticality
	Intake	Treatment	Storage	Distribution	Reaction		
Water treatment plant	H	H	H	H	H	1	
A-Wells	N/A	N/A	M	M	M	2	5
B-Wells	N/A	N/A	N/A	M	M	6	4
A-Boosters	N/A	N/A	H	M	M	5	
B-Boosters	N/A	N/A	N/A	L	L	7	4
A-Storage	N/A	N/A	M	M	M	3	3
B-Storage	N/A	N/A	M	L	L	4	3
Well #8 booster	N/A	N/A	H	M	M	8	6

	Consequences to the System				
	Intake	Treatment	Storage	Distribution	Reaction
Water treatment plant	H	H	H	H	H
A-Wells	N/A	N/A	M	L	M
B-Wells	N/A	N/A	N/A	L	M
A-Boosters	N/A	N/A	M	M	M
B-Boosters	N/A	N/A	N/A	L	L
A-Storage	N/A	N/A	M	L	M
B-Storage	N/A	N/A	M	M	H
Well #8 booster	N/A	N/A	M	M	H

Based on the facility's ranking, criticality and impacts on operations, evaluate the consequences to the community, where:

High—Very serious economic, monetary, and illness (or death) losses would occur

Medium—Moderate economic, monetary, or illness losses would occur

Low—Some economic, monetary, or illness losses would occur

Finally, the results of the incident analysis and the consequence analysis were entered into the risk table for calculating the Phase I risk values, resulting in Table 4.9. In this table, the H, M, and L values for the incident and consequence relative probabilities were replaced with numerical values. For a high chance of occurrence, H was replaced with 0.9, for M, a medium chance was replaced with 0.5, and for L, a low chance was replaced with 0.1. For Phase I, $(1 - P_E)$ was set equal to 1.0 to establish base-line risk values for all the critical operations (intake, treatment, storage, distribution or discharge, and negative public reaction) associated with each facility.

The final risk data for this process were summarized in Table 4.10. In the actual workshop, it turned out that the facilities' ranks, as determined by the attendees, corresponded closely with the facilities risk sum values. In other words, the higher ranked (more critical) facilities were also at high risk, or more vulnerable, whereas most of the lower ranked facilities were at lower risk, or less vulnerable.

It is felt that this initial risk analysis was successful in developing the Phase I risk values. In the next section of this report, Phase II results (risk values calculated based on current security practices) and Phase III results (risk values calculated after all recommended security practices are in place) are presented.

Table 4.9
Risk calculations

| | Probability of Undesirable Event | | | | | | Consequences to the System | | | | |
| | Impede, Lose, or Negatively Impact the Below Operations | | | | | | Impede, Lose, or Negatively Impact the Below Operations | | | | |
	Intake	Treatment	Storage	Distribution	Reaction		Intake	Treatment	Storage	Distribution	Reaction
Water treatment plant	0.90	0.90	0.90	0.90	0.90		0.90	0.90	0.90	0.90	0.90
A-Wells	0.00	0.00	0.50	0.50	0.50		0.00	0.00	0.50	0.10	0.50
B-Wells	0.00	0.00	0.00	0.50	0.50		0.00	0.00	0.00	0.10	0.50
A-Boosters	0	0	0.9	0.5	0.5		0	0	0.5	0.5	0.5
B-Boosters	0	0	0	0.1	0.1		0	0	0	0.1	0.1
A-Storage	0.00	0.00	0.50	0.50	0.50		0.00	0.00	0.50	0.10	0.50
B-Storage	0.00	0.00	0.50	0.10	0.10		0.00	0.00	0.50	0.50	0.90
Well #8 booster	0.00	0.00	0.90	0.50	0.50		0.00	0.00	0.50	0.50	0.90

| | Probability of Event Success $(1 - P_E)$ | | | | | | Risk to System | | | | |
| | Impede, Lose, or Negatively Impact the Below Operations | | | | | | Impede, Lose, or Negatively Impact the Below Operations | | | | |
	Intake	Treatment	Storage	Distribution	Reaction		Intake	Treatment	Storage	Distribution	Reaction
Water treatment plant	1.00	1.00	1.00	1.00	1.00		0.8100	0.8100	0.8100	0.8100	0.8100
A-Wells	1.00	1.00	1.00	1.00	1.00		0.0000	0.0000	0.2500	0.0500	0.2500
B-Wells	1.00	1.00	1.00	1.00	1.00		0.0000	0.0000	0.0000	0.0500	0.2500
A-Boosters	1.00	1.00	1.00	1.00	1.00		0.0000	0.0000	0.4500	0.2500	0.2500
B-Boosters	1.00	1.00	1.00	1.00	1.00		0.0000	0.0000	0.0000	0.0100	0.0100
A-Storage	1.00	1.00	1.00	1.00	1.00		0.0000	0.0000	0.2500	0.0500	0.2500
B-Storage	1.00	1.00	1.00	1.00	1.00		0.0000	0.0000	0.2500	0.0500	0.0900
Well #8 booster	1.00	1.00	1.00	1.00	1.00		0.0000	0.0000	0.4500	0.2500	0.4500

Final risk = $P_A \times (1 - P_E) \times C$
Where P_A is the relative probability of an undesirable event; $(1 - P_E)$ is the probability of the undesirable event's success; C is the probability of having high negative consequences.

199

Table 4.10
Summary of risk analysis

	Risk to System					Risk Sums	Facility Rank
	Impede, Lose, or Negatively Impact the Below Operations						
	Intake	Treatment	Storage	Distribution	Reaction		
Water treatment plant	0.8100	0.8100	0.8100	0.8100	0.8100	4.05	1
A-Wells	0.0000	0.0000	0.2500	0.0500	0.2500	0.55	2
A-Storage	0.0000	0.0000	0.2500	0.0500	0.2500	0.55	3
B-Storage	0.0000	0.0000	0.2500	0.0500	0.0900	0.39	4
A-Boosters	0.0000	0.0000	0.4500	0.2500	0.2500	0.95	5
B-Wells	0.0000	0.0000	0.0000	0.0500	0.2500	0.30	6
B-Boosters	0.0000	0.0000	0.0000	0.0100	0.0100	0.02	7
Well #8 booster	0.0000	0.0000	0.4500	0.2500	0.4500	1.15	8

Results of the Wastewater System Risk Analysis Workshop

A similar workshop was conducted for the wastewater system, however, it was not conducted at the same level of detail as the water system workshop since in general wastewater operations are not considered as serious a security threat as raw and finished water facilities.

The wastewater facilities considered were

WWF Facility
A-Lift Station
B-Lift Station
C-Lift Station
Collection
NR—Recharge
Reservoirs
RP—Recharge
Waste Water Treatment Plants

where the A, B and C Lift Stations were defined as:

A-Lift Station—Lift stations with generators
B-Lift Station—High capacity lift stations – no generators on site
C-Lift Station—Low capacity lift stations – no generators on site

It turned out in this analysis that the facilities' ranks, as determined by the attendees, corresponded closely with the facilities risk sum values (as was the case for the water system analysis). In other words, the higher ranked (more critical) facilities were also at high risk, or more vulnerable, whereas most of the lower ranked facilities were at lower risk, or less vulnerable.

200

PHASE II AND III RISK ASSESSMENTS

For the Phase II analysis, it was assumed that the countermeasures currently installed (or currently being installed) and in working order (fences, cameras, alarms, etc.) at each surveyed (security audited) site reduced that probability by some proportion of the total possible risk reduction—how much security is now provided compared to how much is possible. For Phase III, it was assumed that all the current countermeasures were in working order (e.g., fences repaired as opposed to replaced) and that the newly recommended countermeasures were installed and operational. In any case, the final calculated risk values, based on the newly calculated values of $(1 - P_E)$, can never be made equal to zero for the more complex sites because certain risks (e.g., earthquakes, monsoons) are not affected by security countermeasures. Thus, the risk reduction strategy focuses on "relative" risk reduction, and is based on ordered, but subjective, judgments about current and recommended security measures. "Absolute" risk reduction can never be known because one never knows the real probability of an incident. In the case of an incident caused by a disruptive individual, security measures were assumed to increase the risk to the perpetrator, and therefore reduce the probability of his/her success, and thereby reduce the risk of such an incident to the facility. In the case of a terrorist attack, security countermeasures were assumed to reduce the probability that such an event would be successful (e.g., target hardening). In all cases, the strategies were selected to affect prevention, detection, and response. Finally, a conservative approach was taken in assigning the prevention, detection and response values as show in Table 4.11. As before the data in this has been fabricated for security reasons. In this example only wastewater sites are shown. Since these values are all relative values it was more important to consider the change in values over Phases I, II and III.

Calculation of $(1 - P_E)$—Vulnerability

Using the results of the security audit, the data collected for each site, the fault trees (which further supported the premise that it would be almost impossible to protect all the sub components of a major facility) and the security countermeasures recommended, data were entered into Table 4.11 This table is a possible example for the Wastewater Sites and would be used to calculate new values of $(1 - P_E)$, a measure of the sites vulnerability, for the Phase II and III analyses.

In Table 4.11, three criteria were used to calculate $(1 - P_E)$: (1) the effectiveness of the countermeasures that are in place to prevent either physical or electronic access to the site, (2) the effectiveness of the countermeasures to detect or delay an intrusion, and (3) the effectiveness of the countermeasures to respond and recover from an intrusion or disruption. Based on the security audit and the recommended countermeasures for each site, values between 0 (for essentially no countermeasures in place) and 10 (for essentially 100% effective countermeasures) were entered in the appropriate cells. Once the data were entered a new value of $(1 - P_E)$ for each site was calculated using the following normalized model:

$$\left[\frac{(10 - A)}{10}\right] \times \left[\frac{1 - (D + R)}{20}\right] = (1 - P_E)$$

where
A = access prevention
D = intrusion detection/delay
R = response and recovery

201

Table 4.11
Vulnerability calculations

Calculation of relative probability of disruptive event's success $(1 - P_E)$—All wastewater facilities.

Enter a number on a scale of 0 to 10 where 10 = 100% successful practices and 0 = No security practices.

| Site | Category | Current Status | | | | Final Status (new practices) | | | |
		Prevention (access)	Detection	Respond and Recover	Current $1 - P_E$	Prevention (access)	Detection	Respond and Recover	Final $1 - P_E$
WWTP/WRP	WWTP	2	2	1	0.680	3	3	6	0.385
Crossroads	A-Lift station	3	2	4	0.490	4	4	5	0.330
Islands	A-Lift station	2	1	1	0.720	8	6	8	0.060
Gilbert Common	A-Lift station	2	4	4	0.480	8	8	5	0.070
Neely	B-Lift station	3	2	4	0.490	7	7	5	0.120
Candlewood	B-Lift station	4	2	2	0.480	4	6	6	0.240
Western Skys	C-Lift station	1	2	3	0.675	4	4	6	0.300
Spring Meadows	C-Lift station	1	3	2	0.675	4	3	5	0.360
Rancho Verde	C-Lift station	3	2	2	0.560	6	6	6	0.160
Commerce	C-Lift station	2	2	3	0.600	5	3	5	0.300
Santana	C-Lift station	2	3	2	0.600	3	5	5	0.350
Effluent reservoirs	Reservoirs	1	1	0	0.855	3	3	2	0.525
Riparian preserve	RP-Recharge	1	0	1	0.855	3	1	2	0.595
Neely recharge	NR-Recharge	2	2	3	0.600	4	4	4	0.360
Collection	Collection	1	2	1	0.765	3	4	3	0.455

An estimated value for sites in a facility category was made based on the surveys of representative, active sites within those categories. Although there were some large variances for some sites in some categories, it is felt that the estimates for all the categories seemed to reasonably represent the site category and is adequate for this analysis. Once again, if a particular site has unique characteristics that warrant an individual consideration, then that consideration should be made.

Upon completion of the table in the section "Vulnerability Calculations," the $(1 - P_E)$ calculations for Phase II and Phase III were entered in the previously developed risk analysis table (see Table 4.9) to generate the Phase II (current security practices) risk sums, and Phase III risk sums—risk sums after implementing the recommended security countermeasures.

It was felt that the Phase II and III risk assessment/vulnerability analysis conducted on all the sites was a reasonable process for decision assisting as to prioritizing and scheduling security countermeasures.

Final Risk/Vulnerability Values

The initial relative risk values calculated at the workshop took into account a large number of criteria and the values calculated for risk were summed over the individual critical operations' risk values for each facility.

Table 4.12 presents an aggregated summary, in the form of an average risk sum, weighted by each asset's importance sum for all the water sites. The theoretical maximum value could be as high as 5.0, but it is seen in this table that the Phase I value is 1.64. In other words, even without

Table 4.12
Average weighted risk sums

	Phase I	Phase II	Phase III
Weighted risk sums	1.640	0.773	0.796
Percent reduction	N/A	52.8%	74.6%

considering any security practices in place, the final weighted risk sum for the Gilbert water system is 1.64.

Recall that although each relative probability of risk calculated for the critical operations can be 1, final summed values of 5 are possible (i.e., maximum values of 1 for each of the five critical operations). This is not a problem since throughout the process a relative (not absolute, and not necessarily a numerically measured) value of risk was always the goal.

In Table 4.12 the columns headed by Phase I, Phase II and Phase III show the average weighted risk sums (weighted by the importance sums demonstrated in Table 4.4 along with the percent reduction in risk from the previous Phase. The values in this table are aggregated from the actual Gilbert risk data and were not calculated from the fabricated data presented, as an example, earlier. For instance, it is seen that the average weighted Phase II risk sum is around 53% lower than the Phase I average weighted sum, and the Phase III average weighted sum is around 75% lower than the Phase II sum. This demonstrates that by performing all the security recommendations presented the average weighted risk sum can be reduced around 75% over the current security practices in place (remember, a 75% reduction over Phase II is a relative measure of risk or vulnerability reduction not an absolute measure).

In the report presented to the Town of Gilbert additional information was presented that assisted them in making their security plans. Each critical asset (site) was organized into four vulnerability quadrants. Vulnerability quadrants were determined by the site's rank and the facility's risk sum. A site that is highly ranked (top 50%) and is at high, or moderately high, risk (risk sums greater than the mid-point risk sum) were in quadrant 1 and represents the greatest vulnerability to the entire system, whereas sites that are low ranked and not at significant risk were placed in quadrant 4, and represented the least vulnerability to the system. Low ranked facilities at high risk were in quadrant 2 and facilities that were high ranked but at low risk were designated third quadrant facilities.

Based on the Phase II analysis all the facilities in the first vulnerability quadrant were to receive top priority for implementing the suggested security countermeasures, followed by the third, and then the second quadrant facilities (quadrants 2 and 3 were considered of equal priority). The fourth quadrant (lower ranks, low risk sites) were the last sites to be considered. Obviously, any single site, regardless of what vulnerability quadrant it is in, may be selected for receiving higher priority security measures based on other unique site-specific factors. The goal is to make certain that after the installation of all the security countermeasures (Phase III) the average weighted risk sum was as low as possible (currently around 0.2 for the Phase III recommendations).

Additional guidelines for prioritizing individual sites were also presented to the Town based on information obtained from the audit and the workshop. It should be noted that when implementing security countermeasures, water system facilities are in general considered to be more important to a community's well being than wastewater facilities.

VULNERABILITY ANALYSIS—SUS REDUCTION STRATEGY

In this section the linkage between SUS systems and the goal of reducing the vulnerability of water utility systems to disruptive events is established. An operational risk reduction strategy for the Town of Gilbert's water system is suggested that could benefit other water utilities that elect to develop SUS facilities.

Model for SUS Risk Reduction Strategy

In the Appendix the actual Town of Gilbert's finished water treatment and storage facilities along with their storage or pumping capacities are listed. The two recharge facilities (the Neely recharge and the Riparian Preserve) are also listed and are assumed (for the purposes of the model) to be potable SUS facilities.

The model developed is based on the premise that SUS facilities must be potable (or as close to finished water as possible) to be able to reduce the vulnerability of the system. As potable water facilities then they are effectively a redundant storage site adding additional capacity to the system in the event of a disruption. For the Town of Gilbert that would mean an additional 9 mgd pumping capacity (or around 25% of the water treatment plant's capability). One could also treat the sites as raw water facilities (since they have been treated in preparation for recharge) that could be pumped to the WTP for additional treatment and distribution.

Operational Model

In "*Risk Analysis*" the model used for calculating a numerical value for risk at each asset, and for each critical operation was given as:

$$R = P_A \times (1 - P_E) \times C$$

where P_A is the relative probability of the undesired event, $(1 - P_E)$ is the relative probability of the event's success based on security countermeasures, and C is the relative measure of the seriousness of the consequence associated with the events. Values for $(1 - P_E)$ were set equal to 1.0 (totally vulnerable to a disruptive event) for the Phase I, or base-line data, and then calculated using the previous model in the "Calculation of $(1-P_E)$ Vulnerability" section for Phase II and Phase III risk sums.

One way of determining how much reduction could be obtained is to try and estimate values for the normalized model in section "Phase II and III Risk Assessments," but that may prove to be unwieldy for using in a general model that could have national (and international) implications. Rather than adjusting the vulnerability—$(1 - P_E)$—it was decided to establish a general multiplier for each risk sum over each asset that would best represent the reduction for Phase II and Phase III.

Results of Risk Reduction Model

Recall that the values given in Table 4.12 are the aggregated sums of the asset risk values weighted by their importance sums developed in the risk analysis workshop. By modifying the risk sums by an SUS multiplier and then taking the weighted average of those values a new value

204

Table 4.13
SUS multipliers

Critical Assets	Phase II Multiplier	Phase III Multiplier
A-WTP	0.65	0.55
B-WTP	0.7	0.6
Public works facility	1	1
Water treatment plant	0.9	0.8
WPF-System	0.5	0.4
WPF-Plant reservoir	0.5	0.4

Table 4.14
Recalculated average weighted risk sums

	Phase I	Phase II	Phase III
Weighted risk sums—No SUS	1.640	0.773	0.196
SUS considered	N/A	0.644	0.133
Percent reduction without SUS	N/A	52.8%	74.6%
Percent reduction with SUS	N/A	60.7%	79.3%

for the average weighted risk sum is obtained. By working with an expert in water security, Professor James C. Snyder, Co-director, Studies in Urban Security Group, University of Michigan, Taubman College of Architecture and Urban Planning, Table 4.13 was obtained. The critical assets in this table are defined by:

WTP—Water treatment plant including reservoirs and boosters
A-WTP—Sites having active wells and above ground reservoirs
B-WTP—Sites having active wells and below ground reservoirs
WPF-PLANT RES.—Wells pumping into plant reservoir
WPF-SYST.—Direct pumping well sites

Applying the multiplier values from Table 4.13 and re-calculating the average weighted risk sums for each Phase, the data in Table 4.14 were obtained.

It is seen from this table that by employing the two SUS facilities the Phase II average risk sums are reduced by almost 61% and the Phase III risk is reduced by over 79%. These values represent around an 8% reduction for Phase II over the non SUS incorporated water system, and once the recommended security measures (described in the following section) are implemented, an additional 5% reduction for Phase III.

Recommended Security Practices for SUS Facilities

The general and site specific recommendations that follow do not include all of the measures that might be appropriate to higher levels of security, but focus on measures that might be typical of,

205

and appropriate to the SUS facilities at various public water systems. A more detailed study would be required for each system as security measures are designed and implemented.

Neely Reserve and Reclamation Reservoir

The Neely Reserve (recharge area) and the Reclamation Reservoir are located adjacent to, and south of, the WWTP. The entire site is bounded by a chain link fence, and there is no general public access. The reclamation reservoir and associated pumps are located in the northeast corner of the site on a compacted gravel pad. The north and east sides of this facility have a new six foot masonry wall; each side has a new iron gate that will be equipped with a keypad and remote access control. The west and south sides of this pad have no fence between it and the Reserve.

The pumps are located in a masonry structure with key locked metal doors; transformers are exposed on site. The reservoir has vents, and one access hatch.

Given the somewhat non-critical nature of this facility (use of reclaimed water and recharge), it is not clear that a high level of security is warranted. However, if the site is to be used as an SUS facility, then a higher security level would be desired and consideration should be given to the following:

- Provide a chain link fence between the recharge area and the reservoir/pump station (south and west sides).
- Secure the reservoir vents and hatch with appropriate locking devices.
- Provide motion detection lighting and intrusion alarms for the reservoir/pump area.
- Increase wall height to eight feet.
- Conduct periodic site patrols.

Riparian Preserve

The Riparian Preserve is essentially open to the public, and is therefore difficult to secure (see Figure 4.1 and Figure 4.2). Although this facility plays an important role in the system, additional security may be unwarranted, but if the site were to be used as an SUS facility, then the following should be considered:

- Fencing in the concrete distribution structure.
- Provide motion detection lighting and intrusion alarms for the area within the above fence.
- Conduct periodic site patrols.

CONCLUSIONS AND RECOMMENDATIONS

It is important to note that the percent reduction in the average weighted risk sums obtained by considering SUS facilities as a method for reducing risk are not absolute measures. In other words, a 5% reduction in the average weighted sum does not mean that risk/vulnerability has been reduced by 5%. As pointed out earlier these are relative measures and should be treated in that fashion. What it does mean is that some security practices at important critical assets (most important assets for carrying out all the critical operations) may also show the same level of reduction. As a result, if one is considering a security plan for a water system, they can compare a

206

Figure 4.1 Neely Reserve: fences (top left), main gate (top right), boosters (lower left), and reservoir hatch (lower right)

Figure 4.2 Riparian Preserve: vehicle entrance (left) and distribution control (right)

variety of physical and policy based security practices using the model discussed in this report, to SUS facilities as an adjunct to those practices. In so doing, additional vulnerability reduction could be obtained, allowing for a greater sense of overall security for the entire system.

Since a reduction in vulnerability is established (and measured) by employing SUS facilities, it is recommended that SUS be considered in any comprehensive security plan for water systems. Furthermore, the results of the analysis presented in this report demonstrates that for

207

Table 4.15
Water facilities including SUS sites

Designation	Description	Capacity	Category
Water Production Facilities		mgd	
WTP	Water Treatment Plant	40.0	WTP
Well #3	pre-1977	2.4	A
Well #19	July-99	2.2	A
Well #20	July-99	2.2	A
Well #7	May-87	2.1	A
Well #12	July-91	2.1	A
Well #4	pre-1977 re-2001	3.1	B
Well #18	May-96	2.9	B
Well #8	July-89	2.8	B
Well #14	December-92	2.6	B
Well #16	October-94	2.5	B
Well #17	June-95	2.5	B
Well #15	June-94	1.9	B
Reservoirs		MG	
Well #7	Aboveground steel tank	2.0	A
Site #20	Aboveground steel tank	2.0	A
Well #3	Aboveground steel tank	1.0	A
Site #5	Aboveground steel tank	1.0	A
WTP	Belowground reinforced concrete	16.0	B
Site #21	Belowground reinforced concrete	2.2	B
Well #12	Belowground reinforced concrete	2.0	B
Well #19	Belowground reinforced concrete	2.0	B
Booster stations		gpm	
WTP	West zone	23,000.0	A
WTP	East zone	14,500.0	A
Well #12		6,000.0	A
Well #3		2,500.0	A
Well #7		2,500.0	A
Site #5		2,100.0	A
Well #19		2,000.0	A
Well #20		3,900.0	B
Well #16	Direct pumpage well	2,150.0	B
Well #18	Direct pumpage well	2,000.0	B
Well #14	Direct pumpage well	1,800.0	B
Well #17	Direct pumpage well	1,720.0	B
Well #15	Direct pumpage well	1,300.0	B
Well #8	6,000 gpm to Zone 1 or 4,200 to Zone 2	6,000.0	B-U*
Underground Storage		mgd	
NR	Neely recharge	3.0	2 bg St†
RP	Riparian preserve	6.0	

* Underground storage.
† 2 bg St = 2 billion gallons, storage capacity.

208

sites currently employing SUS facilities with potable water systems, that they should be considered as part of their vulnerability reduction strategy and given appropriate consideration when implementing physical and policy related security practices.

Following is a list of definitions for the disruptive events (malevolent acts) used in the incident analysis are defined. It is important to note that risk is the probability of the event multiplied by the cost (or consequences) associated with that event, and that some components in the water system are more vulnerable to these events than others. However, it is the actual emergency that will dictate the response actions, and not necessarily what caused the emergency.

A. Assault: the threat of, or the carrying out of, physical harm on an employee while the employee is on duty.
B. Civil Unrest: any situation involving riots, strikes, public demonstration or protest leading to social instability.
C. Computer Hacking: any damage or disruption to the system resulting from the unauthorized entry into critical computer operations (e.g., SCADA).
D. Purposeful Cross-Connection or Contamination: any indirect or intentional contamination of the system resulting from an action by an individual or group.
E. Extortion: a threat to disrupt the system for purposes of monetary gain.
F. Sabotage: any act carried out by system personnel intended to damage, disrupt, or destroy the system.
G. Terrorist Acts: the use of force or threats directed against the system, system components, or personnel, for political or other purposes.
H. Theft: illegally removing property or equipment from a facility where the removal of such items could potentially disrupt normal operations.
I. Vandalism: any act intended to damage, disrupt or destroy the system (other than by illegal computer entry).
J. Accidents: any unintentional, unforeseen or unexpected act or event that could damage or disrupt the system that results in the failure of a system component to function as designed. This includes incidents resulting from faulty system component design and/or fabrication.
K. Human Error: a mistake or incorrect action, or response to an action, resulting from carelessness, inattention, or misunderstanding.
L. Natural Disaster: any damage or disruption to the system resulting from a meteorological or geophysical event or other environmental stress.
M. System Aging: any damage or disruption to the system resulting from the breakdown of a component due to an age related problem.
N. System Malfunction: any damage or disruption to the system resulting from a component malfunction during normal operations.

Table 4.15 lists the current finished water treatment and storage facilities along with their storage or pumping capacities. The two recharge facilities (the Neely recharge and the Riparian Preserve) are also listed as potable SUS facilities.

209

CHAPTER 5
KNOWLEDGE GAPS, EMERGING ISSUES, AND RESEARCH NEEDS

While good progress has been achieved with regard to development of the science and technology for Sustainable Underground Storage (SUS), several areas require further research and development so that SUS strategies can be effectively applied to new applications. These may be broadly classified as:

- Water quality
- Facilities design and operation
- Economics
- Regulatory framework
- Water system reliability improvement

WATER QUALITY

Probably the most significant knowledge gap, and consequently the greatest research need, is for an improved understanding of subsurface geochemical, microbial and physical processes that contribute to changes in water quality. The complex interplay of these various processes under changing conditions of pH, ORP, temperature, salinity and other conditions occurring during recharge and recovery are not well understood. As a result it is currently not possible to predict, with any high degree of confidence, the likely contribution of natural subsurface treatment processes to improvement of water quality, and the sustainability of such processes. The natural tendency is to err on the safe side, requiring pretreatment of recharge water to high standards prior to recharge and considering subsurface treatment as an additional, incidental buffer to protect groundwater quality and public health. This is certainly safe, but in many cases it is also very expensive. With improved understanding of subsurface natural treatment processes it would be possible to tailor pretreatment requirements to the local hydrogeology, source water quality and variability, and available recharge processes. While surface recharge processes have been shown to provide excellent soil aquifer treatment under oxic conditions in the vadose zone, well recharge processes also provide treatment under anoxic conditions that generally occur or develop in deeper, confined aquifers.

Selected potential research topics within the broad area of subsurface natural treatment processes may include the following:

- Mobilization and attenuation of metals (particularly arsenic) and phosphate during recharge and recovery. Are these processes sustainable? What will be the impact of these processes on soil hydraulic conductivity, soil aquifer treatment and infiltration rates?
- Microbial processes occurring during recharge and recovery, and their impact upon the quality of recharge water and ambient groundwater.
- Achieving an improved understanding of the interplay of physical, geochemical and microbial processes occurring within the proximal zone, within a few feet to a few tens of feet from an ASR well. Sometimes, concentrations of normally biodegradable organic compounds in water used for SUS are so low that bacteria can no longer

211

develop the enzymes and the "energy" to biodegrade these chemicals in solution. However, when these chemicals are adsorbed by soil particles, their concentrations in the adsorption layers around the particles will be much higher than in the soil water itself. Thus, more active biodegradation could occur in the adsorption layers, which are most likely to be found in the proximal zone.

- Biodegradation, sorption, desorption and ion exchange processes for attenuation and remobilization of trace organics during SUS. What are the resultant end products (anions, cations, gases, minerals, etc.)?
- Pretreatment of recharge water to enhance the effectiveness of subsurface microbial, geochemical and physical processes to achieve specific treatment design objectives. Such pretreatment processes may include filtration, pH adjustment, dissolved oxygen removal and other approaches.

Once a better understanding has been gained regarding these subsurface physical, geochemical and microbial processes and their interactions, it will be possible to improve existing computer simulation models so that they provide a more useful tool for evaluation of SUS management options.

FACILITIES DESIGN AND OPERATION

As the applications of SUS increase in number and also in variety, several technical frontiers are emerging. A common theme, nationwide, is the rapidly increasing difficulty for obtaining suitable sites for SUS facilities, considering constraints of competing land uses, economics, setback distances from property lines and potential contamination sources, aquifer contamination, environmental and other regulatory constraints. Consequently a major thrust of proposed research and development should be for achieving the greatest amount of long-term value from a small recharge area. Accordingly, the following potential topics for research and development are proposed:

- Application of horizontal directional drilling (HDD) technology for the design, construction and operation of high capacity water supply wells, aquifer storage recovery wells and wellfields
- Alternative materials of construction and design concepts to achieve longer service lives for ASR facilities
- Improved understanding of the multiple causes for, and control of, corrosion in ASR wells
- Subsidence control and ground level movement during ASR operations in unconsolidated clayey aquifers.
- Improvements in well development and rehabilitation techniques.
- Integration of bank filtration, soil aquifer treatment and SUS to achieve more efficient utilization of seasonally available stormwater sources and conventional water treatment processes in order to achieve reliable, sustainable water supplies.

Research is also needed regarding the movement of fine particles during surface recharge and the accumulation of such fine particles on deeper, denser or finer layers to form mini-clogging layers that must be broken up by deep-disking or ripping before the recharge basins are filled again.

ASR wellfield design is another suggested research topic. For many areas a new application of ASR is to recharge primarily in one well and to recover primarily from a second well,

taking advantage of the natural treatment ability of the aquifer between the two wells. This is known as ASTR ("Aquifer Storage Transport Recovery"). The ASTR science and technology is primarily being developed in Australia and the Netherlands. The appropriate design of such wells and associated wellfields, including well spacing, is a subject of great significance globally in areas where the regulatory framework provides for soil aquifer treatment.

ECONOMICS

There exists widespread misunderstanding of the economics of SUS. This is due to the dearth of studies on this subject; the wide variety of units that are utilized for water measurement in different parts of the United States and other countries; and the lack of concensus regarding how to properly compare the economics of storage options (diurnal, monthly, seasonal, long-term or water banking) with continuous, reliable sources of water supply.

Common units include $/acre foot per year; $/million gallons or $/1,000 gallons; and $/MGD of recovery capacity for ASR wells. For a water utility considering a plan for SUS to meet a projected short term peak or emergency period, the volume of water required for storage may be relatively small. Unit costs expressed in terms of $/AFY or $/MG would therefore tend to be quite high. If compared to an alternative water source such as seawater desalination, the SUS unit cost might initially appear to be not cost-effective. However, at no additional capital cost, the same SUS facilities could be utilized to store much larger water volumes to meet long term water banking goals, storing water in wet years for recovery in later drought years. Unit costs for such a wellfield would tend to be much lower. The problem is with the units of measurement, not with the technology. Comparison with other options should be on the basis of $/MGD of reliable recovery capacity.

In some cases the full cost of other water utility operations is included with the aquifer recharge components, tending to create very high unit costs for SUS. In other cases SUS costs include only the marginal costs required to achieve increased water supply during peak or emergency periods, yielding much lower unit costs. Several sites have achieved remarkable success in achieving recharge objectives, but at very high cost for many years of initial demonstration testing at steadily increasing scales of operation in successive development phases. Once established, long term operations at these sites tend to be less expensive. A real need therefore exists to develop a nationwide database of recharge economics, using a common method for comparison of multiple SUS data sources. With such a database it would then be possible to establish typical cost ranges to guide future planning, and the sensitivity of these costs to variability in key contributing factors. Use of the case study sites presented in the Appendix of this report would provide a useful starting point for conducting such an economic analysis.

REGULATORY FRAMEWORK

Probably the greatest constraint upon achieving widespread successful and effective application of SUS facilities is the regulatory framework, which varies widely between states and also within different parts of many states. The regulatory framework is clearly evolving, with regulatory officials in some states seeking out comparable regulatory approaches to similar issues in other states.

Of considerable value would be an updated assessment of the regulatory framework in each state, as developed to manage and control aquifer recharge and SUS. This would also include

<center>213</center>

consideration of the federal regulatory framework, as administered by EPA pursuant to the Underground Injection Control program and the 1974 Safe Drinking Water Act. Key regulatory issues would be identified that are common to many states. The variety of ways that each of these issues is addressed by different states would then be considered. Some typical regulatory issues include the point of compliance measurement for drinking water standards such as disinfection byproducts and arsenic; the ability to store water for periods exceeding one year; monitoring and reporting requirements, and recovery efficiency.

A suggested approach for an effective SUS regulatory framework is to move in the direction of a risk-based regulatory program, the goal of which would be to achieve a reasonable balance of risk, benefit and cost, with full consideration of the needs, constraints and opportunities in each state. Such an effort is currently underway in Australia to develop such a regulatory framework. This could provide a basis for development of a similar program tailored to meet SUS needs in the United States.

A fundamental challenge in the United States is the discontinuity between federal law (1974 Safe Drinking Water Act) and the 1981 EPA Underground Injection Control program, which was implemented to promulgate the federal law. The law provides for measurement of compliance with drinking water standards at a monitor well however the UIC requirements remove that provision, requiring compliance at the wellhead prior to recharge. This constrains the viability of SUS in the United States and greatly increases the cost in order to pay for pretreatment for constituents that may otherwise be removed through natural subsurface processes. It is anticipated that considerable time and effort will be required before this discontinuity can be corrected. In the meantime efforts should continue to better define subsurface processes and their effectiveness in providing water treatment.

Previous efforts (Pyne 1995, 2005) have been directed to developing a proposed model regulatory framework for ASR wellfields. A suggested research need is to update the current status of regulatory programs governing SUS in the various states, and also other countries, and to develop an updated suggested regulatory framework for application in the United States.

WATER SYSTEM RELIABILITY IMPROVEMENT

Water utility systems throughout the United States have conducted "Vulnerability Assessments" during recent years, the objective of which has been to "harden" these facilities against potential disruption due to a variety of possible causes. A very broad range of security measures and other procedures have since been implemented, greatly reducing the vulnerability and improving the reliability of these water systems. In general, however, these vulnerability assessments have not integrated SUS concepts into their procedures. The ability to create several days' to several months' supply of drinking water, safely stored below ground and capable of recovery at a few minutes' notice for disinfection and distribution, is a powerful tool for achieving water supply reliability.

Research is needed to take the concepts presented in Chapter 4 of this report and apply them to several existing or proposed SUS sites in order to demonstrate the relative value of SUS as one of the many "vulnerability reduction" strategies that are typically considered. Selection of a representative variety of sites, and tailoring the analytical approach to each site, would provide a strong basis for extrapolation of the results to additional water utilities. Providing security cameras, strong fences and gates, and other such devices is helpful but is only part of the solution. Providing an emergency reserve of drinking water, available for recovery at short notice, could be a valuable and cost-effective additional measure for many water utilities.

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CHAPTER 3: SELECTED CASE STUDIES

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228

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ABBREVIATIONS

AC	alternating current
ADEQ	Arizona Department of Environmental Quality
ANSI	American National Standards Institute
AOC	assimilable organic carbon
AOP	advanced oxidation processes
APHA	American Public Health Association
API	American Petroleum Institute
ASR	aquifer storage recovery
ASTM	American Society for Testing and Materials
ASTR	aquifer storage transport recovery
AWWA	American Water Works Association
AwwaRF	Awwa Research Foundation
BDOC	biodegradable organic carbon
BFT	bypass filter test
BSPT	British Standard Pipe Taper
°C	degrees centigrade
CD	compact disc
CD-ROM	compact disc read-only memory
CFR	Code of Federal Regulations
cm/d	centimeters per day
Co.	company
DC	direct current
DOC	dissolved organic carbon
ed.	edition, editor
eds.	editors
ENP	Everglades National Park
°F	degrees Fahrenheit
fph	feet per hour
ft	foot
ft^2	square foot
ft^3	cubic foot
ft^3/h	cubic feet per hour
ft^3/min	cubic feet per minute
ft/s	feet per second
GAC	granular activated carbon
gal	gallon

231

gal/ft/d	gallons per foot per day
gph	gallons per hour
gpm	gallons per minute
gpm/ft^2	gallons per minute per square foot
h	hour
ha	hectare
HDD	horizontal directionally drilled
HDPE	high density polyethylene
hp	horsepower
hr	hour
ID	inner diameter
in	inch
in^2	square inch
K	hydraulic conductivity
kg/d	kilograms per day
km^2	square kilometer
kPa	kilopascal
L	liter
lb/d	pounds per day
L/h	liters per hour
L/m/s	liters per meter per second
L/m^2/s	liters per square meter per second
L/s	liters per second
m	meter
m^2	square meter
m^3	cubic meter
m^3/d	cubic meters per day
m^3/hr	cubic meters per hour
MAR	managed aquifer recharge
MFI	membrane filtration index
mg	milligram
mgd	million gallons per day
mg/L	milligrams per liter
m/h	meters per hour
mi^2	square mile
MIT	mechanical integrity test
m/km	meters per kilometer
mL	milliliter
ML	megaliter
ML/d	megaliters per day
mL/s	milliliters per second

232

mm	millimeter
mm^2	square millimeter
Mm^3	million cubic meters
m/s	meters per second
mV	millivolt
μ	micron
N/A	not applicable
nm	nanometer
No.	number
NOM	natural organic matter
NPSH	net positive suction head
NPT	National Pipe Taper
ntu	nephelometric turbidity unit
OD	outer diameter
PAC	Project Advisory Committee
P.E.	professional engineer
PFI	parallel filter index
Ph.D.	doctor of philosophy
PM	project manager
ppm	parts per million
Proc.	Proceedings
psi	pounds per square inch
PVC	polyvinyl chloride
Q	flow rate
RO	reverse osmosis
rpm	revolutions per minute
SAT	soil aquifer treatment
SCADA	supervisory control and data acquisition
SDI	silt density index
SDR	surface dimension ratio
Sec.	section
SEM	scanning electron microscopy
SI	Système International (International System of Units)
spp.	species
SS	stainless steel
Stat.	statute
SUS	sustainable underground storage
TDS	total dissolved solids
THM	trihalomethane

TOC	total organic carbon
TSV	target storage volume
U.S.	United States
U.S.A.	United States of America
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
UV	ultraviolet
VFD	variable frequency drive
WEF	Water Environment Federation
WTP	water treatment plant
WWTP	wastewater treatment plant

234

CONVERSION TABLE: SI UNITS

	To convert		
	From customary units	To SI units	Multiply by
Area	mi^2	km^2	2.590
	or	m^2	2.590×10^6
	acre	ha	4.047×10^{-1}
	ft^2	m^2	9.290×l0^{-2}
Flow rate	mgd	ML/d	3.785
	acre foot/year	cubic meters/day	3.38137
	acre foot/year	gallons/day	8.9268×10^2
	gph	m^3/h	3.785×10^{-3}
	gpm	m^3/s	6.309×10^{-5}
	or	L/s	6.309×10^{-2}
	ft^3/h	m^3/h	2.832×10^{-2}
	ft^3/min	m^3/s	4.720×10^{-4}
	or	L/s	4.720×10^{-1}
Head loss	ft	kPa	2.989
	or	m	3.048×10^{-1}
Horsepower	hp	metric hp	1.014
	or	kilowatt	7.457×10^{-1}
Length	mile statute (m)	kilometer	1.609
	yd	meter	9.1440×10^{-1}
	ft	m	3.048×10^{-1}
	in	millimeter	2.540×10^{-1}
Pipe cross-section	in^2	mm^2	6.452×10^2
Pressure	psi	kPa	6.895
	or	bar	6.890×10^{-2}
Velocity	ft/s	m/s	3.048×10^{-1}
Volume	Acre foot	US gal	3.258×10^5
	US gal	m^3	3.785×10^{-3}
	or	ML	3.785×10^{-6}
	or	L	3.785×10
	or	Imperial gal	8.330×10^{-1}
	yd^3	m^3	7.645×10^{-1}
	or	*L*	7.645×10^2
	ft^3	m^3	2.832×10^{-2}
	or	ML	3.785×10^{-5}
	in^3	cm^3	1.638×10^1
	or	L	1.638×10^{-2}
Weight	lg ton	metric ton	1.016
	short ton	metric ton	9.070×10^{-1}
	lb	kilogram	4.536×10^{-1}
Temperature	°F = (9/5) (°C + 32)		

235

APPENDIX

UTILITY FILES

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

EQUUS BEDS
GROUND-WATER RECHARGE DEMONSTRATION PROJECT
CASE STUDY FOR AWWARF PROJECT #3034*
*(INFORMATION ADAPTED FROM A CONVERSATION WITH JERRY BLAIN ON JUNE 9TH, 2006)

Background

In 1993, the City of Wichita, Kansas approved a water supply plan, titled "The Integrated Water Supply Plan" (ILWS Plan), to evaluate the City's water supply needs through the year 2050 and determine how potential shortfalls would be met. The artificial recharge of the Equus Beds aquifer was identified as one potential alternative to meet those future water-supply demands. The Equus Beds Ground-Water Recharge Demonstration Project was initiated in 1995 to examine artificial recharge techniques and their effects on water quality in the Equus Beds aquifer. The demonstration project conducted as a collaborative effort between the city of Wichita, the Bureau of Reclamation (U.S. Department of the Interior), and the U.S. Geological Survey (USGS).

The Demonstration Project sought to determine if excess flows could be diverted from the Little Arkansas River, either from wells drilled adjacent to the river or an intake in the river, and then used to recharge the aquifer. The project included the following key components:

- An evaluation of several recharge techniques;
- An extensive water quality monitoring program;
- And an extensive public information program.

The Demonstration Project served as an effective tool to not only establish the technological components of an ASR project, but also provided essential information to regulatory agencies and the public, to instill confidence in the safety and viability of the City's proposed ASR project and the entire Integrated Local Water Supply Plan. The following sections review the results of this project.

PROJECT OBJECTIVES

The overall objective of the Equus Beds Ground-Water Recharge Demonstration Project was to identify the best sustainable aquifer storage and recovery technology for use in full-scale implementation. The goals of this facility would be to meet the following criteria:

- Provide seasonal storage of excess runoff from Little Arkansas River;
- Restore groundwater levels in the Equus Beds Aquifer;
- Expand the usability of local well fields;
- Prevent the migration of nearby chlorides and saltwater plumes towards the well fields;
- And conserve water in the Cheney Reservoir.

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DEMONSTRATION PROJECT REVIEW

General

Water from the Little Arkansas River was diverted for artificial recharge when flow in the river exceeded base flow in accordance with the Kansas Department of Agriculture, Division of Water Resources, permit conditions (Burns and McDonnell, 1998). Water was artificially recharged to the Equus Beds aquifer, which is part of the High Plains aquifer and consists of alluvial (river-deposited) sediments of sand and gravel interbedded with clay and silt.

HALSTEAD SITE

Diversion

At the Halstead diversion well site, water was diverted from the Little Arkansas River by pumping a diversion well completed immediately adjacent to the river that induces the surface water into the well. This diverted source water then was pumped to the Halstead recharge site and recharged to the aquifer by injection well, basin, trench. Artificial recharge of the Equus Beds aquifer began at the Halstead site in May 1997.

Injection Well

One recharge well was constructed at the Halstead recharge site. The recharge well was 32-inches in diameter, had a total depth of 215 feet deep, and had an 18-inch casing. In order to enhance hydraulics and infiltration, the space between the casing and the edge of the well was filled with gravel. The well passed through three levels of water bearing strata. The well was screened at the lowest strata layer, below an extensive clay barrier. The well was equipped with three recharge tubes that discharge into the casing approximately five feet above the screens. Three tube sizes were used: 2 inches, 2-1/2 inches, and 3 inches. The variation in tube size allowed the well to accommodate a range of flows while keeping the tubes full. This decreased air entrainment and reduced the potential for dissolved iron to oxidize. The total well capacity was 1,000 gpm.

Recharge basins.

Two recharge basins were constructed at the Halstead recharge site. Each basin was dug to the depth of the first significant sand layer, which was approximately 12 feet below land surface. The basins had bottom surface areas of 0.35 acres and 0.2 acres. The predicted recharge rates for the basins was approximately 4 feet per day, or around 500 gpm. Unfortunately, the clay layer located below the first sand layer significantly restricted the flow of the recharged water, yielding recharge rates of only 1 to 2 feet per day.

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Recharge trench

One recharge trench was constructed at the Halstead site. The trench was 100 feet long, 3 feet wide, and 12 feet deep. A concrete frame was constructed for the top of the trench and to reduce the growth of vegetation, a simple removable roof was placed over the trench. Optimum flow rates of 60-75 feet per day were achieved when the flow was able to cross the entire trench. The water supplied to the trench contained significant amounts of dissolved iron, which oxidized as it flowed across the trench. The oxidized iron precipitated onto the permeable membrane and plugged the membrane. This plugging significantly reduced run times for the trench.

Passive wells

After the recharge basins proved unsuccessful, five wells were drilled 200 feet below the clay layer in each basin. Each well was 2-inches in diameter and constructed of PVC pipe and screen. The wells in each basin were connected together with a 4-inch diameter horizontal screened pipe. The screened pipe served as a collection system to collect water and transport it to the wells and then the water would flow down the wells into the aquifer. These wells improved the recharge rates of the basins by over 400 percent.

SEDGWICK SITE

Diversion and Pretreatment

Recharge water for the Sedgwick recharge site was diverted directly from the Little Arkansas River. It was treated with Lamella Gravity Settler to reduce turbidity (the cloudy appearance of water caused by suspended matter) and powdered activated carbon (PAC) to remove organic compounds, including the herbicide atrazine. Artificial recharge of the Equus Beds aquifer at the Sedgwick site began in April 1998.

The peak atrazine concentrations found in the Little Arkansas River were as high as 47 µg/l, which far exceeds the drinking water standard of 3 µg/l. Turbidity in the river ranged from a low of 20 NTU to a high of over 1,000 NTU. A treatment goal of less than 30 NTU was chosen for water before it was discharged to recharge basins. Polymer was added to promote flocculation and to improve settling characteristics.

Recharge Basins

Three recharge basins were constructed at the Sedgwick site. Each recharge basin was approximately seven feet deep and had a total bottom area ranging from 0.42 to 0.49 acres. The basin sides were lined with a geotextile fabric and riprap, and had access ramps for maintenance. The recharge rates of these basins were much higher than the Halstead site, between 8 to 9 feet per day. Due to the higher recharge rates for these basins it was difficult to determine optimum treatment parameters for the pretreatment facility. It was anticipated that less turbid water would reduce maintenance of the basins; however, treatment also increased the operational costs. It was observed that lower recharge rates resulted in longer recharge times and higher recharge volumes than higher recharge rates. In addition, the higher flow rates appeared to clog the sand. It was observed that raw water with high turbidity was easier to treat and to recharge than low turbidity raw water.

Modified Recharge Trenches

The modified recharge trenches were composed of four circular steel tubes, 11.5 feet in diameter, that were set approximately five feet into the sand bottom of the recharge basins at the Sedgwick site. Three of these tubes had permeable membranes fitted on the bottom, the other was used as a control. Each of the membranes had a different porosity. During the test period the fabrics were washed once to remove sediment buildup. The observed recharge rates in the simulated trenches varied from 5 to 30 feet per day. These results indicate that the recharge trench technology may be a viable option for appropriate geological sites, even if surface water is used.

LESSONS LEARNED

HALSTEAD SITE

Recharge Basin

The following bullets summarize the lessons learned for the recharge basin testing conducted at the Halstead site:

- The clay layer located below a recharge basin reduced the recharge rate.
- The side walls of the basin were lined and ripraped to prevent topsoil erosion during heavy rainfall.
- The installation of access ramps into the basin allowed easy access for the equipment used to clean the sediment and debris.
- Recharge rates of 1 to 2 feet per day were achieved.

Recharge Trench

The following bullets summarize the lessons learned for the recharge trench testing conducted at the Sedgwick site:

- Optimum flow rates were achieved when the flow was able to cross the entire trench.
- Dissolved iron oxidized as it flowed across the trench. The oxidized iron precipitated onto the permeable membrane and plugged the membrane, reducing the recharge run times.
- Recharge rates of 60 to 75 feet per day were achieved.

Passive Wells

The following bullets summarize the lessons learned for the recharge passive well testing conducted at the Halstead site:
The installation of passive wells that penetrated the clay layer of the recharge basins significantly increased recharge rates (400 percent).

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Pretreatment

The following bullets summarize the lessons learned for the pretreatment testing conducted at the Sedgwick site:

- If the PAC is used as a pretreatment step, it must be removed prior to recharge to prevent clogging.
- Lower recharge rates yielded longer recharge times and higher recharge volumes than higher recharge rates.
- High turbidity raw water was easier to treat and to recharge than low turbidity raw water.

Modified Recharge Trenches

The following bullets summarize the lessons learned for the modified recharge trench testing conducted at the Sedgwick site:

- A modified recharge trench with a membrane with a porosity of 100.0 gpm/ft2 performed the best during testing.
- Recharge rates varying from 5 to 30 feet per day were achieved.

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AWWARF Case Study:
Orange County Water District
Ground Water Management Program,
Southern California
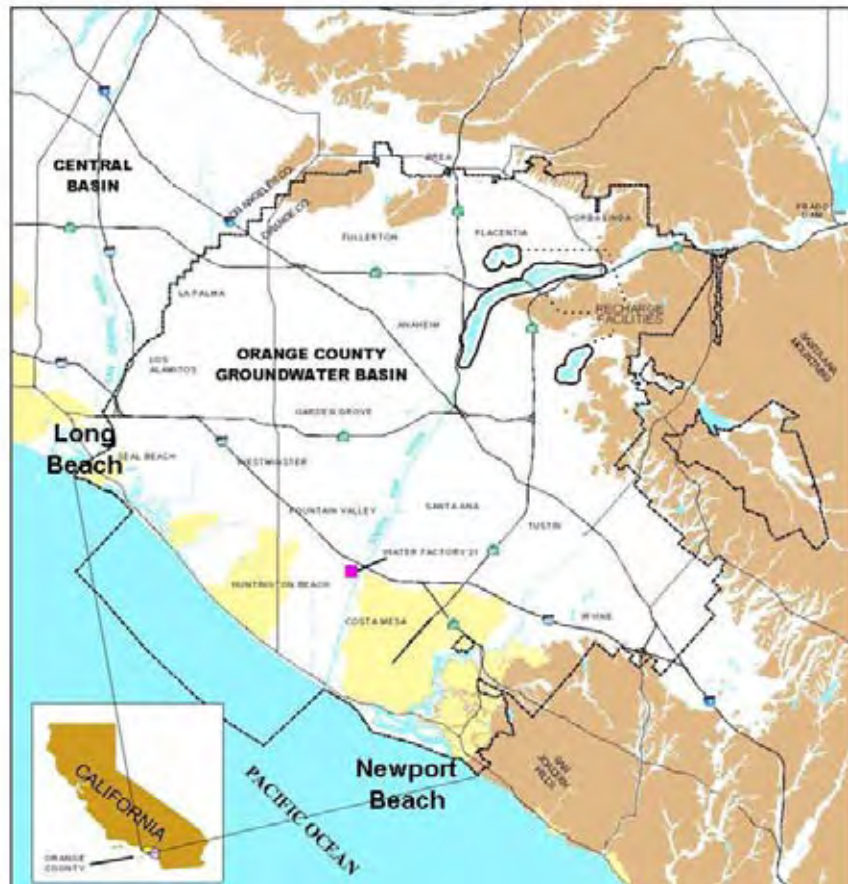
Prepared by: ASR Systems LLC

Prepared for: American Water Works Association
Research Foundation

May 2006

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Introduction

The Orange County groundwater basin, located in southern California, is bound by Los Angeles County on the north; San Diego County on the south; the Pacific Ocean on the west; and the Santa Ana Mountains on the east. The setting is nearly 800 square miles of interconnecting urban sprawl with 50% of the land occupied by development, 4% by agriculture, and 46% open area, mostly located in the Cleveland National Forest. Current 2006 work force is 1,586,500 in accordance with demographics (Orange County, 2006) with an estimated populous exceeding 2.2 million. The Orange County Water District (OCWD) is chartered with managing the groundwater basin, not distribution of potable supplies through pipes to service customers. Thirty four individual cities within the county either draft water from the groundwater aquifers and/or take piped deliveries from Metropolitan Water of Southern California (MET) for a total demand of 512,000 AF in 2001/02, and an extrapolated demand of 600,000 AF by 2020.

In 1933, the OCWD was established by state legislature to protect the Orange County Groundwater Basin. This responsibility was further increased in 1955 to provide water management of the basin. OCWD secured the rights to all water in the Santa Ana River (SAR), enabling the capture of all flows prior to loss to the Pacific Ocean. OCWD developed a two fold objective to replenish the aquifer with SAR water and prevent the intrusion of saltwater from the coastline into the groundwater basin.
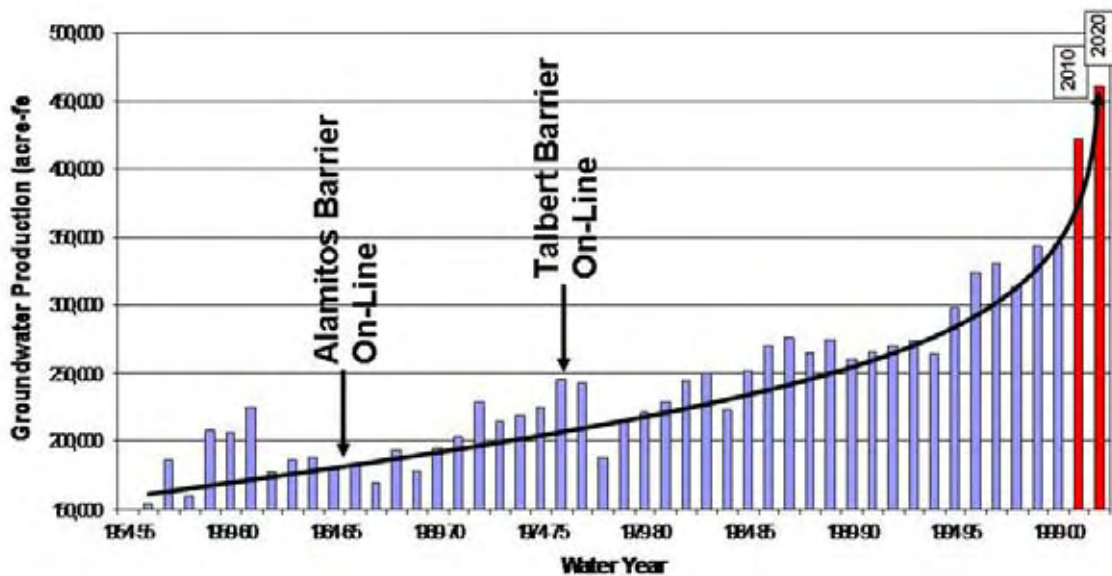
Figure 1 – Pumping History for Orange County Basin (Bonsangue, 2006)

Over the 50 years of management, the groundwater pumping has increased from 150 k-afy to over 350 k-afy. Today, nearly 200 small scale and 300 large scale wells are permitted by the OCWD for operation in the basin. Note the pumping history in Figure 1. OCWD has successfully counterbalanced this increased pumping with the infiltration basin and barrier recharge facilities. The Talbert Barrier wells are capable of injecting 12 mgd into 4 aquifer zones to hold seawater back. Infiltration operations in the upper basin are designed to percolate 191 mgd to 479 mgd, depending on the degree of basin

clogging at the time (OCWD, 2006). The current system is capable of capturing and recharging over 350,000 AF of water in the aquifer in a single year. The system is still unable to capture the full resources before it is lost to the ocean, though improvements are under way to increase the capacity by over 70 mgd with a new advanced waste water treatment plant. OCWD is still issuing groundwater use permits so long as the use can be protected through guaranteed recharge capacity. The program is considered highly sustainable in the view of capturing localized effluent and having proprietary rights to all flows on the SAR.

Site Setting - Hydrogeologic Framework

The Orange County groundwater basin is characterized as a north-south trending synclinal depression filled with stratified sedimentary sands, silts, and clays. Figure 2 shows a west-east cross-section of the basin.
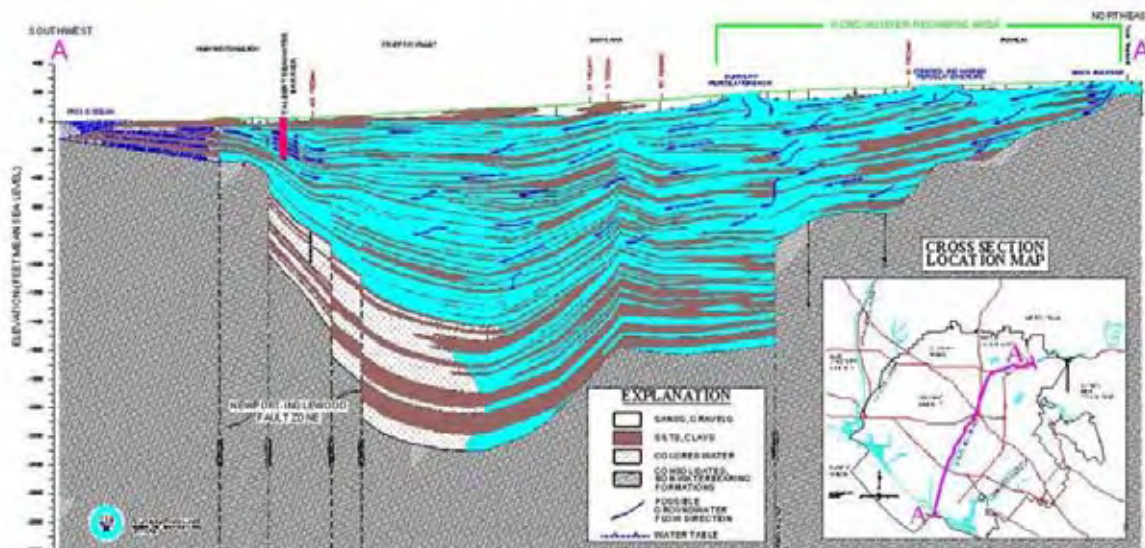
Figure 2 – Orange County Groundwater Basin Cross-Section (Bonsangue, 2006)

The basin is bound by impermeable highs on the east forming the Santa Ana Mountains and on the west forming the Newport Beach and Huntington Beach mesas (buried). The synclinal axis rises to the south-east resulting in pinching and termination of the water bearing strata in the southern basin. Figure 3 shows the surface geologic setting of the basin, depicting the mesas and impermeable bedrock areas. Troughs are cut through the mesa ridgeline by creeks to form gaps filled with porous sedimentary fill. The basin has a large surface stream feature, the Santa Ana River, flowing northeast to southwest, terminating at the Pacific Ocean. Several other minor parallel or tributary creeks pass through the hydrologic basin including Santiago Creek, Carbon Creek, Coyote Creek, and San Diego Creek.

The basin fill is divided into three aquifer zones as depicted in Figure 4. The Principal System is primarily used for pumping and is composed of four aquifers. At the Talbert Barrier site for instance, the zones and depths are: Alpha Aquifer with a base of 100-150 feet; Beta Aquifer with a base of 200 feet, Lambda Aquifer with a base of 300

feet, and the Main Aquifer with a base of 800 to 1000 feet. Water pumped from portions of the Deep System produce red colored water. In the lower, or western portion of the basin, the Principal System is confined, requiring recharge water to be placed directly through wells as demonstrated at the seawater intrusion wells. In the upper, or eastern portion of the basin, the aquifers are unconfined. The unconfined sections will allow infiltration of water that will benefit the Principal System. OCWD has carefully mapped out the extent of the unconfined basin and has titled this the "Forebay". The Forebay became the target for groundwater spreading basins. To date, over 1,200 acres of infiltration basins are in service in the Forebay recharging the groundwater basin.
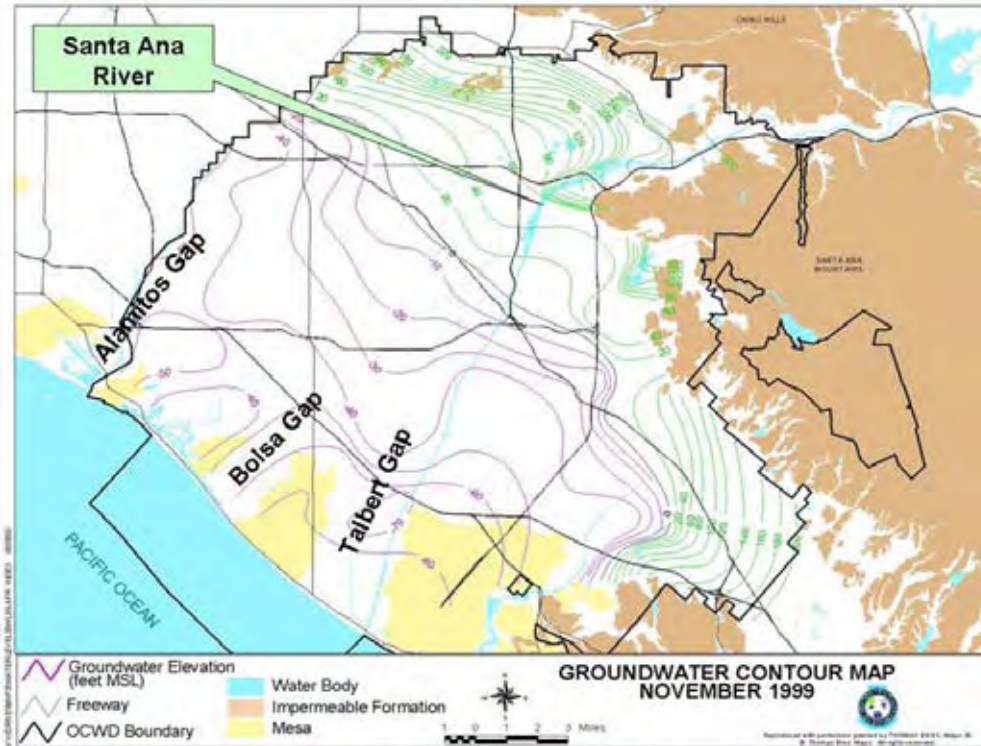
Figure 3 – Hydrogeologic Setting (Bonsangue, 2006)
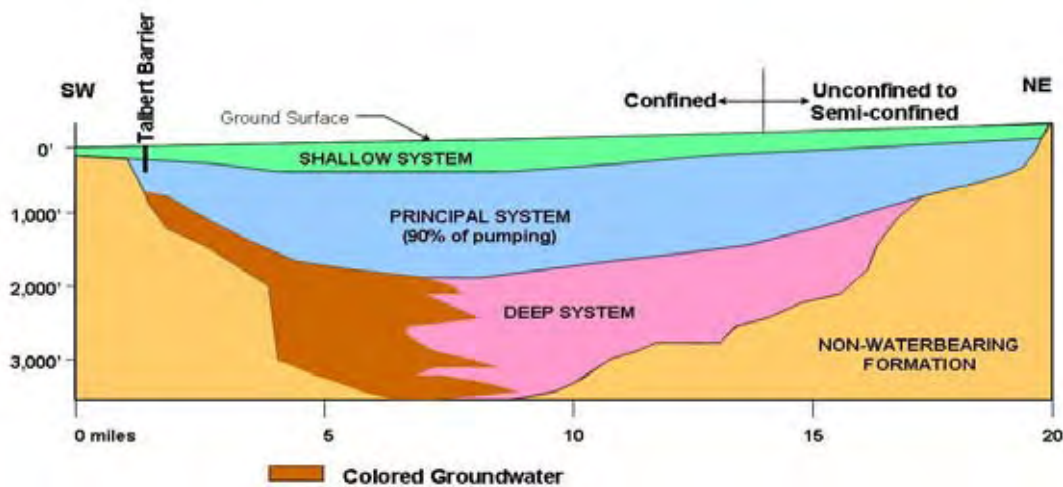
Figure 4 – Basin Aquifers and Hydrology (Bonsangue, 2006)

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Source Water Characteristics

As early innovators, OCWD is a good model for the progressive use of non-native surface base-flows and storm water surplus flows. With the urban development of the upper basins, a consistent base flow of nearly 150,000 AF of treated effluent is observed entering the basin from upstream communities (OCWD, 2003) Storm flows into the basin average 65,000 to 75,000 AF with peaks around 150,000 AF. OCWD attempts to capture and divert these flows to spreading basins for percolation into the aquifer. The infiltration basin system is unable to capture 100% of the basin inflows, indicating more resource availability than facilities. The infiltration system will receive an additional 40 mgd in highly treated effluent from the new county treatment plant starting in 2007.

Water availability for the barrier wells is derived from multiple sources to maintain the barrier. The base source is highly treated effluent mixed with Deep Aquifer System well water, imported water from MET, and local purchased water from the City of Fountain Valley consisting of a combination of well water and MET water. To date, the creative balancing of water sources has allowed the sustainability of the system. An on-going improvement project scheduled for completion in 2007 will increase treated effluent volume from 4 mgd to over 30 mgd. This should sustain the current 12 mgd demand of the barrier system and the necessary improvements.

Water Quality

The quality of the water used for direct injection at the barrier wells is of many makes. The treated water component is clarified secondary effluent obtained from Water Factory 21 (WF21) where the final stages include multi-media filtration, reverse osmosis, and ultra-violet. This plant is being replaced with a new 70 mgd state-of-the art treatment facility utilizing reverse osmosis, ultra violet, micro-filtration, and advanced oxidation processes to polish the water. Other water used for blending included Deep Aquifer System water high in iron and diversions from the potable water system. Potable water contributions consist predominantly of MET water, mostly treated Colorado River water, and some portions of local groundwater

The quality of the surface water used for infiltration can vary widely. Upstream effluent discharges must travel some distance down the riverbeds before entering Orange County. This travel changes the identity of the water to SAR water, altering its appearance from a nearly clear effluent to a brown-green mixture of sand, silt, algae, fish detritus, bacteria, organic material and trash. OCWD employs automated debris removal racks and pre-infiltration settling basins to remove the bulk of the material. During storm events, high sediment load in the SAR water may require reduction or suspension of SAR diversions to protect the basins from clogging. When the SAR flows return to acceptable quality, then the infiltration system inlet flows are re-established. During times when MET surplus water is available, potable water is purchased to fill unused capacity in the infiltration basin system. With the completion of the new wastewater treatment plant, nearly 40 mgd of highly treated effluent will be piped to the basins for recharge.

Sea Water Intrusion Barrier System

Feasibility and Design Studies: The Orange County Groundwater Basin is in direct contact with the high chloride sea water at three gaps cutting the beach front mesas noted in Figure 3. As groundwater production increased in the basin so did the invasion potential of sea water. The first demonstration testing was conducted in 1966 to 1968 at the Alamedas Gap. The project proved successful in holding the migration of sea water through the hydro-gap. This system is still on-line today, though is operated by the Los Angles County Department of Public Works using MET water and was not a part of this study. In the early to mid 1970's, the OCWD drilled the Legacy wells along the east side of Talbert Gap to hold the chloride migration from entering the groundwater basin. In 1975, WF21 construction was complete providing 4 mdg of highly treated effluent water for the Legacy Barrier wells. To date, the recharge rate at the Talbert Gap barrier wells has been increased in response to basin pumping to 12 mgd using a unique combination of water. The wells are capable of injection or back-flush rates from below 100 gpm to near 1000 gpm.

Facility Development: The Legacy wells were installed as clusters to target four unique aquifer layers. Figure 5 shows the typical Legacy well site completion. A total of 26 well sites were drilled reverse circulation rotary at 30-inches in diameter and equipped with up to four, 6-inch casing tubes. Screens are louvered and completed with a gravel envelope. Grout seals were placed between intervals and surface. All material is stainless steel, with exception of the centralizers and collars. In recent years, leakage has occurred at the casing couplings due to casing deterioration. It is suspected this is due to poor welding and dielectric differences of the mild steel components at the casing connections. Special removable insert tubes had to be designed to sleeve between the target screen and the surface sanitary seal, providing a leak-tight seal.
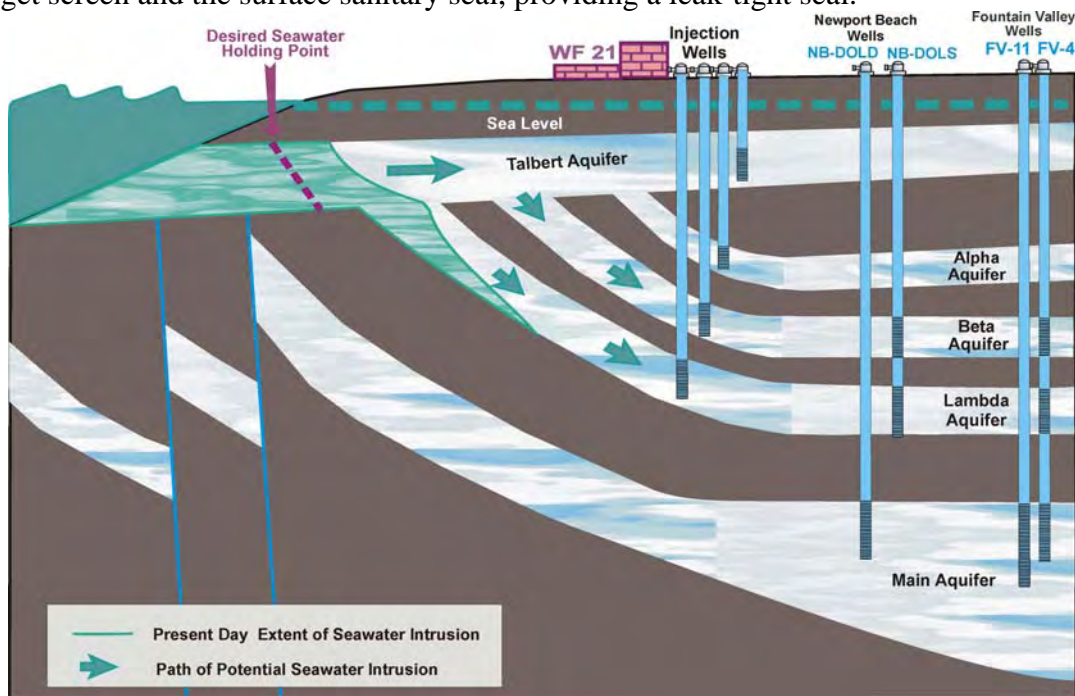
Figure 5 – Principal System Aquifer Zones and Barrier Wells (Bonsangue, 2006)

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The wellheads are equipped with casing vent and pressure gauges, injection tubing vent and pressure gauge, magnetic flow meters, and globe style control valves with an automated pilot circuit as noted in Figure 6. The 3-inch injection tubes extend to below the water table and in some cases are sleeved to the screen due to casing corrosion in the blank sections. Some tubes have orifices, most are open ended, and no borehole control valves are used in the Legacy wells. Backpressure is maintained in the wellhead piping and injection tubing during operation. The water level rises above static, but only in a few cases does it rise to the surface inducing pressure on the wellbore casing.
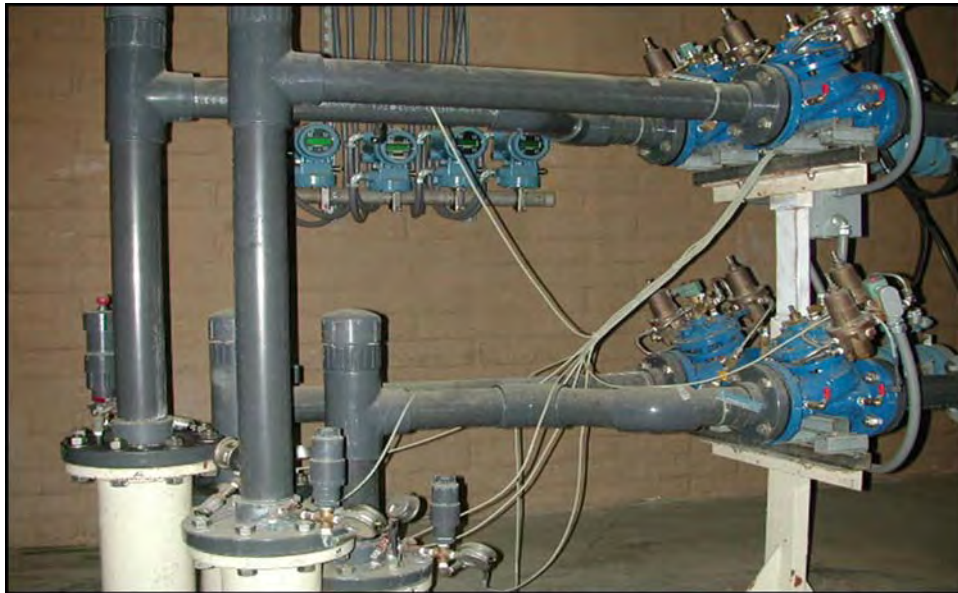
Figure 6 – Four Well Nest Injection Controls (Bonsangue, 2006)

Recently, five new wells have been constructed to support the Talbert Gap barrier program. These wells were drilled reverse circulation rotary at 18 to 22-inches and cased with 12-inch stainless steel casing and screen. The wells are constructed with wire wrap screen, gravel enveloped, and a grout seal to surface. Each borehole is constructed to a unique aquifer zone. The construction included a 3-inch camera tube tying back into the central casing below the pump, setting. Two of the new wells are equipped with pumps for full volume back-flush ability. Each well is equipped with water level transducers and a borehole flow control valve. The flow adjustment is tied to level in the wellbore through logic control programs. The wells employ conventional casing and column air release valves, bi-directional magnetic flow meters, and stainless steel column pipes.

Operation Data: The demand on the groundwater basin has increased 125% since the inception of the Talbert Gap barrier wells 40 years ago. To overcome the pull of high chloride water into the basin, the rate of injection has increased three fold. This requires many of the wells to operate full time, year round. In some select cases, wells are cycled on and off based upon monitoring data of the select aquifer. In the worst case, a well may be stopped 4 times a month with 2 to 3 days off per cycle. The wells are controlled by a SCADA system allowing automated on/off control. With individual

injection tube control at each wellhead, select wells can be placed into service to achieve specific hydraulic responses.

Startup of the older wells will entrain the column of air in the injection tubing into the well column, typically hundreds of feet above the first screen. Performance problems have not been attributed to entrained air at startup, indicating this air may be mostly rising back up the casing rather than down into the aquifer. A positive pressure is achieved and maintained on the column pipe during operation, regardless if the end of the pipe is equipped with an open end, orifice, or flow control valve.

The new wells are manually valved to back-flush the well. This operation component is not automated. Waste discharge is to the local roadways and storm water collection system. After a few hours of monitored flushing, typically 100 well volumes, the pump is stopped and the well valved to recharge. The flow control valve is manually opened to the desired range, and then SCADA control fine tunes the operation rate based upon water level. On/off control is automated through a surface motor operated butterfly valve. Automated starts and stops have the potential to push a full column of air into the wellbore. In this case, the control valves are located on top of the pumps, deep in the wells, near the well screens. This provides a higher potential of air entering the aquifer rather than migrating up the well casing.

Water pressure is always available at the wellhead in this closed loop system. The system operator must be cautious to maintain injection capacity in balance with WF21 output, Deep Aquifer production, and imported water availability. A sophisticated SCADA pump system assists in this operation.

Performance Evaluation: The OCWD operates a year-round maintenance program. Performance of the wells is tracked through flow declines and excessive water level rises as compared to historic trends. Well clogging is mostly attributed to gallinules and slime bacteria growth in the wells, with little problems from iron or sulfate related bacteria. The agency has found a successful combination of jetting and air-lifting to return the performance of the wells. Jetting is accomplished with chemical solutions containing acids and/or strong chlorine.

Due to the wells being located in vaults in the roadway, OCWD has taken special steps to make the well rehabilitation process efficient. They use a large truck mounted reel to un-coil, or coil, the rigid well development pipe into the wells. This requires no pipe support trucks or assembly of pipe sections. The truck can pull up, open vault, swab-jet-air lift well, pull out, close vault and drive away. A daily routine can be setup at low traffic hours and removed for peak traffic flow periods.

The OCWD has an extensive network of monitoring wells around the Talbert Gap and buried mesa areas to track the relationships of the Shallow, Principal (Alpha, Beta, Lambda, and Main Aquifers), and Deep Aquifer Systems. This includes heads and high chloride migration. Figure 7 shows the extent of chloride migration into the basin aquifer system. The chloride rich sea water has pushed past the desired hold point and migrated around the barrier through the low permeability, bedrock mesas. The monitoring data indicates a need to increase the barrier pressure and look for ways to form a new barrier in the mesa bedrock areas.

The OCWD plans to expand the operation on three fronts. The expansions will be supported in 2007 by the addition of 26 mgd of supply from the new plant replacing

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WF21. The first improvement will be to install more large diameter, ASR type wells with higher capacity and less maintenance. The second project is to determine the best approach to stop the migration of high chloride water through the low permeability mesas. The third priority task is to address a new barrier system in the Bolsa Gap, located between the controlled Talbert Gap and Alamitos Gap.
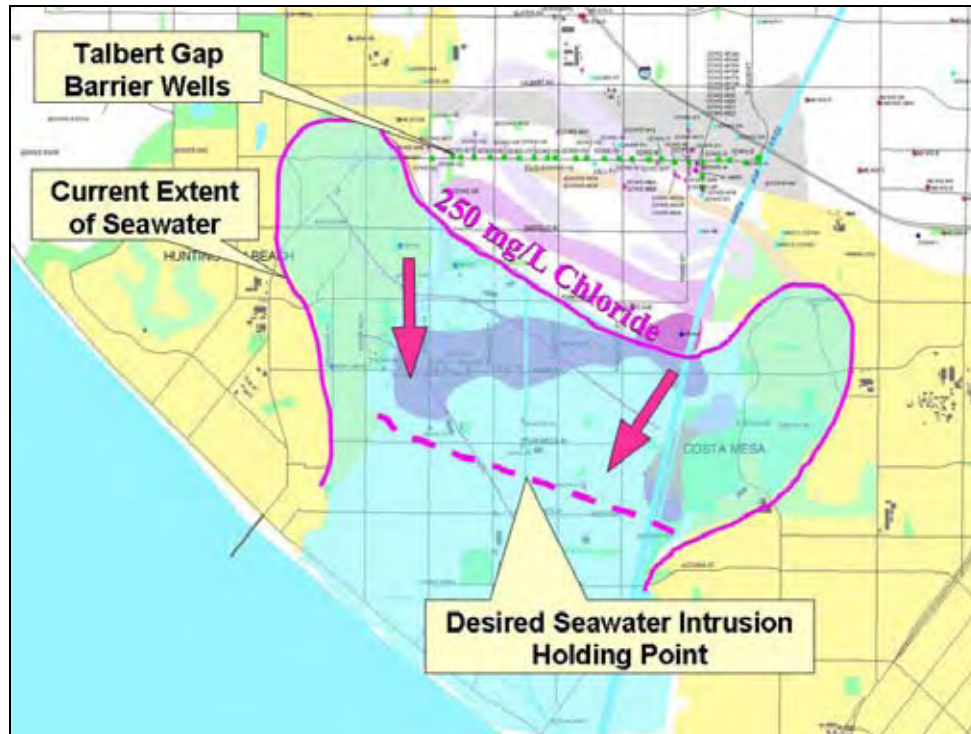
Figure 7 – Chloride Migration At the Talbert Gap (Bonsangue, 2006)

Problems Identified, Lessons Learned, and Recommended Operational Practices: Key lessons learned for the barrier well system are:

- Water placed in a sea water intrusion barrier system replenishes the groundwater aquifer, providing recharge water available for beneficial use within the basin.
- Changes in conventional effluent treatment to include RO, UV, MF, and AOP (no cl2) can render approximately 85% of the water useful as purified for direct injection withy no argument by regulators about quality.
- A groundwater barrier can be successfully developed and maintained for over 40 years to hold back a non-potable source from a potable basin. The barrier under monitoring and controlled injection, can be sustained with groundwater production increasing over 125%, so long as the basin is replenished each year through natural and managed practices (no adverse water level decline rates per year).
- Talbert Gap barrier can be maintained using approximately 8-10% of the annual basin pumping. The current application of 4% annual pumping is losing ground with the high chloride migration.

– Cluster well construction in a single borehole was not the most successful path to take. Though grout was placed between aquifer zones, pipes reaching to deeper zones corroded in their passage through adjacent zones. This resulted in zonal leakage.
– Cluster well use of 6-inch casing proved too small for the repair of the holes with a swedge. The casing is too small for development pumps providing the necessary yield.
– Installation of a single well per borehole is the approach with the highest success and longetivity.
– Well casing is increased to 12-inch, allowing adequate room for recovery pumping equipment and flow control valves. Construction is all stainless steel, including collars, centralizers, sounding tubes, and gravel feed tubes.
– Target zone wells are individually controlled allowing select zone injection to produce an isolated, desired aquifer response.
– Back-flush preformed when performance drops 50%. Back-flush typically requires removal of 100 well volumes over a few hours of pumping. Back-flush is constant pumping without surging.
– Low permeability barriers (mesas) are still transmissive. They will slowly leak. Need to account for this low-permeability leakance in the original design. Now have to develop a catch-up plan since leakage has occurred into the mesas.
– Specialized well rehabilitation equipment has been incorporated into a costume truck allowing daily work at well sites located in traffic corridors. The work can proceed during low traffic hours and easily demobilize during high traffic use hours.

Infiltration Basin Recharge System

Feasibility and Design Studies: To match the groundwater basin demand, the OCWD operates over 1,200 acres of spreading, or infiltration facilities located in and adjacent to the SAR and Santiago Creek (Figure 8). The percolation spreading grounds consist of 17 major facilities grouped in four major components: the Main River System, the Off-River system, the Deep Basin System, and the Burris Pit/Santiago System (OCWD, 2003). The four component types are designed and managed uniquely different. The facilities are located within the Santa Ana River bed, parallel SAR flood plane, side tributary creek channels, off-river basins, and old quarry pits. In 1953, OCWD began improvements in the SAR bed and created off-channel spreading basins. With improvements over the years, the system has expanded surface area to meet increased groundwater pumping. Now land is at a premium, requiring use of tributary channels, old quarry pits, and possibly right-of-way galley wells. In the interim, unique basin operation methods and cleaning efficiency methods are providing increased recharge capacity with the existing system.

Prado Dam, an upstream SAR reservoir on the county line, can be used in a limited capacity to capture and slow release storm water to downstream Orange County. The dam is designed for flood protection and maintains a minimal pond for environmental protection of the wildlife and plant. When possible, controlled releases are placed in the SAR to be captured by OCWD. Prior to storm events, the pond will be

lowered to minimal level to allow maximum pond capture. A small off-channel series of shallow ponds are maintained for recharge in the Prado Dam flood impoundment area. This recharge water enters the head of the Orange County groundwater aquifer system.
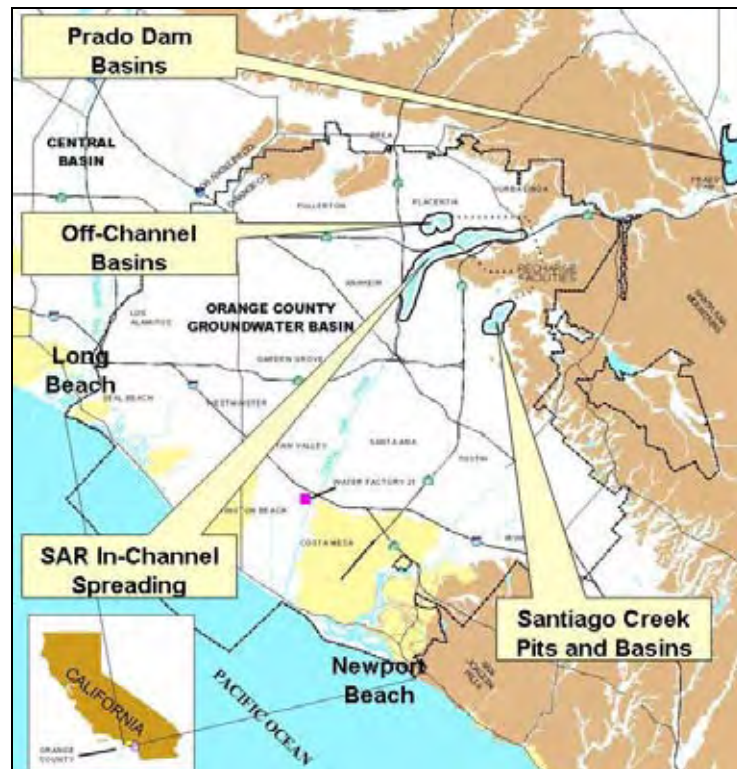
Figure 8 – Infiltration Basin System (Bonsangue, 2006)

In-Channel Facilities: The SAR forms a broad flood bed as the creek exits the Santa Ana Canyon and sweeps into the basin. The SAR flood bed has been separated into two channels with a 15 to 20 foot high levee. The primary channel is capable of handling all normal and flood release flows from Prado Dam. The other half of the flood bed is then used for recharge operations. Some fraction of recharge occurs in the primary channel, but the operation and maintenance is focused on diversions into the side channels and basins. The primary channel "armors" itself through the loss of sediment, leaving behind cobbles and rocks. During low flows, silts fill between the rocks and quickly clog the percolation ability. Cleaning the rocky bottom is difficult.

At the upstream recharge diversion point in the SAR, a 24-inch inflatable dam spans the width of the primary channel. The dam diverts most all normal flow to the side channel, allowing higher base flows to cross over the top of the dam. The diversion structure consists of automated trash removal racks, large concrete channels, automated sluice gates, and a bypass galley to allow return flow to the SAR primary channel. The trash racks are a series of chains with spikes that roll upwards on a time interval. Tree branches, soda bottles, plastic wrappings, get picked up and removed into a disposal bay.

The diverted water enters a sediment settling basin to drop out the suspended solids. The basin is nearly 3000 feet long by a few hundred feet wide maintaining a water depth of 3-4 feet. Percolation rates are low, though beneficial since the duty of the

basin is to clarify the water. The water is then diverted through a series of small channels and pipes to in-channel and off-channel infiltration facilities.

The next several thousand feet of (parallel) flood bed channel is used for stream flow percolation. Flow can be taken from the de-silting pond section or directly diverted from the primary SAR channel. The flows are routed through a torturous path of shallow "T" levies to increase surface area contact. If the water is flowing in a shallow path and maintaining some velocity, silt will not be deposited and the percolation rates will stay high. If the area is flooded, the resulting velocity is low resulting in the settling of sediment and reduced percolation rates. The "T" levy walls are no more than 12-18 inches high.

The next few thousand feet of flood bed channel is used for the Subsurface Collection And Recharge System (SCARS). This is a project looking at improved de-silting and rehab methods through shallow bank filtration. In this project, the upper stretch of the channel base contains three 1000 foot perforated collector pipes buried one foot below bed grade. The pipes empty into surface flow and diversion levies in the lower section of the reach. The concept is to de-silt the raw SAR water in the upper reach, collect it in an under-drain system, and route the filtered water to an area with high percolation rates. With the buried pipes, mechanical scrapping and replacement of sand on the first few inches of the surface can be done simply.

Below this facility is another 24-inch diameter inflatable dam spanning the SAR. This diversion structure is also constructed with automated trash removal and diversion control as the upper diversion structure. The water is routed to multiple basins for de-silting and shallow flood percolation in several channel bed sections. Flows terminate at the old Burris Pit quarry where the water is pumped to the Santiago Recharge System.

The channel, basin, and pit bases are sand, not rock and gravel as found in the "armored" primary SAR channel. Infiltration is greatest in the sand matrix. Routine maintenance and cleaning of the recharge facilities removes the first few inches of sand that must be de-silted and recycled for replacement back in the channel bed. The program could not afford a continual loss of material. To this extent, a small scale sand washing plant is in operation with plans for expansion in the near future.

Off-Channel Deep Basin and Pit Facilities: Off-channel basins constructed in the 1960's and 1970's were generally deep (35-60 foot) to allow more volume storage per acre. Recent basins are constructed as shallow ponds to reduce the hydrostatic pressure on the clay platelets, allowing greater infiltration rates per acre.

The water that exits the upstream de-silting pond is channeled to a series of three large infiltration basins, and one small basin. These basins contain water at a depth of 10 feet to 35 feet. The largest of the basins, Warner Basin, is co-used for recreation and fishing. The water is continually pushed through these basins to maintain freshness. The outflow of the near channel basins is piped over a half of mile north to Carbon Creek to the next series of four large infiltration ponds. The largest basin, Anaheim Lake, has also been historically co-used for public recreational use. The lake even includes islands for wildlife habitat. Two additional small infiltration basins have been built within the flood bed of Carbon Creek about a mile below the larger basins. These basins passively pick up outflows down the creek bed from the Anaheim Lake facilities. OCWD co-uses these

two lower basins with Orange County Public Facilities and Resources Department for flood control, limiting recharge use to non-flood periods.

In or near the SAR, a few very deep pits were excavated up to 60 feet below surface. Increase the capacity and lessen the travel to the aquifer. The percolation of these pits is low and clogging rates are high. This is attributed to the steep walls and high hydrostatic pressure on the soils. These pits are therefore used for de-silting operations since volume also means low velocity movement. The side walls of the pits are being re-structured to form gently sloping bank walls to increase the percolation rates and subsequent benefit while clarifying the water.

Burris Pit and Santiago Creek Recharge Facilities: The terminus point for the SAR diversions is Burris Pit. This deep pit, historically used for infiltration, is used as a supply pond for the Santiago Creek Recharge System pumping plant installed in the 1990's. A 235 cfs intake and pumping plant moves the water from Burris Pit, 4.5 miles south through a 66-inch pipeline, to Santiago Creek. At this location, the water is placed in three deep quarry pits and released down a stretch of Santiago Creek. Outflows from the Santa Ana Mountains in Santiago Creek are minimal (about 350 AF per year).

To increase percolation rates in the Santiago Pits, the near vertical side walls were reconstructed to angled slopes. Fill was placed and compacted from bottom up to provide stable slopes. Local quarry aggregate and disturbed material was used to re-structure the pits into effective infiltration basins.

Operation Data: Each basin in the string of channels acts as a clarifier, releasing progressively cleaner water to the next. This results in higher percolation rates in the final basins. The system is controlled by a SCADA system to track dam statuses, inflate/deflate dam(s), monitor SAR flows, meter diversion flows, meter by-pass return flows, control the maze of valves and sluice gates, and operate the pumping systems in unity with the percolation rates at the receiving facilities. The control allows selective diversion of flows based upon the quality of the water in the SAR. During peak flooding events, or after major mountain fires, the intakes are closed and the turbid flows are allowed to proceed to the ocean. The dams are deflated to reduce sediment trapping. When the water quality improves, the diversion intakes are re-activated. As the flow diminishes, the dams are inflated to maintain a pond and hydrostatic head to drive the recharge system. Future improvements will include the installation of in-line turbidity monitoring equipment integrated with the SCADA system for tracking the SAR quality and making diversion open/close decisions.

The bulk of the recharge facility depends on gravity fed supplies from the de-silting pond at the upstream dam diversion. This basin becomes a critical link in the operation chain, making it difficult to take this pond out of service for cleaning. The agency developed a system to continually clean the basin with an automated dredge, while keeping the basin in operation. The dredge is a cable/winch guided barge with a control system that progressively moves the barge over the length and with of the basin. The unit excavates 10-12 inches into the basin base with a beater bar, pumps off the suspended silt, and allows the sand to settle back to the channel base. The silt slurry is piped to adjacent drying basins. The dried debris is hauled to the sand wash plant for recovery of the sand from the silt. The silt is disposed.

The operation of the Warner off-channel deep basins is a chain-linked system. Taking any basin out of service could result in the lower elevation basins going dry as well. The Carbon Creek and Santiago Creek basins are plumbed with diversion piping allowing selective use of basins. The in-channel basins leading to Burris pit have proven difficult to take out of service for cleaning.

Basin cleaning operations may as frequent as twice a year. This poses other issues with basins co-used for public use. Minimum water levels must be maintained for boating activity. Frequent draining of basins does not allow the fish to grow to larger sizes. And in contrast, the fish detritus and public waste creates higher clogging rates, requiring more frequent basin cleaning. The fish alone represent a potential high rehab cost. As a basin is drained, the fish die and quickly rot, creating a fly and odor issue. The material must be removed like a hazardous waste and disposed of at an acceptable location in Arizona. An innovative solution is to lower the basins to minimal fish survival levels, about 12-inches, and let the local birds feast on the fish. They will devour the live fish in less than a week, relieving the agency of the cleanup step.

Under basin rehabilitation, the basin floor ands walls are excavated a few inches down to remove the surface silt and debris. This is accomplished with heavy equipment. In the next step, cleaned sand is brought back to make up for the material loss and the basin base and walls are re-contoured with heavy equipment. The process of draining, drying, scrapping, and reforming takes about a month.

OCWD has developed some exotic Basin Cleaning Vehicles. These remote devices, about the side of a back-hoe, can operate partially submerged in the shallower basins. The unit roams the basin floor in a systematic pattern. It uses a roto-tiller type of head to break up the floor and suspend the silt. The silt is pumped off and sand is returned to the basin floor. The OCWD currently operates three automated basin cleaning vehicles with future plans for newer versions.

Future operational adjustments may include operating the deeper basins in the shallow mode. The reduced basin water levels will provide greater percolation rates and allow cleaning during operation using automated basin cleaning vehicles.

Sand recycling is a critical issue in the operation of the basins. After 40 years of operation, the gradual loss of material through rehab could have produced tremendous scours. To prevent this, a small sand wash plant is used to recover as much original sand as possible. The agency has plans to build a large scale sand handling facility to make this end of the program more efficient.

With over 1,200 acres of infiltration pond, insect control becomes an operational issue. The most significant pest has been the Midge Fly. They swarm together in small clouds of insects. The OCWD combats this with a routine cloud spraying operation. They use a non-toxic substance that globs to the wings leaving the insects unable to fly. Mosquito control has not been an issue since most all water is kept in motion.

Performance Evaluation: The OCWD, in conjunction with the USGS and other agencies, maintains a water level monitoring network to enable modeling of the recharge impact on the select major aquifer layers. OCWD has installed single point and multi-point monitoring wells to fill in critical data. The data indicates that the recharge program has had a significant benefit to the north and east side of the basin (Figure 3). The central basin remains unchanged, and the western side of the basin has declined 1

over 60 feet. The water recharged in the un-confined basin section on the east is not traveling fast enough to the pumping centers in the western basin. In order to make up for the slow aquifer travel time, the agency is looking to expand efforts through direct injection in the central and western portions of the basin. With groundwater pumping (by others) increasing almost three-fold over the last 40-years, the agency was able to hold most of the basin in balance. Recognizing the hydraulic impacts by recharge allows better modeling for the best location to place the 70 mgd of highly treated effluent becoming available in 2007.

OCWD does not pump water from the groundwater basin, avoiding recovery sampling programs. The surface water is naturally infiltrated into the aquifer with no chlorine or chemical additives, relieving a lot of injection regulatory water quality monitoring requirements. The OCWD must ensure the water has a minimum 6-month residence time in the ground before it is pumped out for domestic use. The agency has proven to the California Regional Water Quality Board through the use of iodine tracer studies, that their infiltrated water indeed has the required residence time in the ground before detection at the closest producing wells.

Problems Identified, Lessons Learned, and Recommended Operational Practices: Key lessons learned for the basin infiltration system are:
- The water utility does not deliver the water through pipes to the customers, it places the water in an underground reservoir for the user to removal at a location of their desire through their own reservoir connection – a well.
- Analyzing drill logs and map the confined and un-confined sections of the basin. This will allow definition of the target area for infiltration recharge facilities. Land acquisition should include: the largest flood plane area possible around creeks and rivers; adjacent and tributary creek beds; recreation lake opportunities; flood control detention basins; and old quarry pits.
- Separating the river bed flood-plane into two channels allows normal and controlled release flood flow down one half of the river bed, while the other half is being groomed for high percolation rate recharge.
- The natural, or primary, channel "armors" its-self through seasonal erosion of silt and sand, leaving rocks and gravel behind. Silt clogs the areas between the rocks resulting in low percolation rates. This type of riverbed is difficult to clean. All the infiltration basins are groomed to remove the rocks leaving a sand bed. The highest percolation rates will be achieved in the sand.
- Installation of an inflatable dam in the main river supports the operation by: impounding the river to increase surface area contact and in-channel recharge rates; divert water to a side channel and basin system; and provide the driving head for the side channel and basin system. The dam can allow overflow down the river bed, or be deflated during flood events to prevent the trapping of sediment.
- The development of automated trash racks to remove "urban" trash form the intake grates maintains a consistent operation flow. Trash clogging the intake grates will reduce the head on the recharge system, forcing flow over the dam. The chain and spike system routinely pulls the vegetation, plastic bottles, and plastic bags off the grate and deposits it in a drying area.

- The channel and de-silting basins for each diversion are critical components, required for any part of the system to operate. Two such structures should be planned at each diversion to allow maintenance of this piece of the system without having to take the entire system off-line.
- The agency has been progressive in designing automated basin cleaning vehicles. These machines clean the basin floor while the facility is maintained in service.
- The use of "T" diversion levies provides the highest percolation rates. The torturous path allows directing the shallow flow across the entire surface area. The increased velocity prevents deposition of silt. This results in higher percolation rates with lower clogging rates per operation time.
- Bank filtration to subsurface collector pipes may provide an alternate solution to de-silt the water to a higher quality than through the use of settling basins. The high quality water could then be piped to off-channel basins. The superior quality water would reduce the annual clogging and cleaning frequency of the basin.
- In a surface recharge basin design, the chain linking of the basins allows for progressively cleaner water moving to the final basin. This results in the highest overall system recharge performance. Diversion channels or pipes should be installed for each basin to allow selective removal of basins for maintenance without disturbing the chain-filtration system.
- Basins should not be designed as dead end sinks. A flow must be maintained through the ponds at all times to keep the water alive. This may require outflow from the final facility to a discharge sink, an open dry creek bed stretch, or returned to the native river.
- Deep basins can be co-used for public recreation use. This has drawbacks: the basin level must be maintained for boating; the fish detritus and other bacteriological organisms supporting a healthy fish environment increase the basin clogging rate; and draining the basin for maintenance requires costly fish kills, removal, and re-stocking.
- Flood control basins should be planned for co-use as infiltration basins as well. This way precious land in creek paths could be used for recharge facilities most of the year. During storm season, the recharge ponds can be drained to allow increased storm capture.
- The horizontal recharge rate in pit walls is low. Re-structuring pit walls to a gentle slope will increase the surface area for downward infiltration and subsequent percolation rates for the basin.
- Historic deep pits used for recharge make good de-silting basins. The volume and low flow allow residence time to settle finer material.
- The system should be designed to allow emergency closure of the main river intakes at such times the water quality is un-acceptable. During storm events, the sediment load in the water may not settle in the conventional de-silting system resulting in clogging of connecting basins. These flows must be blocked, either by: water quality sensors and automated diversion gate control; 24-hour operator observation and control; or a midnight call that a storm is developing and maybe someone should close the gates.
- Designing the basins to operate off of river level and gravity feed may result in limitations. Can only use facility locations down stream and generally in that

valley. To divert excess flows to other tributaries, off-channel basins, or upstream locations, pumping plants and pipelines would be necessary and should be considered in the master plan.

- For insect prevention, deeper water that is moving has lower potential for larvae development. Shallow water with slow movement increases the fly and mosquito larvae potential.
- The managed aquifer recharge to the basin must be spread out. Placing all the facilities in an isolated area may result in water stuck in slow movement in the aquifer, not fully balancing the effects of pumping. Recharge facilities may need to be in close hydraulic connection with the major pumping centers if possible. This may require piping the source water considerable distances for infiltration or direct injection at the location of need in the basin.

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AWWARF Case Study, Scottsdale Water Campus, Scottsdale, Arizona

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Prepared for: AWWA Research Foundation

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Introduction

The City of Scottsdale is located in the northern part of the Salt River Valley in Central Arizona. The city adjoins Phoenix to the west in this Senora Desert environment. The boundaries encompass approximately 185 square miles with the Continental Mountains and McDowell Mountains on the north and east, the Salt River on the south, and the Phoenix Mountains to the west. The current water supply is derived from a combination of local groundwater, surface water from the Salt River Project, and surface water from the Central Arizona Project aqueduct. Overdrafting of groundwater between 1950 and 1980 resulted in the State of Arizona enacting the 1980 Groundwater Management Act. Provisions in this regulation require cities to achieve safe yield by the year 2025. To meet this requirement, the city must acquire or develop new water sources. The Water Campus Project will provide the city with the ability to treat municipal sewage effluent to potable water quality as well as to recharge and store the product water within the city boundaries. (Small, 1993)

The Water Campus is a new state of the art water and wastewater treatment complex located on a 141-acre parcel. Construction and baseline testing started in 1993 with facility operational testing being conducted in late 1998. It includes a 54 MGD surface water treatment plant to treat imported waters for potable delivery; a 12 MGD water reclamation plant producing tertiary treated water for reuse at 17 golf courses; and a 10 MGD advanced water treatment plant including microfiltration and reverse osmosis producing water for aquifer recharge at 27 vadose zone injection wells. A total of 28 additional vadose zone wells are installed to capture discharge from the reclamation plant during potential over-runs. At the heart of the complex is a major indirect potable reuse program with an annual groundwater recharge target of 150,000 AF.

The plant relies on a series of three sewer pump stations to provide the flows. Effluent from the plant meets standards set by the Arizona Department of Environmental Quality, which allow reuse for open access irrigation. The indirect potable reuse portion of the complex operates under an Aquifer Protection Permit and an Air Quality Emissions Permit, regulated by three different agencies; Arizona Department of Environmental Quality, Arizona Department of Water Resources, and Maricopa County Environmental Department. A considerable amount of data accumulation and reporting is necessary to maintain compliance with each of these permits. (Nunez, 1999)

Site Setting - Hydrogeologic Framework

The site is approximately two miles west of the McDonnal Mountains within the Paradise Valley Basin. The Paradise Valley Basin is an elongated, northwest trending topographic depression formed by a series of block faults between the Phoenix Mountains on the west and the McDonnal Mountains on the east. This valley is part of the southern Basin and Range Geologic Province.

The Paradise Valley Aquifer is composed of three stratigraphic units. The Upper Alluvial Unit is found at the surface of the throughout the basin and consists of unconsolidated sands and gravels with thickness as great as 600 feet. This is the target aquifer for the recharge operations. The Middle Fine Grained Unit consists of clays, silts and evaporite deposits. The unit varies in thickness to as much as 800 feet in the

southern part of the valley. The Lower Conglomerate Unit consists of semi-consolidated sands, gravels, and some stringers of basalt. The unit varies in thickness up to 900 feet. Figure 1 displays a generalized cross-section showing the conceptual geology and sedimentary sequences of the Paradise Valley Aquifer.
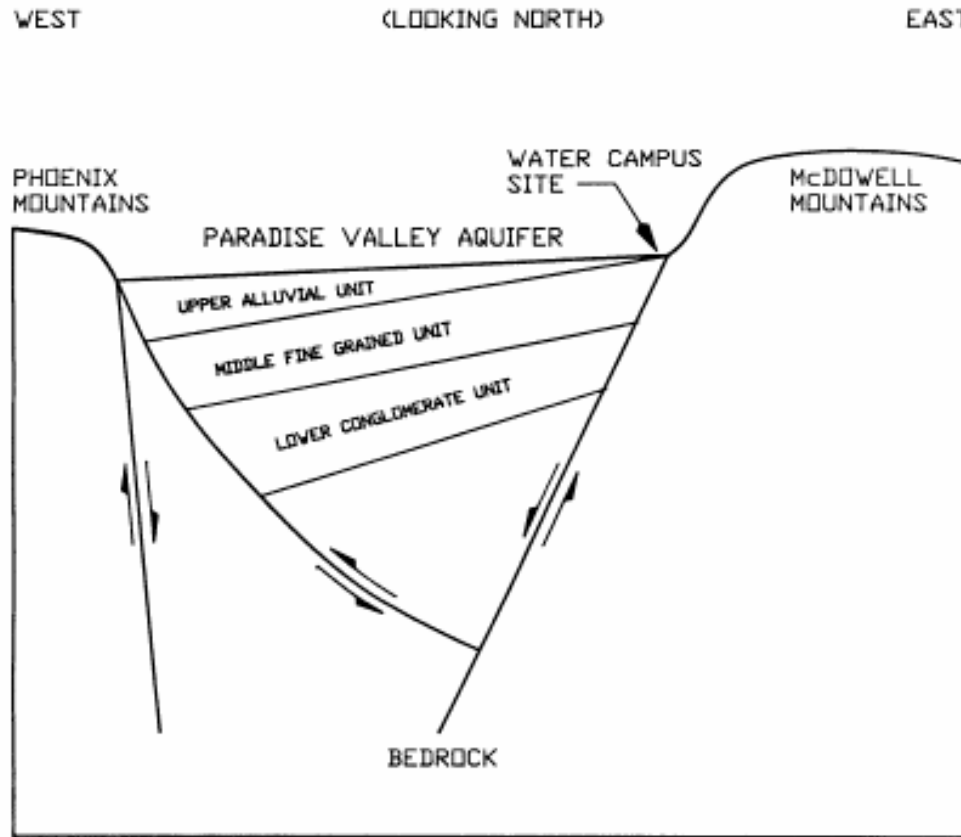
Figure 1: Conceptual Subsurface Geology and Sedimentary Sequence
(Small 1993)

The depth to groundwater in the Scottsdale area ranges from less than 200 feet in the south to as much as 900 feet in the north, with the depth below the site exceeding 600 feet. The majority of the groundwater that is being pumped within the city boundaries is from the southern portion, where water levers are relatively shallow. The city pumps an average of slightly more than 16,000 AF per year for use in its municipal system. (Nunez, 1999)

Source Water Characteristics

In balancing water resources to provide safe yield by 2025, the City of Scottsdale recognized that wastewater effluent was a sustainable commodity that paced growth. In developing the Water Campus, city leaders were innovative in placing surface water treatment, wastewater treatment and advanced water treatment components into the same facility.   The operations ensures that all water entering the city, or attempting the leave

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the city boundary, is treated and placed into beneficial use. Treated waste water is used for golf course turf irrigation or further treated for recharge through wells, providing the link for an indirect potable reuse program.

Water is available for recharge after the demands of the golf courses is met with the tertiary treated effluent. The surpluses are further treated with advanced methods to produce ultra-pure water for recharge at the primary injection wells. Plant outflows are routed to a storage tank prior to injection. The tank buffers the injection wells from plant back-flushes and low source availability periods, allowing continuous recharge operation.

Flow availability for treatment and recharge can vary greatly during the day or by the season. Higher volumes of effluent is available during the winter months whereas during the summer days, the effluent flow will decline and stop for short periods. These low influx periods cause havoc with the treatment trains, resulting in elevated turbidity in the output.

During periods of low plant output, the facility has been constructed with an additional advanced treatment leg to prepare surface water imports for recharge. This water can be added into the recharge program to either sustain flows during low wastewater input periods or to utilize surface water system surpluses through underground storage.

The emergency recharge wells will only be operated during periods of excess flow. This occurs when the reclamation plant output exceeds the golf course demand and advanced treatment plant ability. Until the inflows can be reduced and balanced, the excess tertiary treated water is routed to these emergency wells.

Water Quality

The quality of water used for recharge is among of the highest noted. The water is virtually purified through the treatment processes prior to recharge. The sewer effluent is separated sending the solid mass further down the line to the main treatment facility on the southwest side of the Phoenix Valley. The secondary fluids are processed at the Water Campus to tertiary standards using conventional methods. The tertiary treated effluent water is chlorinated and further treated with advanced methods creating potable quality water for recharge into the groundwater aquifer for storage.

Figure 2: Microfiltration and Reverse Osmosis Trains (Pugh, 2006)

The advanced water treatment plant includes in order: a 500 micron strainer, 18 continuous microfiltration units and 10, 1 MGD, 3 stage reverse osmosis (RO) trains set in a 24-10-5 array with a target of 85% recovery. The microfiltration unites are staged in three "6-packs" with the primary function of two clusters to pre-treat the wastewater effluent prior to reverse osmosis. The additional microfiltration cluster is for filtering imported surface water prior to recharge into the aquifer for storage. Following the RO train, the wastewater is routed to degassifier towers to reduce the carbon dioxide and further routed through lime feed silos for RO permeate pH stabilization. The water is routed to a storage tank prior to injection into the vadose recharge wells.

Feasibility and Design Studies

The City of Scottsdale has evaluated an innovative way of recharging the aquifer through shallow vadose zone wells. The agency completed nearly 5 years of tests and studies prior to developing a full scale program. Early feasibility studies included the installation of four soil borings to a depth of 350 feet and one monitoring well to 540 feet deep, all fully penetrated the Upper Alluvial Unit targeted for recharge, though none intercepted the water table. Drill cutting samples, geophysical logs, and borehole tele-viewers indicated a complex interbedding beneath the facility. Correlation over the 800 foot triangular area for the monitoring wells and soil borings showed poor association of the beds. With multiple small lenticular beds of impermeable silts and clays, the recharge water is expected to perch and move laterally to seek a deeper position. Placement of this water below the surface layers would ensure this travel is generally downward and not sideways resulting in surface seepage down slope of the facility.
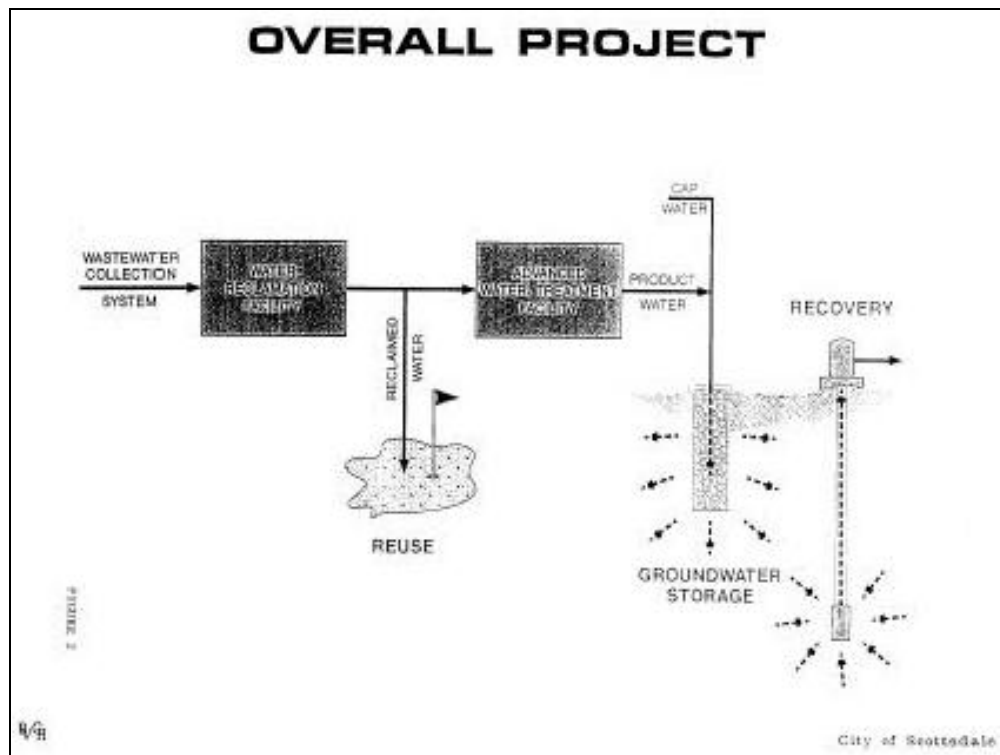
Figure 3: Conceptual Design Philosophy of the Water Campus (Small 1993)

Multiple dry well designs were evaluated and tested. The initial two wells were drilled 48-inches and telescoped to 30 inches in diameter to a depth of 150 feet. One was cased with a 16-inch casing with a perforated fabric wrapped screen and a thick gravel envelope. The other design utilized a 6-inch PVC casing with factory slots and a thick gravel envelope. Additional design considerations include a gravel filled borehole equipped with a 20 foot eductor pipe, a removable filter sock filled with gravel placed into an open or cased borehole, and a borehole cased to bottom with screens some distance above the base. The preliminary testing aided in refining the final facility well design and provided insight to operational issues such as wellbore clogging, maintenance needs, and recharge equipping methods.

The recharge wells were installed in clusters for a current total of 55 wells. This allowed testing and improvement during the phases. Total build-out of the system could allow as many as several hundred recharge wells spaced 50 feet apart. The wells were recharge tested with treated surface water and filtered surface water for periods of 30 to 45 days. The final plant design treats the surface water through advanced microfiltration before injection.

Facility and Operational Data

In the final facility design, the vadose zone recharge wells were drilled to a depth of 180 feet with groundwater at approximately 600 feet below surface. The aquifer below the site is dry with the projected recharge flow to be to the southwest of the site to saturated sections of the Upper Aquifer Unit and shallower water levels. Three deep groundwater penetrating monitoring wells at depths of 900 feet are located in the potential directions of vadose flow. Depending on the receiving water quality, two different types of recharge wells were designed and installed.

Figure 4: Standard Recharge Wellhead Comnfiguration (Pugh, 2006)

The primary, or standard recharge well is used for daily recharge operations. This well type is drilled at 48-inches in diameter, equipped with a 10-inch PVC casing to base, and gravel enveloped. The well screens are located some distance off the base to allow for an injection chamber at the base of the well. The 6-inch injection tubes are inserted to near the base and equipped with either an orifice or a flow control valve. Injected water

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then rises to the screens, reducing turbulence in the well casing and screen area. Each well is equipped with a flow meter, water level transducer, and surface flow control valves. A total of 22 of the 27 wells are equipped with fixed orifices and five are equipped with variable orifice flow control valves. The injection tube maintains a positive pressure at all times during operation at approximately 300 to 450 gpm per well. During operation, the water level rise and flow rate is monitored by the operating system. As the wells reduce their recharge ability, a water level alarm limit of 70-80 feet below the surface is set to take the well off-line and rotate operation to some other well. The wells are operated in clusters of 9 with rotation between all 27 primary wells.
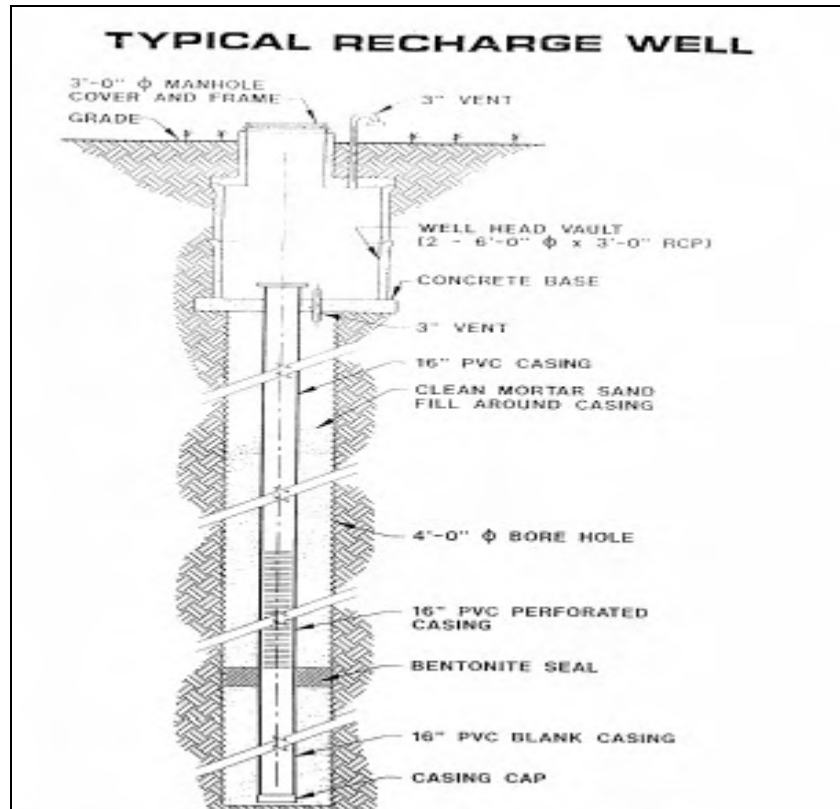
Figure 5: Typical Recharge Well Design with 10-Inch Casing

The emergency wells are a simpler design. A 48-inch diameter bore hole was drilled to 180 feet below surface. A fiber "sock" is then inserted in the hole and filled with ½-inch to ¾-inch gravel. The fiber provides a barrier between the soil and the gravel. A 6-inch PVC inductor penetrates the gravel pack to a depth of 20 feet. Three 2-inch PVC vent tubes are placed to near the base of the well to vent trapped air under startup operation. The facility has 28 emergency type vadose recharge wells. These wells are only placed into temporary service when the output of the treatment plant is hydraulically overloaded. Under these conditions tertiary effluent is diverted to the emergency wells to reduce the hydraulic loading. Each of the emergency wells was flow tested averaging 300 gpm while not experiencing excessive mounding. Since the intent of the emergency wells us strictly under adverse conditions and expected to be rare,

automation and control is minimal. For rehabilitation, the air release tubes and socks can be removed form the wells for cleaning and re-installed.

Extensive testing was conducted during the commissioning of the wells. The biggest discovery was the need to modify the size of the fixed orifice plates on 17 of the 22 wells. Changing the orifice plates, to a certain extent, was a hit and miss process. If a well received in excess of 450 gpm and the water levels climbed to less than 70-80 feet within land surface, a small orifice size was installed. With frequent orifice changes, the agency developed a simpler approach. New orifice plates were fabricated and installed with a basic fishing hook design and 200 feet of cable. New plates were set directly on top of the existing orifice plates. Testing suggested a substantial impact on adjacent wells when operating side by side. For this reason, the wells are operated in an "every other one" philosophy whenever possible.

The basic dry well technology, modified for site-specific hydrogeological considerations, was selected as the recharge strategy of choice for cost and land availability reasons. Dry wells constructed and fully equipped to depths of 150 feet cost $14,000 to $16,000 per well whereas, deep injection wells of equivalent capacity are estimated to cost ten to fifteen times as much. Therefore more wells can be constructed initially with the same funding and individual dry wells can be replaced more economically should it become necessary due to clogging or diminished recharge rates. With regard to land use, dry wells and associated piping and operational facilities can easily be placed in narrow linear corridors which already exist along road right-of-ways. This minimal land use requirement is in contrast to needs of infiltration basins for recharge through the unsaturated zone.

Problems Identified, Lessons Learned and Recommended Operational Practices

Some of the key lessons learned from the evaluation of the Scottsdale Water Campus and vadose zone recharge program are noted below.

- Nearly 5 years of pilot studies on filtration methods, dry-well design and operation led to the final design of the water campus recharge component.
- To ensure percolation of the recharge water, the water needed to be placed below the surface layers and soils.
- Recharge to the aquifer is monitored by three water table penetrating wells located in the three potential flow directions.
- Two injection well designs are used based upon the source water quality. The wells using polished water are cased with PVC where as the emergency plant overflow wells receiving higher particulate matter are constructed with socks filled with gravel pack. The socks are removed for cleaning and re-inserted into the wells.
- In the 28 emergency wells, the injection tube extends 20 feet into the gravel with vent pipes plumbed to within 40 feet of the 180 foot base.
- In the primary vadose zone wells, the injection tubes extend into the well through the center of the 10 inch casing to near the 180 foot base. The tubes are fitted with designed orifices or automated borehole flow control valves to maintain a positive back-pressure under operation.

- Flow testing and design of the emergency wells resulted in average recharge rates of 300 gpm with maximum mounds rising to 70 feet below surface. Flow testing and design of the primary wells provided average rates up to 450 gpm with water level rises to within 70-80 feet of surface.
- During peak wet weather conditions when the wastewater plant is overloaded, the effluent is diverted to the emergency wells to reduce the loading on the advanced wastewater treatment plant.
- The emergency wells operate in clusters of 9 at 300 gpm. Other clusters are rotated into use as necessary.
- Fixed orifice plates required frequent changing as water mounds develop. To avoid rig and labor costs for removing and re-installing the recharge tubing, wire line techniques were developed to set-in a smaller orifice plate.
- Operation of the vadose zone wells indicated a performance impact of wells operating side by side. Since only a fraction of the wells are used at any one time, the optimum operational is to use every other well at a minimum.
- The sustainability of the source is very good with growth spurring additional wastewater supplies in the future. Having the surface water plant for treating imported Colorado River water on the same property offers an additional source of recharge supply. The plant is designed with a leg in the treatment train specifically to treat surface water for vadose well recharge.

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AWWARF Case Study, Calleguas ASR Project, Thousand Oaks, California

Prepared by: LJHB Partners LC

Prepared for: ASR Systems LLC Team and the AWWA Research Foundation

March 2006

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Introduction

The Las Posas Basin ASR project is located in southern California near Thousand Oaks within the isolated Las Posas groundwater basin. The Las Posas Basin ASR project is a joint-venture project with costs and responsibility split 50-50 between the Metropolitan Water District of Southern California (MWD) and Calleguas Muncipal Water District (Calleguas). MWD supplies water to 26 cities and water districts serving about 17 million people in Los Angeles, Orange, San Diego, Riverside, San Bernardino, and Ventura counties (Susan Mulligan, 2006). MWD's sources of supply are the Colorado River and the California State Water Project. Calleguas supplies water to 22 water agencies and cities serving about 550,000 customers in southern Ventura County including the cities of Oxnard, Camarillo, Thousand Oaks, and Simi Valley (Susan Mulligan, 2006). Investigations have shown that the Las Posas groundwater basin has available storage capacity of approximately 300,000 acre-feet, primarily due to historic groundwater production (Pyne, 2006). ASR investigations were initiated during 1990 to determine the feasibility of using ASR to emergency, seasonal, and long-term drought storage needs (Pyne, 2006). A map of the Calleguas water distribution system and service area is shown on Figure 1.
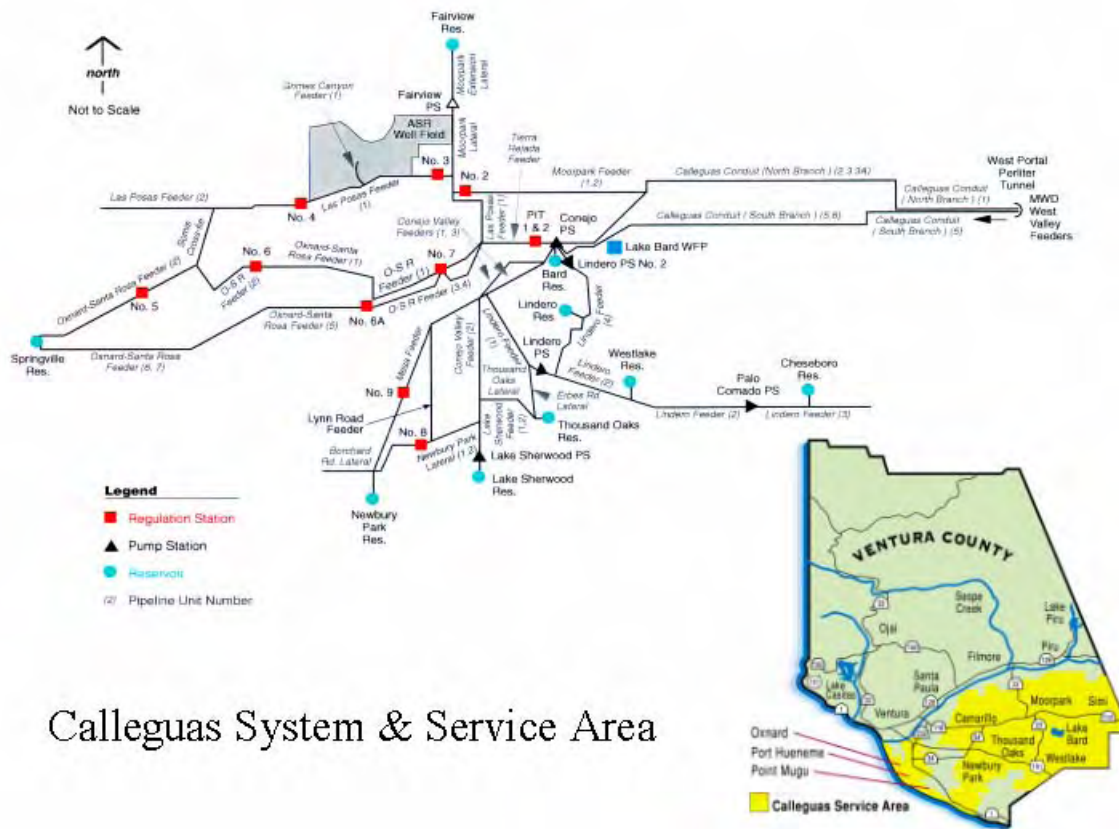
Figure 1 Calleguas system and service area (Source: Susan Mulligan Calleguas)

MWD identified a need for the project in order to have a reliable dry season supply while extracting maximum use of existing pipeline and aqueduct infrastructure. Calleguas identified a need for the project in order to ensure system redundancy as well as provide for emergency water supply since the current system was susceptible to long outages (George Mulligan, 2006). Further impetus for the project was provided by the 1994 Northridge earthquake which caused extensive damage to Calleguas pipelines resulting in up to six weeks of partial system outages (Susan Mulligan, 2006).

The Las Posas ASR project currently consists of 18 wells capable of recharging up 48 MGD and recovering 64 MGD (Kristine McCaffrey, 2006). One additional well, the Fairview ASR well, is located in another location within the Calleguas distribution system. Planning is underway to further expand the system to 26 wells over the next two years. Currently, the 18 wells are split between the older well field # 1 and the newer well field # 2. This case study report is based upon existing project literature and results of a site visit and tour conducted on March 6, 2006. Those present for some or all of the site visit and tour included:

- Chris J. Brown, LJHB Partners LC
- Tom Mooris, ASR Systems LLC
- Susan Mulligan, Calleguas Manager of Engineering
- George Mulligan, Calleguas Manager of Operation and Maintenance
- Kristine McCaffrey, Calleguas Project Manager
- Tony Goff, Calleguas Operations Supervisor & Compliance Officer
- Don Kendall, Calleguas Superintendent

The site visit focused upon the site history, operating practices, engineering data, water quality issues, operational problems, innovative site practices, and lessons learned. Results of the site visit are discussed throughout this report and referenced as "personal communications" with one of the attendees.

Site Setting - Hydrogeologic Framework

The ASR wells are constructed in a hydrogeologic basin filled with alternating layers of marine sands, marine gravels, and marine silts/clays (Pyne, 1995). The basin contains sediments of Pliocene to Recent Age, including two primary confined aquifers that are separated by 50 to 100 feet of low-permeability units (Pyne, 2006). The Fox Canyon Aquifer is the shallow most aquifer and is the primary storage zone for the ASR wells. The Fox Canyon Aquifer consists of a confined aquifer composed of 200 to 400 feet of marine sand and non-marine sand and gravel. The aquifer is highly productive with aquifer transmissivities ranging from 600 to 20,000 ft^2/day. Pyne (1995) reports an average value from initial site testing of 19,385 ft^2/day along with a storage coefficient of 4×10^{-6}. Pyne (1995) also reports an aquifer porosity of 23% (as determined from geotechnical testing of soil cores) and a dispersivity of 22 feet. The deeper aquifer in the basin is the Grimes Canyon aquifer, which consists of marine sands with minor gravel deposits ranging from 100 to 400 feet thick (Pyne, 2006). Both of the aquifers have small recharge zones located along narrow margins of the basin walls amounting to almost 6%

of the recharge into the basin (CH2M Hill, 1993). Figure 2.x provides a geologic cross section of the entire Lost Posas Basin.
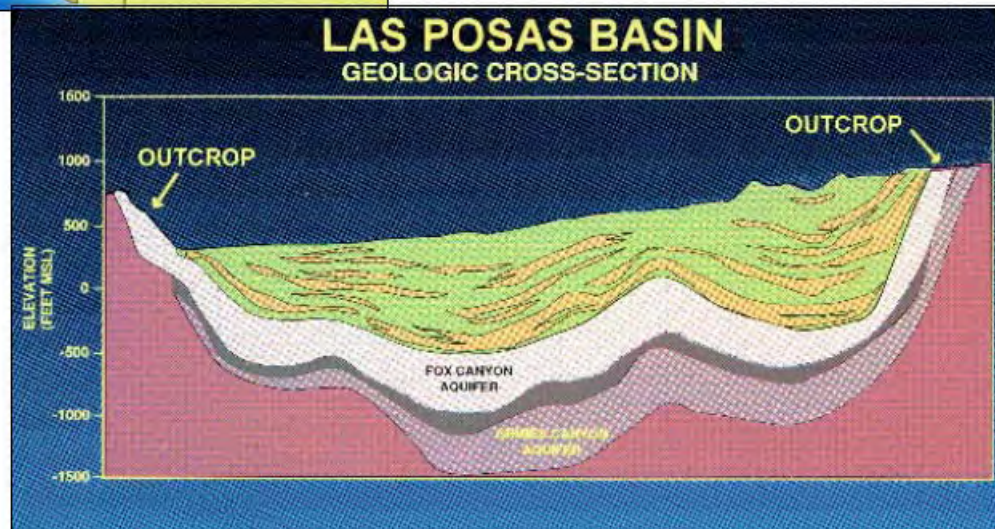
Figure 2 Las Posas groundwater basin location and geologic cross-section (Source: Calleguas)

More significant recharge (47% of the total) comes through the undifferentiated surface sediments into the Fox Canyon Aquifer (CH2M Hill, 1993). Recharge also is derived from the Arroyo Las Posas (25% of the total) as well as inflow from the Oxnard Plain Basin (22% of the total). A calibrated numerical model of the basin developed by CH2M Hill estimated that the average annual recharge was approximately 16,825 acre-feet while calculated discharges were approximately 26,850 resulting in a net overdraft of the aquifer of almost 10,000 acre-feet per year (CH2M Hill, 1993). Hydrographs presenting water level data in the basin show a general decline from the 1970s to the 1990s, further supporting the aquifer overdraft concept (CH2M Hill, 1993). As urbanization continues within the basin, this pattern is expected to continue.

Source Water Characteristics

The Calleguas water supply is derived from the west branch of the State Water Project where it is treated in the MWD Jensen filtration water treatment plant and conveyed by pipeline through the San Fernando Valley (Susan Mulligan, 2006). The water is then transmitted through the Santa Susana Mountains via a one-mile long tunnel

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where it then enters the Calleguas distribution system. The Calleguas distribution system consists of 120 miles of large-diameter pipeline connected to various surface reservoirs and water filtration plants (Susan Mulligan, 2006). The average annual water demand approaches 120,000 acre-feet. The imported source water is also typically colder than ambient surface water in the basin. Data from 2004-2005 reveal that the average source water temperature measured at the Jensen filtration plant was 16 degrees Celsius. Groundwater in the basin has an ambient temperature of approximately 25 degrees. This temperature difference is significant enough to affect the viscosity of the water as well as the "apparent" transmissivity of the aquifer.

Water Quality

The source water quality for the Calleguas ASR wells is fairly consistent as it is discharged from the Jensen filtration plant. Table 1.x displays relevant source water quality characteristics from 1990 as reported by Pyne (2006) and MWD (2005). Ambient groundwater quality is also shown on Table 1. Major water quality differences between the source water and groundwater are noted for calcium, bicarbonate, sulfate, chloride, hardness, alkalinity, nitrate, iron, manganese, and total THMs. Chloride and nitrate in

Table 1. Water quality characteristics measured at Jensen filtration plant.

Parameter	Units	8/1/90	Average for 2004-2005	Native GW Well 97, 12/11/90
Calcium	mg/l	25	27	68
Magnesium	mg/l	16	13.5	13
Sodium	mg/l	76	52	36
Potassium	mg/l	4	2.8	3
Carbonate	mg/l	2	0	0
Bicarbonate	mg/l	99	102	189
Sulfate	mg/l	53	64	121
Chloride	mg/l	111	61	13
TDS	mg/l	355	288	375
Hardness	mg/l	128	123	222
Alkalinity	mg/l	85	84	155
Color	TON	15	3 #	10
Nitrate	mg/l	3.15	2.70	0.18
Fluoride	mg/l	0.12	0.15	0.25
Boron	mg/l	0.158 *	0.180	0.085
Iron	mg/l	0 *	NR	0.242
Manganese	mg/l	0 *	NR	0.043
Ammonia	mg/l	0.19 *	NR	0.17
Total THMs	ug/l	74 *	NR	0
Total Organic Carbon (TOC)	mg/l	2.9 *	2.37	1.1
pH		8.40	8.29	7.52
Silica	mg/l	15.1	13.8	37.5

Notes: NR means not reported; * means recorded at Vault-1 downstream ; # color units

particular could be utilized as injection tracers in order to monitor water quality changes over time throughout the aquifer.

The recharge water quality is characterized by moderately high TOC ranging from 2.37 to 2.9 mg/l and moderate levels of total suspended solids (TSS) of 3 mg/l. These values are moderate but may be high enough to result in aquifer clogging and biological growth around the ASR well screens. In addition to constituents displayed on the table, the ambient groundwater is also characterized as having moderate amounts of radionuclides including gross alpha and radon.

Feasibility and Design Studies

Feasibility and design studies at the site have been thorough and included detailed hydrogeolgic investigations, geochemical studies, numerical modeling, water availability studies, and ASR pilot testing.

A series of recovery and injection tests have been completed at the project. First, Ventura County Water Works existing well # 97 was tested using both injection and recovery. Well # 97 is screened within the Fox Canyon aquifer, which is 450 thick at the site (Pyne, 2006). Initial testing revealed a well efficiency of 87 % as well as a specific capacity of approximately 23 gpm per foot. A second injection test cycle indicated a lower in-situ transmissivity of 44,000 gpd/feet that was 329% less than during cycle # 1. At the time, it was hypothesized that the cause of the reduction was a combination of well plugging, hydraulic interference from adjacent wells, and the 11 ^{0}C temperature differential between the source water and the ambient groundwater (Pyne, 2006). Recovery testing included recovery of 138% of the injection water to ensure ambient groundwater quality was reached as well as to restore well efficiency and specific capacity of 23 gpm/ft.

During storage of the cycle test water, TTHM concentrations were reduced and iron and manganese concentrations dropped also possibly indicating precipitation or some other geochemical reaction. Unfortunately, during recovery, iron and manganese concentrations still remained higher than desired by Calleguas. Therefore, Calleguas is considering additional treatment of the constituents (Calleguas, 2004). Lastly, moderate concentrations of radon were observed in some recovered water data.

ASR system design was aided by the use of computer models of the subsurface aquifer as well as the Calleguas distribution system. Both groundwater modeling simulations and hydraulic simulations of the entire system were performed in order to ensure a robust design. The instrumentation and control of the whole well system has progressed over time leading to a mostly automated system design for both well fields and the Fairview well. The wells themselves initially included a variety of pump types and sizes due to the heterogeneous nature of the aquifer storage zone. As a result of the different well designs, the well casing diameters range from 12 to 20 inches and the well capacities also range from 0.5 to 2.0 MGD. Recently, an effort has been undertaken by facility staff to further standardize instrumentation and control of key components (e.g., flow meters, valves, wells) in order to improve reliability and minimize down-time (George Mulligan, 2006).

Facility and Operational Data

The ASR well system is spread between 19 separate wells located in two distinct well fields and a stand-alone well at Fairview. The wells were constructed using stainless steel casing and each well is equipped to recharge through either the pump column or the well annulus around the pump column. All wells are equipped with vertical turbine pumps with motors ranging from 600 to 800 horsepower. The entire system can be remotely operated through a complex instrumentation system (Calleguas, 2004).

Both the distribution system and the ASR system are highly automated and controlled by a state-of-the-art SCADA system. Well field # 1 started as a more manually controlled system but has evolved to a full SCADA system. The newer well field # 2 has always utilized a SCADA system to manage its operations (George Mulligan, 2006). One important issued noted by facility staff is that the full SCADA system was designed and constructed by a trio of consultants/contractors. This has led to some system integration issues that must be handled by the onsite staff (George Mulligan, 2006). Due to concerns over terrorism at all water plants in the United States, security infrastructure has become more important. The Calleguas facility has invested in security fencing, alarms, and motion detectors (Kristine McCaffrey, 2006).

In general, operations follow an accepted operational plan developed by the facility staff in conjunction with various external consultants. The Fairview well is operated separately from the remaining 18 onsite ASR wells. These wells are operated in groups based upon observed performance. Four groups of wells are operated together in order of priority for recharge or recovery operations. The four operational groups include:

- Wells # 1, 2,3, 4, 17, 18 (priority 1)
- Wells # 6, 15, 16 (priority 2)
- Wells # 5, 10, 11, 12, 13, 14 (priority 3)
- Wells # 7, 8, 9 (priority 4)

In addition, each well within a particular group is utilized in a priority order for both recharge and recovery operations. For example, the first well operated for each of the four groups is 18, 16, 14, and 8. Operational data provided by Calleguas confirm this operational plan; each of these four wells has recharged at least 175 million gallons of MWD water. However, the data seems to also indicate that well group # 1 has gotten the most overall use with a combined recharge total of approximately two billion gallons out of the well field total of approximately five billion gallons (e.g., 40% of total recharge). If the Fairview recharge quantity is removed from the system total, the combined use of group # 1 wells represents almost 51% of the total recharge for the remaining 18 ASR wells. This pattern may not be sustainable over the long term.

Performance Evaluation

The Calleguas ASR system (including the Fairview ASR well) has posted a good performance record. Over a 13-year operational period, the ASR system has successfully stored approximately 5,216 million gallons of MWD water and recovered approximately 732 million gallons consisting of a mix of MWD water and native groundwater. This represents a net recovery utilization of only 14%, however, it should be noted that the system is mainly designed to help meet peak demands and for use during emergencies. As noted above, the distribution of the system recharge has been unequal and dominated by wells # 1, 2, 3, 4, 8, 9, 16, and 18. The remaining ASR wells have not been utilized regularly. Figure 3 displays the cumulative system performance over the 13-year period.

Calleguas ASR System
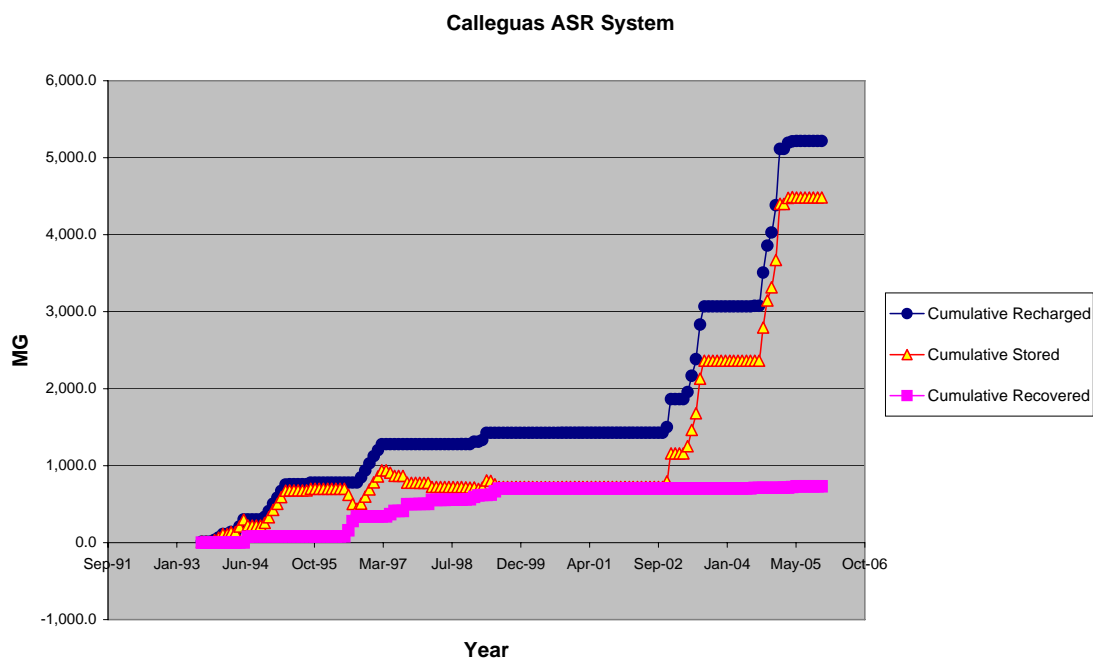
Figure 3 ASR Recharge and Recovery Volumes over project life

An economic analysis completed by the author estimated long-term unit costs of the project to be approximately $0.51 to $0.65 per 1,000 gallons recovered. The costs were estimated based upon standard water treatment costs, well costs, and pipeline costs at similar projects. Wolcott (2003) reports a value of $0.72 per 1,000 gallons recovered compared to a nearby dam project where the costs are 10 to 15 times higher per gallon. Calleguas staff tracks operational and maintenance costs routinely in order to get reimbursement from MWD. Projected costs for 2005 were approximately $1.6 million dollars. The distribution of the ASR O&M costs is shown on Figure 4.x. It is important to note that well maintenance and rehabilitation costs represented the largest fraction of the annual O&M costs followed by regulatory compliance costs (e.g., sampling, analytical, and labor).

Projected 2005 O&M Expenditures for Calleguas ASR Project
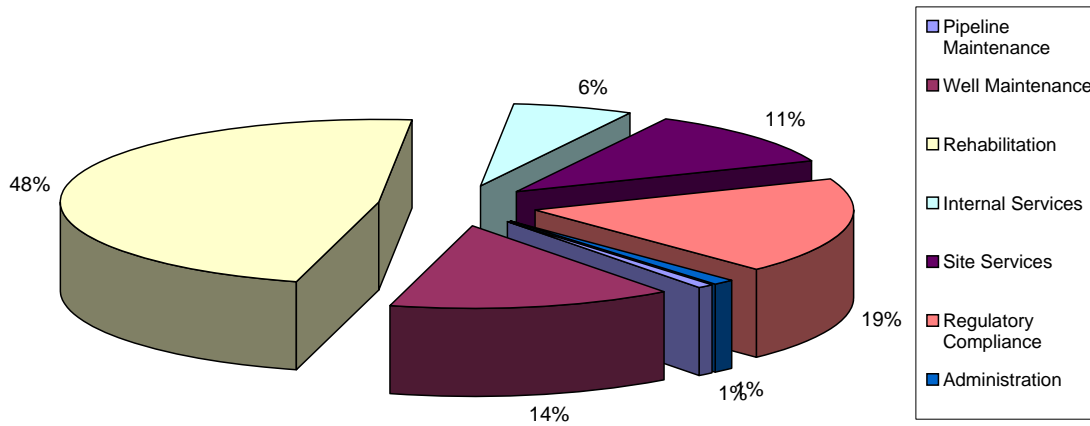
Total Estimated O&M Expenditures = $1,569,403

**Does not include system water treatment costs

Figure 4.x. Calleguas ASR O&M projected costs for 2005.

One interesting economic item to note is that each vertical turbine pump is designed to produce electricity as water is injected. According to Wolcott (2003), the amount of power produced is more than enough to run the project and excess power is sold back to the power grid. Discussions with site staff carried out during the AWWARF project site inspection indicated that the power generation feature is actually rather small and probably is not worth the original expense (George Mulligan, 2006).

Problems Identified, Lessons Learned and Recommended Operational Practices

The most significant problem at this project has been periodic well clogging. The ASR system is located within a portion of the distribution system that has little circulation (Pyne, 1995). Well clogging can be severe since the various supply pipelines and tunnels can get entrained with rust and sand over time without water circulation. In the past, entrainment of these materials has lead to major well clogging during recharge operations (Pyne, 1995). This well clogging mechanism was minimized with the incorporation of strainers within the wellhead. This innovation was developed by Calleguas in-house staff and is shown in figure 5.x below. A regular program of well re-development is usually required at the site to maintain high inflow rates. Recently, additional loss of well capacity has been noted at the facility (George Mulligan, 2006). Several possible mechanisms could explain the recent loss of well injection capacity. First, the entrainment of air is still possible due to the large column diameters and substantial well depths, even though Calleguas staff go to great lengths to ensure that all air is purged through air release valves and that a vacuum is maintained at all times during recharge. Second, although influent total suspended solids concentrations are low, clogging due to particle accumulation is still possible given the large recharge rates in use

at the Calleguas ASR system. More frequent backflushing events should help in alleviating this problem. In addition, site operations revealed that flushing the well and surrounding pipelines to waste for a short duration before recharge or recovery operations can reduce well clogging problems.

Staff at Calleguas has compiled a valuable list of operational best practices and lessons learned. Some of the key best practices are further illustrated in figure 5.x. The best practices recommended by Calleguas include:

- Be sure that you have the right to pump out the water stored
- Be sure to inform other groundwater users in the area of your intentions and potential impacts to ensure that you retain their trust
- Address all environmental issues properly
- If you have water-lubricated pumps, keep water flowing through them all of the time, otherwise, mineral deposits may form on the pump shaft contributing to overheating
- If water-cooled motors are utilized, separate that piping from the water lubrication piping
- Meter cooling and lubrication water flows as they may be significant
- Recover water to a single location. A ground storage reservoir is ideal for this application since it provides a single location for disinfection of recovered flows and for any other treatment process that may be required. Additional benefits include release of any air pumped from the wells, hydraulic equalization, and control of well operations
- Install a backup tube to allow manual water level measurements as a validation check for downhole pressure transducers
- Install a strainer on the recharge piping to keep debris from the distribution system from traveling down the well and plugging them

The Las Posas Basin ASR project administered by the Calleguas Muncipal Water District appears to have a bright future. Additional system expansions are in the planning phases to expand the water conveyance capacity as well as the ASR recharge and recovery capacity. The proposed expenditures under consideration are significant and will further expand the Calleguas system. Future expansion of the ASR system will also be considered as additional information and knowledge are gained through operation of the existing ASR wells.

(a) Install extra water level tubes

Best Operational Practices
at Calleguas

Motorized valve on air vac for annular space

Motorized valve on air vac for pump column
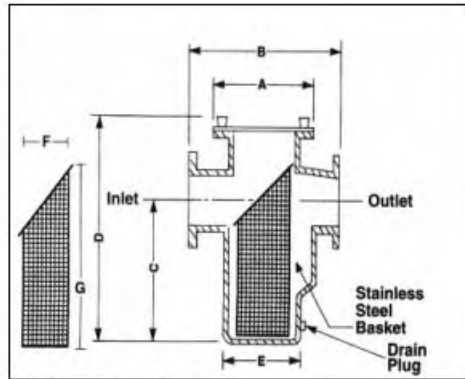
(b) Install strainers to protect pump bowls

(c) Use MOVs to minimize air entrainment

Figure 5.x. Best operational practices developed at the Calleguas project site (Source: Susan Mulligan Calleguas)

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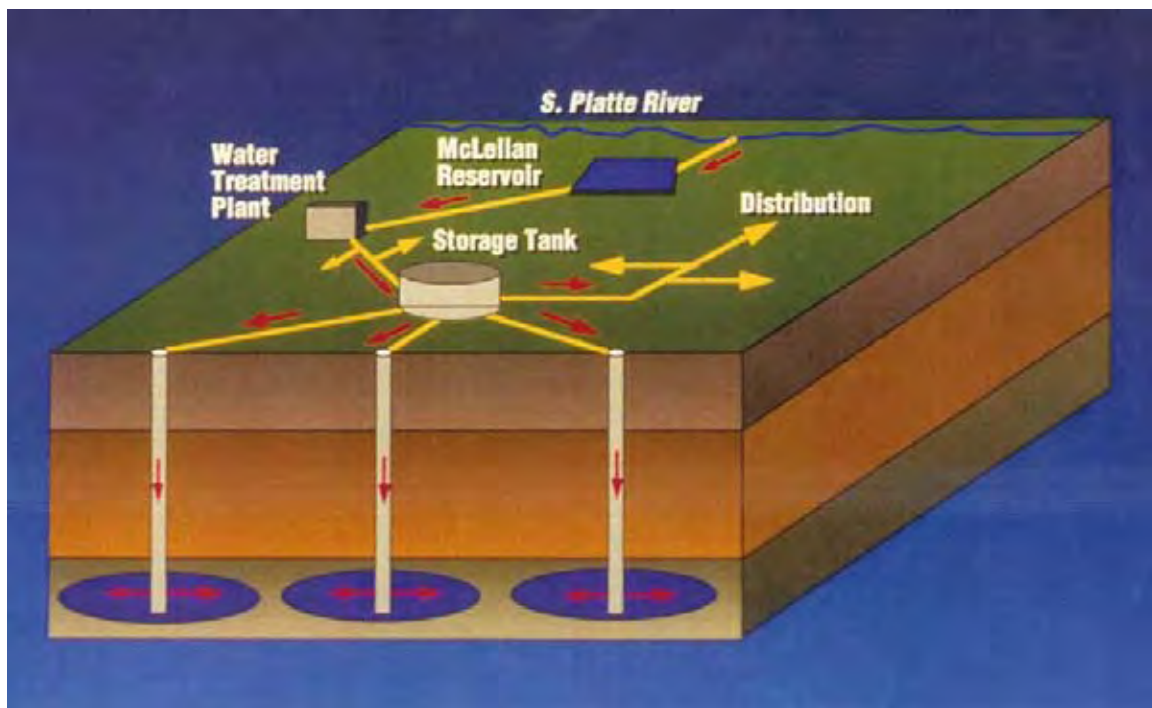
Prepared by: ASR Systems LLC

Prepared for: AWWA Research Foundation

August 2006

Introduction

The Highlands Ranch groundwater recharge program is located in the south Denver metropolitan area in Colorado. Highlands Ranch is a master planned suburban community with a current population of 22,000 and an expected build-out of 36,000 residents by 2022. In order for the county to authorize the population increase in the early 1980's, the development had to demonstrate the water supply scenarios to sustain the community. Aquifer Storage and Recovery (ASR) is included as part of this master water plan. To meet demands, the agency uses water from McLellan Reservoir supplemented by groundwater. Available surface flows captured in McLellan Reservoir during wet years could be placed into storage underground to provide summer peaking supplies and long-term drought storage. At full build-out, over 75 percent of the supply will be creatively derived from surface water and 25 percent from native groundwater.

The Centennial Water and Sanitation District provides treatment and delivery of water to the community. spanning nearly 9,000 acres. The water system was developed in stages as the demand grew. Initially, 29 production wells were installed to prove full recovery capability of the groundwater allocation. The wells were installed quickly and poorly, resulting in operation and sanding problems. Today many of these wells have been replaced. By 1983, a 3-million gallon per day water treatment plant was constructed to treat and deliver surface water from McLellan Reservoir. ASR feasibility studies were conducted in 1990 to 1991, with demonstration testing on an older well in 1992 to 1993. In 1994 the facility began operation and up through 2002 has successfully recharged 6,228 AF.. Currently the Highlands Ranch wellfield consists of 53 wells with a recovery capacity of 11-million gallons per day, 23 of which are ASR wells with an injection capacity of 5 to 6 million gallons per day.
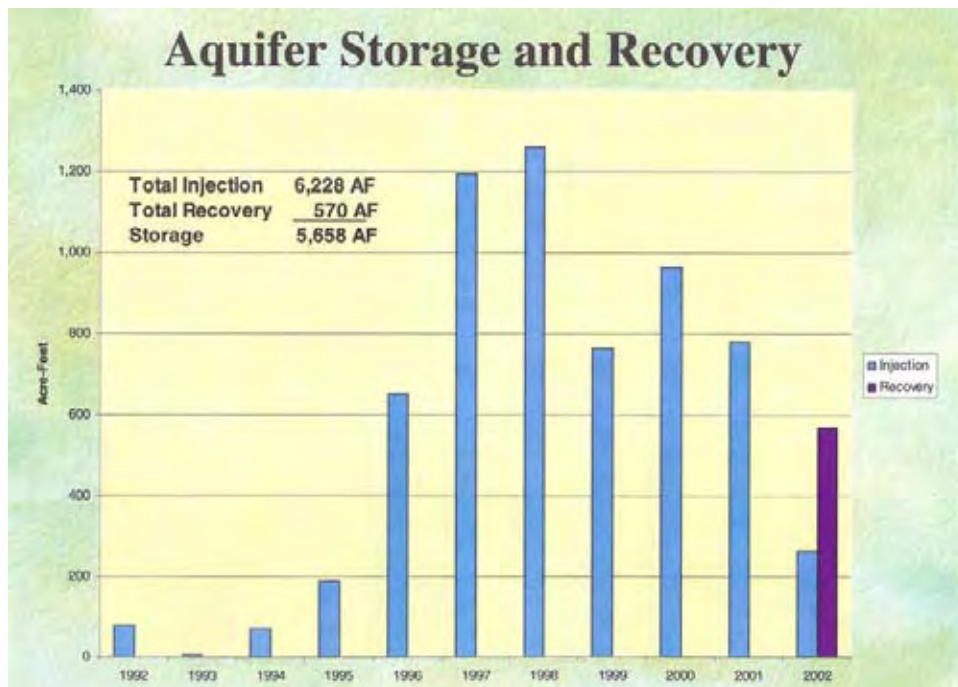
Figure 1: Yearly Recharge Volumes (Grundemann, 2004)

Depending on source water availability, the yearly recharge volumes vary from 200 to 1200+ AF (see Figure 1). To increase this ability to capture seasonal flows, an old gravel quarry will be used to develop Chapman Reservoir. This reservoir will be in line with the existing McLellan Reservoir, boosting the capture volume and sustainability of the recharge source. To meet growing summer demands, the water treatment plant was planned for expansion in 1996. The ASR program has allowed the agency to defer the water treatment plant expansion, reducing capitol costs.

Site Setting - Hydrogeologic Framework

The facility is located in the Denver Basin on the East Slope of the Rocky Mountains. The Denver Basin aquifer system is comprised of four deep, bedrock aquifer units: the Dawson, Denver, Arapahoe, and Laramie-Fox Hills Formations. The Dawson Formation is shallow and primarily utilized for domestic use. Due to declining water levels in the principal aquifer system, the saturated thickness of the Denver Aquifer varies and is undependable. The water produced from the Laramie-Fox Hills Aquifer is of poor quality and must be blended with surface water. This leaves the Arapahoe Formation as the primary storage zone.

The Arapahoe Formation is a sandstone bedrock unit composed of loosely cemented quartz sands with potassium feldspars increasing to nearly ten percent at depth. Calcite is present throughout as a cementation agent with traces of Smectite and Illite clays up to two percent in the lower section. Pyrite is present as a trace constituent throughout.

The formation is 476 feet thick in the wellfield area and contains ten major water bearing zones totaling a combined 200 feet. The Arapahoe Aquifer is an anaerobic, confined artesian aquifer with a current depth to water in excess of 900 feet below surface. It has a transmissivity of 8,500 gallons per day per foot and a storage coefficient of 3.36×10^{-4}. This site operates with one of the lowest aquifer transmissivities in the United States. Due to increased groundwater production water levels have been declining rapidly in recent years and are approaching the top of the first water bearing unit at a depth of 922 feet below surface.

Source Water Characteristics

Potable distribution system water is used for recharge at the Highlands Ranch facility. Flows are reversed through the normal distribution system piping to deliver the water to the wellheads. Water is available for recharge during times of low community demand creating excess in the treatment plant capacity. This is further limited by available supplies from the surface water source. During spring snow melt and storm events, flood water flows are diverted from the South Platt River to McLellan Reservoir. This reservoir provides drinking water supplies for a number of communities, including Highlands Ranch. The source water is then piped to the community and treated to drinking water standards prior to distribution. A chlorine residual is maintained in the distribution system water.

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

The sustainability of the current program is sporadic but good. Excess allotments will dwindle as populations reach peak capacity.. The ability to capture occasional flood water events through the diversion from the South Platt River to McLellan Reservoir will provide long-term sustainability for the water banking program. The concept is to better balance the high and low precipitation years with underground storage while providing a more efficient use of the existing supplies.

Additional source water for recharge is derived from the low quality Laramie-Fox Hills Formation. This water is pumped into McLellan Reservoir to blend with the surface water. An equivalent volume can then be removed from the reservoir, treated, and recharged into the Arapahoe Formation. This component of the recharge program has good sustainability since it effectively moves a volume of water from one aquifer to bolster another.

Water Quality

The quality of the recharge water is good. The water is fully treated to potable standards prior to release to the system for consumer use or recharge. The water is chlorinated for general disinfection. There is little variability in the quality over time. The water is being injected into an aquifer with good water quality. Table 1 displays the water quality of the recharge source water and receiving aquifer.

Parameter	Average Recharge Water Quality (a)	Average Groundwater Quality (a)
Alkalinity	92	123
Aluminum	0.05	0.06
Ammonia	0.43	0.17
Boron	0.05	0.10
Calcium	51	32
Chloride	28	3.25
Eh, millivolts (estimated)	400	200
Fluoride	0.95	0.75
Iron	0.015	0.13
Magnesium	10	1.85
Manganese	0.005	0.02
Nitrate	0.05	0.05
pH, laboratory	7.6	8.1
Phosphorus	0.02	0.005
Potassium	2.85	3.75
Silica	4.95	12.85
Sodium	29.5	27.5
Sulfate	91	16.5
Total dissolved solids	288	180

Note: All values expressed in milligrams per liter (mg/L)unless otherwise specified.
(a) To compute the average values, half the detection limit value was used when the value for the parameter was below the analytical detection limits.

Table 1: Source Water and Native Aquifer Water Quality (Pyne, 1995)

Initial geochemical investigations indicated that wellbore plugging resulting from the precipitation of calcium carbonate and ferric hydroxide in the upper intervals of the aquifer was a concern. The results indicated that the pH of the recharge water should be maintained between 7.5 to 8.3 to minimize the potential for precipitation.

Prior to delivery to the community, the recovered water quality is acceptable and meets drinking water quality standards. No treatment is necessary other than chlorination. Initial concerns of iron and calcium precipitation were not observed in the water pumped during testing and recovery. The pH buffered naturally to near formation levels within the mixing zone of the recharge bubble. Trihalomethane and organic carbon concentrations declined very rapidly in the recovered water.

The Highlands Ranch Project set the precedence in the Colorado area of direct recharge and recovery of potable water through wells. In response to this program, State regulations were developed defining operation and water quality parameters of the program. The wells are permitted under a Class V Underground Injection Control permit requiring routine water quality sampling and reporting. Only select wells representing sections of the operation are required to be sampled, streamlining the sampling and reporting procedures. The Colorado Office of the State Engineer requires reporting the volume of water displaced to ensure adherence to existing water rights. Water level and operational data must be collected and retained for review upon request, but are not required for yearly reporting. Recently the Colorado Division of Public Health and Environment adopted more stringent organic requirements for recharge recovery water. Concerns addressed in the new regulations include disinfection byproducts in the recovered water. Although the existing facility is exempt from the modified requirements, the level of organics in the recovered water is not elevated enough to be a concern.

Feasibility and Design Studies

The availability of water resources, including the feasibility of storing water underground, had to be proven initially for the county to approve the Highlands Ranch development. To secure the construction permits, early groundwater flow and geochemical mixing studies were conducted by Willows Water District in the early 1980's. In 1990 the program became active with additional modeling activities, monitoring wells, core samples, and field feasibility testing.

Sidewall cores were taken during construction of an observation well. Four sidewall cores were taken from the Arapahoe Formation. The use of sidewall cores instead of continuous wireline coring is unique. The small volume and disturbed nature of these cores limited their usefulness. X-ray diffraction, cation exchange capacity, and acid insoluble residue tests were conducted. Geochemical modeling of the core data, source water chemistry, and native aquifer water chemistry indicated that the pH must be maintained between 7.6 and 8.3 to prevent the precipitation of calcium carbonate or ferric hydroxide. No metal or arsenic mobilization was expected.

Full scale field testing was conducted on existing A-6. The 10-inch mild steel well is screened with wire wrap between 922 and 1354 feet below surface. The well is constructed with no gravel pack and has a discrete zone monitoring well cluster 77 feet away. Over a period of six (6) months, three cycle tests were conducted on the well. The

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first test included recharging at 198 gpm for five (5) days, no storage time, and then recovery at 309 gpm for four (4) days. The second cycle, water was injected for 15 days at 263 gpm, allowed to stay in storage five (5) days, then recovered through 12 days pumping at 410 gpm. Cycle 3 included recharging at 274 gpm for 45 days, 30 days storage time, and then recovery for 40 days at 376 gpm. Water quality data from the cycle tests indicated that metals and salts were not leached out of the formation. No precipitation of calcium carbonate or ferric hydroxide was observed during testing. Trihalomethane and organic carbon concentrations declined rapidly during recovery.

The test data was further used to evaluate the aquifer response to recharge and the potential for long term drought storage. Further modeling of the hydraulic operational data and chemistry data indicated that the wells would need to be backflushed every four (4) weeks for a period of six (6) to eight (8) hours. As a result of excessive driving head during cycle test 3, the test well experienced hydraulic fracturing of the aquifer.. The water level had risen to nearly 410 feet above static when this occurred. The result was a notable decline in recharge and production performance.

Facility and Operational Data

In the early 1980's, 19 groundwater production wells were installed: 13 completed in the Arapahoe Formation, two in the Denver Formation, and four in the Laramie-Fox Hills Formation. These wells were completed without gravel pack, resulting in sanding issues and pump problems. Due to the casing reductions and slim hole design, pump installation difficulties were also encountered.. In the mid 1990's the agency replaced many of the wells with larger casing, improved screen design, and gravel envelopes. The newer wells utilized highflow wirewrap well screen. Some of the wells were installed with stainless steel well screen and others with mild steel. The blank casing is all mild steel. The agency has not had a history of casing corrosion warranting corrosion resistant construction. The casing and pump column are subject to mild Iron Related Bacteria (IRB) and Sulfate Reducing Bacteria (SRB) attack forming minor nodules and pitting on the pump column and well casing. Slyme Bacteria has been noted in one well and a pipeline system, although it is atypical.

The ASR wells are equipped with submersible pumps equipped with variable frequency drives. The pumps are set to nearly 1,300 feet below surface with the local static water level ranging from 800 feet to in excess of 900 feet below surface. Due to the great depth to the static water level and concerns of maintaining hydraulic control over the recharge flows, the agency teamed with local manufacturer, Baski Incorporated, to pioneer the development of a borehole recharge flow control valve for the water industry. This piece of equipment is placed in the pump column string below the water table, but above the pump bowls and foot valves. This valve eliminated the issues of cascading water, flow control, and the injection of 900 feet of air from the pump column at startup. With no flow through the pump bowls and foot valves, the recharge flow control valve is closed, allowing the column to be prefilled through flooding or pumping. The borehole flow control valve can then be slowly opened to a controlled recharge flow rate while maintaining the desired backpressure on the injection column.

A total of 19 wells are equipped with borehole flow control valves. The majority of the valves are operated manually for conversion from recharge to production and back.

These wells use a solenoid controlled diaphragm valve in the wellhead piping to provide recharge on and off control through SCADA. Bypass valves to waste (atmosphere) are manually operated. Operators observe the discharge until clean and rawhide the well as necessary through multiple restarts to waste.

At two well sites, the flow control valves have been automated. Initially, the finite on and off control was automated through application of preset nitrogen gas pressures on the equipment. Adjustments to the set points were manual. The agency has recently advanced to full flow control automation of the borehole recharge flow control valves. The operator selects the desired flow rate and the valve slowly opens to that rate and maintains it regardless of water level changes. The system utilizes existing flow meter input pulses in a PLC program to momentarily pulse solenoid valves that control the opening or closing of the valve. The automation allows the operator to change the rate at any given time remotely or close the valve entirely. Other than the addition of signal lines for the pulse controls, the supporting nitrogen tank was equipped with a pressure sensor and alarmed just above the pressure required to close the valve.

During the initial startup of the recharge system, sand that had settled in the distribution system was carried into the wells. The current practice is to pump the well to waste to clear the wellbore and pump column of any turbidity or material. This also pre-fills the column with clean water prior to injection. If necessary to clear the line, the distribution line is blown to waste at high velocity. With the entire system full and holding back system pressure, the borehole flow control valve is opened to start recharge. Most of the wells are stopped with the surface diaphragm valve. Subsequent restarts will follow the same procedure of pumping to waste to prefill the column. The wells are backflushed to waste every four (4) to six (6) weeks to maintain recharge efficiency and exercise the pump units. To conserve on monthly electrical demand charges, the agency pumps to waste at the beginning and end of one month, skips a month, then repeats.

The agency has been able to bank most of the water recharged since 1992 with the exception of some minor recovery in 2000. With nearly 7-million AF of water below Highlands Ranch, a recharge program banking a few thousand AF of water is negligible. With a regional gradient of 50 feet per year, there is little concern of losing the water or benefiting a neighboring community in this metropolitan setting. Colorado law allows clear ownership to that volume of water, enabling the agency to recover it anywhere in its jurisdiction, provided it is in the same aquifer basin.

The Centennial Water and Sanitation District has plans to equip three more wells for ASR. They also are planning on converting more of their manual borehole flow control valves to full automation. With recovery imminent and to allow precise use of the resource, the agency is investigating the development of a SCADA driven pump prioritization program managing the associated judicated water rights per well.

Problems Identified, Lessons Learned and Recommended Operational Practices

Some of the key lessons learned from the evaluation of the Highland Ranch recharge program are noted below.

- ASR water is delivered through normal surface water conveyance pipelines to local storage reservoirs and then reverse flowed to the wells.

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- The community is located on the upstream side of a major urban area, providing optimum location for first capture of storm surplus on the river system.
- All water banked can stay any length of time in the aquifer and is fully removable at any time or rate. The banked water is to be used make up for drought shortfalls.
- The first 29 wells were drilled to secure water rights, though were not constructed to compliment ASR. The wells were constructed undersized and were built with mild steel screen with no gravel pack. Due to sanding issues, most of the wells were replaced within ten (10) years.
- Replacement wells were constructed up to 18-inches in diameter with stainless steel wire wrap screen and a gravel pack envelope. Mild steel is still used for the blank sections.
- Wells are completed to very deep aquifer sections exposed at 900 feet to 2000 feet below surface. The water table is 800-900 feet. Submersible pumps are used with settings up to 1350 feet.
- In the early 1990's, a borehole flow control valve was developed with Baski Incorporated.. The valve allows for a controlled flow out the side of the pump column into well without having to pass through the pump bowls. The valve can be used to throttle and hold specific flow rates.
- The recharge borehole control valve is placed just above the pump, 300-400 feet below the water table. This reduces the hang weight of column sections and torque potential by the pump on the valve.
- The borehole control valve is controlled by a nitrogen gas actuating system. Pressure is then monitored and alarmed in SCADA to ensure enough cylinder pressure is available to operate the valve.
- Of 23 equipped ASR sites, two wells are equipped for full automation through program logic control of the borehole flow control valve.. Control includes open, close, and automated adjustment of valve to maintain the set flow rate. Future improvements include equipping more sites in this manner.
- All wells are equipped with wellhead airvacuum valves, electromagnetic flow meters, and a globe check valve. The globe check valve is piloted for recharge on and off control.
- In preparation for recharge startup, each well is pumped to waste to purge stale water from the well casing and any loose sand.
- The initial recharge startup of the wells includes displacing all air in the pump column with water prior to starting injection.
- The borehole control valve is opened manually to the desired operation rate or water level. The above ground globe valve controls the start and stop of recharge. Under automated stops, the airvacuum valve fills the column with air. Automated restarts can force the column of air into the wellbore and aquifer.
- During recharge operation, the wells are backflushed once a month to clear any clogging and improve operational efficiency. Purging is completed at the beginning and end of every other month to avoid power company demand use charges in the center month.
- The recharge water is recovered directly to system with no treatment other than chlorination.

- The deepest aquifer has poor quality water that is pumped up to the surface stream system and reservoirs for blending. This provides surface credits. The blended surface water is removed downstream for recharge in the shallower aquifer units.
- The recharged water does not have to be removed at the point of injection, rather it can be removed at any location in the same aquifer body. This reduces pipelines and distribution system improvements.
- The performance evaluation of the recharge wells is based upon increased changes in water level or injection rate.
- Continued modeling is employed to incorporate recharge technology with growth and community planning.
- The hydraulic impact of the Highlands Ranch Recharge project is very small on the vast aquifer system.

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AWWARF Case Study, Peace River ASR Project, DeSoto County, Florida

Prepared by: LJHB Partners LC

Prepared for: ASR Systems LLC Team and the AWWA Research Foundation

February 2006

Introduction

The Peace River ASR project is located in southwest Florida within the Peace River watershed in Desoto County near the City of Arcadia, Florida (CH2M Hill, 1985). The ASR project is administered by the Peace River/Manasota Regional Water Supply Authority (PRMRWSA) and in conjunction with traditional surface water supply, provides a dependable and cost effective potable supply to over 200,000 residents in Charlotte, Desoto, Manatee and Sarasota Counties including the city of North Port (Sam Stone, 2006). The original Peace River facility was constructed, owned and operated by General Development Utilities and started operation in 1980 (PRMRWSA, 2006). General Development Corporation declared bankruptcy in 1990 and the facility was taken over by the PRMRWSA in June 1991. The Peace River ASR project is one the oldest ASR projects in Florida with initial investigations commencing in 1983 and full-time operations beginning in 1988 (Lehman and Waller, 1996). The ASR system is designed to provide long-term and seasonal water supply in conjunction with the Peace River (Lehman and Waller, 1996). The facility was originally envisioned to be able to supply up to six months of water supply that meets drinking water requirements during a long-term drought. A map of the Peace River facility is shown on Figure 1.
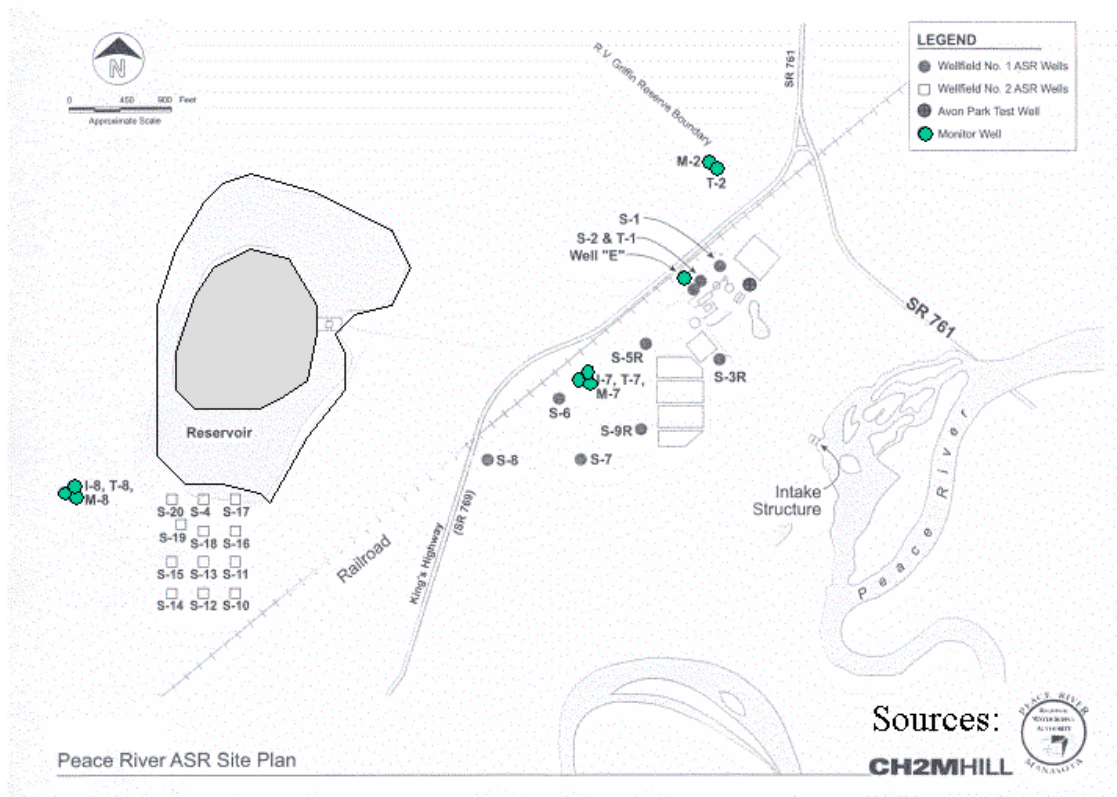
Figure 1. Peace River site plan

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The six-month time period criteria for sizing the facilities was based upon a period of historical record analysis for the Peace River in 1985 (CH2M Hill, 1985). In addition, besides the low river flows recorded during long-term droughts, the Peace River also experiences extensive algae blooms that result in difficult water treatment and taste and odor problems (CH2M Hill, 1985). As a result, there are times when water is available for diversion from the river by permit but has such excessive algae concentrations, that the PRMRWSA chooses not to pump water from the river. Raw water from the river is stored in an 85 acre man-made off stream reservoir. The Authority operates a 24 MGD water treatment plant at this location. Currently there are 21 ASR wells onsite used for potable water storage.

Over the last 20 years, the facility has been expanded several times and now consists of a large system composed of 21 ASR wells with 20 wells completed in the Suwannee Limestone Formation, one well completed in the Tampa Limestone Formation, and one test well completed in the Avon Park Formation (Morris, 2004). The 21 ASR wells and one test well are located in two distinct well fields located east and west of Florida Route 769 (Kings Highway). The east well field was constructed between 1984 and 1995, while the newer west well field was constructed in 2000 (Pyne, 2005; Sam Stone, 2006). This case study report is based upon existing project literature and results of a site visit and tour conducted on February 21, 2006. Those present for the site visit and tour included:

- Chris J. Brown, LJHB Partners LC
- Tom Dobbs, PRMRWSA
- Sam Stone, PRMRWSA

The site visit focused upon the site history, operating practices, engineering data, water quality issues, operational problems, innovative site practices, and lessons learned. Results of the site visit are discussed throughout this report and referenced as "personal communications" with either Tom Dobbs or Sam Stone.

Site Setting - Hydrogeologic Framework

Below the surficial soil horizons are Plio-Pleistocene-aged sand, sandstone, clay and shell beds present to a depth of approximately 30 feet below lands surface (Water Resource Solutions, 2000). These sediments are representative of the Caloosahatchee and Tamiami Formations. These formations were deposited from between one to five million years before present. The surficial sediments unconformably overly the dense, phosphatic clays and limey silts/sands of the Miocene to early Pliocene-aged Hawthorn Group (Scott, 1988; Miller, 1997). The Hawthorn Group sediments, including the Peace River and Arcadia Formations, extend from between 30 to 580 feet below land surface (bls). Near the base of the Hawthorn Group, the limestone and phosphate content of these sediments increases, causing the natural gamma ray log to record high emissions through this interval. This has resulted in the creation of distinctive "marker beds" that can be correlated throughout south Florida (Reese and Memberg, 2000). In the study area, the

permeable Tampa Limestone member lies within the Hawthorn Group. This unit is frequently included as part of the Intermediate Aquifer System (IAS).

The Suwannee Limestone is interpreted to be present in the project area and continuing in a northwest-southeast trend through the northeast portion of Glades County and trending diagonally through the west and central part of Palm Beach County (Miller, 1986). The depth to the top of the Suwannee range from approximately 550 to greater than 900 feet bls. The dominant lithology of this unit in the project area is pale-orange to tan, fossiliferous, medium-grained calcarenite with moderate amounts of quartz sand and little or no phosphatic mineral grains (Reese and Memberg, 2000). At the Peace River site, the Suwannee Limestone is encountered from 560 to 860 feet bls.

Lying below the Hawthorn Group and Suwannee Limestone sediments at depths of approximately 600 to 1,200 feet bls are the Eocene-aged Ocala Limestone, Avon Park Formation, and Oldsmar Formation. The Ocala Limestone is typically recognized as being present within the uppermost 200 feet of the combined section. At the project site, the Ocala Limestone is rather thin in composition with only 50 feet present (Water Resource Solutions, 2000). These formations are characterized by pale orange to brown, poorly-cemented, granular limestone, and are occasionally micritic. At the project site, a dense dolomite unit separates the Suwannee Limestone and the underlying Ocala Limestone. This unit is variably fractured and is highly permeable within fractured areas (Water Resources Solutions, 2000). The Avon Park Formation lies beneath the Ocala Limestone and is typically recognized as white to cream foraminiferal limestone, with the majority of foraminifers being *Dictyconus* (Miller, 1997). The Formation is also comprised of dark brown to tan crystalline to saccharoidal dolomites.   These formations are present to a depth of approximately 1,900 feet bls. Lying below the Ocala Limestone and Avon Park Formations is the Oldsmar Formation. This formation contains significant quantities of hard, yellowish brown finely crystalline dolomite. In some places it is termed the "boulder zone" due to its cavernous nature and propensity to generate cobbles and boulders during drilling operations. Anyhdrite is also found to be present in some local areas. These formations are present to depths of nearly 3,500 feet bls.

Three major aquifer systems have been identified in South Florida; the Surficial Aquifer System (SAS), the Intermediate Aquifer System (IAS) and the Floridan Aquifer System (FAS), (Southeastern Geological Society, 1986). Within the Intermediate Confining Unit is the IAS, as well as a few other aquifers of limited transmissivity and extent. The IAS is located in southwestern Florida, and consists of beds of sand, sandy limestone, limestone, and dolostone of Oligocene to Pliocene age that dip and thicken to the south and southwest. These aquifers are typically leaky artesian with moderate transmissivity and are located primarily in Polk, Sarasota, Highlands, Hardee and DeSoto Counties. The IAS includes the Tampa Member, the Arcadia Formation, and the Peace River Formation, all of the Hawthorn Group.

The FAS is the primary source of water supply for the northern counties of Florida, and is used as a source of supplemental irrigation water as far south as Martin County. The system receives direct recharge along a structural high, known as the "Ocala High", which occurs in the west-central part of the State. From this area, the FAS dips southward where it is overlain by clays and silts of the Hawthorn Group, which form a confining layer over the FAS. The potentiometric surface is also highest in the central portion of the State and decreases radially to the south, east and west.

During the well site selection exercise completed during initial investigations, a complete well inventory was completed of all local well users. The inventory was utilized in order to aid with selection of an ASR storage zone (CH2M Hill, 1985).
The ASR wells onsite are completed by open-hole in three different aquifers as noted above. Twenty of the wells are completed in the Suwannee Limestone of Oligocene Age. This aquifer is located within the Upper FAS and is characterized as sandy, fossiliferous limestone and is located approximately 550 to 920 feet below land surface. Aquifer performance tests have been completed at multiple ASR wells. One well, S-5, produced objectionable quantities of sand during development and testing operations (CH2M Hill, 1985). The average transmissivity of the confined Suwannee zone is 5,000 to 30,000 ft^2/day with an average storage coefficient of 2.0×10^{-4} and leakance of 1×10^{-3}/day (CH2M Hill, 1985). More recent estimates based upon numerical modeling of the entire well field suggest a transmissivity value closer to 6,000 ft^2/day and a much higher leakance of 5×10^{-3}/day (CH2M Hill, 1998). The Tampa Limestone zone transmissivity was estimated to be approximately 4,000 ft^2/day with a storage coefficient of 8.0×10^{-5} and the deeper Avon Park may have a transmissivity of 150,000 ft^2/day and a storage coefficient of 1×10^{-3} (CH2M Hill, 1985). Static water levels within both the Tampa and Suwannee Limestones range from 40 to 50 feet NGVD, with a predominant gradient of 0.003 WNW

Source Water Characteristics

The Peace River is 120 miles long and drains approximately 2,300 square miles of southwestern Florida. The Peace River is an uncontrolled river that is free flowing to the Gulf of Mexico and experiences about 100 feet of elevation change along its 120 mile run from the Trail ridge area to Charlotte Harbor. The historical average flow rate for the river is 1,060 cfs but the flow can vary widely. Figure 2.x displays the approximate monthly average flow rates.

The Peace River plant has a regulated withdrawal schedule that allows up to 10% of the river flow to be captured if the river flow is greater than 130 cfs. If the flow is less than 130 cfs, then no withdrawals are permitted (Sam Stone, 2006). Also, the maximum permitted withdrawal rate is 130 cfs. In a typical year the river supply is unavailable for 100 to 115 days. During those times, the facility depends upon water stored in an onsite reservoir and the existing ASR wells.

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Water Quality

The source water quality of the treated Peace River water is variable depending upon flow characteristics in the river. The TDS value can range from 160 to 596 mg/l (CH2M Hill, 2003). The average value is close to 260 mg/l TDS (CH2M Hill, 1998). The chlorides range from 30 to 162 mg/l and the sulfate ranges from 32 to 175 mg/l (CH2M Hill, 1985). The source water also contains moderate levels of metals such as calcium and iron.
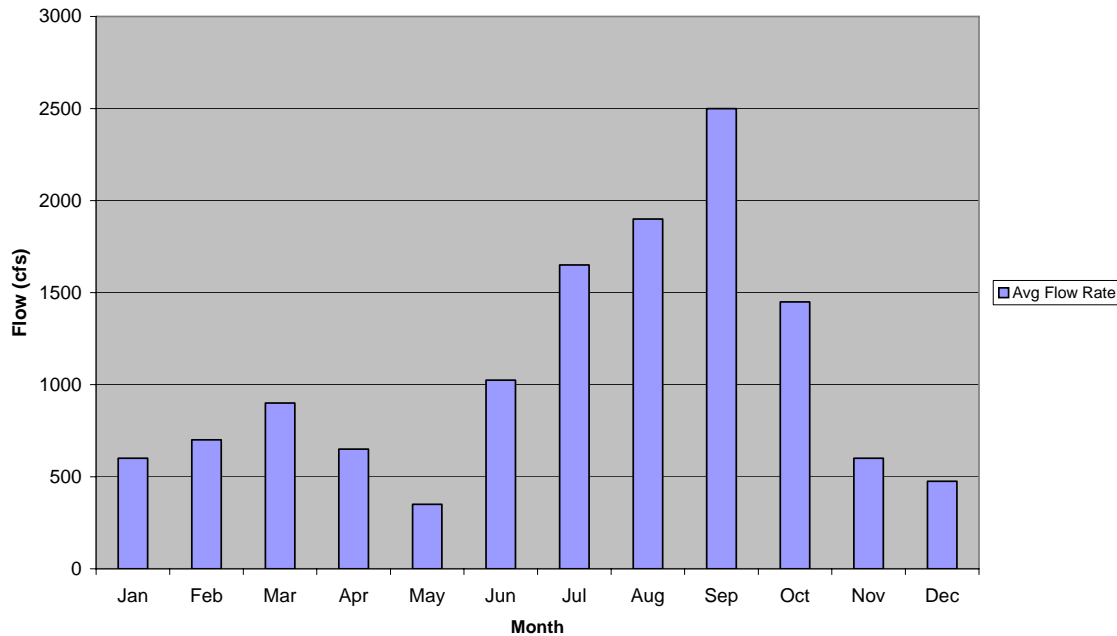
Figure 2. Monthly Average Historical Flows for Peace River at Arcadia Gage (Source PRMRWSA).

After water treatment using conventional alum cogulation followed by filtration, low levels of disinfection by-products are produced including total trihalomethanes (TTHMs) ranging from 25 to 50 ug/l. The treated water has low color and turbidity with values estimated at 0.02 NTU (Tom Dobbs, 2006). The ambient groundwater quality within the Suwannee storage zone is slightly brackish with TDS values ranging from 650 to 800 mg/l, chloride ranging from 151 to 206, sulfate ranging from 216 to 232 mg/l, and low levels of metals including arsenic (7 ug/l), silver (8 ug/l), and calcium (75 mg/l), (CH2M Hill, 1985; Water Resource Solutions, 2000). Representative groundwater quality from three onsite ASR wells is shown in Table 1 (CH2M Hill, 1995).

Cycle testing originally began in 1984 using wells completed in the Suwannee Limestone. Table 2 summarizes data from the first five ASR recharge and recovery cycles. The recovery efficiency (RE) percentage is defined by a TDS regulatory standard of 500 mg/l versus other brackish water sites that use chloride as a regulatory standard.

The pilot testing was very successful and indicated that the potential RE could approach 100%. Subsequent site operations have routinely achieved 100% RE using a blend of stored ASR water and reservoir water (Sam Stone, 2006).

Table 1. Groundwater quality at three onsite ASR wells.

Parameter	Well S-3R	Well S-5R	Well S-9R
Alkalinity (mg/l as CaCO$_3$)	122	140	130
Chloride (mg/l)	173	161	160
Color (APHA)	0	0	5
Conductivity (umhos/cm)	1,200	No Data	1,240
Hardness (mg/l as CaCO$_3$)	376	450	450
PH	7.35	7.50	7.85
Sodium (mg/l)	79.1	78.2	78.6
Sulfate (mg/l)	237	267	252
Total Dissolved Solids (TDS mg/l)	770	838	786
Turbidity (NTU)	<0.2	No Data	No Data
Gross Alpha (pCi/l)	43+/-4	10.7+/-3.5	20.7+/-5.1
Radium 226 (pCi/l)	7.6+/-0.5	3.1+/-0.3	8.2+/-0.4
Radium 228 (pCi/l)	1.0+/-0.8	<1.0+/-0.6	1.7+/-0.8

Table 2 Data from initial cycle testing at Peace River ASR site

Cycle	1	2	3	4	5
Volume In (Mgallons)	3.83	6.38	6.06	6.62	9.78
Storage Time (days)	3	1	1	1	17
Volume out (Mgallons)	12.29	6.75	6.98	5.90	9.58
RE (%)	321	106	115	89	98
Qin (gpm)	200 to 700	200 to 700	200 to 700	200 to 700	200 to 700
Qout (gpm)	200 to 700	200 to 700	200 to 700	200 to 700	200 to 700

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Generally, the recovered water quality data from the initial cycles indicates mixing between the recharge and ambient water quality is the dominant geochemical process, however, some early radionuclide data may reveal some geochemical water-rock reactions occurred during ASR storage. Gross alpha analyses at the beginning and end of each cycle test were tabulated during cycle testing. The end value measured for cycle 2 (21.4 pCi/L) indicates a gross alpha value nearly four times higher than the initial natural background measurement of 5.8 pCi/L. A few other metals also exhibit possible water-rock reactions including iron, which was detected in one recovered water sample at a concentration of 2.35 mg/l, almost ten times higher than background measurements. TTHM concentrations in the recovered water generally declined during recovery operations as the percentage of ambient groundwater in the recovered mixture increased. Typical TTHM concentrations measured at the end of recovery operations ranged from 18 to 28 ug/l (CH2M Hill, 1985). Although TSS measurements were not available, turbidity values were initially high during the early portions of cycle test recovery and then receded quickly.

Most heavy metal data were not recorded during the early cycle recovery tests performed at the original east well field. More recent recovered water data have revealed more serious water-rock reactions that are suspected of releasing arsenic during ASR storage. Cycle testing of the new "west well field" at the Peace River complex has revealed arsenic in the recovered water at concentrations exceeding regulatory standards. Figure 3 depicts arsenic results from well # S11 during cycle 2 recovery. Due to the arsenic contained within the recovered water, the recovered water is re-treated in the onsite water treatment plant at a significantly higher cost. In addition, the arsenic issues have delayed a proposed ASR project expansion at the site (Morris, 2004).

Feasibility and Design Studies

The Peace River facility has been designed and constructed in phases allowing for efficient use of limited capital. Until 2000, the typical strategy has been to increase the ASR well capacity two to three wells at a time. Starting in 1985, the ASR capacity has been incrementally expanded from two wells in 1985, three wells in 1987, six wells in 1989, nine wells in 1995, and finally the current configuration of 21 wells in 2002. The ASR wells are designed to provide water during the drought of record to ensure a reliable water supply for PRMRWSA customers. Typically, design of the wells has been based

Peace River, FL Well S-11 (West wellfield) ASR Cycle 2 Recovery Data
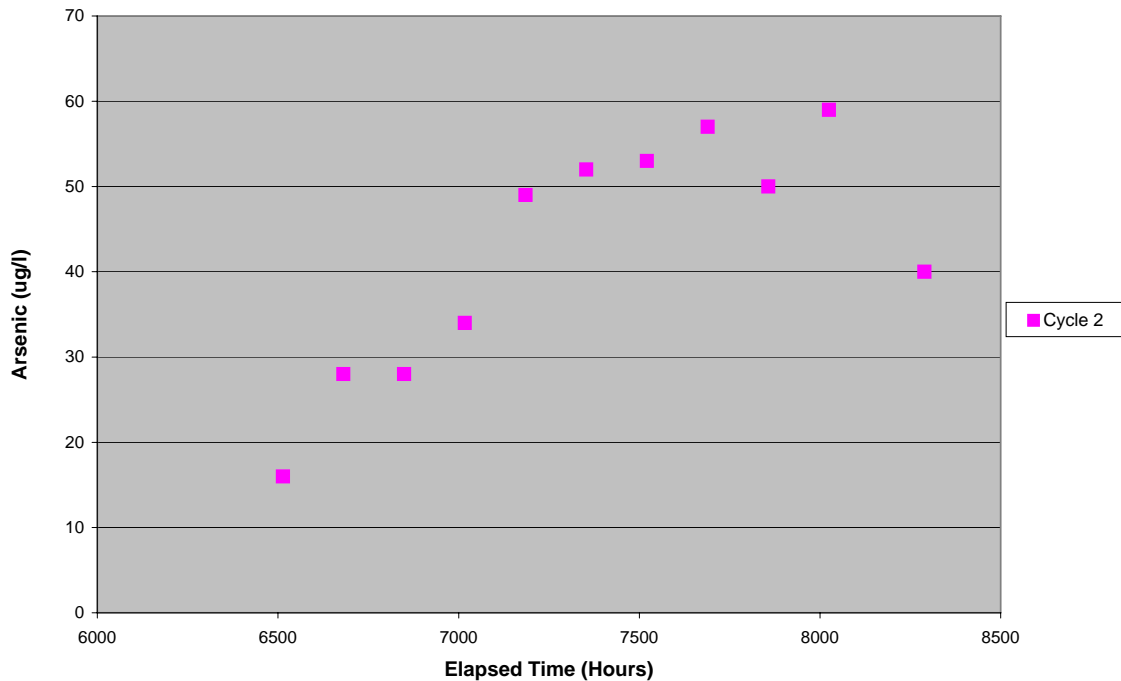
Figure 3 Arsenic concentration in well S-11 recovered water during cycle # 2

upon aquifer performance test data, numerical modeling, and initial pilot testing. The instrumentation and control of the whole well system has progressed over time leading to a mostly automated system design for the new west well field. The wells themselves initially included a variety of pump types and sizes due to the adhoc-nature of the wellfield expansion over the years. Pump manufacturers included both Peerless and Byron-Jackson with motor horsepower ranging from 5 to 50 due to heterogeneous nature of the aquifer storage zone. As a result of the different well designs, the well casing diameters range from 8 to 20 inches and the well capacities also range from 0.5 to 3.0 MGD. Recently, an effort has been undertaken by facility staff to further standardize well equipment, casing size and piping instrumentation (e.g., flow meters, valves) in order to improve reliability and minimize down-time (Tom Dobbs, 2006). The wells are provided water from the onsite water treatment plant that includes the following treatment steps:

- Water intake
- Coagulation with Alum
- Clarification
- Primary disinfection with chlorine
- Residual disinfection with chloramines
- Multimedia filtration
- pH adjustment for stabilization

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The current intake capacity from the Peace River is 42 MGD from the river pump station to the reservoir. The reservoir then pumps up to 24 MGD of raw water through the treatment plant. At this time (early 2006), about 16 to 18 MGD is sent to customers while the remainder is stored in the onsite storage tanks or the ASR system. Similar to the ASR wellfield, the water treatment plant has undergone a series of expansions from its initial capacity of 6 MGD to the current 24 MGD capacity.

Facility and Operational Data

Both the water treatment system and the ASR system are highly automated and controlled by a state-of-the-art SCADA system. The east ASR well field started as a more manually controlled system but has evolved to a full SCADA system. The newer west well field has always utilized a SCADA system to manage its operations (Sam Stone, 2006). Due to concerns over terrorism at all water plants in the United States, security infrastructure has become more important. The Peace River facility has invested in security fencing, remote cameras, and motion detectors (Tom Dobbs, 2006). In addition, due to security and safety concerns, 40-pound chlorine gas cylinders are being replaced with liquid chlorine. This is an important safety concern in a rapidly urbanizing area where risks to offsite receptors is increasing.

Currently, the facility staff operates the ASR system to meet system demands. Typically, when source water is plentiful, the ASR system is recharged continually with the total recharge flow split evenly to all in-service wells. A few poor performing wells and wells lacking regulatory permits have been taken out of service (Sam Stone, 2006). In the original east well field, low levels of arsenic have been measured in the recovered water while the newer west well field has more significant arsenic concentrations. However, the arsenic contained in the recovered water is handled through re-treatment that removes 90 to 95 percent of the arsenic while additional blending with raw reservoir water before treatment reduces the incoming arsenic concentration further. This strategy has been successful and has allowed the ASR system to continue to operate without jeopardizing public safety. The operation of the ASR system is user friendly due to the SCADA control system, however, ongoing regulatory-driven water quality data collection is rather onerous. According to site staff, one full-time person is dedicated to compliance activities including sample collection and data management/reporting. Compiling the monthly sampling reports alone usually requires three to four days of data entry and clerical work (Tom Dobbs, 2006). Arsenic in the recovered water has probably further elevated the importance of the water quality data collection. Inclusion of a tee-connection as a design enhancement at each ASR wellhead has provided considerable flexibility to the compliance staff during sampling events or other important maintenance activities.

Performance Evaluation

The Peace River ASR system has posted an impressive performance record. Over a 19-year operational period, the ASR system has successfully stored approximately 10,807 million gallons of potable (treated river) water and recovered approximately 8,456 million gallons consisting of a potable water and native groundwater. This represents a net recovery utilization of approximately 84%. Figure 4 displays the system performance over the 19-year period.

Well clogging has not been a major problem at the Peace River project although small decreases in specific capacity have been noted (CH2M Hill, 2003b). In general, specific capacity reductions are restored during ASR recovery operations. In addition, periodic well redevelopment activities have been conducted included well acidization treatment and CO_2 treatment. These are discussed in more detail below.
Figure 4.x depicts water recharge and recovery over a 19-year operational period (source: PRMRWSA, 2006).

An economic evaluation of the system completed by CH2M Hill in 1985 has estimated the unit cost of the project to be moderate at $1.00 per thousand gallons recovered (CH2M Hill, 1985). An independent cost evaluation completed by the author for this report has estimated a current unit cost of $0.72 per thousand gallons recovered including re-treatment costs due to arsenic issues. The wholesale rate charged to customers is approximately $1.60 per thousand gallons. The project appears cost effective as compared with other water supply alternatives largely due to the economies of scale derived from the vast amount of water that has been stored and recovered at the site.
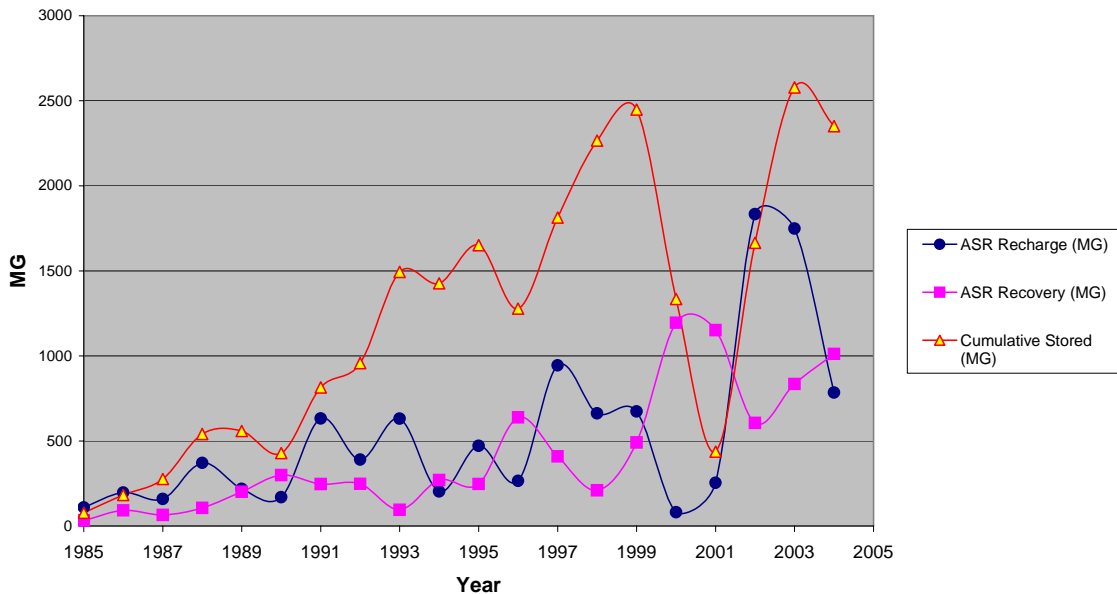
Figure 4 ASR Recharge and Recovery Volumes, Peace Rive ASR System, over project life

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Problems Identified, Lessons Learned and Recommended Operational Practices

Several issues have been evaluated at the Peace River site over the long operating duration. First, TDS values during key recovery periods (1996-97 and 2001-02) have been noted as higher than anticipated (CH2M Hill, 2003). Several contributing factors to this issue include:

- Large recovery volumes without an adequate storage buffer
- Variable recharge water quality
- Lateral displacement of the stored water bubbles around each well caused by storage or recovery operations of adjacent wells
- Lateral displacement of the stored water bubbles around each well caused by operations of new offsite wells
- Possible upward movement of brackish water from beneath the storage zone during extended recovery periods
- Pre-existing groundwater velocity of 15 inches per year towards the west can result in bubble migration

CH2M Hill concluded that the two primary factors were the lateral displacement due to onsite well recharge and recovery operations and large recovery volumes leaving little buffer volumes in the aquifer storage zone. This is an important lesson learned from this site and should be considered carefully at all ASR projects.

The site staff identified several important lessons learned and recommended operating practices. First, acidization of existing wells was completed in 1994 with limited success. The acidization process removes scale from pump shafts as well as clears precipitants from the open-hole interval in the well. The acidization also can expose larger surface areas in limestone aquifers through dissolution. The Peace River staff noted that these activities are difficult and dangerous to complete since pumps must be pulled for their protection and wellheads disassembled. Due to these problems and limitations, CH2M Hill has tried an short term innovative well redevelopment option using CO_2 instead of using strong mineral acid. According to onsite staff, this action was very effective and did not require much system downtime since CO_2 can be bubbled into recharge water (Tom Dobbs, 2006). In addition, the use of CO_2 did not result in any regulatory excursions so there was not need to purge and dispose of the well treatment water as was required using acid.

In reviewing and discussing the merits of either acidization or CO_2 treatments, the impact of these operations to the aquifer geochemical environment should not be discounted. In reviewing the current arsenic issue during the AWWARF site visit, it was hypothesized that well development treatments using acid or carbon dioxide may be exacerbating the situation by exposing more of the carbonate section and by releasing additional oxygen into the aquifer. It is highly recommended that this be evaluated in the future to ensure the long-term sustainability of the project.

The Peace River project appears to have a bright future, more so if the arsenic issue can be solved. Additional system expansions are underway to expand the water

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treatment capacity as well as the water storage capacity. The proposed expenditures under consideration are significant and will further expand the Peace River plant. Future expansion of the ASR system will also be considered as additional information and knowledge are gained through operation of the existing ASR wells.

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Las Vegas, Nevada
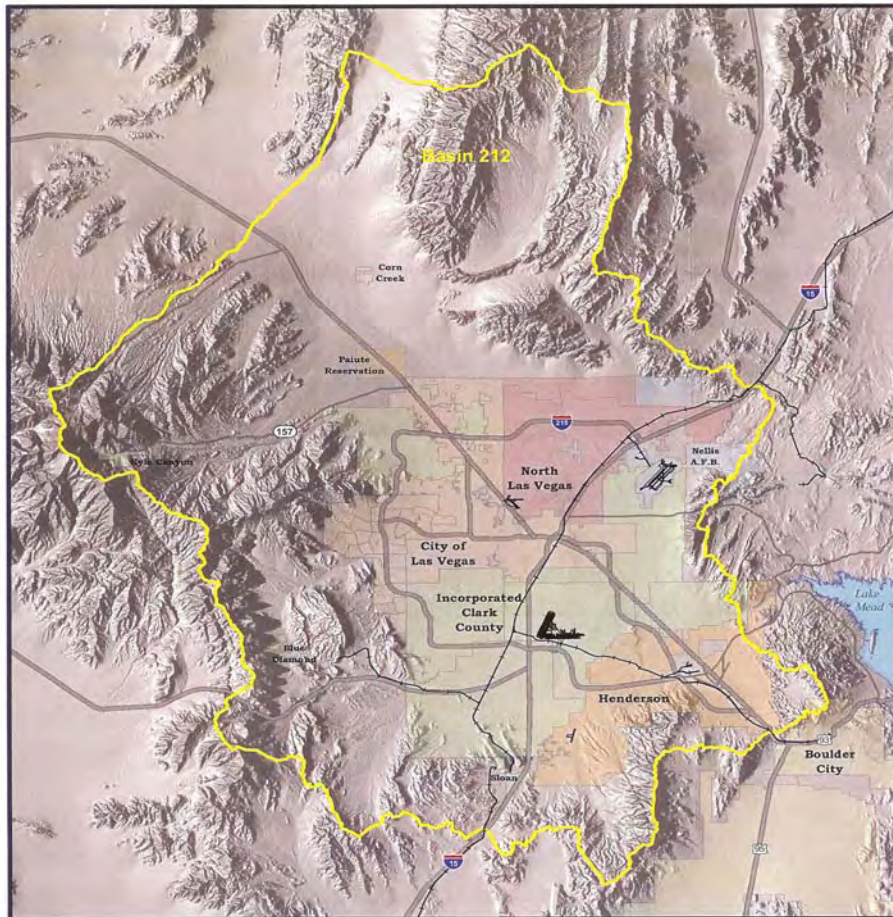
Prepared by: Tom M. Morris, ASR Systems LLC

Prepared for: AWWA Research Foundation

April 2006

Introduction

The Las Vegas Valley is located 20 miles west of the Colorado River and Lake Mead Reservoir, near the southern tip of Nevada. Potable water is provided to the valley from groundwater wells and treated surface water imports from the Colorado River. The Colorado River accounts for about 90 percent of the water supplied, while groundwater supplies the remaining 10 percent. Las Vegas Valley Water District (LVVWD) provides water service to about 1.2 million of Clark County's more than 1.8 million residents, who live in the City of Las Vegas and portions of Unincorporated Clark County.

Over-drafting of the groundwater aquifer from the mid 1950's forward resulted in lowering the water table up to 300 feet in sections of the valley. Nevada began importing Colorado River surface water into the valley in the early 1970's to supplement the use of groundwater. The system capacity was subsequently expanded to improve the ability to deliver the full allotment and any available surpluses. With this delivery potential, instead of watching allotment available Colorado River water flow down the river to be lost, portions could be captured and banked in the partially depleted Las Vegas Valley aquifer. This water could be used to supplement peak demands and to bank for future use bridging forward during development of additional resources for the community.

As the highest groundwater stakeholder, the LVVWD initiated banking in the late 1980's to capture available Colorado River water. This spurred the development of the Southern Nevada Water Authority (SNWA) in 1991 to manage water resources on a regional basis for its seven member water and wastewater agencies. This more regional bank allowed communities without aquifers under their service area to participate in banking activities in return for funding their share of banking costs. The regional role of SNWA allows LVVWD to recharge water as a regional resource for all participating communities. The SNWA additionally has agreements to bank water in Arizona and California. In 1997, the Nevada Legislature directed the SNWA to develop a program to protect and manage the Las Vegas Valley's groundwater supply. Under this program, all groundwater users in the Las Vegas Valley pay a nominal fee of $35.00 per acre foot. This includes municipal, industrial, agricultural, and domestic groundwater users. This helps fund the groundwater recharge program and the sealing of unused wells.

Site Setting - Hydrogeologic Framework

As part of the Mohave Desert, the valley receives an average of 4-inches of rain per year. The area is located in the Basin and Range physiographic province yielding mountains exceeding 11,000 foot above mean sea level (amsl) and a valley floor sloping southeast from 3,500 foot amsl to 1,700 foot amsl. The valley is surrounded by rugged sedimentary and volcanic mountains, forming an isolated bowl of alluvial basin fill thousands of feet thick. The basin fill is composed of predominately limestone and clastic sedimentary rock fragments, sand, silt, and clay sized particles. The formations grade from course material near the mountain edges to very fine grained material at the dry lake bed setting near the center and eastern portions of the valley (Plume, 1984). Multiple cemented gravel layers at various depths form impermeable boundaries. The sub-surface layering is inconsistent with no lateral continuity.

The valley is cut from the northwest to southeast with high angle faults. Several major springs emanate from these faults, discharging ultra-pure water from deep,

pressurized limestone aquifers. This provides the source of natural recharge to the valley fill aquifer, resulting in an upward head gradient. Surface water cannot percolate to the potable aquifer since it is too deep and isolated by impermeable layers. A surface river exits the valley discharging to the Colorado River. This flow is comprised of water from shallow groundwater seepage induced by excess landscape watering, highly treated wastewater, and urban surface run-off. Nevada receives credit for each gallon of Colorado River water returned to the Colorado River. Nevada's annual Colorado River consumptive use allotment is 300,000 acre-feet, plus return flow credits, provides a current delivery of about 500,000 acre-feet of Colorado River water annually.

Nearly two thirds of the Las Vegas Valley aquifer system is not usable for potable production without treatment due to salts, metals, or low yield. The northwest quarter of the valley contains the high yield, potable aquifer systems capable of a sustainable, long term Aquifer Storage and Recovery (ASR) program. Wellfield production yields vary from 750 gpm to over 4,000 gpm, with an average around 2,250 gpm. Recharge rates vary from 100 gpm to over 4,300 gpm, with an average around 2,500 gpm. Static water levels range from 100 feet to over 500 feet below surface. Ground water storage activity is within poor to well cemented alluvial gravels, sands, silts, and clay sized particles with production transmissivities that range 7,000 gpd/ft to over 250,000 gpd/ft. Over the program duration, the aquifer has recovered by as much as 100 feet in sections of the basin. Figure 1 displays the net potentiometric surface changes in the principal aquifer from Fall 1990 through Fall 2005.

Source Water Characteristics

The source water used for recharge is 100% Colorado River surface water treated to potable use standards. The treatment plant and distribution network typically used to meet the needs of customers is used to convey available treated Colorado River water to wells during times of low demand. LVVWD recharge activities occur during the non-summer months when system capacity is available, while demands for customer use are lowest, and during periods when Colorado River water is available. During the summer months, the wells are pumped to supplement system potable supply. Table 1 displays the recharge volumes from 1987 through 2005. The LVVWD recharged nearly 30,000 acre-feet of Colorado River water annually in 1999, 2000, and 2003.

Water Quality

The Colorado River water used for recharge has a slightly higher Total Dissolved Solid (TDS) concentration than the native ground water. This is in part due to the 2-3 times higher concentrations of Sulfate and Sodium. Table 2 displays the native water chemistry as compared to the proposed recharge water. Evaluation of the water mixing using PHREEQE indicated no adverse reactions with a slight potential for minor calcite precipitation (Brothers, 1988). Under the Class 5 UIC permit, the injection and recovery water is sampled routinely to validate the quality meets drinking water standards and no adverse impacts have occurred to the water during storage. The source water is chlorinated, so the development of disinfection by-products is closely monitored by compliance.

During the recharge operation period, the wells are checked weekly to monthly for compliance required parameters, including field water quality (EC, pH, and temperature), aquifer levels, and accurate volumetric measurement. Additional samples are taken from key points in the distribution system for laboratory analysis to validate the quality of the water recharged. Required parameters include:

- trihalomethane ($CHBrCl_2$, $CHBr_3$, $CHCl_3$, $CHBr_2Cl$),
- major ions (EC, TDS, SiO_2, HCO_3, Cl, SO_4, NO_3 as N, Na, Ca, K, Mg, and ClO_4),
- and trace ions (B, F, Ag, As, Ba, Be, Cd, CN, Cr, Cu, Fe, Hg, Mn, Ni, Pb, and Se).

During recovery, compliance data is collected weekly to monthly including flow, aquifer levels, field chemistry (EC, pH, and temperature), and laboratory water samples. Water quality analysis parameters include trihalomethane and major ions as noted above. In addition, perchlorate, a derivative of rocket fuel processing, is monitored.

Feasibility and Design Studies

The LVVWD has conducted many scientific tests on their ground water recharge system over the 18 years of operation. Prior to starting the program, the compatibility of the water chemistry was evaluated. Three chemistry mixing models, WATEQF, MIX 2, and WATMIX were developed to evaluate the mixing impact. The chemistry modeling indicated a slight potential of calcite precipitation that may damage wells (Broadbent, 1982). Additional laboratory tests were conducted with the source water on similar aquifer rock types. These tests also indicated the potential for calcite precipitation.

LVVWD moved cautiously to a pilot test at a single well to monitor the chemical interactions through successive injection and recovery cycles. Recharge was through an 8-inch drop pipe and perforated casing at rates of 500 to 600 gpm. A total of 3 acre-ft was recharged. No performance problems were observed related to the precipitation of calcite. A demonstration test of extended recharge was conducted next on two wells in the main wellfield. The test location would ensure full capture of the recharged water by the recovery well or existing production wells nearby. A total of 1,153 acre-ft was recharged. The wells were subsequently placed into normal summer recovery for five months. The source water was fully recovered by each recharge well with no recovery at other wells in close proximity. Performance was not damaged by precipitation of calcite, though the sanding potential of the wells at startup was increased.

The LVVWD has conducted tests on drilling methods, well construction methods, drilling fluid programs, well development programs, and the operational benefits of joint-use recharge and recovery wells versus dedicated recharge only wells. The research covered reverse circulation rotary, air/air foam, cable tool, drill and drive, and casing rotation well drilling methods. In other locations they experimented with enhancing formation permeability with the application of CO_2 and/or explosives, neither proved a high rate of success.

One notable experiment was a valley wide aquifer hydraulic test using over 50 recharge wells. The test was designed to understand the hydraulic impacts recharge played on the shallow, intermediate, and deep aquifer systems. Smaller scale injection tests were conducted in the three major recharge hydro-stratigraphic zones to define their

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unique response for receiving large volumes of water. Hundreds of wells representing the three valley aquifer layers were monitored during these tests within a 20-30 mile radius. The valley wide hydraulic test results became the basis for managing a mature recharge program, placing volumetric limits on specific geologic locations of the wellfield. The water is now logically proportioned to areas to achieve the desired hydraulic impact.

Groundwater withdraws induced ground subsidence in up to three locations in the Las Vegas Valley. This was mostly attributed to the de-saturation of fined grained layers. With the advent of the groundwater banking program, the majority of the water levels stabilized and, in most cases, reversed to a slow upward climb. Detailed survey work indicates that ground subsidence has stabilized in response to recharge. The use of satellite spectrometry has allowed accurate monitoring of the ground surface rise and fall in response to injection and pumping.

Facility Development

In the beginning, the program expanded a few wells at a time using older, existing wells completed 800 to 1,000 feet deep with 16 to 18 inch mild steel casing and mild steel louvered or mill knife screen. Recharge was through the existing vertical line shaft pump bowls. As long as a back-pressure could be sustained, and the water did not flow out the top of the well, the operation was acceptable. Early operations used labor intensive steps to manually reverse flow meters, remove check valves and drop pump laterals. In the early 1990's, the wells were modified to automate most of the recharge transition efforts and improve the delivery volume and pressure. Distribution system piping for the wells, normally plumbed to low pressure tanks, were cross-tied to high pressure mains. The pump shaft laterals were equipped with ratchets to prevent rotation. Bi-directional flow measuring electro-magnetic meters were installed along with a bypass "U" pipe around the system check valve. The by-pass pipe was equipped with an automated globe valve enabling SCADA start and stop control. The globe valve is piloted for pressure reduction, allowing the applied recharge pressure to be altered to control the performance of the well. The casing water level access ports were equipped with air transfer vents. Figure 2 shows a typical view of a production well converted for recharge. Upon completion of this phase, over 30 wells were available for recharge.

Concern of long term damage to vital production wells, the LVVWD embarked on installing multiple types of dedicated injection wells to assume the duty. Thirty one wells were drilled by a variety of techniques noted earlier. The wells were completed to depths of 800 to 1,200 feet, cased with 12 to 20 inch mild steel blank casing, and wire wrap screen. In 50% of the installations, the screen material was stainless steel. The wells were equipped with 150 to 300 foot grout sanitary seals to prevent the migration of water into overlying formations. Figure 3 shows a view of a dedicated recharge well.

These dedicated injection wells were equipped with a drop pipe and down hole orifice. The wellhead piping was equipped with a globe valve piloted to allow start, stop, and pressure reduction. The globe valves were plumbed with solenoid valves to allow automated SCADA control. Air release valves were moved as close to the wellhead as possible. Flanges and gaskets were placed on the well casing tops to enable sealing the wellbore for full casing pressure injection. Pipeline purge ports were installed and

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discharge paths established. Once placed online, operation of the dedicated injection wells became an issue due to annual well bore clogging. With no ability to pump or purge the wellbore prior to starting injection, residual sand, silt and bacteria are forced into the screen and formation each year the well is placed into operation. This results in a 10-20% loss in efficiency each year, rendering the wells unusable within 3-5 years.

During growth of the recharge program, the agency simultaneously expanded their peak recoverability by installing a new recovery wellfield. A total of 43 new wells were installed at depths of 1,100 to 1,400 feet deep, cased 20 to 24 inch with mild steel, and screened with predominately stainless steel wire wrap. The wells were drilled by reverse circulation rotary and constructed with a gravel envelope. The program increased the recoverability from less than 90 million gallons per day to nearly 160 million gallons per day, allowing simultaneous recovery of existing water rights and banked water under conditions of high demand.   Under demands to expand the recharge program, many of these wells have been converted to ASR wells. With several years of production duty on the wells, they did not clog or increase sanding potential in response to recharge.   Tests indicated favorable results of recharge at rates equal, and in cases nearly twice, the normal production rate. The ability to seasonally pump the wells clear of any un-desirable impacts such as air, sand, and bacteria, helps maintain a high operational performance year to year. The agency has permitted many of these wells with plans to equip then for recharge operation. Currently, 78 wells are permitted for recharge. Of these, 46 will be equipped for ASR activity and 32 of these are equipped for dedicated injection. The agency has utilized up to 66 wells during a recharge cycle with cumulative rates exceeding 100 million gallons per day.

The entire production and injection wellfield is connected through a sophisticated series of on-site signal conversion units transmitting site parameters through micro-wave antenna to the base command center driven by a SCADA system. In the event of main-breaks or large fires, sections, or all of the recharge wells can be stopped within minutes by the command center to preserve system capacity and pressure.

Operation Data

The LVVWD has over 100 wells available for production and recharge in the Las Vegas Valley located within the urbanized section of the community. The agency desires to have manual valves isolating waste, production, and recharge to ensure no accidental operation. Only by intention will select wells be opened for use. Therefore, the switch between recharge and production is a fully manual valve turning exercise. The target wells and the location in the aquifer for recharge are determined ahead of time by modeling and evaluation. This ensures that the volume is placed in specific locations to produce a managed response in the aquifer.

The wells are not purged prior to starting injection, resulting in any developed turbidity (bacteria, disturbed silt, etc.) in the casing being pushed into the formation. Recharge is through mostly pump bowls and orifices with a few borehole control valves in use. Upon startup, the air in the recharge column is pushed into the well, causing some aquifer clogging. A positive pressure in maintained most all times on the recharge tubing. A few wells are constructed and equipped to operate with the water level at land surface and a fully pressurized well casing.

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Water level in the wells is allowed to climb to within 25 feet of surface before the well is stopped and taken out of service for the season. The wells are not back-flushed at any time during the operating season. The recharge wells typically operate continuously for anywhere between 30 days and 7-months with very few interruptions or shutdowns. Mid season maintenance shutdowns usually result in automated restarts with no air valve closure, inducing more air into the formation upon startup. At the end of the recharge season, then wells are isolated with a manual valve.

Each summer, the agency pumped their approximate 40,000 acre-ft per year groundwater allotment, effectively recovering the majority of the injected water "molecules" and leaving the native groundwater as the "banked" component. In some small isolated sections of the wellfield, the water is left in the ground to migrate down gradient to receiving wells. In December 2004, the Nevada State Engineer issued an order creating an in lieu recharge program for the Las Vegas Valley Groundwater Basin. This program allows the agencies to obtain storage credit for permitted water not pumped. Up to 85 percent of the credits earned by not pumping under the program are recoverable, with 15 percent remaining in the aquifer in perpetuity. This "In Lieu Recharge" represents another tool in managing the water resources of Southern Nevada.

The ASR program raises the cost of the water from $199.00 per acre foot to $218.00 per acre foot for an average 20,000 acre-ft storage. This only includes treatment, transmission, maintenance, and technical support costs (Donovan, 2002). This does not include the original facility improvement investment. The cost of constructing two new wells for the program, amortized over 20 years, would increase the cost to $283.00 per acre foot for the 20,000 acre-ft program.

Performance Evaluation

The LVVWD, and oversight agency SNWA, collect hundreds of measurements from the shallow, intermediate and deep aquifers to evaluate the groundwater basin response and operation performance. In a valley wide coordinated effort, the spring and fall data is collected un-influenced by any pumping or recharge to provide high quality correlation points on the hydraulic changes in the individual aquifers. This data is then used for modeling to evaluate aquifer impacts and determine optimum recovery and recharge locations for the next year.

During initial production or recharge well season startup, baseline static water level data is obtained just prior to startup and correlated to 24-hour operation performance data. Following startup, the wells are generally monitored manually once a month, providing "high confidence" data points and field validation of any telemetered SCADA data points. Origionaly, the performance of the wells was based upon declining trends in recharge rates and climbing water levels. With the understanding today of well bore hydraulic actions and mounding, the performance of the wells have been evaluated with the startup Specific Capacity (SC) and Effective Hydraulic Area (EHA) methods. These methods have allowed the determination of clogging and the need for rehabilitation.

Production performance has suffered very little as a result of the recharge program. With no calcite precipitation issues, the main concerns became rising water levels and the increase in sanding potential observed from wells in a specific geologic setting. An unforeseen impact of the banking program included the production pumps

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pumping off their performance curve as a response to rising static water levels. In some cases, this allowed the equipment to over-pump the wells and cause formation damage and excessive sanding. In a mature, managed banking program, the water level impacts must be correlated to the well pump performance to ensure over-pumping does not occur. Tracking well bore performance in an aquifer gaining saturated thickness each year is difficult. This causes the specific capacity to increase, masking any potential losses induced by the recharge cycle. Recharge well performance became the leading indicator for well clogging at the ASR type facilities.

Recovery of the non-native recharged water bubble is near 100% within the year, with exceptions as noted. Pumping of the wellfield for normal summer delivery removes a greater volume than recharged, resulting in un-removed native water becoming the banked portion. Water recharged at the dedicated injection wells must travel various distances through the aquifer to recovery wells located a few hundred feet away to over a mile away. The amount of recovery from the water bank is limited to 10% of the banked volume in any calendar year. The current bank contains 306,808 acre-ft.

Problems Identified, Lessons Learned and Recommended Operational Practices

Over the 18 years of operation, the LVVWD has learned many key points about the design, operation, and maintenance of direct injection and ASR type underground storage programs. The agency has catered to sharing their technical knowledge with many water agencies throughout the world. Many of the early lessons learned are common practice in systems installed today. The key lessons are:

- A ground water recharge program could be successfully implemented using the existing, older wells pumping to reservoirs for sand removal. Recharge could be through the existing vertical line shaft pump bowls and column pipe at normal system pressures. The resulting injection rates could be 50% to 75% of the normal production rate for the well.
- Wells drilled and constructed by the reverse circulation rotary drilling method, gravel packed, and constructed for normal production provide the most efficient recharge and recovery wells. These wells generally can be developed to operate at higher rates than wells constructed by other techniques.
- To allow simultaneous recovery of existing water rights and banked water under conditions of high demand or emergency, the agency installed additional production wells enabling nearly twice the normal recovery capacity.
- The well facilities were modified to improve the delivery volume and pressure. Distribution system piping for the wells, normally plumbed to low pressure tanks, were cross-tied to high pressure mains.
- Recharge through submersible pump bowls was tested with poor success. Four out of five units exhibited repeated pump operation problems and motor failures. The pump motors did not generate stray electric currents during rotation since well pump motors are not typically equipped with "excitation circuits" required to turn the motor into a generator.

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- The location in the aquifer and the target wells for recharge are determined ahead of time by modeling and evaluation. This ensures that the volume placed produces a managed response in the aquifer.
- In the event of main-breaks or large fires, sections, or all of the recharge wells can be stopped within minutes by the command center to preserve system capacity and pressure.
- The dedicated injection wells have no ability to pump or purge the wellbore prior to starting, forcing residual sand, silt and bacteria into the screen and formation each year the well is placed into operation. This results in a 10-20% loss in efficiency each year, rendering the wells unusable within 3-5 years.
- In a mature, managed banking program, the water level rises must be correlated to the well pump performance curves to ensure over-pumping does not occur.
- In a valley wide coordinated effort, all pumping and recharge wells are shut off and the aquifer allowed to stabilize for a spring and fall "static" water level evaluation program. This provides high quality correlation points on the hydraulic changes in the individual aquifers. This data is then used for modeling to evaluate aquifer impacts and determine optimum recovery and recharge locations for the next year.

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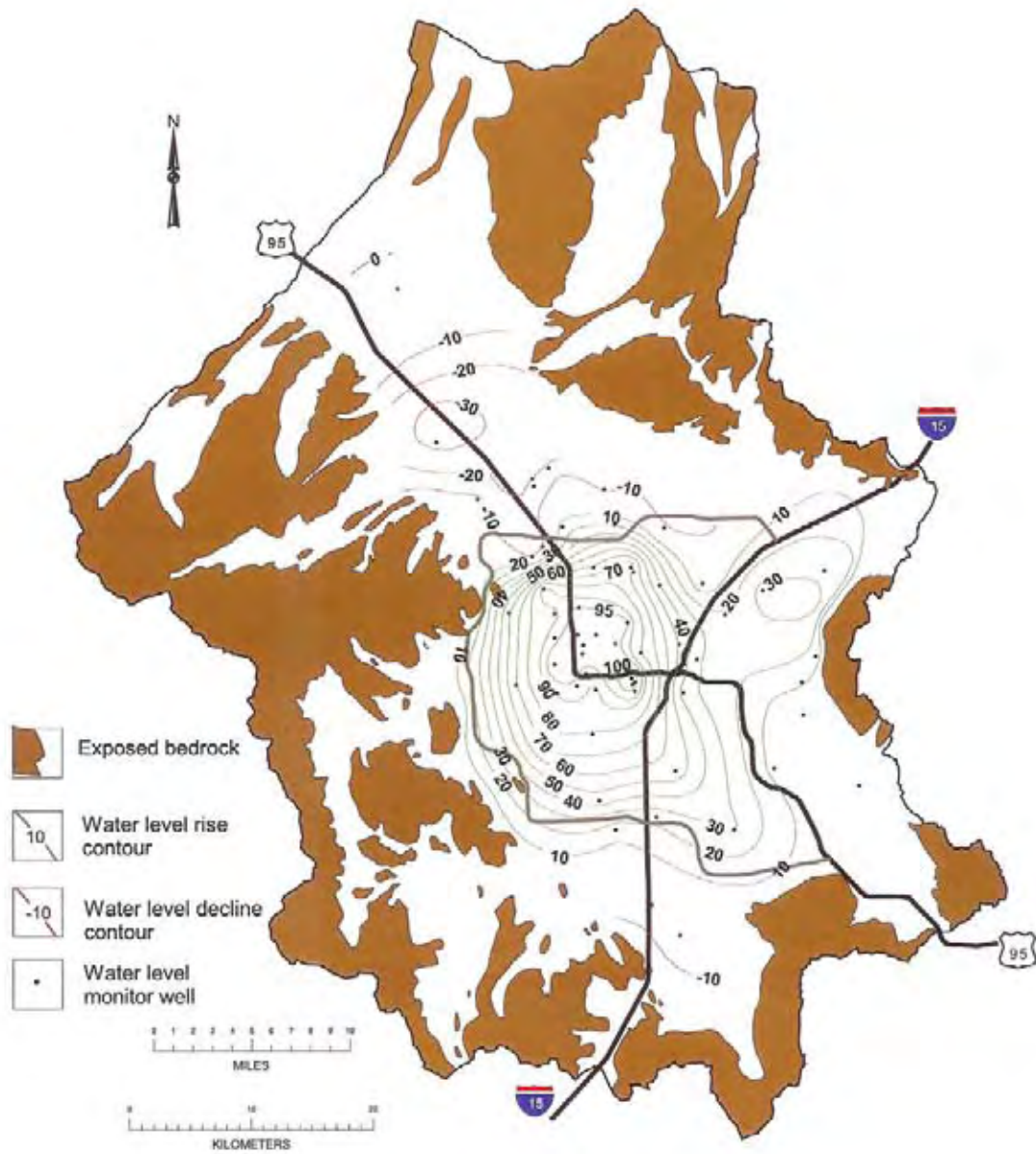
(water level decline, negative: water level rise, positive)

Figure 1. Feet Change in Water Level Surface of Principal Aquifer, 1990 to 2005

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Annual well production, recharge, recovery, and yearly totals since 1987

Year	LVVWD Well Production	LVVWD Colorado River Recharge	LVVWD In-Lieu Recharge Recoverable	LVVWD In-Lieu Recharge Unrecoverable	LVVWD Ground water Rights	LVVWD Recovery	LVPT Recovery
2005	31,661	15,867	7,621	1,345	40,626	0	572
2004	40,877	17,116	0	0	40,612	265	664
2003	40,127	28,540	0	0	40,126	1	985
2002	41,219	2,254	0	0	40,126	1,092	1,178
2001	40,620	21,269	0	0	40,126	494	1,205
2000	38,255	29,721	0	0	40,126	0	0
1999	39,028	32,061	0	0	40,126	0	0
1998	39,857	27,146	0	0	40,126	0	0
1997	40,316	17,791	0	0	40,152	164	0
1996	41,543	12,005	0	0	39,947	1,596	0
1995	42,720	16,661	0	0	40,247	2,473	0
1994	37,907	20,120	0	0	40,314	0	0
1993	35,647	23,868	0	0	40,314	0	0
1992	39,937	15,616	0	0	40,314	0	0
1991	36,653	14,621	0	0	40,314	0	0
1990	33,925	10,389	0	0	39,920	0	0
1989	34,025	3,676	0	0	39,890	0	0
1988	37,096	1,153	0	0	39,772	0	0
1987	37,145	2	0	0	39,682	0	0
Total Recharged: 309,876		7,621	1,345	Total Recovered:	6,085	4,604
Net Cumulative Storage : 306,808						

Table 1 ASR Recharge Volumes Over Project Life (LVVWD, 2005)

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

Chemical analyses of ground water from selected District wells[1].

Constituent	Local Well Number					
	21	1A	17	15A	23A	45
Date of Collection	3/87	10/87	8/87	10/87	10/87	8/87
	mg/L					
Alkalinity (HCO_3)	204	204	235	234	234	235
Calcium	85.0	68.5	54.1	49.8	51.6	49.4
Magnesium	36.0	35.7	27.1	25.4	25.8	24.3
Chloride	10.9	7.0	5.0	3.7	3.6	3.8
Nitrate (N)	0.72	0.94	0.64	0.47	0.49	0.42
Potassium	6.1	3.11	2.26	2.63	2.17	1.92
Conductivity (micromhos/cm)	729	607	461	438	443	424
Sodium	13.50	8.97	7.11	5.77	5.42	6.35
Sulfate	212	147	53.9	45.7	48.8	37.3
pH (units): Lab	7.90	7.84	8.06	7.93	7.60	8.01
pH (units): field	7.36	7.40	7.41	7.65	7.61	7.52

[1] Samples analyzed by Desert Research Institute Laboratory, University of Nevada System, Reno, Nevada.

Chemical analyses of Colorado River water in Lake Mead[1] before and after treatment.

Constituent	Date of Sample Collection							
	6/73		10/75		6/79		11/86	
	Raw	Treated	Raw	Treated	Raw	Treated	Raw	Treated
	mg/L							
Alkalinity ($CaCO_3$)	130	132	140	130	126	127	130	129
Calcium	90	90	86	90	86	86	78	74
Magnesium	26	26	22	29	29	28	18	21
Chloride	90	92	104	108	90	90	53	55
Fluoride	0.37	0.39	1.00	1.00	0.31	0.28	0.28	0.28
Nitrate (N)	1.70	1.70	0.07	0.02	0.48	0.48	0.41	0.44
Potassium	---	---	3.9	4.3	6.0	6.0	3.7	3.7
Conductivity (micromhos/cm)	1072	1057	1077	1035	1118	1107	846	857
Sodium	105	107	109	134	115	115	70	71
Sulfate	290	290	311	309	301	301	232	216
pH (units)	8.20	8.20	8.20	7.80	7.95	7.75	7.92	7.84

[1] Samples collected and analyzed at the Alfred Merritt Smith Water Treatment facilities located at Lake Mead, Nevada.

Table 2 Source and Native Groundwater Chemistry (Brothers, 1988)

Figure 2 Converted Duel Use ASR Well (LVVWD, Well 33)

Figure 3 Dedicated Recharge Only Well (LVVWD, Well AR-1)

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

AWWARF Case Study, City of Beaverton ASR Project, Beaverton, Oregon

Prepared by: LJHB Partners LC

Prepared for: ASR Systems LLC Team and the AWWA Research Foundation

March 2006

Introduction

The City of Beaverton (City) ASR Project is located southwest of Portland, Oregon, at the City's Sorrento Water Works (Groundwater Solutions, Inc., 2004). The ASR project receives its water from the Joint Water Commission (JWC), a joint water agency made up of representative governments including the Cities of Beaverton, Hillsboro, Forest Grove, and Tigard along with the Tualatin Valley Water District (Winship, 2006). The JWC's treatment plant is located near Forest Grove, Oregon, and receives its water from the Trask and Tualatin River basins via the Henry Hagg Lake and Barney Reservoir (impoundments operated by the U.S. Bureau of Reclamation). During the summer, stream flows are maintained by the release of stored water from the reservoirs. Beaverton provides water supply to over 64,000 customers. Beaverton has water rights ownership of 4,300 acre-feet of water from Barney Reservoir and 4,000 acre-feet of water from Hagg Lake (Winship, 2006). During some drought years, this amount of water allocation may not be available, leaving the City potentially vulnerable in its ability to meet summer peak demands. The main surface water source for Beaverton is supplemented by pumping native groundwater from the basalt aquifer interflow zones in the study area. Actually, Beaverton has utilized native groundwater heavily in the past, relying on it from the 1920s to 1983 (Winship, 2006). In 2003, JWC source water accounted for 3,164 million gallons of supply while native groundwater provided 49 million gallons. A map of the Beaverton study area is shown in Figure 1.
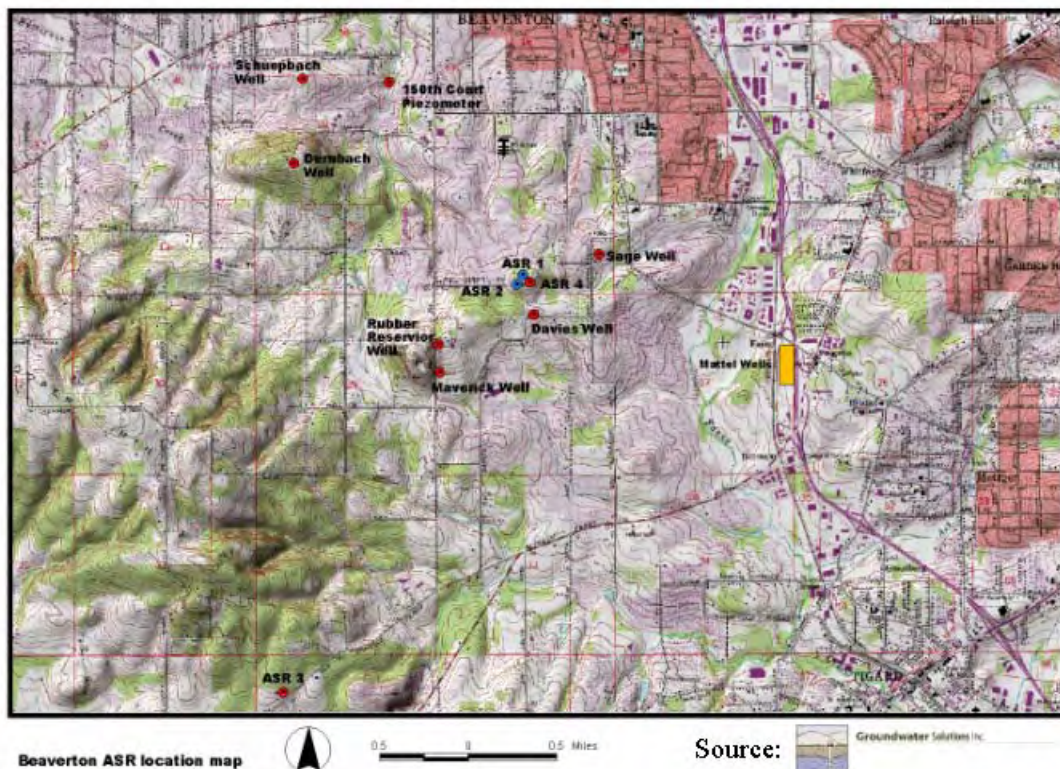
Figure 1 Beaverton Site Plan

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

The current ASR system consists of four ASR wells: ASR No. 1 and No. 2 are operational, No. 4 will be on-line in the summer of 2006, and the City intends to develop ASR No. 3 in either 2007 or 2008. The ASR wells store treated water in the basalt aquifer during winter and spring months when river flows are high. The stored water is recovered in the summer and fall months to help meet peak system demands.

The wells are also available during emergency situations, such as extreme weather or contamination of the primary water supply. The ASR system is designed to provide long-term and seasonal water supply in conjunction with the JWC supply.   Figure 2.x depicts the locations of ASR wells at the Sorrento plant location. It is interesting to note that the three ASR wells (ASR No. 1, 2, and 4) are located within an upscale residential neighborhood.
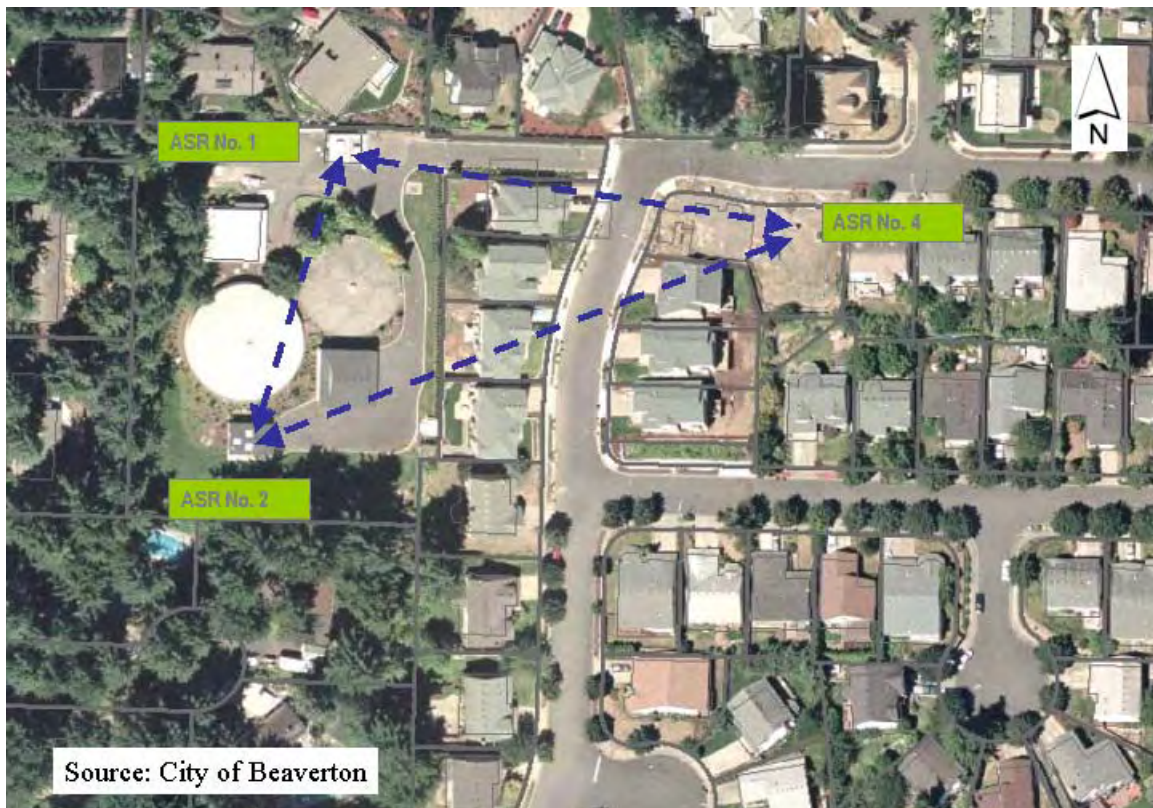
Figure 2 ASR Wells at the Sorrento Facility (not to scale)

Over the last 7 years, the facility has been expanded several times and will include operation of all three wells by the end of 2006, creating a total system capacity of approximately 5 million gallons per day (mgd) (Eaton, 2006). ASR No. 3 is anticipated to be developed as a 0.75-mgd well in the southern portion of the City (see Figure 1.x). ASR No. 1, 2, and 4, located at the Sorrento facility, have capacities of 1-, 2-, and 3-mgd, respectively. This case study report is based on existing project literature and results of a

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site visit and tour conducted on March 13, 2006. Those present for the site visit and tour included:

- Chris J. Brown, LJHB Partners LC
- David Winship, City of Beaverton
- Rick Weaver, City of Beaverton

The site visit focused on the site history, operating practices, hydrogeologic and engineering data, water quality issues, operational problems, innovative site practices, and lessons learned. Results of the site visit are discussed throughout this report and referenced as "personal communications" with either David Winship or Rick Weaver. Prior to the site visit, a detailed discussion of the project also included Tom Ramisch, City of Beaverton Director of Engineering, and Larry Eaton, hydrogeologist from Groundwater Solutions Inc., the primary project consultant.

Site Setting - Hydrogeologic Framework

ASR No. 2 and 4 are lined and screened at interflow zones within the Columbia River Basalt Group (Eaton, 2004). ASR No. 1, which was an existing well operated by the City, was drilled in 1945 and is an open borehole through the basalt section. The aquifer is highly fractured and behaves as a confined to semi-confined aquifer with average transmissivity of 13,000 square feet per day (ft2/day) and a storage coefficient of $1 \times 10\text{-}4$. The basalt section is over 1,000 feet thick in the region; however, fractured and highly permeable interflow zones provide the most productive portions of the aquifer. Intraflow, the basalt colonnade and entablature between interflow zones, is much less permeable by orders of magnitude, but it is believed that a hydraulic connection still exists between interflows as a result of localized fracturing. At the Beaverton study area, three primary interflow zones have been identified. In addition, existing geologic structures, including possible inferred faults, exist in the study area. These faults appear to isolate the groundwater basin. The aquifer is semi-confined with multiple leaky and non-leaky boundaries. Evaluating pump test data can be tricky at the site because the drawdown and recovery curves do not match an expected theoretical confined aquifer (Theis) response; the response is less than that predicted by the Theis formulation in early-time periods (1,000 minutes or less) and the response is greater than the Theis predicted data in late-time periods (Eaton, 2004). Figure 3.x depicts a conceptual hydrogeologic cross section of the study area with a theorized injection bubble superimposed on the cross section.

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.
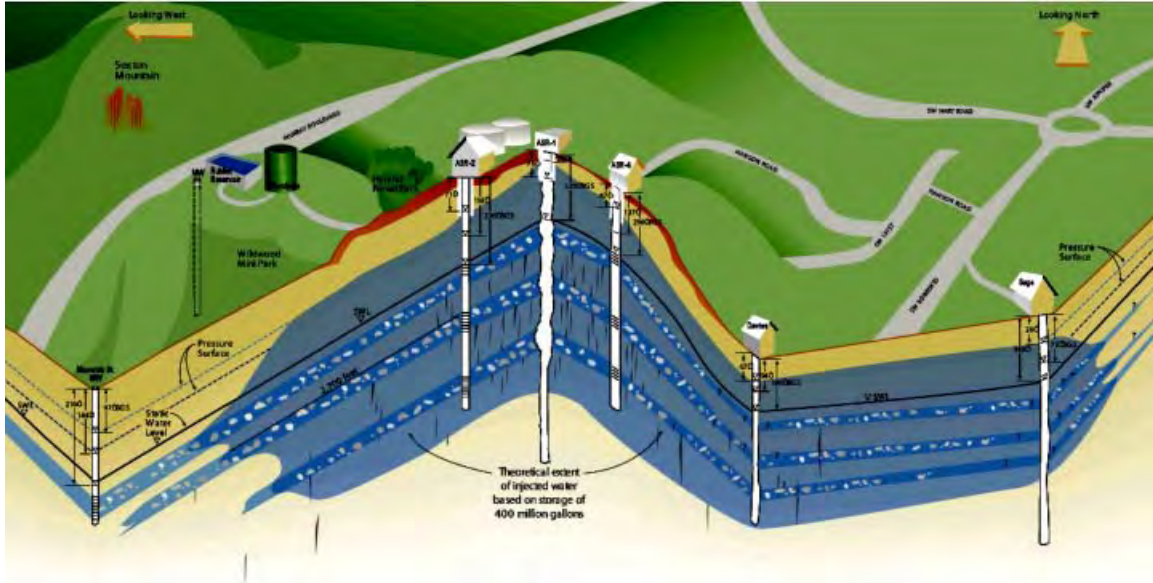
Figure 3 Conceptual Hydrogeologic Cross Section of the Beaverton ASR Project. (not to scale) *Source: Groundwater Solitions and CH2M Hill*

Source Water Characteristics

As mentioned previously, the primary water source for the Beaverton ASR project is water supply from the JWC treatment facility, which derives its water supply from the Trask and Tualatin Rivers. As a JWC owner-member, the City of Beaverton is entitled to up to 15 mgd of treated drinking water from the Forest Grove water treatment plant (City of Beaverton, 2003). The Forest Grove treatment plant includes the following treatment steps:

- Rapid mix adding alum, polymer and chlorine
- Flocculation and sedimentation removing sludge to sludge thickeners
- Filtration to remove suspended solids
- Storage in clearwell where chlorination and caustic soda is applied
- Delivery to in-town storm tanks or reservoirs

The City of Beaverton's water rights in the winter (November to late May) allow daily use of up to 16.2 mgd, while in the summer, its rights to water in both the Hagg and Barney reservoirs are utilized (City of Beaverton, 2003).

During 2003, the average daily water demand for Beaverton was 8.8 mgd while the peak summer demand rose to 16.8 mgd. A total quantity of 3.21 billion gallons of water was provided to customers during 2003 (City of Beaverton, 2003). Future 2020 demands are expected to rise to 12 mgd average and a 24-mgd peak summer value. Because of this anticipated growth, additional water supply projects are in the early planning phases and ASR will continue to play a critical part to help the City meet its current and future peaking demands.

Water Quality

Water quality of the source water and the ambient groundwater is deemed acceptable. The source water is treated at the Forest Grove water plant and usually contains total suspended solids (TSS) of approximately 10 milligrams per liter (mg/L) or higher. The treated water has excellent quality, but includes minor amounts of metals, disinfection by-products, and radionuclides. Table 1 provides a summary of the data for 2003 and 2004 (City of Beaverton, 2003 and 2004).

Table 1 Water Quality Characteristics Measures at Forest Grove Filtration Plant

Parameter	Units	Max Value Recorded for 2003	Average Value Recorded for 2004	Max Value Recorded for 2004
Turbidity	NTU	0.80	0.28	0.95
Barium	µg/L	3.7	2.9	5.9
Nitrate as N	mg/L	1.1	0.57	1.5
Sodium	mg/L	13	9.7	12
Fluoride	mg/L	0.06	NA	0.9
Chromium	µg/L	2	ND	ND
Copper	mg/L	0.37	0.24	0.26
Lead	µg/L	23	5	16
Radon	pCi/l	530	123	420
Gross Alpha	pCi/l	0.999	ND	ND
Total THMs	ug/l	51	35.6	56.3
Total HAAs	ug/l	81	37.4	318

Mg/L = milligram(s) per liter
NA = not applicable
ND = non-detect
NTU = nephelometric turbidity unit
µg/L = microgram(s) per liter

During recovery operations, minor amounts of radon, iron, manganese, and disinfection by-products (total trihalomethanes [THM]) were observed, but at concentrations well below regulatory screening levels. One interesting observation is that the concentration of total haloacetic acids (HAAs) was reduced to non-detectable levels during storage while the total THMs were reduced based on mixing patterns of other compounds. It is possible that biodegradation of the HAAs did occur at the Beaverton ASR site.

Overall, water chemistry data collected over the years on the project indicates a mixing trend from ASR injection through ASR recovery with no significant geochemical reactions observed at the site. Radon is the only constituent that appears to be quickly adsorbed by source water when it is injected into the basalt aquifer. Typically, radon concentration detected in recovered water range from 300 to 450 picocuries per liter (pCi/L), which are similar in native groundwater concentrations.

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Dissolved oxygen levels were usually higher during initial recovery operations than during extended recovery. This may be the result of biological growth within the wells caused via recharge of oxygen rich water. Recovery of injected water approached 100% and provided a high-quality and dependable water supply for the City.

Feasibility and Design Studies

Feasibility and design studies were both undertaken in support of the Beaverton ASR project. An initial feasibility assessment report was prepared by CH2M Hill and later design efforts have been jointly developed between Groundwater Solutions, Inc., and CH2M Hill. Design studies have included an evaluation of existing groundwater users in the area, hydraulic design, hydrogeologic studies, aquifer performance testing, water quality characterization and modeling, geochemical bench-scale testing, engineering design and layout, and initial pilot testing at each ASR well. Also, a limited amount of numerical and analytical modeling was conducted to determine appropriate well spacing and layout and in support of a wellhead protection study. The feasibility and design efforts indicated that the Beaverton ASR project was economically and technically sound. In essence, during the winter months when the Beaverton average demand is approximately 8.8 mgd and the total allocation is 14 mgd from the Forest Grove plant, an excess quantity of source water can be stored using the onsite ASR wells. In the summer time, the ASR wells can help in meeting peak demand or during extended droughts, or they can be used if there is an emergency situation with the source, such as a primary source line failure. In addition, the City has a native groundwater right, and can pump native groundwater with its ASR system when the storage account is depleted, if needed. All of these factors help to make the Beaverton project an ideal application of the ASR technology.

ASR No. 1 included a retrofit of an existing groundwater well that was drilled in 1945, most likely with a cable tool drill rig. This well is not lined, does not have a downhole control valve, and the piping is configured such that injection and recovery occur in the same pipe through a bi-directional clay-valve. This was necessary given the spacing constraints in the existing wellhouse of ASR No. 1. Subsequent upgrades of ASR No. 1 may include some surface casing seal upgrades, and potentially installing a downhole control valve because the static water level at the Sorrento site (host to the ASR wells) is roughly 200 feet below ground surface. Designs were optimized after pilot testing of ASR No. 1 and lessons learned were applied when ASR No. 2 was designed and constructed. However, one item was overlooked at ASR No. 2: The 250-horsepower pump motors were quite noisy because of motor winding. The sound was a nuisance to the nearby residential neighborhood, so additional soundproofing was retrofitted to the ASR No. 2 building. This issue was considered during design of ASR No. 4 and will be considered during design of future ASR wells for the City. One additional innovation at the project is the use of stainless steel liners, casing, and well screens. Because the wetting and drying zone of the aquifer is large, stainless steel is the construction material of choice. Stainless steel lining has been used in-lieu of an open-hole completion (as most study area wells use). Operating experience to date suggests that the smooth liner has reduced turbulent flow down the well and resulted in a higher well efficiency (Eaton,

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2004). A last innovative design/construction service used by the primary project consultant has been the development of a "dummy" submersible pump. Before the final installation of the custom submersible pumps (i.e., at ASR No. 4), a spray-painted steel dummy of similar size and diameter to the pump assembly was lowered down the well to check tolerances and overall spacing (Eaton, 2006). This idea is practical and could be used at any site where submersible pumps are under consideration and tolerances are limited in the well borehole. Use of the dummy ensures that the custom-ordered pumps will fit down the well with no additional retrofits necessary.

Facility and Operational Data

Both the onsite chlorination system and the ASR system are highly automated and controlled by a state-of-the-art supervisory control and data acquisition (SCADA) system. ASR well No.1 initially used manually controlled systems, but has evolved to a control taking full advantage of a robust SCADA system. The newer wells have always used a SCADA system to manage their operations (Weaver, 2006). One concern raised by the operating staff is that sometimes it is difficult to integrate various parts of the SCADA system designed by different firms. Standardization of the system would make future integration efforts easier for the operators. Also, the site operator indicated that future integration efforts would be made easier if the designers further involved the facility staff in the overall operational control concept and design (Weaver, 2006). Because of concerns about terrorism at all water plants in the United States, security infrastructure has become more important. The Beaverton facility has invested in security fencing, remote cameras, locked vaults/manways, and motion detectors (Weaver, 2006). In addition, because of security and safety concerns, 40-pound chlorine gas cylinders are not used at the site. Instead, onsite chlorine generators deliver chlorine residual via a diluted carrier water bubbled into the mains. This is an important safety concern in a rapidly urbanizing area where risks to offsite receptors is increasing. In case of an actual emergency, the control systems for ASR No. 1 and No. 2 are redundant allowing for quick changes at either site.

The Beaverton ASR project is unique in that it contains wells that include downhole control valves in ASR No. 2 and ASR No. 4 (under construction). As previously mentioned, ASR No. 1 was a retrofit of an existing groundwater supply well and was not equipped with a downhole control valve or a liner. Early on in cycle testing of ASR No. 1, air entrainment was problematic because of the depth to the water table (at least 200 feet) and well recharge that occurs down the column (Eaton, 2004). Operations at ASR No. 1 have since been optimized to minimize possible air entrainment, however, to avoid the issue entirely, downhole control valves were included at the other three wells and, as mentioned earlier, upgrades to ASR No. 1 may include installation of a downhole control valve. Besides the obvious prevention of air entrainment into the aquifer, the downhole control valves allow recharge flow to vary over a larger operating range by optimizing the downhole valve, pressure reducing valves (PRV) in the system, and the SCADA system; this provides greater operational flexibility during the recharge period.

Typically, when source water is plentiful in the winter time, the ASR system is recharged continually with the total recharge flow split evenly to all in-service ASR wells. Currently, the facility staff operates the ASR system to meet system summer demands. According to the state-issued ASR limited license or permit granted to the City of Beaverton for the ASR system, only 95% of the stored water can be recovered, ensuring that at least 5% residual is always left in the aquifer (Groundwater Solutions, 2006). This "tax" was added to the limited license to ensure that the regional aquifer (which is closed to further appropriations) could not inadvertently be negatively affected by ASR activities (loss of water from the aquifer caused by head changes). However, this condition may not be needed or added when the City applies for an operational permit in the future because data have shown that the aquifer has not been negatively affected by ASR activities. In fact, the residual amounts left in the aquifer have caused the regional aquifer to rebound enough that future storage volumes may have to be reduced. The operation of the ASR system is user friendly because of the SCADA system, however, ongoing regulatory-driven water quality data collection is rather onerous and can be quite time-consuming and expensive. Reductions in water quality sampling requirements are being negotiated with the state. Moreover, the City is looking at ways to streamline and automate field data collection activities.

Performance Evaluation

The Beaverton ASR system has posted an impressive performance record. During a 6-year operational period, the ASR system has successfully stored approximately 1,629 million gallons of potable (treated) water and recovered approximately 1,389.8 million gallons consisting of stored ASR water and 320 million gallons of native groundwater. This represents a net recovery utilization of approximately 85 percent for the entire project, assuming native groundwater pumping is not included in the calculation. Table 2 provides an operation summary of the Beaverton system since its inception.

Table 2. Beaverton ASR Operational Summary

Cycle Test #	Wells Used	Year	ASR Volume Stored (MG)	ASR Volume Recovered (MG)	Native GW Recovered (MG)	Total Recovery (MG)
1	1	1999	1.7	1.7	0	1.7
2	1	1999	34.5	32.7	37.3	70
3	1	2000	74.7	70.9	27.1	98
4	1 & 2	2001	159.1	151.2	99.8	251
5	1 & 2	2002	310.5	294.9	106.3	401.2
6	1 & 2	2003	394.6	364.9	49.2	414.1
7	1 & 2	2004	444.3	202.6	0	202.6
8	1 & 2	2005	209.5	270.9	0	270.9
Totals			**1,628.9**	**1,389.8**	**319.7**	**1,709.5**

MG = million gallons.
Source: Groundwater Solutions (2006).

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In addition to the full water budget displayed in Table 2, the Beaverton team has developed an accounting of the water demand since 1999 and components of the supply used to meet the demand. Figure 4 displays the system demand and overall performance from 1999 to late 2004. Figure 4.x clearly shows that peak demand is over 14 mgd and that ASR recovery has allowed Beaverton to meet the peak demands in-lieu or more expensive options such as piping upgrades. Figure 4 also reveals that recovery of native groundwater is still an important component of the water supply system. As the ASR system grows further at the Beaverton site, the use of only native groundwater should diminish, which will help maintain water levels in the native groundwater aquifer in this area that has been over-appropriated in the past.
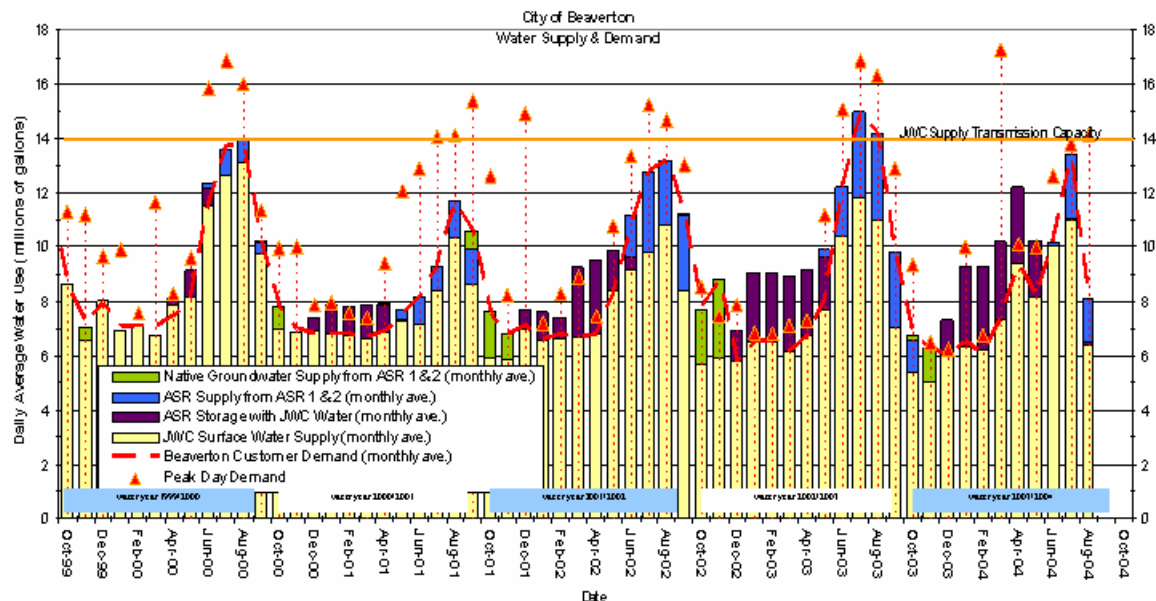
Figure 4 ASR Recharge and Recovery Volumes Over Project Life (1999-2004)
Source: Groundwater Solutions and City of Beaverton

An economic evaluation of the system completed by the City in 2006 has estimated the unit cost of the project to be moderate at $1.26 per thousand gallons recovered (Winship, 2006). The wholesale rate charged to customers is approximately $2.50 per 1,000 gallons. The project appears cost effective when compared to other water supply alternatives, largely because of the economies of scale derived from the large amount of water that has been stored and recovered at the site. The ASR system unit cost was significantly less than the cost of other water supply options considered (Eaton, 2004). Table 3.x displays an annualized cost comparison between the ASR option and the conventional surface water supply option. Beaverton also compared conventional storage tank costs to ASR storage. Table 4 provides a comparison of capital costs.

Table 3 Beaverton Cost Comparison *Source: City of Beaverton – David Winship*

	Cost to Supply 500 MG Storage (annualized)	Unit Cost ($/1,000 gallons)
ASR annual capital cost	$209,749	$0.420
ASR annual O&M cost	$417,303	$0.834
ASR annual total cost	$627,051	$1.26
JWC annual capital cost	$565,516	$1.130
JWC annual O&M cost	$224,635	$0.449
JWC annual total cost	$790,151	$1.58

Table 4 ASR Storage Capacity Comparison of Storage Capital Costs
Source: City of Beaverton

Storage Facility	Year of Cost	Cost (original)	2006 Cost	Storage Volume (MG)	Storage Cost/MG (2006 $)
ASR Well No. 1 (Hanson Road Well)	1997	$724,320	$924,051	100	$9,241
ASR Well No. 2	2001	$1,228,881	$1,428,291	150	$9,522
Proposed ASR Well No. 3 (Estimated Total Cost)	well drilled 2001	$1,032,504	$1,199,259	50	$23,985
Proposed ASR Well No. 4 (Estimated Total Cost)	2004-2006	$2,159,833	$2,273,511	200	$11,368
Sexton Mountain (15 MG) Reservoir, City of Beaverton	1995	$7,289,000	$10,596,284	15	$706,419
JWC Fern Hill No. 2 (20 MG) Reservoir and connective piping to WTP and transmission lines	2005/2006	$25,210,247	$25,210,247	20	$1,260,512

Table 4 clearly shows that the ASR cost per million gallons of storage has a much lower capital cost than comparable steel storage tanks. This also may make ASR options more attractive to users because of cash flow flexibility provided by ASR.

Problems Identified, Lessons Learned, and Recommended Operational Practices

Several issues have been evaluated at the Beaverton site over the operating project life. First, well clogging issues have been managed successfully through the use of a frequent back flushing program. Specific capacity is monitored over time to evaluate declines resulting from well clogging. After the specific capacity has diminished a set percentage, back flushing activities are started. Monitoring also has helped the facility staff determine the extent of local and regional water level changes in the aquifer. During recharge events, high water levels were noted near the ASR wells. The pressure mound resulting from ASR activities has been mapped, using a detailed monitoring network, which has helped to determine where no-flow boundaries are located and where potential leaky boundaries are located. The pressure response does not mimic a typical Theis-like response because of the semi-confined nature of the highly bounded and leaky basalt aquifer. The high water levels (potentiometric surface) were attributed partially to

A-93

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increased flow in an area seep (preferred pathway). In an urbanized environment, seeps can be problematic because of perceived flooding risks and slope stability issues. Monitoring of existing seeps before and during recharge events can aid in the interpretation of the seep flows and help determine what contribution ASR is having on the seep. Ultimately, some seeps related to ASR activities may need to be mitigated if there is a risk of flooding streets or nearby homes.

Second, the static water level of the basalt aquifer is approximately 200 feet below land surface, and without a downhole control valve, air entrainment can be problematic at the site. This problem has been eliminated at the newest ASR wells at the site because they were equipped with downhole control valves. ASR No. 1 still must contend with the air entrainment problems, but operating experience has resulted in minimization of the issue. The Beaverton ASR project primary consultant has provided some additional valuable lessons learned as follows:

- A network of monitoring wells is important to track the dynamic response of the aquifer resulting from ASR operation.
- Tracking water quality in a database has been helpful.
- Back flushing the wells is critical to maintaining head buildup. Recommend frequency of back flushing to be at least every 3 to 4 weeks at each ASR well.
- Installing a turbidity meter to monitor for potential "dirty source water" is important; the system has to be designed to shut down automatically in the event of a turbidity event to minimize any impact to the ASR well.
- ASR wells can be sited pretty close together as long as interference during injection and recovery are taken into account during design.
- It is important to make sure the diameter of the borehole is large enough to accommodate the pump, downhole control valve and its control lines, transducer drop pipes, and water level drop pipe. The Beaverton staff has found that if you try to squeeze too much in the borehole, it can create problems. The use of a "dummy"mock up of the pump assembly or other equipment can be useful during the testing and design phase.
- Cascading water from interflow zones that are dewatered during pumping can result in air entrainment in recovered water, if the up-hole velocities are high, which can be the case in smaller-diameter wells with high-capacity pumps.
- It is important to note that the City also completed a detailed wellhead protection plan and installed several monitoring wells near the leading edge of the 10-year time-of-travel boundary as an early warning system. Upgradient of the ASR facility is an old quarry, where past filling activities left some doubt as to how clean the fill was. This was a very proactive approach by the City to ensure that an early warning system was installed to protect its ASR system and its investment in the system.
- Monitoring of existing contaminated sites that might be affected by the ASR project may be important. At the Beaverton project, the Mattel site, which has solvent-contaminated groundwater, is located approximately 1.5 mile east of the City's main ASR wells. It has been monitored during operation of the City's ASR wells to determine if there are effects from recharging the regional aquifer and to ensure that ongoing mediation is not affected. After several years of monitoring, it is clear that a

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boundary fault hydraulically separates the Mattel site from the City's primary ASR facility.

- Lining old well boreholes can reduce turbulence during injection and reduce head loss during injection and recovery.

The Beaverton project appears to have a bright future, more so if the ASR system is expanded further. Future expansion of the ASR system will be considered as additional information and knowledge are gained through operation of the existing ASR wells.

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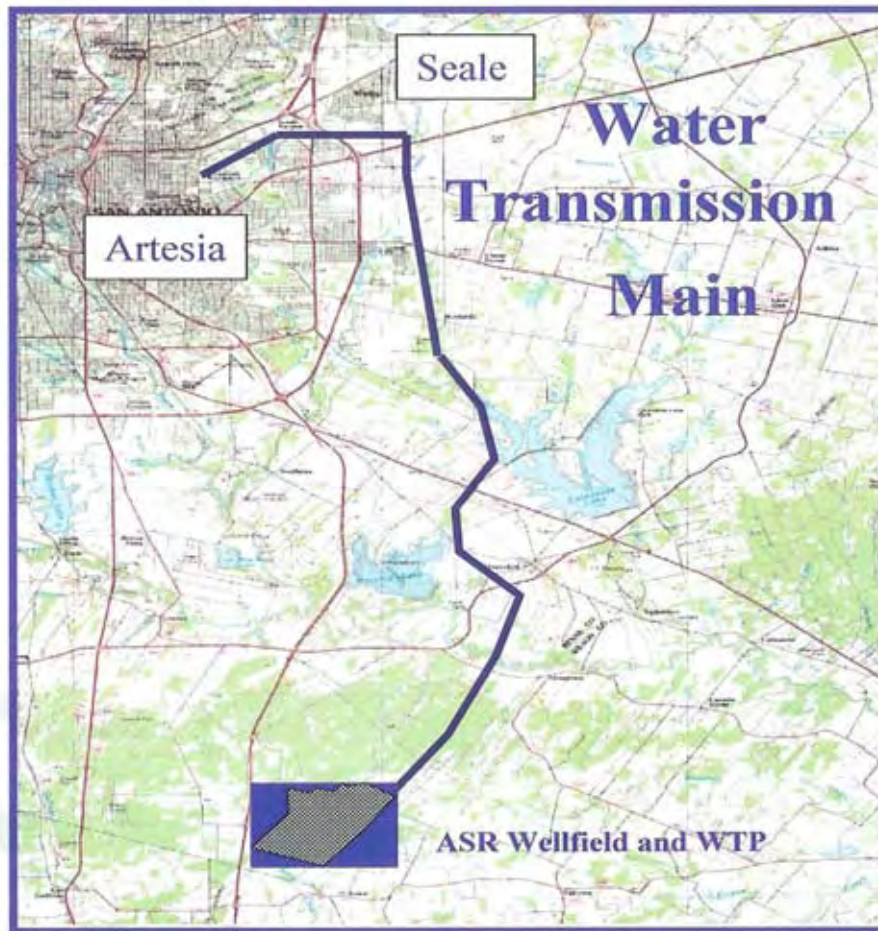
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Introduction

The San Antonio Water System (SAWS) has developed a unique aquifer storage and recovery program banking groundwater from one aquifer into another. In the rural farmland of south central Texas, 30 miles south of San Antonio, the Twin Oaks ASR Facility was built to bank potable water for seasonal recovery during peak demands. The system is designed to recharge excess Edwards Aquifer groundwater during times of plenty into the poor quality Carrizo Aquifer. To ensure no adverse system impacts occur from recovery of the recharge water and any native Carrizo Aquifer water, all water is treated back to the normal system quality before it is pumped back to San Antonio.

The Balcones Fault Zone Edwards Aquifer (Edwards Aquifer) in south central Texas is one of the most permeable and productive aquifers in the United States. The San Antonio segment of the aquifer extends a distance of approximately 180 miles. The aquifer is the primary source of water for approximately 1.7 million people in the region and provides most of the water for agriculture and industry. In addition, the aquifer discharges through a series of large springs that provide aquatic habitat for a number of threatened and endangered species. Spring flow also provides a significant portion of water for downstream interests in the Guadalupe River basin.

The Edwards Aquifer is a karst aquifer, characterized by the presence of sinkholes, sinking streams, caves, large springs, and a well-integrated subsurface drainage system. The aquifer exhibits extremely high porosity and permeability allowing it to transmit very large volumes of water, such as rainfall (recharge) events, through the aquifer very quickly. The flow from two notable aquifer discharge points, Comal Springs and San Marcos Springs, was greatly reduced, or ceased, during drought periods in 1956, 1966, 1971, 1984, 1989, 1990, and 1996. In response, the Texas Legislature passed Senate Bill 1477 in 1993 mandating the reduction in pumping from the Edwards Aquifer. The Edwards Aquifer Authority was formed to protect the aquifer and set staged limits for with-drawls. In order to maintain adequate spring flow for habitat protection, the elevation of the aquifer at select control point wells dictates the pumping limitations. If the water level is above the 70-year mean, then normal allotment pumping is allowed. As the aquifer level drops into concerned and critical levels, allowable pumping volumes are reduced.

The Twin Oaks Recharge Facility is designed to bank water from the Edwards Aquifer during high water level conditions for delivery back to the community during restricted aquifer pumping periods. The $250 million facility began operation in 2004 and consists of 30 miles of 48 to 60 inch delivery pipeline, 17 ASR wells capable of storing 7.3 billion gallons per year; a 30 MGD water treatment plant to remove iron and manganese; a 3-million gallon on-site storage tank; and a bank of 1000+ horse power lift pumps to reverse the flow in the sole supply line and return the treated water back into the San Antonio community water system. The technology allows better management of the year-round water resources passing through the San Antonio territory. It provides an additional source of water in the SAWS portfolio that can be called upon during drought periods.

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Site Setting - Hydrogeologic Framework

Extensive research went into selecting an isolated aquifer to bank approximately 30,000 AF. A unique set of stratum, the Carrizo-Wilcox Formation, was selected for its marginal water quality and low demand, depth and confining nature, and the lack of hydraulic connection to the Edwards Aquifer or regional rivers. The Carrizo Aquifer is a sandstone aquifer with a pH of 5 containing elevated iron and manganese. The aquifer is used locally for farmland irrigation.

The ASR wellfield is located in the Texas Coastal Plain on the downthrown side of the Balcones Escarpment. Formation dip is approximately 150 feet per mile in the south south-east direction. Several mapped and un-mapped extensional faults trend in the north-northeast direction. The target formation, The Carrizo Sand, is a medium to very course grained, noncalcarious sandstone. It is friable to indurated with thick beds and local iron-oxide banding. The Carrizo Sand ranges from 700 to 800 feet thick in the area and yields moderate to large supplies of fresh water. The wells are typically screened between 400 feet and 700 feet below surface in the formation. The Reklaw Formation composes the confining layer over the Carrizo Sand. The 200 foot thick unit is a fine to medium grained sandstone and silty clay with abundant hematite, glauconite, and muscovite. The Wilcox Group underlies the Carrizo Sand and is composed of mudstone and varying amounts of sandstone and lignite.

The local farmers and Bexar Metropolitan Water District had concerns the ASR facility would adversely impacting the local aquifer. This resulted in an agreement with the SAWS to rectify any damage to the aquifer and well production as a direct result of the facility operations. The heart of the concern is in the "big city" folks coming into their small community and over-pumping the aquifer, leaving the livelihoods of the locals dry. Under Texas law, property owners are permitted 2 acre feet per year groundwater production for each acre of land. The facility includes 7,000 acres of retired farmland netting 14,000 AF potential water production rights from the native Carrizo Aquifer. During the recovery of the banked water, some mixing is anticipated resulting in recovery of Carrizo aquifer water as well. Ultimately the banked water as well as the permitted Carrizo water adds to the total water sources for the SAWS.

Source Water Characteristics

The source water availability can be highly variable depending on seasonal weather conditions. Cyclic droughts will result in seasons with little to no recharge water available, providing poor relief to reduced pumping mandates during the summer. During seasons above the mean, water will be placed into storage. Banked water not taken by the summer demand will then be available for protection during drought years.

To boost the water available from the Edwards Aquifer, the regional aquifer authority of the time installed infiltration basins on the exposed sections, or natural recharge sections, of the Edwards Formation in the mid 1970's and early 1980's. The pilot basins recharged over 150,000 AF in the 30 years of operation. This was only a fraction of the nearly 28-million AF of surplus storm water that flowed through the creeks during the same period.

Current research by the Edwards Aquifer Authority, in support of SAWS, is ongoing to evaluate the ability to install more infiltration basins in the Edwards Aquifer recharge zone. Computer modeling is being used to evaluate the hydraulic impact on the aquifer as a result of more recharge. In the karst setting, the recharged water could move quickly through the system. Spring discharges will increase as a result of the added system head. These losses could add up if the water is allowed to sit there too long. This is where the Twin Oaks Recharge Facility allows this managed recharge to the Edwards Aquifer to be wheeled some distance through the aquifer, removed through pumping, and then piped south to be placed back underground in the Carrizo Aquifer for extended storage. Development of this program will increase the sustainability of the Twin Oaks operation.

Water Quality

Typically the concern is if the source water will plug the receiving aquifer and what level of pretreatment is necessary to prevent this. In this case, native groundwater pumped from one aquifer is piped and stored in another aquifer of lesser quality. The storage aquifer has much slower groundwater movement allowing retention of the water for later recovery. The source water for recharge is taken from the potable distribution system containing a minor dose of chlorine for disinfection. The water has very low turbidity, total suspended solids, salts and metals.

The Twin Oaks Recharge facility must be maintained in some sort of recharge or production operation most all of the time. During non-operational periods, there is a chance the slow or non moving water in the large diameter transmission main between the facility and San Antonio will stagnate. The volume of water stored in the pipeline is significant, in the order of 18 to 20 million gallons. Therefore, purging this water to waste on any routine basis is a large loss. During the two years of operation so far (2004 to 2006), the regional precipitation has been above normal allowing the facility to be operated in the recharge mode to develop a base water bank in the Carrizo Aquifer.

Figure 1: Water Treatment Plant (Morris, 2006)

The water quality issues for the recharge program are tied to the marginal quality of the receiving aquifer and the impact this plays on the recovered water. The native Carrizo Aquifer water has a pH of approximately 5.5 due to the extremely high levels of carbon dioxide, Iron at 50 times over the safe drinking water standards, Odor at 67 times

the recommended limit, Hydrogen Sulfide at 40 times the limit, and Manganese at 6 times the recommended maximum concentration level for drinking water. In order to recover all the recharged water, some portion of the native Carrizo Aquifer water would be removed from the transition zone of the two water types, degrading the resulting water quality. The 14,000 AF of water rights the SAWS has on the Carrizo Aquifer supports the volume of native water removed for mixing, though the primary intent for this right is to serve as an additional water supply to the City of San Antonio. For this reason, a full scale 30 MGD water treatment plant was constructed at the Twin Oaks Facility to treat the native Carrizo Aquifer water and recharge water blends to normal distribution system standards.

Contaminant	Units	Raw Water Design	MCL	Finished Water Goal
Primary Drinking Water Contaminant Levels				
Radium-226&228, Total	pCi/l	4.50	5.00	<3.0
Arsenic	mg/l	<0.01	0.05[1]	<0.01
Secondary Drinking Water Contaminant Levels				
Color, True	color units	20	15	<3.00
Hydrogen Sulfide	mg/l	2	0.05	<0.03
Iron	mg/l	15	0.3	<0.2
Manganese	mg/l	0.30	0.05	<0.03
PH		5.50	>7.0	>7.8
Odor	T.O.N	200	3	<2.5
Other Parameters				
Alkalinity, Total	mg/l as $CaCO_3$	30	NA	>100
Calcium	mg/l	23	NA	>40
Dissolved Oxygen	mg/l	0.10	NA	>2.0
Hardness, Total	mg/l as $CaCO_3$	35	NA	>100
Radon	pCi/l	160	300[2]	<100
Turbidity	NTU	0.20	<0.5	<0.1
Corrosion Indices				
Langlier Saturation Index		-2.0	NA	>0.20
Ryznar Index		10.1	NA	<7.1
Disinfection By-Products				
Bromate	mg/l	NA	0.010	<0.005
Total Trihalomethanes (TTHM)	mg/l	0.007	0.080	<0.040
Haloacetic Acids (HAA5)	mg/l	0.004	0.060	<0.030

[1] A lower arsenic MCL of 0.01 has been proposed but not yet approved and implemented.

[2] Radon is not currently regulated, MCL shown is proposed

Table 1: Receiving Aquifer Water Quality Issues (SAWS, 2005)

The water treatment plant processes include alkalinity adjustment, aeration, chemical addition, solids content clarification, filtration, and then disinfection. Back-flush fluids are routed to lagoons for evaporation and solids separation. The plant output meets or exceeds all Texas Commission of Environmental Quality regulatory requirements including all national primary and secondary drinking water quality standards. This ensures that the water provided by the Twin Oaks Facility does not impact or corrode the distribution system resulting in water that is not aesthetically pleasing to the customer.

Feasibility and Design Studies

The Twin Oaks Recharge facility ranks within the top few facilities in United States utilizing most all the current technology advancements for a large scale aquifer storage and recovery program. The agency did their homework by interviewing many other agencies performing aquifer recharge through wells. This compilation of the 2000 era technology allowed SAWS to conduct a site characterization and feasibility program; design an isolated recharge and recovery facility; and develop the program into a full scale operation by the year 2004.

Site characterization for an isolated facility was conducted between 1996 and 1998. As a result of the investigation six potential storage zones, designated by unique geographic area and aquifers, were selected for more detailed assessment. This included three distinct aquifer bodies in the Trinity Aquifer, the brackish Edwards Aquifer, the Wilcox Group, or the Carrizo Aquifer. Preliminary compatibility evaluation of the five potential source waters with each of the potential storage zones was completed using the generalized native water quality information and aquifer mineralogy. From this study, a development and operating cost analysis was conducted to rank to options. The Carrizo Aquifer option provided an estimated operations and maintenance cost of $0.11 per 1,000 gallons as compared to the worst case at $0.34 per 1,000 gallons. The marginal cost of water produced from the Carrizo Aquifer was estimated to be $82 per AF as compared to $398 per AF for the Lower Trinity Formation.

By the spring of 2000, the SAWS completed the preliminary field investigation program to define the characteristics of the Carrizo Formation and aquifer at the proposed facility location. This included the installation of 5 test borings from 700 feet to 1,900 feet below surface. The test borings were completed as monitoring wells. Chemical and geochemical analysis on the water and boring core samples concluded that there were no significant concerns regarding implementation of an ASR project in the Carrizo Aquifer using Edwards Aquifer water as the source water.

The aquifer capacity evaluation suggested that up to 60 MGD of storage capacity could be developed. Assuming an average seasonal recovery cycle of 4 months, the wellfield could provide approximately 22,400 AF of supply to meet peak demands. The analysis also indicated that continuous 2-year recovery at 60 MGD could be achieved with no adverse impacts to the aquifer or up to 30 MGD for a 50 year yield of native Carrizo Aquifer water.

By October 2000, a preliminary design was completed for the wells, treatment plant, on-site piping, and supply line. A total of 11 options for the supply line connection to the San Antonio source water were considered. The 30-mile pipeline route was

selected based upon project schedule, property acquisition, ease of constructability, costs, and integration of the returned water at points of greatest need to support growth.

The facility was designed and constructed by the spring of 2004. Three day, 10 day, and 20 day recharge and recovery cycle tests were conducted on two wells using Edwards Aquifer water. The tests indicated that the recharge water bubble did not absorb constituents such as arsenic with the recovered water meeting drinking water standards. Water levels did not exceed land surface during any of the testing periods. There was no notable changes in well efficiency during injection with any loses being fully restored following recovery pumping.

Facility and Operational Data

A total of 17 wells were installed to form the Twin Oaks wellfield. Each well site included a small diameter pilot exploratory well drilled mud rotary and completed to depths of 600 to 775 feet. These bore holes were used for lithologic control and geophysical logging. Nearly half of the facility ASR wells were drilled mud rotary with bentonite and synthetic polymers. The other half were drilled with flooded reverse circulation utilizing a light-weight water, polymer, and bentinite fluid. Each drilling method resulted in substantial formation invasion by the drilling fluids. The reverse drilled wells generally produced a slightly higher efficiency well.

Figure 2: ASR Wellfield Layout (SAWS, 2005)

The 24-inch boreholes were cased with 16-inch epoxy lined mild steel blank pipe and stainless steel, wire-wrap well screen. One well utilized a pre-pack gravel screen. Well screens are located in select segments between 400 and 700 feet below surface. Dielectric couplings were placed between dissimilar metal casing sections. The wells were gravel packed with silica based sand and gravel feed tubes were placed into the

annular space (except for the pre-pack screen well). In two wells, gravel feed tube failures resulted in cement grout flowing into the tubing breaks and down into the screen zone. One well was damaged beyond repair, leaving 16 wells available for service.

Figure 3: Well Equipping (Morris, 2006)

Each well was pump developed and step tested between 500 and 2,500 gpm. Continuous pumping tests were conducted at the design rates between 1,300 and 1,500 gpm. The wells were equipped with vertical line shaft turbine pumps fitted with reverse rotation ratchets to allow recharge through the pump bowls. Five wells are equipped with downhole flow control valves to allow recharge at greater rates than that achieved through the pump bowls. The wellheads are equipped with pressure gauges, vents, and water level transducers to support a fully automated operation. The above ground piping layout is one of the best observed. The distribution system water line enters the site to an above ground piping manifold with four ports controlled by pressure operated globe valves and butterfly valves. The ports are for recovery pumping from well, recharge supply to well, pipeline pre-startup purging to waste, and pressure blow-off to waste. The well collector line also contains a waste discharge line and control valve. The wellhead and associated conduits are fully sealed to allow full well bore pressure recharge. In support of this operation, an additional by-pass pipe and isolation valve has

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been placed on the flow line enabling the recharge water to be re-directed to the well casing annular space rather than down the pump column. This would avoid the head losses of injection through the pump string, allowing higher rates.

Unlike a typical potable water system where the tanks and reservoirs provide pressure stabilization, the Twin Oaks wellfield is a closed system. Pressure spikes from the startup and stopping of recharge are relieved to waste at the well site relief valve. If the distribution line contains un-desirable water, the system operator at the command center has the ability to purge the distribution system water to waste until it is acceptable for recharge. The well is then automatically placed into the injection mode. With most all but one automated air release valves closed, the recharge startup forces the column air into the formation. A positive back-pressure is achieved and maintained on the column throughout the recharge operation. If a borehole control valve is used to increase the base recharge rate, the valve is manually opened in the field to the desired position and locked into place. The finite start and stop control is by the surface globe valve at the manifold's recharge port. Recharge is stopped twice a month to back-flush the wells and check the operation of the pumping equipment. With the routine pumping activity, the wells are maintained in a high efficiency state. Very little to no declines in recharge performance have been observed.

Conversion between recharge and production is an automated routine. The system control calls for a stop of recharge by closing the supply globe valve. After a stabilization period, the waste valve is opened and the pump is called to start. After discharging to waste for a predetermined time, the logic control switches the flow to the distribution system line, or stops the pump and returns the site back to recharge. To date, recovery pumping has been limited to monthly preventive maintenance events and original facility testing. In the two years of operation, recharge water has been available to sustain the plant in the recharge mode banking over 20,000 AF.

The facility is in its infancy in developing long tern operational practices, purge durations, equipment calibration procedures, and field monitoring programs aimed at wellbore performance. Unlike typical utility based systems that grow a few wells at a time allowing fine tailoring of the operations, this facility is a stand-alone 30 MGD recharge, recovery, and treatment facility that is new end to end.

Problems Identified, Lessons Learned and Recommended Operational Practices

Some of the key lessons learned from the evaluation of the San Antonio Water System recharge program are noted below.

- The below ground storage location does not need to be in the local community, or even within the local municipal supply aquifer. The location could be in some other basin, valley, or hydrologic system connected through pump stations and pipes.
- A separate aquifer 30 miles south was selected with native water pH of 5 and high in iron and manganese. Thus, an aquifer with non-compatible water quality. Aquifer is used for local residential water and agricultural use.

- Purchase enough land upfront to encompass the current and build-out needs. The agency purchased ~7,000 acres, providing them 14,000 AF per year water rights from the native aquifer.
- Developed agreement with local water agencies representing domestic and agricultural interests to take full responsibility to fix, repair, or deepen the local community wells should such damage be caused by the operation of the ASR facility. This would ensure the "big city" agency would not pump their small community dry.
- Agreement should also account for water ownership and benefits of ASR program to the aquifer. State law does not prevent the local groundwater producers from drawing from the fresh water bubble. So facility design must account, in modeling, the maximum concentrated pumping effort that may occur on the facility boarder by others before the fresh water bubble is drawn into their production.
- Groundwater barriers using several ASR wells on their critical boarder can be formed with the intent of injecting native water. This will feed the local over-pumping with native water, and force the fresh water bubble back to the recovery wellfield.
- A treatment plant is built to ensure water developed from wells meets the normal quality of the system, avoiding distribution system problems and corrosion. The facility allows altered recharge water, mixed recharge and native water, and (in times of great need) native water to be pumped and treated to meet the system quality needs.
- Most all wells were drilled with flooded reverse circulation rotary methods for ease of development and high efficiency. The first facility well was drilled mud rotary resulting in a low efficiency well with persisting mud issues.
- Blank casing is mild steel with epoxy coating. No issues to date, though wells are only a few years old. Pump columns are mild steel with no special protector for the epoxy coating.
- The wells are equipped with all the manual and electronic devices. Gauges and transducers monitor the wellhead and system pressure. Bi-directional electro-magnetic flow meters are used. The boreholes are equipped with water level transducers. (no automated air release valves) Most all the valves have position indicators. All this, along with motor performance data, is being transmitted back to the command center and alarmed for operation.
- Borehole flow control valves are equipped in five of the wells. They are used to provide flow in addition to that going through the pump bowls. The valves are operated manually to a set orifice size when used.
- The well site piping configuration is by far the best. The system line enters the site to an above ground manifold with four ports controlled by pressure operated globe valves and butterfly valves. The ports are for recovery pumping, recharge supply, pipeline pre-startup purging, and pressure blow-off. The well collector line also contains a waste discharge line and control valve.
- The wellhead and associated conduits are fully sealed to allow full well bore pressure recharge. In support of this operation, an additional by-pass pipe and isolation valve has been placed on the flow line enabling the recharge water to be

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directed to the well casing annular space rather than down the pump column. This would avoid the head losses of injection through the pump string, allowing higher rates.

- Large "Y" strainers have been placed on the well recharge lines to prevent the introduction of anything larger than 1/8-inch into the pump bowls.

- The wells are designed to recharge at a rate lower than the production rate to allow pumping purge forces to be greater than injection forces. This provides some comfort of being able to back-flush develop the well with the existing equipment and not having to rehabilitate the well with a higher capacity development pump.

- The system is fully monitored and controlled by SCADA. The system operates by a "scheduler program" placing the most efficient unit or well in a specific location, brought online first and least needed unit last.

- All wells are equipped with recovery pumps. The pumps are operated twice a month for preventive maintenance, vibration, and performance checks to ensure operation when needed. This purge cycle helps back-flush the wells and maintain efficiency.

- A heavy reliance on instrumentation and automated operation must be supported by real-time data collection of the parameters. Quarterly electronic checks preformed by the agency do not catch mechanical drift or defects. Water level transducers must be accompanied by a physical level measurement, as well as pressure and flow.

- Common operation parameters such as waste time change with operation of ASR wells. Waste discharge quality and duration must be frequently checked (sand and turbidity) to ensure proper flush times to remove the harmful debris. Premature shut-down during wasting and conversion to recharge could cause well or pump damage. This may result in changing automated waste timers on a routine basis.

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AR Case Study
Nassau County Department of Public Works
Recharge Basins
Nassau County, New York

Introduction

Stormwater recharge basins were initially designed by Nassau County in the 1930's as an inexpensive means of reducing flooding problems in roadways. With the prospect of urban growth and water shortages during the growth season, Nassau County set forth in 1935 a measure to expand the number of recharge basins to offset anticipated water supply shortages. The process of water delivery in urban areas during storms involves runoff flow across hard or paved surfaces into gutters that lead to street inlets. The inlets are connected by storm-sewer pipes, which eventually carry the water to recharge basins where it infiltrates moderate to highly permeable sand and gravel deposits to the underlying aquifers. Recharge basins on Long Island are generally open, unlined pits that range in size from 0.1 to 30 acres. On average, most are 1.5 acres with depths ranging

Figure 1: Recharge Basin Locations
Source Ku and Aaronson, 1992

from 10 to 40 feet (Ku and Aaronson, 1992). As of 1992, more than 800 (Figure 1) were distributed throughout the County's interior (Ku and Aaronson, 1992). The main benefit of recharge basins has been that they enable groundwater recharge to occur during the growing season and increase annual recharge. A co-benefit of this method has been to help lengthen the lag time (the time it takes for surface water and groundwater to enter a watercourse) for storm peak flows and reduce the possible flooding caused by storm sewers during intense rainfall or sudden snowmelt periods. With the introduction of recharge basins into central Long Island where urbanization is most predominant, the water table as of 1992 was approximately 5 feet above pre-urban levels (Ku and Aaronson, 1992). This is expected to relieve concerns of water shortages and allow more focused attention on pressing water quality issues. However, recharge basins have caused uneven water table levels across Nassau County, and have altered the seasonal and spatial distribution of recharge within the County. This opens the prospect of water table elevations that may cause flooding during intense weather events such as hurricanes. Hence there is a requirement for the effective, strategic allocation of recharge basins, especially in coastal areas where surface elevations are generally low.

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Recharge Basin Design and Construction

Recharge basins use a simplified design approach containing several steps; including:

1. Area Reconnaissance, Site Selection, and Testing
2. Design Criteria for Recharge Basin
3. Construction of Basin

Area Reconnaissance, Site Selection, and Testing

A reconnaissance survey is conducted to identify locations that contain relatively permeable surficial soils and a minimum depth of at least 10-feet between the bottom of the basin and the water table based on knowledge of the surficial soils and the position of the water table. This work would be followed up by drilling borings to determine the depth to the water table and collecting and analyzing soil samples to determine the permeability of the unsaturated soils and the infiltration rate (Weaver, 1971).

Design Criteria for Recharge Basin

The design criteria for recharge basins in Nassau County have evolved for over 50 years. These criteria are based primarily on engineering experience including a generous safety factor of over flow. In Nassau County, the required capacity of a recharge basin is based on the following assumptions:

1. the basin has no outlet,
2. 5 inches of rainfall per storm, which has an occurrence interval of 10 years,
3. the service area of the basin's storm sewer system is 40 percent impervious in residential areas, 60 percent impervious in business areas, and 90 percent impervious at shopping centers, and
4. any area that drains to a recharge basin but is not served by a storm sewer system is considered to be 20 percent impervious.

The recharge basin capacity calculated by this method is called the "required volume" and is calculated by:

$$V_r = 3,630\ P\ (0.4\ A_r + 0.6\ A_b + 0.9\ A_s + 0.2\ A_n)$$

where: V_r = required volume, in cubic feet;

A_r = contributing residential area, in acres;
A_b = contributing business area, in acres;
A_s = contributing shopping center area, in acres;
P = precipitation, in inches (assumed to be 5-inches)

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The value of 3,630 is a dimensionless constant used to convert acres to square feet and inches to feet. The rate of storm water infiltration through the floor of the recharge basin is omitted from the design equation to provide an additional safety factor in basin construction (Ku and Aaronson, 1992).

Construction of Recharge Basin

The design of a recharge basin has evolved from the 1930s and can include various design components. Recharge basins are excavated to the design acreage calculated above with sidewall slope that ranges from 2:1 to 1:2. Some of the excavated material could be used to construct a surrounding berm to channel overland runoff into the basin in a controlled fashion to prevent scouring and destabilizing the recharge basin walls. The slope walls are typically stabilized to prevent erosion and sloughing. This is often done by spreading stockpiled topsoil from the excavation along the slope walls followed by seeding with fescue (Weaver, 1971).

Inlets are designed to reduce the velocity of the incoming water and prevent scouring. This is typically completed using various techniques such as rip-rap or baffles. Many recharge basins are designed to be multi-level containing a settling area near the inlet for the collection of debris and fine-grained materials and an auxiliary infiltration area to allow for increased infiltration (Aronson and Seaburn, 1974).

Some basins that experience clogging are designed to contain either a separate overflow basin or diffusion well typically constructed with 10-foot diameter pre-cast perforated concrete cylinders backfilled with coarse sand and gravel and are installed to a sufficient depth to penetrate the restricting layer (Aronson and Seaburn, 1974).

Infiltration Rates and Hydraulic Conductivity

The vertical hydraulic conductivity was measured at 14 locations in 9 recharge basins. Data from infiltration tests and tensiometers were combined to trace water movement through the saturated and unsaturated zones. The vertical hydraulic conductivity ranged from 0.08 to 5.33 feet per hour with a median of 1.63 feet per hour with a water temperature of 71.6 °F.

The infiltration rate was measured at 51 recharge basins in Nassau County. The tests were conducted over a 2 to 4 hour period and averaged for each recharge basin. The results show the average infiltration rate for each recharge basin ranged from 0.13 to 5.63 feet per hour with a median of 1.83 feet per hour (Ku and Aaronson, 1992).

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Water Table Mounding

Aronson and Seaburn studied the water table mounding beneath two recharge basins. They measured the mound height in response to precipitation events. The results show the water table rose 0.5 feet in response to 1-inch of precipitation and 2 feet in response to 2-inches of precipitation. Aronson and Seaburn also measured the time it took for the mounds to dissipate. The results show it took from 1 to 3 days for the mound to dissipate at Deer Park and from 7 to 14 days for the mound to dissipate in Syosset, depending at least partially on the height of the mound.

Recharge Basin Clogging and Factors that Affect Infiltration Rates

Clogging recharge basins are defined as those that retain water for more than 5 days after 1-inch of rain. In 1974, Seaburn found that about 9 percent of the recharge basins on Long Island were clogged. Examination of aerial photographs (April 11, 1969) six days after 1.5 inches of rain revealed that 62 basins or 12 percent of the basins still contained water. In 1986, 106 out of 598 recharge basins (18 percent) were considered to be clogged using this criteria. These data show that the number of clogged basin has increased considerably (Figure 2).

Figure 2: Clogged Recharge Basin Locations
Source Ku and Aaronson, 1992

Clogged basins on Long Island are determined by four main factors:

1. permeability of the basin materials,
2. land use within the basin's drainage area,
3. age of the recharge basin, and
4. intersection of the basin floor with the water table.

Each is discussed in more detail below.

Permeability of Basin Materials

Most of Nassau County, Long Island, especially the southern part, is underlain by highly permeable surficial outwash deposits. The central and northern parts of Nassau County are underlain by morainal deposits, which have lower infiltration rates. The area of low hydraulic conductivity soil as determined by the U. S. Soil Conservation Service (1982) does not necessarily indicate the permeability of the basin bottom because the basins are generally at least 10 feet deep and beneath the surficial soils.

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

The northern part of Nassau County is composed mainly of morainal and ice-contact deposits that typically have a higher silt and clay content and lower permeability than well-sorted outwash deposits in the southern part of Nassau County. The geologic control of the permeability and therefore the infiltration rate of the recharge basins becomes evident when the basin soil grain size (Figure 3), surficial geology (Figure 4), and perched water (Figure 5), are viewed together.

A comparison of the three data sets will show that surficial geology had the largest affect on the infiltration rates and the percentage of clogging basins. The areas of perched water had the second largest affect followed by the grain-size distribution of the soil. Figure 6 shows the correlation of percent clogging with the surficial units. This data shows that the percent clogging is the lowest in the Outwash deposits (17%) followed by the Harbor Hill Drift (22%), Ice-contact deposits (28%), and Ronkonkoma Drift (36%) (Ku and Aaronson, 1992).
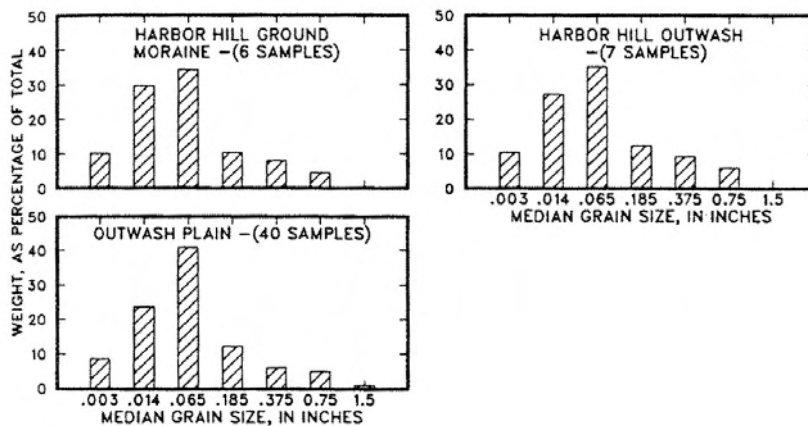
Figure 3: Grain Size Distribution in Recharge Basin
Source Ku and Aaronson, 1992

Land Use in the Drainage Areas

The land use in the drainage area of a recharge basin is a major factor in the clogging of a recharge basin. Aronson and Seaburn (1974) found that 28 percent of the basins in commercial and industrial areas were clogged. The high percentage of clogged recharge basins in commercial and industrial areas is probably dues to the relatively large inflow of asphalt, grease, oil, tar, and rubber particles in the runoff from adjacent parking areas (Ku and Aaronson, 1992).

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.
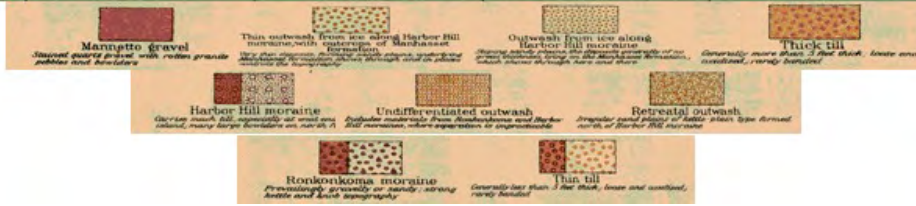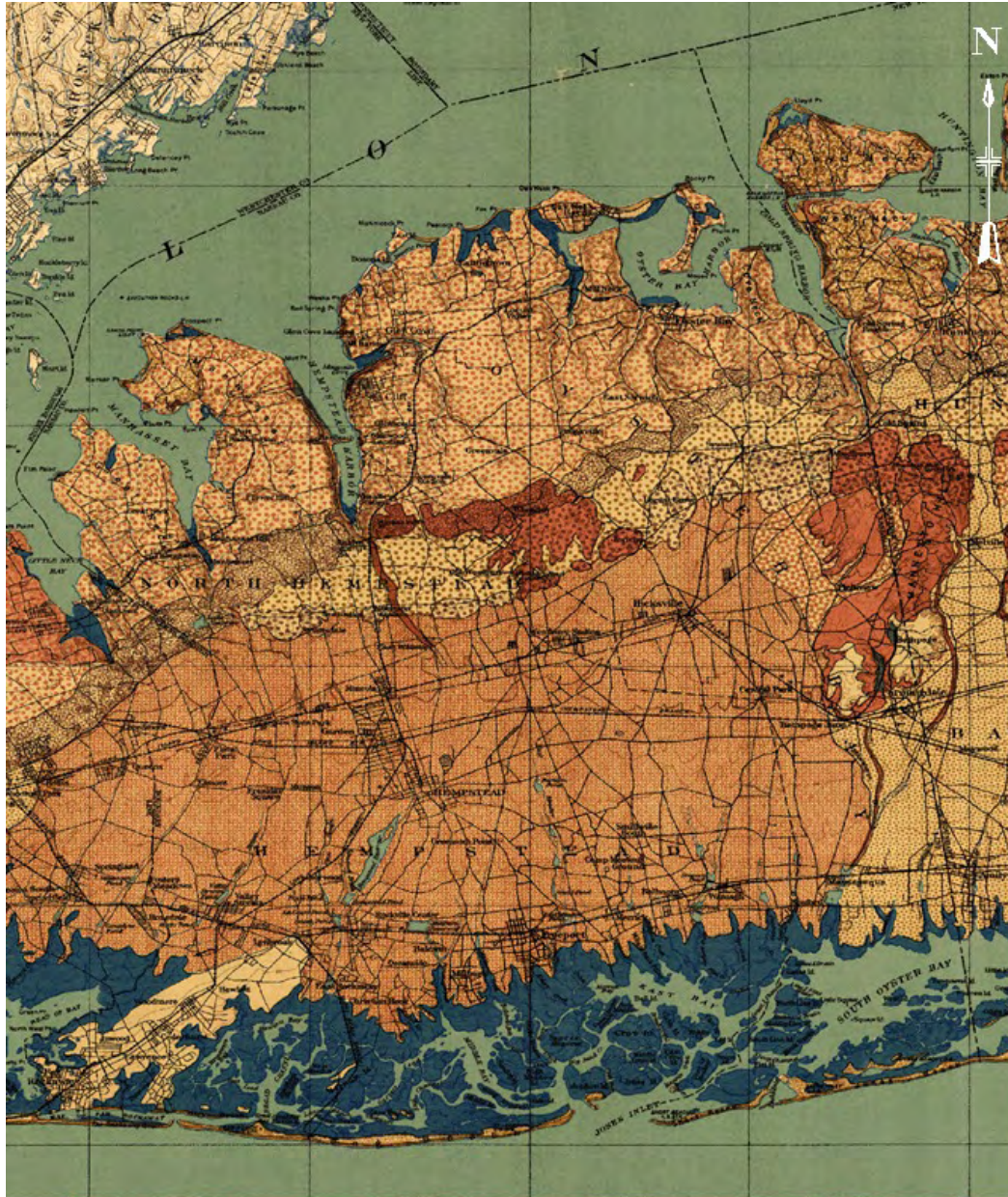
Figure 4: Surficial Geology of Nassau County   Source M. L. Fuller, 1914

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.
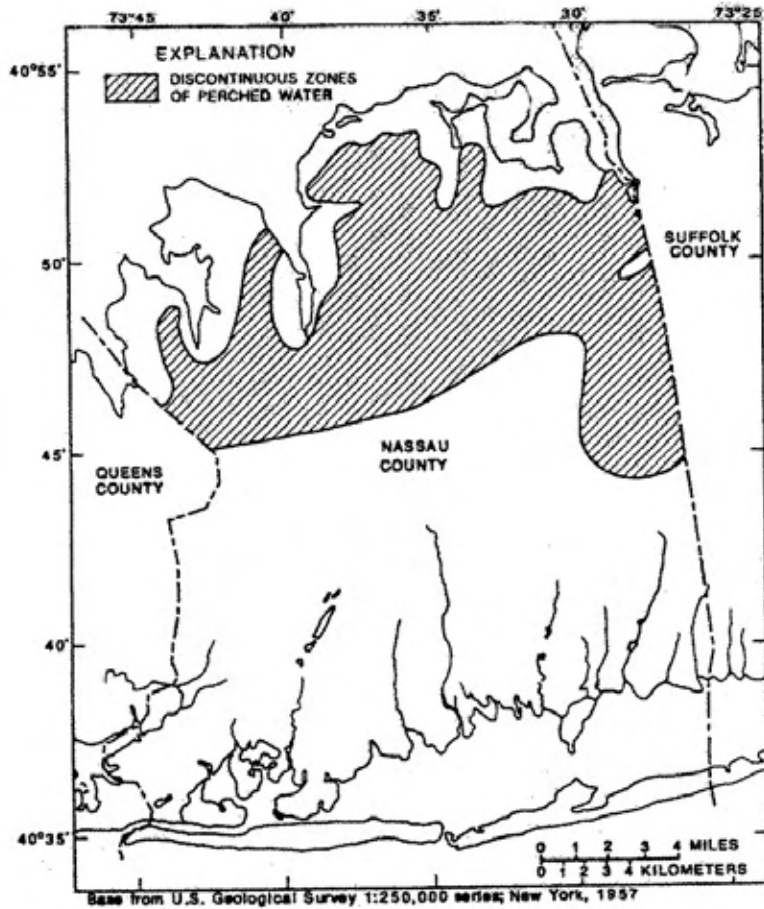
Figure 5: Perched Water Locations
Source Ku and Aaronson, 1992

Geologic unit	Percentage clogged	Total number of basins in unit
Ice-contact deposits	28	36
Harbor Hill Drift		
Harbor Hill end moraine	19	21
Harbor Hill ground moraine	25	61
Harbor Hill outwash deposits	22	77
Ronkonkoma Drift		
Ronkonkoma terminal moraine	36	25
Outwash plain deposits	14	355

Figure 6: Correlation of Geologic Unit and Percentage of Clogged Basins
Source: Ku and Aaronson, 1992

Age of Basins

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

Nassau County started constructing recharge basins in the 1930s. By 1988, Nassau County Department of Public Works had constructed and maintained 598 recharge basins. Operation and maintenance data collected by Nassau County shows that over 50 percent of the recharge basins constructed before 1950 were clogged and approximately 17 percent of the recharge basins constructed after 1950 were clogged. This data suggests that the efficiency of the recharge basins decrease with age. The cause of the clogging is not clear but may be due to the accumulation of fine-grained sediment and debris on the basin floor and microbial activity within the surface sediment (Ku and Aaronson, 1992).
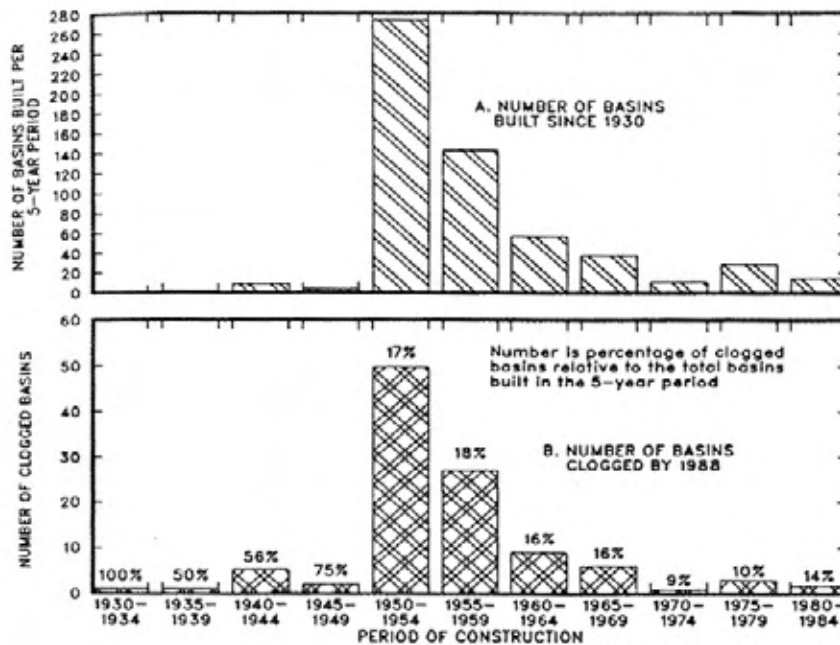
Figure 7: Correlation of Recharge Basins Age and Percentage of Clogged Basins
Source Ku and Aaronson, 1992

Intersection of the Basin Floor with the Water Table

A few basins that have been determined to be clogged may not actually be clogged but rather intersect the water table. Most of these are near the south shore where the water table is just below land surface. Also included in this category are basins that were constructed during the 1962-1966 drought, when the water table was lower than normal. As the water table recovered, the basin floors became flooded (Ku and Aaronson, 1992).

Summary

The distribution of clogged recharge basins shows that surficial geology plays a key role in determining the infiltration rate of recharge basins. From 19 to 36 percent of recharge basins in morainal deposits are clogged as compared to 14 percent of clogged recharge basins in outwash deposits. Age also plays an important role in the clogging of basins. Over half of the recharge basins built before 1950 are considered clogged.

The average infiltration rate of 51 studied basins ranged from 0.13 to 5.63 feet per hour. The recharge basins in northern Nassau County that are built in the morainal deposits tend to have slower recharge rates than the recharge basins built in the southern portion of Nassau County that contains outwash deposits. The vertical hydraulic conductivity of the initial foot of soil beneath the bottom of the recharge basins range from 0.08 to 5.33 feet per hour (Ku and Aaronson, 1992).

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AWWARF Case Study, Central Avra Valley and Sweetwater Recharge Facilities, Tucson, Arizona

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Introduction

The City of Tucson is located in the northern semi-arid reaches of the Sonoran Desert in eastern Pima County, Arizona. Very few surface streams contain perennial flow and most of these are effluent-dominated streams located downstream from municipal wastewater treatment plants. Until the early 1990s, the Tucson community relied almost exclusively on pumped groundwater to meet water demand. Due to rapid growth in population and associated water demand following World War II, the groundwater system transitioned from an approximate state of equilibrium to one of accelerating depletion. Despite the successful implementation of water conservation programs and the "desert landscape" ethic of Tucson residents, groundwater withdrawals for municipal use continued to increase through the year 2000. Rapidly declining water levels in the metropolitan area as well as in surrounding areas have resulted in land subsidence, increased pumping costs, and the gradual loss of native riparian habitat.

Tucson Water's need to develop renewable water supplies in order to reduce reliance on groundwater and meet projected future demand has long been recognized and is a critical goal of *Water Plan: 2000-2050* (Tucson Water, 2004). In an earlier resource planning process (CH2M Hill, 1989), the City of Tucson identified two available renewable water resources which will be increasingly utilized in order to satisfy projected water demand: treated municipal wastewater and Colorado River water delivered through the Central Arizona Project. The process of recharge and recovery has been, and will continue to be, a critical water resource utilization strategy for both of these renewable water supplies.

Effluent Reuse: Recharge for Water Quality Enhancement

In the early 1980s, the City of Tucson constructed one of the first reclaimed water systems in the country. This system provides tertiary treatment of secondary effluent derived from Pima County Wastewater Department facilities to produce water of sufficient quality to be used for landscape irrigation and certain industrial uses. The system began operation with 10 miles of pipeline and only one customer—a destination resort golf course. Since then, the system has grown to include over 100 miles of transmission pipelines and serves about 600 customers including multiple golf course facilities, parks, schools, industrial sites, and certain residential sites. Tucson Water's reclaimed water system remains an industry leader and serves to meet approximately eight percent of Tucson's total water demand. This reuse of wastewater effluent reduces groundwater pumping and conserves higher quality water sources for potable supply.

The secondary effluent that is received from Pima County's treatment facilities is either filtered at the Tucson Reclaimed Water Treatment Plant or recharged in a number of facilities. The recharge facilities include the Sweetwater Recharge Facilities, the Santa Cruz River Managed Underground Storage Facility (Santa Cruz Phase I) and the Lower Santa Cruz River Managed Recharge Project (Santa Cruz Phase II) as shown in Figure 1 (Tucson Water, 2004). The Santa Cruz Phase I facility is co-owned with the U.S. Bureau of Reclamation and the Santa Cruz Phase II facility is jointly owned by multiple parties.
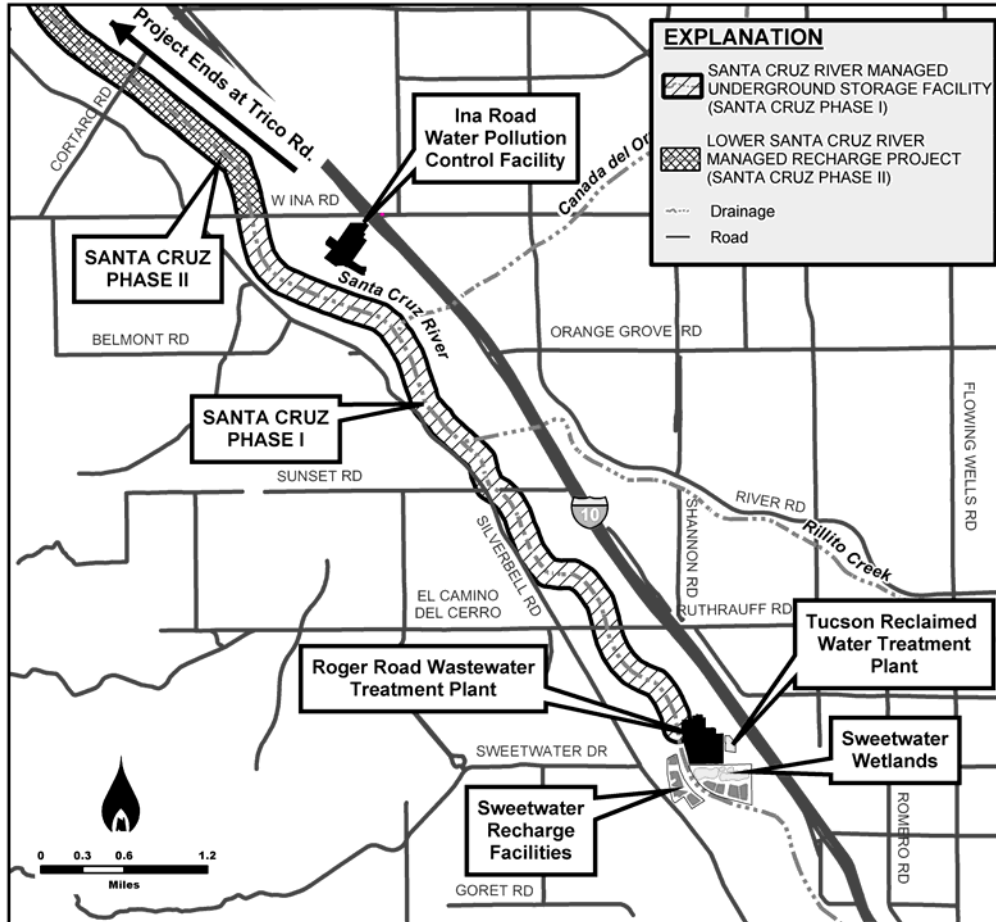
Figure 1: Sources of Supply to the Reclaimed Water System.

Sweetwater Recharge Facilities

The Sweetwater Recharge Facilities (SRF) are located on the west side of Tucson, north of Prince Road along the Santa Cruz River (Figure 1). The SRF are composed of eight recharge basins with four on the west bank of the Santa Cruz river and four on the east bank, the Tucson Reclaimed Water Treatment Plant, a constructed wetland primarily used to treat plant filter backwash water, twelve monitoring wells used for compliance sampling, eighteen piezometers monitored for water levels, and six high capacity extraction wells. Secondary effluent from Pima County's Roger Road Wastewater Treatment Plant (Roger Road Plant) is delivered directly to the recharge basins. Currently, recharge operations take place eleven months of the year, with one month reserved for routine recharge basin maintenance to maximize infiltration rates.

The depth to water at the SRF site is about 120 feet below land surface (bls). Infiltration rates for the SRF have ranged from a few tenths of a foot to several feet per day over the years. In general, the transmissivities calculated for areas along the Santa Cruz River are quite high, ranging from 150,000 to over 300,000 gallons per day per foot width of the aquifer (Light et al, 1997).

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

The quality of effluent is improved by the percolation of the water through the basin alluvium and aquifer. This process is referred to as soil aquifer treatment (SAT). The process of soil aquifer treatment has been studied in detail at the SRF. The alluvial sediments serve as an effective filtration mechanism to significantly reduce turbidity and bacteria levels in the recharged effluent. Total organic carbon levels in the secondary effluent are reduced on average from about 20 mg/L to less than 1 mg/L. Source water containing total nitrogen levels on the order of approximately 25 mg/L has been effectively treated to about 7 mg/L. While attenuation and other mechanical processes play a role in soil aquifer treatment, biological activity is a key mechanism. The biological processes are renewable and have been effective for over 15 years of project operations. The use of alternating wet and dry cycles at the SRF facilitates the processes of soil aquifer treatment by providing alternating aerobic and anaerobic conditions for the biological activity.

The Sweetwater Recharge Facilities are permitted to annually recharge and recover up to 6,500 acre-feet of reclaimed water to meet seasonal peak demand requirements. The recovered effluent is blended with filtered water from the reclaimed plant, disinfected with chlorine, and boosted to customers through the reclaimed water distribution system. The total delivery capacity of blended water from the reclaimed plant and the Sweetwater Recharge Facilities is 27 MGD.

Managed Effluent Recharge Facilities

Santa Cruz Phase I is located in the channel of the Santa Cruz River from the Roger Road Plant to Ina Road in Pima County, Arizona (Figure 1). The facility includes approximately 5.1 miles of unmodified stream channel, one stream gage, two secondary effluent outfalls, and four water level monitoring points. Infiltration rates are not artificially maintained. Periodic natural storm flow events scour the channel bottom, removing built-up fine sediments and organic matter that form a clogging layer.

Santa Cruz Phase I is permitted to recharge 9,307 acre-feet of effluent annually. The regulations that govern managed recharge facilities award credits for only 50 percent of the effluent that is recharged. The City of Tucson and the Bureau of Reclamation (BOR) evenly share the credits accrued at Santa Cruz Phase I; therefore, Tucson Water can accrue approximately 2,300 acre-feet of recharge credits per year. During the first full year of operation, from April 2000 through March 2001, the facility recharged a total of 9,245.1 acre-feet of effluent. This total compared very favorably with the permitted facility volume. As a result of the 50% cut to the aquifer and the division of credits with the BOR, the City of Tucson accrued about 2,310 acre-feet of recharge credits during this time frame (Thomure and Wilson, 2003). It is expected that the recharge rate of a managed project will vary widely from month to month due to the sporadic occurrence of runoff scour events. While the initial 12 months of operations yielded close to the permitted volume, the subsequent 12-month periods have been less productive. The time period of April 2001 through March 2002 produced 5,366.6 acre-feet of recharge while the 12-month period of April 2004 through March 2005 recharged only 1,183.6 acre-feet.

The average annual recharge rate for the first five years of operation has ranged from 0.64 to 4.97 acre-feet per mile per day. Significant storm runoff events have been

lacking in the Santa Cruz River in recent years. This has allowed a persistent clogging layer to form resulting in the low recent infiltration rates.

Effluent recovery from the SCRMUSF commenced in April 2002 from one well. In an ideal world, the maximum permitted volume would be recharged annually and be available for recovery. Since this level of recharge will not be attained consistently, recovery operations for any given year must be planned to not exceed the annual and accumulated long-term storage credits at the site.

The Santa Cruz Phase II facility is co-owned by several local entities and recharges effluent on behalf of Tucson Water as well as others with effluent entitlements. This facility is located immediately down-gradient from Santa Cruz Phase I in the channel of the Santa Cruz River. Santa Cruz Phase II runs from Ina Road to Trico Road (Figure 1). The main components of the facility include approximately 17.9 miles of unmodified stream channel, one additional stream gage, and six water level monitoring points. Santa Cruz Phase II is permitted to recharge up to 43,000 acre-feet of effluent per year with a cumulative 20-year recharge limit of 680,000 acre-feet (average of 34,000 acre-feet of effluent recharge per year).

In 2004, the first full year of operation, Santa Cruz Phase II recharged 17,628.7 acre-feet of effluent. While this total was well below the permitted volume, this was not unexpected due to the same conditions that have limited recharge in Santa Cruz Phase I. After distributing the credits amongst the various cooperators and accounting for the cut to the aquifer, the City of Tucson accrued 2,813.8 acre-feet of effluent recharge credits. The average annual recharge rate for the first year of operation was 2.69 acre-feet per mile per day. The Santa Cruz Phase II facility does not currently have a direct (wet-water) recovery component.

Effluent Recharge in the Future

In addition to the continuing operation of current facilities, effluent recharge for non-potable reuse will increase over the next few years. Discussions are currently underway to develop wet-water recovery facilities to utilize Phase II effluent. Currently, the credits accrued in this project can be recovered outside of the area of impact; however, recovery of this type does not provide a hydrologic balance. One possibility is for the parties that store effluent in the project to utilize infrastructure that already exists in the area of impact for recovery. The existing infrastructure is largely controlled by the Cortaro-Marana Irrigation District (CMID). If an agreement can be reached, CMID could recover the stored effluent and wheel it to the effluent owners at a negotiated rate. The alternative is that each storer could attempt to implement independent recovery systems. Failing either of these options, the incentive for effluent owners to utilize the Santa Cruz Phase II facility will diminish and other recharge options would likely be pursued.

As Tucson Water's Reclaimed Water System grows in the coming years, additional access to tertiary-treated effluent will be required. The increasing demand is not only within the Tucson Water service area, but also in areas served by others. For instance, Tucson Water will wheel others' effluent through its Reclaimed Water System to their facilities (Pima County and Oro Valley are examples). The expansion of constructed recharge facilities will be evaluated as a way to provide this additional supply. Currently, a series of possible ways to expand the existing SRF are being

evaluated. In addition, Tucson Water is participating in two multi-benefit studies that could include recharge components for water supply. These US Army Corps of Engineers projects, *Tres Rios del Norte* and *El Rio Medio*, could become attractive to Tucson Water if water supply development components are included.

Even though effluent will continue to be used to meet non-potable demands, a large volume of effluent will be available to augment the potable supply. Two critical decisions are outlined in *Water Plan: 2000-2050* regarding the future use of effluent:

> "Should current effluent disposal practices continue or should Tucson Water maximize the use of effluent as a water supply?" and "If the use of effluent is to be maximized, should it be stored in long-term banking facilities or should it be used to augment the potable water supply?"

The role of recharge is a critical element under both decisions. Essentially, the community has three options for the future of effluent: 1) limit effluent to non-potable reclaimed use with managed recharge/disposal of the unused portion, 2) continue non-potable use of a portion of the effluent and store the remainder in constructed long-term banking facilities, or 3) continue non-potable use of a portion of the effluent and treat the remainder to a very high quality and convey it to recharge and recovery facilities that will contribute to the blended water supply for indirect potable reuse.

Option 1 does not make full use of the Utility's effluent supply and does not aid in meeting its projected growing water demand. Absent the ability to acquire additional alternative water supplies or greatly increased effectiveness of conservation programs, Tucson Water could begin to have a shortfall in renewable water supplies as early as the 2020s without the expanded use of effluent. The decision to bank effluent in long-term recharge facilities but not use it to augment potable supply (Option 2) would provide the opportunity to preserve the effluent for use in the future. This decision would allow for the accrual of storage credits that would allow the utility to pump additional groundwater. However, storing the effluent outside the area where pumping occurs would result in a resumption of ground-water mining in the Tucson Water service area that would cause renewed water-level declines in the regional aquifer, and would increase the risk of additional land subsidence in the area. It is projected that Option 2 would add about 20 years to Tucson Water's sustainable water supply with a shortage occurring in the 2040s. Option 3, however, would allow Tucson Water to meet demand through the current planning horizon of 2050 while avoiding the negative impacts of resumed water level declines and land subsidence. The recharge process plays a key role in all of the possible water-use futures that Tucson Water envisions for effluent. Under Option 3, recharge would be a key component to converting effluent from simply a non-potable supply to a resource that can meet a much more significant share of total water demand.

Colorado River Water: Recharge for Public Acceptance and Reliability

In order to address continued groundwater depletion, the City has also pursued efforts to utilize its allocation of Colorado River water. The City of Tucson has an annual Central Arizona Project allocation of 135,966 acre-feet. This volume, if fully utilized, would be sufficient to meet the utility's current water demand.

Direct Use of Colorado River Water in the 1990s

The initial plan for Colorado River water use in Tucson was similar to programs being implemented by other water providers. A large conventional surface water treatment facility, the Hayden-Udall Water Treatment Plant (WTP), was constructed in Avra Valley adjacent to the Central Arizona Project canal. Colorado River water was treated through filtration, ozonation, chemical corrosion control, and chlorine disinfection prior to delivery to the main potable distribution system. The plant was designed to process and deliver Tucson's entire Central Arizona Project allotment. From 1992 to 1994, deliveries of Colorado River water through the Hayden-Udall WTP satisfied the demands of approximately 50 percent of the customers formerly served by Tucson Water's well fields. In addition, excess Colorado River water was conveyed through the distribution system for recharge via injection wells located in areas with the largest water level declines. This was being done to reduce the potential for future subsidence in the metropolitan area and to store excess water where it could readily be recovered. This recharge program resulted in rising water levels in the Central Wellfield for the first time in over four decades.

However, a series of operational problems besieged the Colorado River water delivery system. Customer complaints of rust-colored water, taste and odor concerns, and damage to household appliances were widely profiled. These issues ultimately resulted in the passage of a local ordinance, the Water Consumer Protection Act (1995), which prevented Tucson Water from continuing the direct delivery of Colorado River water unless certain conditions were met (Johnson et al, 2003).

Clearwater Program and the Birth of Blended Water

In order to utilize Colorado River water in compliance with the constraints imposed by the Water Consumer Protection Act, Tucson Water constructed a large-scale recharge and recovery facility in central Avra Valley. The Central Avra Valley Storage and Recovery Project (CAVSARP) was designed and built to recharge and recover up to 60,000 acre-feet of Colorado River water per year (54 MGD). The facility consists of 11 recharge basins, 27 recovery wells, a 54-MGD booster station, a 10 million-gallon reservoir, and approximately 25 miles of pipelines. The depth to water at the CAVSARP site is about 350 feet bls. Infiltration rates for the CAVSARP basins range from less than one to several feet per day. Through recharge and recovery, Colorado River water mixes with Avra Valley groundwater to produce a blended water supply. CAVSARP and the Hayden-Udall WTP currently form the core of the Clearwater Program (Figure 2).

Owing to the issues surrounding the initial introduction of directly treated CAP water, Tucson Water has invested significant resources into researching and testing the blended water program. Major research and public outreach activities included the "At the Tap" Program, water flavor panel tests, iron release rate studies, and the Ambassador Neighborhood Program. The research programs were designed to ensure that the problems which occurred in the direct delivery of Colorado River water did not re-occur under the blended water program which relies on recharge. The Ambassador Program provided samples of the blended water in bottles; also, blended water was delivered to four volunteer neighborhoods. Initial plans were to distribute 40,000 sport bottles of

blended water; however, the program was so well received that nearly one million bottles had been distributed by the end of the program in late 2000 (Light et al, 2001). The Clearwater Program eventually gained wide community support and regular deliveries of the blended water began in May 2001.
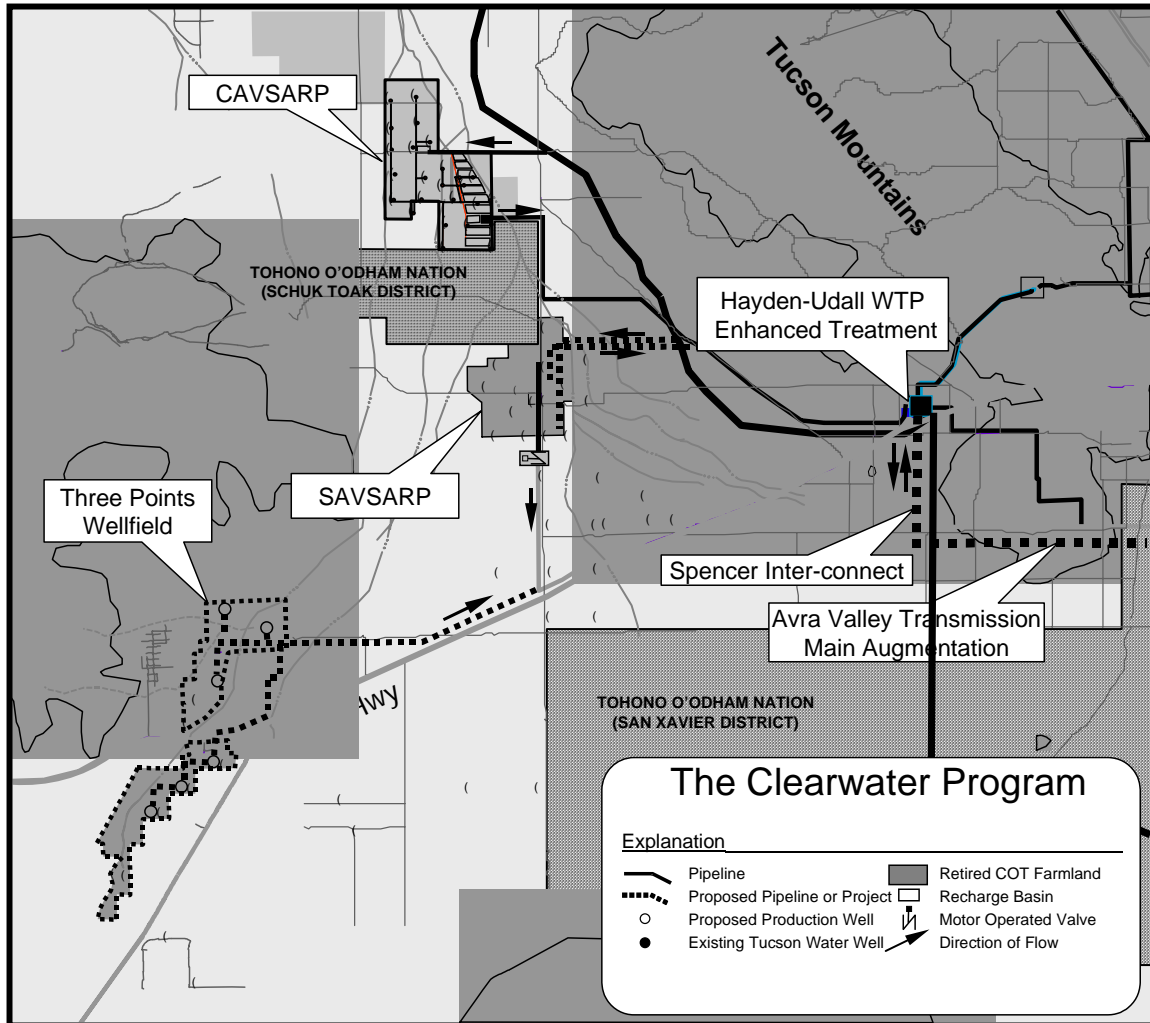
Figure 2: The Clearwater Program

Colorado River Water Recharge in the Future

Building upon the success of the Clearwater Program's recharge and recovery facility at CAVSARP, plans have been initiated to assess additional program elements to fully utilize the City's entire Central Arizona Project allocation as soon as possible. As outlined in *Water Plan: 2000-2050*, several projects are being evaluated which are shown on Figure 2 and include:

- Expanding the permitted recharge capacity of CAVSARP.
- Implementing the Southern Avra Valley Storage and Recovery Project (SAVSARP) Phase I and Phase II.

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

- Developing a new well field near Three Points in Avra Valley.
- Constructing the Spencer Interconnect pipeline.
- Augmenting the Avra Valley Transmission Main.
- Incorporating additional treatment processes at the Hayden-Udall WTP

Some of these projects are considered to be common elements: projects that provide a high level of flexibility to remain viable into the future despite the inherent uncertainties in water resource planning. The common elements are the CAVSARP expansion, SAVSARP Phase I, the Spencer Interconnect, and the Avra Valley Transmission Main Augmentation. Both CAVSARP and SAVSARP rely on recharge.

The CAVSARP facility is currently permitted to recharge and recover up to 60,000 acre-feet of Colorado River water per year. The existing recharge facilities, however, are physically capable of recharging up to 80,000 acre-feet per year. An application to expand the annual permitted recharge capacity to 80,000 acre-feet has been submitted to ADWR to allow additional annual recharge at CAVSARP by 2005.

SAVSARP, another CAVSARP-type facility that may be located several miles to the south, may have sufficient annual capacity to recharge and recover 30,000 to 100,000 acre-feet of Colorado River water. The upper end of this range could provide Tucson Water with the physical ability to fully utilize its annual Central Arizona Project allocation. SAVSARP is divided into two phases with Phase I considered the common element. SAVSARP Phase I would be sized to recharge 45,000 acre-feet of Colorado River water per year with 30,000 acre-feet of recovery. The size of the recovery component fits into the existing Clearwater Program infrastructure and the excess recharge capacity could be used by the Arizona Water Banking Authority to store excess Colorado River water for use in future years. SAVSARP Phase II is contingent upon a community decision that must be made as described in the following section. Depending on the ultimate sizing of SAVSARP, construction efforts would include 150 to 400 acres of recharge basins, 15 to 40 recovery wells, a reservoir/booster station, and many miles of pipelines to convey this additional blended supply to the Hayden-Udall WTP.

In order to determine which final projects should be implemented in addition to the identified common elements, two critical resource-management decisions will be made regarding the use of Colorado River water:

"What is an acceptable long-term mineral content target for the Clearwater blended water program?" and "Should Tucson Water expand the Clearwater recharge program by building SAVSARP to maximum capacity or rehabilitate the Hayden-Udall WTP to perform direct treatment?"

Water Plan 2000-2050 provides a detailed explanation of the positive and negative aspects of each of these decisions. The second listed decision will determine whether SAVSARP Phase II would be constructed or whether the surface treatment capability of the Hayden-Udall WTP will be restored. Tucson Water recommends that SAVSARP Phase II be constructed. Recharging all of the City of Tucson's Colorado River water will provide water quality enhancements, increased availability for banking excess water, and better drought management capabilities since the recovery component can continue to provide water supply even when flows from the river would be reduced.

Problems Identified, Lessons Learned and Recommended Operational Practices

Some of the key lessons learned from the evaluation of the Central Avra Valley and Sweetwater recharge facilities are noted below.

- Three years of pilot operations were conducted in three 20 acre basins, recharging 47,000 acre-feet of surface water at an average rate of 1,100 acre-feet per month.
- Each basin is a shallow rectangular box with shallow entry sides, concrete water entry aprons, and staff gauges with dimensions of 660 feet wide by 1200 feet or greater in length. Access ramps are provided for heavy equipment for scrapping and rehabilitation.
- A pump station at the canal delivers water to the basins at 13,000 gallon per minute (gpm) through a 36 inch line.
- Hydraulic responses at the project site are monitored in 17 ground water wells and 32 vadose zone viscometers. The piezometers are completed on top of clay or fine-grained layers that were potential perching zones for recharged water. Each basin has a similar array of monitoring points and the same general patterns of response have been noted for each site.
- Pilot recharge operations and public outreach programs are contributing to the success of the recharge facility. A primary goal was to deliver water that has an acceptable quality to customers which means it should taste good and should have a corrosion rate that is equal to or less than occurs with groundwater.
- The experience gained from the operation of the pilot program has been integrated into the design of the full-scale facility.
- At the beginning of recharge activities, the depth to ground water was about 370 feet bls. The current depth to water under the central basin is approximately 300 feet bls.
- The regular drying of the recharge basin allows cracking of the fine surface sediments and opens up potentially cemented surfaces. Normal operational wet cycle lengths have varied between 5 and 15 days, with interceding dry cycles of 3 to 14 days.
- At an intermediate depth of 100-140 feet bls, the perched water resulting from successive wet cycles tended to coalesce in a more sustained presence of perched water. The greatest lateral extent of perched water on the finer-grained layer was observed to extent 1,300 feet from the basin.
- Over time, the vadose zone's ability to transmit recharged water downward appears to improve; perched zones at the intermediate and deeper depths become less pervasive even as recharge activities continue.
- Iron release studies showed that pH was the single most important factor in controlling corrosion in galvanized pipelines. All water types with a pH of 8.5, disinfected with chloramines, and which had polyphosphate added at a concentration of 3 – 5 mg/L produced water with the very low median iron concentration of 0.7 mg/L.

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- The average infiltration rate at the pilot recharge project is 1.2 feet per day. At a hydraulic loading rate of about 220 feet per year, recharge water is transmitted to the water table at 370 feet bls resulting in a mound that is about 56 feet high.
- As the recharge water recharge front advances through the vadose zone, dissolved salts in the soil moisture are flushed to the water table. The salts are diluted as the recharge front reaches the water table. Groundwater quality showed a sharp increase in TDS concentrations as salts from the vadose zone where flushed to the water table. As recharge water reached the water table, the sulfate concentrations increased to 240 mg/l.

Summary

Tucson Water's planning efforts are geared toward providing flexibility into the future. This will allow the Utility to plan for and manage the uncertainties that often drive water resource utilization. The recharge and recovery process is one technology that provides needed flexibility. The treatment of effluent for non-potable use can be effectively accomplished through recharge. In addition, as effluent transitions toward becoming a contribution to indirect potable usage, recharge will provide a necessary barrier between water that is effluent and water that is acceptable by the public as part of the blended water supply. Recharge plays a vital role for Tucson's Colorado River water as well. The public has accepted the use of Colorado River water as part of the Clearwater blend largely due to the use of recharge. As the Clearwater Program expands into the future, its use of recharge will expand as well providing increased operational flexibility, system reliability, and enhanced drought management opportunities.

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AWWARF Case Study, C-111
Project, Homestead, Florida
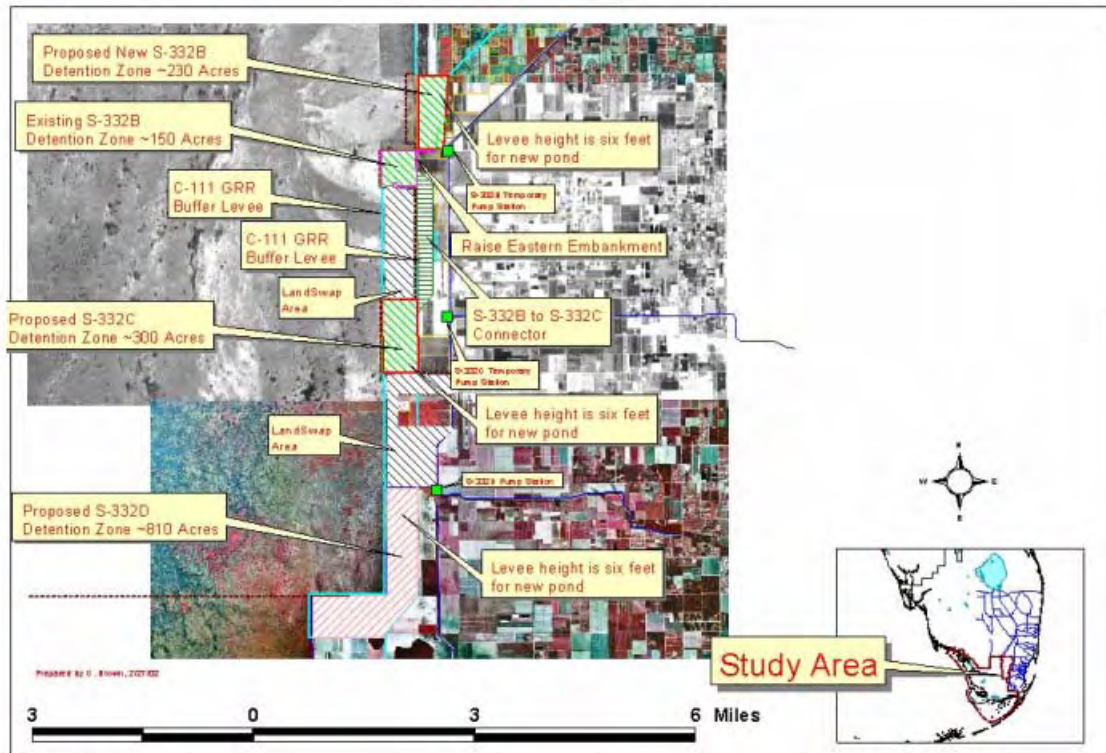
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Introduction

Freshwater is the liquid of life (People&Planet, 2002). Freshwater is a necessary ingredient to life on earth. The supply of such water is limited and finite. As the population of the world grows, water demand grows also. Unless steps are taken now, the world is headed to a major water supply crisis. Even today, the World Health Organization (WHO) estimates that 1.1 billion people around the globe lack access to safe water systems (IFPRI, 2002). According the same report from WHO, parasites, bacteria, and chemicals are killing 3.4 million persons each year. Globally, the annual population increase of 80 million persons per year implies an increased water demand of approximately 64 billion cubic meters per year – an amount equivalent to the annual flow rate of the Rhine River in Europe (People&Planet, 2002). The growing water demands has placed many traditional water supplies in jeopardy. The depletion of surface water and groundwater supplies threatens the entire world since no region is immune. In addition, competing demands for water (irrigation, drinking water, industrial cooling, in-stream environmental uses, etc.) have led to political upheaval and multiple lawsuits. In the United States, lack of water within the Columbia River Basin has pitted farmers versus environmentalists (Oregonian, 2001). In South Florida, these issues are especially troublesome due to the presence of urban, agriculture and pristine park environments in close confines. Since the 1940s, the U.S. Army Corps of Engineers has constructed, maintained and operated a huge water resources project called the Central and Southern Florida Project (C&SF). One important portion of the C&SF system is the Canal 111 project.

The Canal 111 project is located in southeastern Dade County, Florida, adjacent to the eastern boundary of Everglades National Park (ENP). C-111 was originally authorized by Congress as an extension to the Central and Southern Florida Project. The authorization for this modification was given to the U.S. Army Corps of Engineers, Jacksonville office through the Flood Control Act of 1962. In the late 1960s and early 1970s, additional authorizations permitted additional modifications to C-111 to allow construction of the ENP-South Dade Conveyance project. This project provides water to urban and agricultural Dade County as well as to lower ENP. The ENP Protection and Expansion Act of 1989 added additional language and purposes to the regional water control project (U.S. Congress, 1989). The Act stipulated that continued studies/plans for project works within the L-31N/C-111 basin should include all measures (feasible and consistent with project purposes) that would protect and enhance the natural values associated with ENP. The C-111 project is intended to provide multi-purpose project benefits including water supply, flood control and ecological restoration benefits. The project proposes to construct a parallel levee system west of the C-111 Canal between the Canal and ENP to infiltrate massive amounts of storm water into the underlying porous limestone aquifer. To date, implementation of project has led to the development of a General Re-evaluation Report (GRR) in 1994 (USACE, 1994) and a supplement to the GRR in 2002 (USACE, 2002). The C-111 project study area is shown on Figure 1.x.
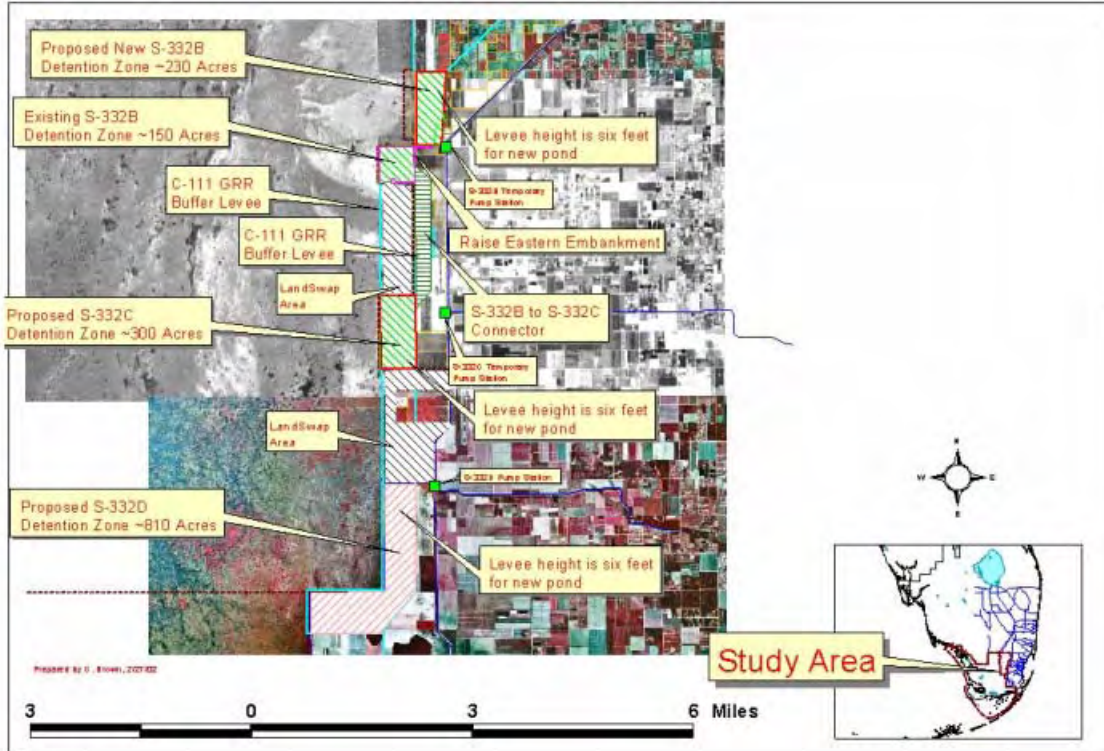
Figure 1. C-111 Project Study Area

The U.S. Army Corps of Engineers, Jacksonville District, is responsible for completing construction and testing of the C-111 Project. The C-111 project involves construction and operation of a series of infiltration basins located along the eastern boundary of Everglades National Park (ENP). Excess basin storm-water is pumped into the infiltrations basins and allowed to percolate into the subsurface soils and bedrock. As the storm-water infiltrates, the groundwater elevations rise and create a groundwater "mound". This mounding effect ultimately reduces easterly seepage emanating from ENP. It is hoped that through long-term operation and optimization of the infiltration basins, deleterious annual water losses from ENP can be minimized. When completed, the project would consist of several thousand acres of infiltration basins along with associated water-control structures and pump stations. Several portions of the entire project have already been constructed and tested. The S-332B North basin is located in the northern portion of the study area and is approximately 230 acres in size. The S-332B West basin is located in the northern portion of the study area and is approximately 150 acres in size. The S-332B West basin started operation in late 2001 and can infiltrate up to 125 cfs, while the S-332B North basin started operation 2003 and can infiltrate up to 225 cfs. The S-332C and S-332D infiltration basins are located in the central and southern portion of the study area respectively, and were constructed in 2003. The S-332C infiltration basin is approximately 300 acres in size, while the S-332D basin is approximately 810 acres. Eventually, once the entire project has been completed, a continual series of infiltration basins will provide a buffer between the ENP and more

urbanized areas to the east. Two areas of the project labeled as part of the ENP "landswap" on the figure, are currently part of the ENP but will become part of the C-111 project when equal size lands are provided within the Model Lands in southern Miami-Dade County. This action is awaiting Congressional action to be completed.

Each of the basins is connected by pipeline or canal to the main C-111 Canal system. Powerful pump stations capture C-111 canal water and covey the water to the infiltration basins. S-332B West, S-332B North, and S-332C basins are designed as typical infiltration basins while S-332D is a flow through system that still results in considerable infiltration. Figure 2.x provides an aerial photo of the completed S-332D basin along with a general flow diagram. Water pumped from the pump station is conveyed to the high head cell for gravity flow through the series of cells to the flow way into ENP.

Figure 2. S-332D Basin with flow diagram and features (not to scale)

Site Setting - Regional Geology and Soils

South Florida is underlain by Cenozoic-age rocks to a depth of approximately 5,000 feet below land surface (bls) and is comprised of various percentages of sand, limestone, clay and dolomite (Miller, 1997). Geologic units exposed at the land surface in southern Florida have been mapped by Brooks (1981), and have been presented in

detailed county maps by the Florida Geological Survey (FGS). An understanding of the surficial geology is an extremely important component of the Everglades ecosystem. The geology in this area influences land use, waste disposal, groundwater quality, drainage, recharge, and ecological zonation. Thin strands of sand and the Miami Limestone underlie most of the lower Florida East coast and form the highest elevations in the area corresponding to the Atlantic Coastal Ridge physigraphic province (Enos and Perkins, 1977). These near surface units are highly permeable and higher elevations are well drained. The oolitic facies of the Miami Limestone are breached by numerous shallow sloughs, further enhancing drainage.

In agricultural areas of Miami-Dade County, it is common to encounter mixed soils called "rock plowed" soil. This is a man-made material created by farmers excavating and crushing soft Miami Limestone and mixing/tilling it along with the natural overburden silty-sand soils. Consequently, the overburden thickness is somewhat higher in these areas. In most cases however, the underlying Miami Limestone still controls the infiltration of rain or introduced storm water. One key design parameter that must be known is the spatial extent and vertical conductivity of fine-grained mudstone stringers contained within the Miami Limestone. These layers effectively retard infiltration and in some areas cause the surface water and groundwater systems to be completely separate.

Site Setting - Regional Hydrogeology

Much of the work in identifying, mapping and describing the geologic units within southern Florida has been completed in an effort to understand the hydrologic resources of the area. The relation between the carbonate units and the interfingering siliciclastic sediments (quartz rich sands) controls numerous environmental aspects of the south Florida ecosystem mainly due to the profound influence these units have on the movement of water. Along with stratigraphic sub-divisions, these units are also delineated as hydrologic units based on their hydraulic properties. These hydrologic sub-divisions are referred to as aquifers. It is not uncommon for the top and base of an aquifer to include multiple stratigraphic units. Three major aquifer systems have been identified in South Florida; the Surficial Aquifer System (SAS), the Intermediate Aquifer System (IAS) and the Floridan Aquifer System (FAS) (Klein, et al. 1975; Causarus, 1985; Miller, 1986; Fish and Stewart, 1991). The C-111 infiltration basins are only connected to the SAS so the IAS and the FAS will not be discussed further.

In the study area, as with the rest of Dade County, the most important hydrogeologic unit is the Biscayne Aquifer that is a sub-division of the larger SAS (Fish and Stewart, 1991). The Biscayne aquifer is the most prolific aquifer in Dade County and contains highly permeable sands and limestone (Fish and Stewart, 1991). It is an unconfined or "water table" aquifer and provides virtually all of Dade County's water supply. Because it is the principal aquifer for several million people, the United States Environmental Protection Agency (EPA) has declared it a sole source aquifer in 1979. Within close proximity to C-111 basins, the Biscayne Aquifer is comprised, from top to bottom, Miami Limestone, Fort Thompson Formation, and contiguous, highly permeable

beds of the Tamiami Formation. The hydraulic conductivities of these permeable units range from 100 feet per day to over 30,000 feet per day where voids exist within the limestone.

Source Water

The primary source water for the infiltration basins is from the C-111 Canal itself supplemented by frequent summer precipitation. During the Florida wet season, ample water supplies are available to provide water supply for the infiltration operations. Without the infiltration basin system, this water was being sent to Florida Bay and Barnes Sound. In the past when flows were very high, the damaging freshwater pulse releases from the C-111 project into the estuary were quite problematic. Since the introduction of the infiltration system, the frequency of large pulse releases has diminished. Once the entire project has been constructed, it is expected that the Barnes Sound portion of the Florida Bay should thrive again. The water quality of the source water is variable but generally contains low concentrations of total dissolved solids (TDS) typically less than 500 mg/l and low nutrients (less than 20 ppb of total phosphorus). Generally, the groundwater within ENP is similar except that TDS and nutrient values are even lower, approaching concentrations typical of precipitation in the study area.

Feasibility and Design Studies - Field Investigations for Design

Extensive field investigations were conducted in the study area prior to construction of the infiltration basins. The field investigations included installation of numerous core borings, twenty-seven groundwater monitoring wells, and five test pits (located near to S332C basin but not directly onsite) along with in-situ geophysical logging and large-scale percolation tests. Soil and rock samples were also collected and tested at a reputable geotechnical testing laboratory.

The wells were constructed as four-inch diameter wells and are available for both collection of water levels and groundwater quality. The wells are screened within the Miami Limestone and the Fort Thompson Formation. The wells were utilized to collect continuous water levels during one of the infiltration basin system tests.

The exact screen elevation setting for each well was determined through geologic logging of rock cores and interpretation of downhole geophysical logs. The logging was usually completed in the deepest well at each well cluster.
The geophysical suite consisted of the following:

- Acoustic televiewer (ATV)
- Natural gamma
- Resistivity
- Caliper
- Spontaneous potential
- Temperature

A gyrocompass was used during the logging of the well bore so that an accurate orientation of the identified features (fractures and voids) could be determined. At location S332C MW-0007, natural gamma logs and rock cores were utilized to determine that the Miami Limestone is approximately 21 feet thick in the study area (elevation 3 to elevation –18 feet NGVD) and that it is underlain by 35 feet of the Fort Thompson Formation (elevation –18 feet to –53 feet NGVD). These limestone formations are mantled by approximately one to two feet of overburden soils. These soils appear to be a mix of natural silty-sand materials and rock plowed soils.

A total of 15 geotechnical soil samples were collected from five test pits, and submitted for analysis of moisture content, Atterberg limits, compaction, and dry sieve analysis. The information was utilized to classify the soils according to the Unified Soil Classification System (USCS) that is the Corps of Engineers preferred classification system. Table 1.x presents a summary of the geotechnical analysis.

Table 1 Overburden Soil Testing Results

Soil Sample #	Unified Soil Classification	% fines
TP-01-0001 (top of pit)	GW-GM	11.4
TP-01-0002 (bottom pit)	GW	2.30
TP-01-0003 (bottom pit)	GW	10.13
TP-02-0001 (top of pit)	SP-SM	9.60
TP-02-0002 (bottom pit)	GP	4.90
TP-02-0003 (bottom pit)	GW	1.75
TP-03-0001 (top of pit)	SM	14.80
TP-03-0002 (bottom pit)	GW	4.61
TP-03-0003 (bottom pit)	GW	2.07
TP-04-0001 (top of pit)	SM	28.20
TP-04-0002 (bottom pit)	GW	1.58
TP-04-0003 (bottom pit)	GP	2.11
TP-05-0001 (top of pit)	GM	48.20
TP-05-0002/3 (mixed sample)	GW	1.80

Evaluation of the geotechnical data indicates there is a large difference in the physical properties between the near surface soils and those three to four feet below the ground surface. The near surface soils appear to contain much higher percentages of fines while those at depth are most likely in-situ, soft limestone that was crushed during the test pit excavation process. The near surface materials are classified as sands or silty sands (SW, SP or SM). The deeper materials have been classified as gravels or silty gravels (GW, GP or GM), but they probably should be classified as soft, highly-vuggy limestone. Soil classification data that is discussed in later sections of this report utilizes soil data from the U.S. Department of Agriculture classification system. Under that classification system, the onsite soils would be classified as coarse sands, loamy sands or sandy loams. The underlying limestone is not really covered by the classification system since it is devoted entirely to overburden soils.

Onsite Percolation Testing

Standard percolation tests based upon U.S. Department of Agriculture (USDA), Soil Conservation Service, guidelines have been performed for many years. This quick and simple in-situ test measures water percolation rate in inches per hour or inches per day. Much of this information has been compiled by the USDA and is available in County Soil Survey reports.

Standard percolation tests in South Florida are less reliable due to man induced changes to the surface soils (farming, roads, etc..) and the nature of the porous fossilliferous limestone that forms the majority of the Biscayne Aquifer. Standard small-diameter tests will tend to give skewed results because of the heterogeneous nature of the limestone material. One test may be performed in an area of fractures and solution cavities where the percolation rate will be high, while other tests will be performed on top of finer-grained sandy limestone where the percolation rate may be very slow. In order to reduce some of the uncertainty in percolation rate values, a new large-scale percolation test procedure was developed for this investigation (Brown, 2001).

A total of three paired percolation tests (each pair consisting of a large-scale percolation test and a comparative standard double-ring infiltrometer test performed per the ASTM D3385 method) were performed to determine percolation rates within the study area. Two large-scale percolation tests were conducted within the proposed S-332B basin area (nearby the main study area). One large-scale percolation test was conducted within the S-332C basin area. The three comparative double-ring infiltrometer tests performed per ASTM D3385 were performed adjacent to the large-scale tests. The percolation tests were performed in accordance with ASTM D3385 and the *Procedure for Custom Large Scale Percolation Tests*, developed by the Jacksonville District, U.S. Army Corps of Engineers (Brown, 2001). The procedure for the large-scale test is summarized below.

A 15-feet by 15-feet square area was cleared and grubbed. The percolation test was performed on top of existing topsoil and limestone; therefore, scraping down to bedrock was not required.

Once the soil surface was exposed, a trench approximately 1 foot deep with centerline dimensions 10 feet by 10 feet was excavated. A 10-feet by 10-feet steel percolation cell with an open bottom was installed within the trench. The box sides were constructed of welded steel and measured 5 feet high and 0.25 inch thick. Steel cross-braces were welded to the top and bottom of the box for structural integrity and to allow a crane operator to load and place the box using steel cables attached to the cross-braces. A steel splash plate was installed in the center of the box to disperse water evenly while filling the cell.

Once the steel box was placed in the trench and leveled, the side-walls were sealed to the bedrock with a water-tight joint by pouring grout into the trench on both sides of the box. Loose soil on the inner bank of the trench was washed or brushed away

to expose a relatively clean rock face and ensure an adequate seal at the trench/grout interface. The inside steel/grout joint was caulked with an industrial-strength silicone gel caulk. Once the percolation cell was completed, the required instrumentation (piezometers, water level indicators, etc..) was installed (SAIC, 2002). Pressure tranducers were utilized to measure the water level in the test cells over time. A two-channel instrument was utilized to record water levels in the well and the percolation cell. Regular water level measurements were recorded until the percolation cell was dry or until the water level in the percolation cell did not drop for 1 hour. Following completion of the test, 1 hour of post-test measurements were recorded at 15-minute intervals (SAIC, 2002).

The test cell data was then reduced and estimates of the infiltration rate were developed. The infiltration rate was calculated using water budgets and water level recession rates. The water budget analysis was fashioned after procedures developed for ring infiltrometers. Basically, the flow in minus the change in storage equals the flow out of the system or the infiltration into the ground. Evaporation out of the system was ignored due to the short testing period (less than one day). The results are available in Table 2.x below. It is interesting to note that the shorter tests (8 hours or less) provided infiltration rates much higher than the longer term 40-hour tests. This observation is discussed in more detail later in this report. As was explained previously, standard double-ring infiltrometer tests were also run as companion work at each large-scale test site. The standard infiltrometer test results are summarized in Table 3.x below.

Table Results of Large-Scale Percolation Tests

Test Box – Test	Tank Water Elevation at Start (feet msl)	Tank Water Elevation at End (feet msl)	Elapsed Time from Start of Test to End (hours)	Relative Infiltration Rate (feet/hour)
Box 1 – Test 1	8.940	3.138	46.92	0.124
Box 1 – Test 2	8.895	6.367	40.42	0.063
Box 2 – Test 1	8.38	6.37	7.0	0.287
Box 2 – Test 2	8.63	6.46	9.0	0.241
Box 3 – Test 1	8.44	6.52	5.42	0.354
Box 3 – Test 2	7.862	5.892	7.83	0.251

msl = mean sea level.

Table 3 Standard Double-ring Infiltrometer Test Results

Test Location	Infiltration Rate (feet/hour)
Test Box 1	0.174
Test Box 2	0.053
Test Box 3	0.137

Basin Construction

The C-111 basins were planned as a shallow impoundment surrounded by earthen levees. The levees are approximately six feet high and are designed to hold up to four feet of excess storm water pumped from the C-111 Canal via a water control pump station and corrugated metal conduits. In order to build the levees, the interior of the basins was scraped of some of its surface overburden. The result of this action left the basin surface as a mix of fresh limestone bedrock and thin pockets of overburden soils described previously in this report. The completed levees were fairly impermeable and served to force infiltration vertically into the Biscayne Aquifer instead of exiting as levee seepage. The infiltration basins were also fitted with a concrete emergency overflow spillway so that the levees would not be overtopped in a severe storm event. Following construction of the basins and the associated pump stations, a period of prove-out testing commenced immediately.

Two system tests have been run at the S-332C basin to evaluate its actual infiltration capacity and to determine its capability to raise groundwater levels sufficiently high to reduce easterly seepage emanating from ENP. The first test was carried out during the wet season during July 2002. The second test was carried out during the dry season during February 2003. These two tests are discussed in more detail in the following paragraphs. System testing at full-scale is recommended as a best practice for infiltration basin operational design. Full-scale testing allows the designer or owner to develop a final operating manual for the project.

July 2002 System Test

The July 2002 system test commenced on July 10, 2003 and ended on July 13, 2002. During this time, the pump station delivered an average inflow of 245 cubic feet per second (cfs). Several large rains during the testing interval contributed another 7 cfs of average flow. Therefore, the total inflow during the test averaged approximately 252 cfs. It is important to note that the inflow distribution was not even throughout the test and that more inflow entered the basin during the first half of the test than during the second half. One observation by the field staff monitoring the pumping was that the operation of two diesel pumps each pumping at 125 cfs resulted in a steady pool within the basin. This observation of apparent steady-state conditions matches well with the calculated average inflow during the test. A total of approximately 546,000,000 gallons (73,000,000 cubic feet) of water was delivered to the S-332C infiltration pond during the testing cycle.

Since the July test occurred during the Florida rainy season, antecedent moisture conditions within the study area were expected to exert a strong influence on the basin infiltration capacity. In fact over 10 inches of rain had fallen on average during this time. Additional large rains occurred in early June 2002 also. The resulting hydrologic conditions were very wet indeed. Surface stages within ENP to the west were higher than normal and the C-111 Canal system had been working incessantly to remove flood waters from the agricultural areas of Dade County, Florida.

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So how much of this inflow water infiltrated into the ground ? In general, a majority of the water infiltrated into the ground with some water evaporated (minor component compared to water infiltrated). The infiltration rate was calculated on an hourly basis for the test using a water budget approach applied to ring infiltrometers. The governing equation utilized for the S-332C basin is:

$$f(t) = (W - Qout - DH*A)/Dt$$

where f(t) is the infiltration rate; W is the volume applied during the time interval; Qout is the volume of ponded water removed from the plot by a small pump in the case of an infiltrometer; DH is the change in ponded-water level during the time interval Dt (Dingman, 2002). For this analysis Qout was set to zero since there is no outflow pump at the S-332C site. Therefore, in order to determine the infiltration rate for each time interval, only the inflow volume and the change in basin storage was required. In determining the steady-state infiltration rate, numerous boundary effects had to be analyzed resulting in additional uncertainty, however, the water budget analysis did reveal that the average steady-state infiltration rate was approximately 0.06 to 0.07 feet per hour.

February 2003 System Test

The February 2003 system test commenced on February 5, 2003 and ended on February 12, 2002. During this time, the pump station delivered an average inflow of 222 cubic feet per second (cfs). Several small rains during the testing interval contributed another 1 cfs of average flow. Therefore, the total inflow during the test averaged approximately 223 cfs. It is important to note that the inflow distribution was not even throughout the test and that more inflow entered the basin during the first half of the test than during the second half. One observation by the field staff monitoring the pumping was that the operation of two diesel pumps each pumping at 125 cfs resulted in a near steady pool within the basin. This observation of apparent steady-state conditions matches well with the July 2002 system test. A total of approximately 1,054,680,000 gallons (141,000,000 cubic feet) of water was delivered to the S-332C infiltration pond during the testing cycle.

Since the February test occurred during the Florida dry season, antecedent moisture conditions within the study area were expected to exert a strong influence on the basin infiltration capacity. The rainfall recorded during the February test was minimal and resulted in less interference from boundary conditions. As the Everglades ecosystem was dry also, stages within the S332C infiltration pond had to be reduced as compared to the July 2002 test, in order to minimize un-naturally high ENP stages. The lower pond stage may have contributed to a lower apparent steady-state infiltration capacity. On average, the February test had S-332C basin stages that were 0.5 to 0.75 feet lower than those recorded in July 2002. One other item of note is that the length of the February test was about twice that of the July 2002 test. The longer testing period may also have led to a more stable system response.

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So how much of this inflow water infiltrated into the ground for the February test ? In general, a majority of the water infiltrated into the ground with some water evaporated (minor component compared to water infiltrated). The infiltration rate was calculated on an hourly basis for the test using a water budget approach applied to ring infiltrometers as presented above. The test revealed that the average steady-state infiltration rate was approximately 0.05 to 0.06 feet per hour. In addition, the data appears to vary more on shorter time intervals than over longer periods. The moving average shows that the infiltration rate for the last 80 hours of the test was approximately 0.06 feet per hour.

Performance Evaluation

The C-111 system (including all four infiltration basins) has posted an excellent performance record. The S-332B West basin has been in operation since 2001 and has been operating successfully. The remaining three basins have been in operation since 2003 and have operated intermittently during that time. The S-332C basin has been utilized most frequently of all of the basins. As part of this case summary, a complete operational assessment of the four basins was completed. As part of the operational assessment, a water budget evaluation was completed for the four basins in order to determine the total amount of water recharged in the year 2004. In addition, the total number of operational days was determined for each basin for 2004. The data is presented in Table 4.x.

Table   Water budget and operational summary for 2004.

Basin Name	# Operational Days for 2004	Mean Flow Rate into basin during Operations (cfs)	Mean Infiltration Rate during Operations (ft/hr)	Total Water Infiltrated into basin (Gallons)
S-332B West	153	101	0.056	9,531,374,561
S-332B North	112	114	0.041	8,266,846,452
S-332C	157	170	0.047	17,239,267,632
S-332D	76	232	0.024	11,348,024,688
			Total for 4 basins =	46,385,513,333

 The 2004 water budget clearly depicts a highly successful project recharging vast quantities of storm water into the Biscayne Aquifer. The analysis also reveals that the basins have not been utilized equally such that S-332C basin is used more often than any other basin. In addition, the mean infiltration rate for all of the basins is less than observed during large-scale system tests. It is possible that sedimentation within the basins has slightly reduced the capacity or hydraulic interference effects among all of the basins has raised aquifer water levels further than the basins themselves. This so called "superposition" effect is an accepted fact of groundwater hydraulics and is the most likely cause of observed infiltration rate reductions at S-332B West, S-332B North, and

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S-332C. The low infiltration rate at S-332D is probably a result of insufficient use of the basin due to groundwater stage constraints related to the protection of the Cape Sable Seaside Sparrow (CSSS). The endangered CSSS is a ground-nesting bird whose existence depends upon the ability to have dry nests for an extended period of time during the Florida dry season. Therefore, water levels in the vicinity of the S-332D basin cannot be raised as much as technically possible due to the environmental constraint. Without this constraint, it is likely that the S-332D basin could have infiltrated at least 25,000,000,000 gallons of storm water during 2004. The presence of endangered species in the vicinity of any infiltration basin is an important feasibility and design consideration.

Problems Identified, Lessons Learned and Recommended Operational Practices

The C-111 infiltration basin system has performed well since operations began and have generated a number of valuable lessons learned and recommended operational practices.

First, several lessons learned can be gleaned related to feasibility or design studies of potential infiltration basins. Site selection of potential basin locations should include consideration of impacts upon threatened or endangered species, otherwise, the resource in question could lead to under-utilized infiltration systems. Second, the site selection study should also consider current landuse of the proposed basin areas. In the case of the C-111 project, the presence of contaminated soils and citrus canker in orange groves located at the project area lead to considerable delays during construction and additional unplanned costs. Third, subsurface explorations within prospective infiltration basins should utilize continuous sampling drilling techniques to fully characterize the existence of fine-grained impermeable lenses or low-permeability mudstones. If these materials are present, the project may not perform as well as expected. In some cases it may be possible to remove some of these deleterious materials by excavation and replace them with clean sand or gravel.

After construction of the infiltration basins, it is recommended that full-scale infiltration tests be carried out including appropriate monitoring of water budgets, precipitation, ET, as well as groundwater levels around and below the basin. The full-scale testing should provide a final check allowing for completion of an updated operational plan for the project.

The two S-332C system tests support many of the concepts outlined in the Sustained Underground Storage guidance document. First, the depth to the water table for any infiltration basin is very important. Even though the foundation was as permeable as any existing project surveyed, the steady-state infiltration capacity was constrained by the high water table. The February test conditions were much drier than the July test conditions and initially, this resulted in rapid infiltration rates of up to 1 ft/hr, however, as the water table rose beneath the basin to meet the bottom of the infiltration pond, hydraulic limitations constrained the infiltration rate so that the steady-state infiltration rate was generally the same for both tests (~0.06 feet/hour). Due to the high

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water table, the flow from the basin was dominated by lateral flux rather than vertical flux. The two S-332C tests exhibited lateral flow similar to that presented earlier in this report on figure 1.15.

In order to develop a higher degree of confidence in design estimates, the use of the "large-scale percolation tests" appears promising under certain circumstances. As the reader will recall, Table 2.x detailed results of large-scale percolation testing completed for the S-332C basin study area. The results indicate that longer duration versions of these tests may provide decent preliminary design infiltration values. The average infiltration rate determined for the two long duration tests was approximately 0.09 feet per hour. Although this value is 50% higher than those calculated for the full basin tests, it appears to be in the "ball park".

The standard USDA infiltration tests presented in Table 3 resulted in an average infiltration rate of 0.12 feet per hour (100% higher than the full basin values). The results indicate that the smaller-scale tests overestimate the actual basin-scale infiltration rate. The reasons for this observation include the depth to the water table during the small-scale tests and the scale of the tests themselves. During the small-scale tests, the water table elevation did not rise to the bottom of the test apparatus, thus allowing more vertical infiltration and less lateral infiltration. Perhaps some type of "scale-factor" is needed to allow use of the field percolation tests as surrogates for the large infiltration ponds. The sheer size of the full-scale infiltration ponds and the magnitude of flow rates into the ponds generally resulted in a rapid rise of the aquifer water levels. Since this is one of the primary purposes of the basins, this is a positive. However, the rapid rise of the water table resulted in less infiltration capacity since the vertical infiltration component was limited.

Conclusions

The C-111 project has been highly successful and should continue to operate for many years to come. The design of successful infiltration basins within heterogeneous limestone is more complicated than similar design in unconsolidated sediments. The scale of field-scale percolation tests appears to be an important factor in estimates provided. In the end, the best indication of the long-term infiltration rate can be provided by a full-scale system test that is fully instrumented so that sufficient data can be collected and interpreted. Ultimately, the operation of any infiltration basin is dependent upon the site soils, groundwater hydraulics, and operational regime chosen by the owner.

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American Water Works Association Research Foundation
Design, Operation and Maintenance Considerations for Sustainable Underground
Storage Facilities

Artificial Recharge Case Study
Artificial Recharge of Tertiary Treated Wastewater at the
Fort Dix Army Base/Maguire Air Force Base
Fort Dix, New Jersey

OVERVIEW

Fort Dix – McGuire Air Force Base (Fort Dix) has artificially recharged from three (average) to 12 mgd (peak) of tertiary treated wastewater to the 12 recharge basins at the Land Application Site (LAS) since 1994. Prior to discharge, the wastewater is treated with the following treatment processes: primary mechanical screening; secondary treatment by the 'Bardenpho' process; settling; gravity filtration; and disinfection. According to verbal communications with the plant operator, the water discharged to the LAS consistently meets the New Jersey Department of Environmental Protection (NJDEP) Discharge to Groundwater monitoring requirements summarized on Table 1 (verbal communication with David Baril, April 7, 2006). In addition, the NJDEP also requires Fort Dix to collect groundwater samples from __ monitoring wells that surround the LAS to document that the tertiary treated effluent does not negatively impact groundwater (Table 2). According to verbal communication with the plant operator, the groundwater sample results show that groundwater has not been negatively impacted by the operation of the LAS (verbal communication with David Baril, April 7, 2006).

The LAS includes 12 basins each about four acres in size (Figure 1) grouped into six pairs, with the discharge controlled manually by sluice gates to the first basin in each pair. Water may flow from the first basin to the second basin in a pair by gravity through a weir. The original design called for a 12 day operational cycle for each basin pair, with two days of in-flow and ten days draining and drying while the flow is directed to another pair. However, the infiltration rate in the basins is significantly higher than designed. Consequently, a basin receives at least four days of in-flow, two to four days draining and drying while the flow is directed to another basin. Water depths in basins do not exceed 24 inches. The best performing basins are A1, A3, B1 and B2. Groundwater ambient flow direction is to the southeast and wetlands bound the LAS to the southwest and northeast (Figure 1). Depending on location, depths to water vary from 13 to 33 feet below ground surface. The basins were constructed in the Cohansey Sands aquifer on the Atlantic Coastal Plain. Based on slug tests, hydraulic conductivity ranges from 2.6E-4 to 1E-3 cm/s, with a geometric mean of 8.4E-4 cm/s. There is a network of monitoring wells around the LAS (Figure1).

LAS PERFORMANCE

Key performance points are listed as follows.

- Additional sluice gates were subsequently added to many of the second basins in a pair allow all basins (except B6) to be operated individually.
- Groundwater mounding has not been reported to be an operational concern.

- There have been no exceedences of the discharge monitoring requirements, or of impact to groundwater.
- Infiltration rates for most basins exceed the rates originally designed.
- Basin B6 is never used due to slow infiltration, proximity to wetlands (Figure 1) and inadequate vertical separation with the water table. Standing water due to precipitation (Figure 2) has been common since construction.
- Embankment sloughing or instability occurs (a small patch is visible on Figure 3). The embankments were constructed from sediment removed from the basins, and are therefore comprised entirely of Cohansey Sand. Fine-grained silt and clay or top soil was not used as a surface cap on the embankments.
- Scour at the base of embankments is common as shown by Figure 4. This is due to the loose Cohansey Sand taken from the basin excavations.
- The embankments have a thin grass cover. Consequently the embankments are not adequately stabilized by grass. Attempts to establish a strong grass cover have met with mixed success.
- Plastic mesh was tried on the embankments, in order to provide a stable base for re-seeding. However the mesh frequently entangled the mower blades.
- Some embankment slopes are steep enough as to be close to the safe operating limit of the mowers.
- Poorly constructed riprap at the egress of the sluice gates is shown in Figure 3. The 'stones' were too small and were set in a cement/concrete which has broken down in many places. The aprons were also not wide enough. Consequently, with the high exit velocities, many 'stones' have been washed to the basin floors where they cause damage to scarifying equipment.
- Old riprap aprons are being replaced by wider and thicker aprons of much larger stones, as shown by Figure 5.
- Minor seepage through embankments (Figure 6) occurs due to lack of clay cores. This is considered by the operator to be a minor issue.
- Algae and vegetation growth is a problem in basin A1 (Figure 7); however, it is not enough to decrease the permeability of this basin. Basin A1 needs to be scarified and the growth removed more often than the others. The operator is uncertain of the reason for growths in Basin A1.

LESSONS LEARNED
The primary lessons are listed as follows.
- The original sluice gate riprap aprons were clearly inadequate. The operator suggests that the problem may have arisen with inadequate construction oversight.
- Embankment erosion and toe scour is a significant issue. The costs of supplying a cap of fine-grain soil or top soil (loam) may have been high due to a lack of local sources. However, considering that there was additional land area that could have been used, the embankment slopes, with resultant wider embankment footprints, could have been reduced and lower slopes may have decreased embankment erosion.

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

Legend

- - - Distribution Pipe
···▶ Weir & flow direction

● Monitoring well
● Piezometer
✦ Sluice Gate Valve
● Valve pit 3

Note:

Locations of monitoring wells and piezometers are approximate. Not all may be monitored.

Wetlands indications based on James C. Anderson Associates, Inc. (1991) aerial photograph indications, and communication from David Baril, (Op-Tech Fort Dix Plant Manager).

Basin A1 — Bottom El. 142
Basin A2 — Bottom El. 140
Basin A3 — Bottom El. 138
Basin A4 — Bottom El. 132
Basin A5 — Bottom El. 128
Basin A6 — Bottom El. 126
Basin B1 — Bottom El. 130
Basin B2 — Bottom El. 126
Basin B3 — Bottom El. 113
Basin B4 — Bottom El. 112
Basin B5 — Bottom El. 116
Basin B6 — Bottom El. 108

Force main, from TWTF

Fort Dix Landfill

CANNONDALE RUN

WETLANDS

FORT DIX AND MCGUIRE AIR FORCE BASE
FORT DIX, NEW JERSEY

Land Application Site, Basins and Distribution

MALCOLM PIRNIE

MALCOLM PIRNIE, INC.

Figure 1

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

Basin B6

Note: Basin B6 usually has some standing water. The basin is close to wetlands and is never used, although it is scarified yearly.

Basin B4

FORT DIX AND MCGUIRE AIR FORCE BASE
FORT DIX, NEW JERSEY
Land Application Site, Basin B6

MALCOLM PIRNIE, INC.

Figure 2

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

Embankment Slough.

Eroding riprap, Basin B5. Small 'stones' set in cement.

Note: This old riprap is eroded, damaged and in need of replacement.

FORT DIX AND MCGUIRE AIR FORCE BASE
FORT DIX, NEW JERSEY

Land Application Site, Basin B5 Sluice Gate Outlet Old Riprap

MALCOLM PIRNIE

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Figure 3

Scour of Basin B3 Toe.

FORT DIX AND MCGUIRE AIR FORCE BASE
FORT DIX, NEW JERSEY

Land Application Site, Scour of Basin B3 Toe

MALCOLM PIRNIE, INC.

Figure 4

MALCOLM PIRNIE

Tertiary Treated Water flowing into basin from sluice gate.

Note: This riprap replaced the previous eroded and damaged riprap. Flow is approximately 3 MGD.

FORT DIX AND MCGUIRE AIR FORCE BASE
FORT DIX, NEW JERSEY

Land Application Site, Basin A3 Sluice Gate Outlet Riprap

MALCOLM PIRNIE, INC.

Figure 5

Seepage from Basin B2 to Basin B5.

FORT DIX AND MCGUIRE AIR FORCE BASE
FORT DIX, NEW JERSEY
Land Application Site, Seepage From Basin B2 to Basin B5

MALCOLM PIRNIE, INC.

Figure 6

FORT DIX AND MCGUIRE AIR FORCE BASE
FORT DIX, NEW JERSEY
Land Application Site, Basin A1 Algal and Sod Growth

MALCOLM PIRNIE, INC.

Figure 7

MALCOLM PIRNIE

EDWARDS AQUIFER AUTHORITY
GROUNDWATER RECHARGE THROUGH SURFACE INFILTRATION: RECHARGE DAMS AT SECA CREEK, PARKER CREEK, VERDE CREEK, AND SAN GERONIMO CREEK*

*Information adapted from an interview with Geary M. Schindal (Chief Technical Officer and Aquifer Science Program Manager, Edwards Aquifer Authority) on June 6, 2006 and from two reports: 1. Edwards Aquifer Authority Hydrologic Data Report for 2004, produced by the Edwards Aquifer Authority, and 2. Analysis of Recharge and Recirculation Edwards Aquifer Phase 1, produced by Todd Engineers.

Background

The Edwards Aquifer is one of the largest groundwater systems in Texas, extending approximately 180 miles long, 5 to 40 miles wide, covering parts of nine counties, and supplying water to 1.7 million people. It is also one of the most permeable and productive aquifers in the United States, delivering between 300,000 and 500,000 acre-feet of water per year through well pumping. In addition to supplying potable water, the Edwards Aquifer provides the vast majority of agricultural and industrial water in the area, supports a delicate aquatic ecosystem through ~300,000 acre-feet per year of artesian spring discharges, and supplies recreational water as part of the Nueces, Medina, Guadalupe, and San Marcos River basins.

Hydrogeology

The Edwards Aquifer lies within the Cretaceous age Edwards Group limestone and associated units, is capped by the Del Rio Clay unit, overlays the Glen Rose Formation, and varies from 450 to 600 feet thick. Due to a series of faults, there are approximately 1,250 square miles of Edwards limestone exposed at the ground surface, which serve as the recharge zone for the Aquifer (Figure X). Various streams flow south through the Texas Hill Country and drain most or all of their baseflow to the recharge zone through sinking streams, sinkholes, and caves. Hydraulic residence times in the Aquifer vary from a few hours to many years depending on location and depth of circulation.

Engineered Surface Recharge

The Aquifer's large land surface exposure and high transmissivity and the presence of numerous sinkholes and caves make groundwater recharge inherently efficient. However, recharge is controlled by surface stream flow, which is ultimately controlled by the highly variable rainfall in the region. During dry periods, streambeds dry up, and minimal recharge occurs. Therefore, it is important to capture a high percentage of stream flow as recharge during periods of high rainfall intensity (i.e., flooding). To that end and to complement the various modes of natural recharge, the Edwards Aquifer Authority has constructed four recharge dams, one each at Parker Creek, Verde Creek, San Geronimo Creek, and Seco Creek (see locations on Figure X). Water that builds up behind the Parker, Verde, and San Geronimo Dams readily percolates down to the groundwater. Water that accumulates behind the Seco Creek Dam decants slowly over the Dam into a 10'x10' channel, travels horizontally in the channel for about 700 feet, and dumps into a 100' natural shaft (Figure Y).

Construction of the Parker, Verde, San Geronimo, and Seco recharge dams was completed in 1974, 1978, 1979, and 1982, respectively. Annual recharge volumes are calculated using data from level and flow recorders located at each site. Cumulative, combined recharge from the four dams from construction through 2004 totals 152,877 acre-feet, and combined annual recharge has varied from 0 acre-feet in 1988, 1989, and 1996 to 21,993 acre-feet in 2004. Photos of the San Geronimo Creek and Verde Creek Dams taken in February 2006 (Figures Z and AA, respectively) confirm level and flow recorder data indicating that 2006 engineered recharge totals are near zero.

There is very little required routine maintenance on or repair of the recharge dams. Once per month, someone from the Edwards Aquifer Authority visits each of the four sites, downloads data from the level and flow recorders and checks for any visible structural concerns such as cracks. Some mild subsidence of the dams has been observed, but only minor crack repair has been required.

Lessons Learned

By design, the Edwards Aquifer Authority engineered recharge practice is passive. Recharge is driven by rainfall and streamflow and requires only monthly tracking. Furthermore, no design or construction has occurred at the sites since they were first built. However, there are a few recharge issues with which the Edwards Aquifer Authority must contend. A summary of those issues and outcomes follows.

Level and flow data from on-site recorders can only be gathered manually, making data management slow and bulky. To streamline data management and to add the ability for real-time tracking, the Authority is adding telemetry to the on-site recorders by the end of 2007.

Sediment builds up behind the Seco Creek Dam, which can spill over the Dam and eventually discharge into the 100' recharge shaft. If too much sediment is discharged, plugging of the shaft can be a problem. To avoid sediment discharge, the Authority monitors the depth of the sediment behind the Seco Creek Dam. When the sediment reaches 2-3 feet, they excavate the sediment and reapply it to local ranch land. Not only does this practice address the issue of recharge plugging but it also alleviates some anxiety of local ranchers who are concerned about erosion of their land.

The transmissivity of the Aquifer is so high that during wet periods, artificial recharge (i.e., recharge from the four dams + recharge from individuals who have been granted permits for pumping to the aquifer from surface water bodies located on their land) can be observed in downstream artesian spring discharges within hours. In fact, the Authority has observed pressure waves traveling 70 miles along the aquifer over a 12-hour period. The main problem with this phenomenon is that storage of "extra" water in the Aquifer does not last long. In other words, above some threshold recharge volume, artificial recharging of the aquifer does little to augment groundwater availability during dry periods. To address this issue and to maintain required downstream minimum flows during all periods, the Authority uses a recharge/withdrawal credit system.
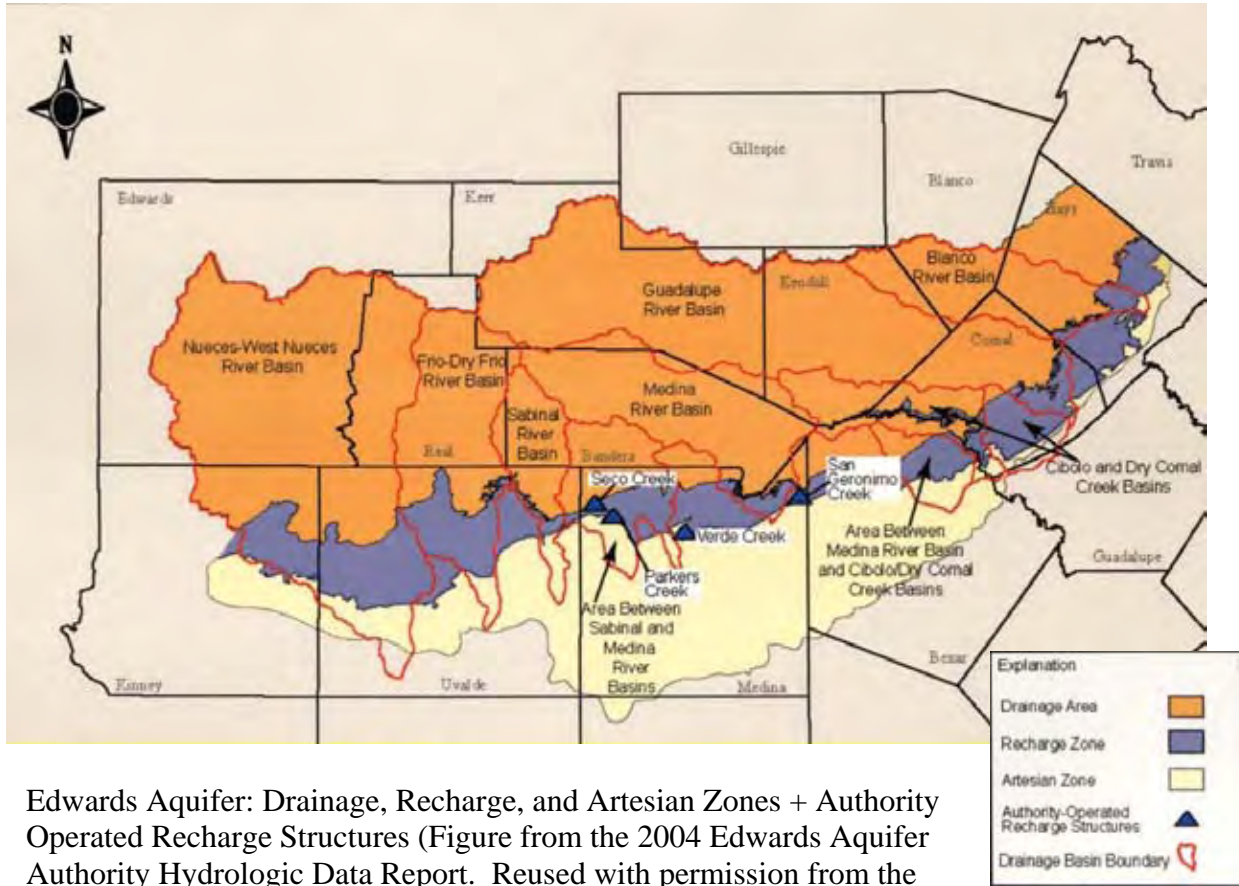
Edwards Aquifer: Drainage, Recharge, and Artesian Zones + Authority Operated Recharge Structures (Figure from the 2004 Edwards Aquifer Authority Hydrologic Data Report. Reused with permission from the Edwards Aquifer Authority)

Natural Recharge Structures Associated with the Seco Creek Dam

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

San Geronimo Creek Recharge Dam (February 2006)

AA Verde Creek Recharge Dam (February 2006)

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

A Case Study of an ASR Failure at the Urrbrae Wetland

Paul Pavelic[1], Peter Dillon[1], Karen Barry[1], Don Armstrong[2], Allin Hodson[3],
Jim Callaghan[4] and Nabil Gerges[5]

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Introduction

The ASR literature contains many case studies that demonstrate the success of field trials and established operating schemes and generally promote the positive aspects of ASR (eg. Pyne, 1995 and the references cited therein). Documented evidence of ASR failures, and the underlying causes for failure, have been far less common and probably cannot be explained by an absence of intractable problems (eg. Pavelic and Dillon, 1997). Two specific examples of such problems include excessive well clogging due to injection of wastewater into a fractured rock aquifer (Lakey, 1978), and the rupturing of the overlying clay layer due to injection of surface water (Ramnarong, 1989). Hesitance to report on negative aspects of ASR may lead to the perception that ASR is a fail-safe technology under all circumstances. Only through the dissemination of both the positive and negative information can the issues and failures of the past be avoided and the knowledge base of ASR advanced.

ASR operations in Australia have largely focussed on limestone or fractured rock aquifers and the results have generally been successful (Martin and Dillon, 2002; Hodgkin, 2005). From a well clogging perspective, limestone aquifers are the more tolerant of poorer source water quality due to the offsetting effect of matrix dissolution. Although fractured rock aquifers are more complex to characterize in terms of their permeability structure and storativity, detailed studies have not yet been conducted, apparently because existing sites have been operating successfully (Hodgkin, 2005).

Unconsolidated fine-grained aquifers present challenges to maintain adequate rates of injection in ASR wells. In Australia and elsewhere, opportunities to enhance groundwater resources through ASR have been foregone due to lack of knowledge in semi-arid regions where unconsolidated alluvial aquifers represent the predominant target unit and excess waters are in seasonal abundance.

In 1997 an ASR trial was initiated at the Urrbrae Wetland site in metropolitan Adelaide, South Australia to test the viability of injecting wetland-treated urban stormwater into an unconsolidated siliceous aquifer so that the recovered wetland water could be used for landscape irrigation of adjacent school grounds. The trial failed due to irreversible clogging of the ASR well. This report retrospectively documents, six years after the trial was suspended after all viable options were exhausted, the main outcomes and lessons learnt during the ASR trial and attempts to remediate the ASR well. Although the ASR trial did not succeed, this examination of the causative factors of failure may have a positive spin-off for proponents contemplating ASR under

similar circumstances and for our own current research activities which are intended to create the conditions necessary to ensure success of future trials at this site and others that are similar.

Local hydrogeology

In broad terms, the regional hydrogeology of the Adelaide Plains is comprised of a Quaternary alluvial sequence of low yielding aquifers, overlying a Tertiary limestone sequence of higher yielding aquifers and Pre-Cambrian bedrock, with a combined thickness of up to 500 metres in the west part of the Plain (Figure 1) (Gerges, 1996; 1999).

The late Quaternary/Tertiary aquifers targeted for ASR at the trial site consist of inter-fingered marine sands that probably represent the margins of extensive sandy deposits common along the eastern margins of the Adelaide Plains that form the intermediate (trough) zone between the hard-rock aquifers of the Mount Lofty Ranges and the Tertiary sequence of the Plains proper. Groundwater flow direction is generally westward, towards the coast. Isotopic data from Tertiary wells within two kilometres of the study site suggest groundwater velocities of around 1-2 m/year and a carbon-14 age of around 3000 years (Dighton *et al,* 1994).

Local drilling at the trial site identified the upper 63 m to be Hindmarsh Clay, a fluvial Quaternary unit comprised of stiff clay inter-bedded with thin aquifers. This was underlain by around 8 m of Carisbrooke Sand, the oldest Quaternary deposits, then, in turn, by 23 m of Port Willunga Formation, which is of the Tertiary period (John Botting and Associates Pty Ltd and Lisdon Associates, 1998; Gerges, 1999). The Carisbrooke and the Port Willunga formations are the most productive aquifers and were targeted for ASR. The Carisbrooke consists of medium to fine grained calcareous sand with some ferruginous and possibly some inter-bedded silt layers. The Port Willunga formation consists of coarse sands and gravels with varying lignitic content.
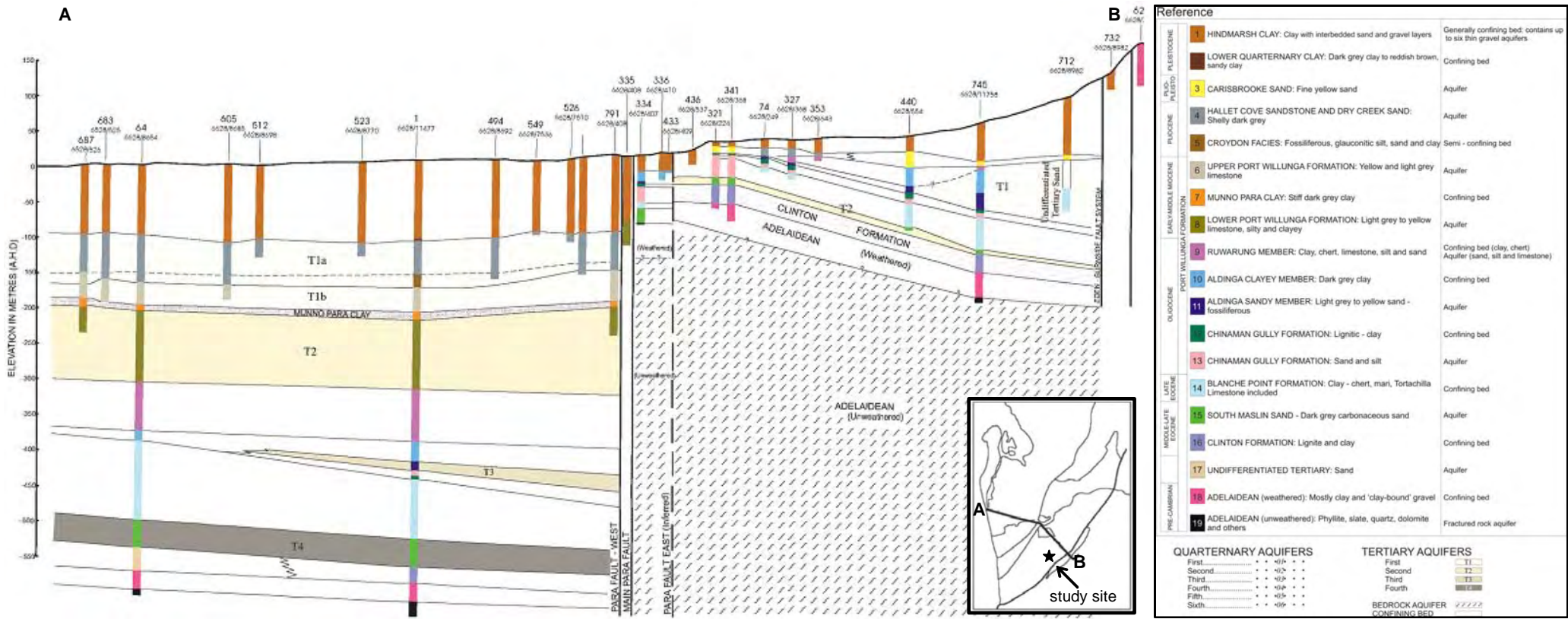
Figure 1 Hydrogeological transect across the Adelaide Plains (from Gerges, 1996)

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

ASR system design

In July 1997 the ASR well (Unit Number 18576) was drilled to a depth of 93.6 m using the rotary mud drilling method. Drilling ceased at this depth due to increasing lignite content and the well was later backfilled to 84.7 m (John Botting and Associates Pty Ltd and Lisdon Associates, 1998). The original pilot hole was reamed to a diameter of 229 mm (9-inch) then the well was cased in 203mm (8-inch) UPVC and cement grouted to the surface, with a larger, 298 mm (12-inch) UPVC collar casing in the top 5 m. A 152 mm (6-inch) wire-wrapped stainless steel screen assembly was installed on the basis of geophysical logs and a limited amount of sample cuttings. Active screen was installed over three intervals with blanks fitted to avoid the more lignitic layers (Figure 2). The uppermost screen aperture was 0.5mm to the finer-textured Carisbrooke sand and the lower two screens were 1.0mm for the coarser textured Port Willunga Formation. The well was extensively airlifted and backwashed to dislodge residual drilling muds and develop a natural gravel pack. The airlift yield of the well was 3 L/s and discharge testing with the pump positioned immediately above the screens at 63.5 m (>30m below the standing water level) led to cavitation of the pump and a flow rate of 4.3 L/s. The anticipated yield based upon well coefficients derived from step testing and the 33m of available drawdown was estimated to be 3 L/s. The combined transmissivity of the aquifers was estimated to be around 6 m^2/day.

The ASR well is situated on the south-western corner of the holding pond, within close proximity to the source water and necessary power supply (Figure 3).

The components of the ASR system include the ASR well fitted with a Calpeda submersible pump positioned at 80 m depth, an 8 m^3 ferro-cement storage tank, two rapid sand media filters (Yamit 600 series), on-line cumulative flow meter, manual flow control valves, electrical control system and cement footing for the sand filter and proposed irrigation pumps (Figure 4).

Water was pumped from the holding pond through the sand filters and into the holding tank at a rate of 1.25 L/s using a submersible pump (Calpeda MXS 204) mounted on a float positioned just below the pond surface. This water was then gravity-fed into the ASR well. The sand filters were programmed to backwash every two hours for five minutes, with the waste stream returned to the main lagoon. A recharge line composed of 20 mm UPVC pipe was installed to a depth just below the standing water level to control clogging by aeration. The depth to standing water level, which ranged from 30-34 m below ground surface (bgs), provided latent storage capacity and was at a sufficient depth to provide a good driving head from the tank.

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.
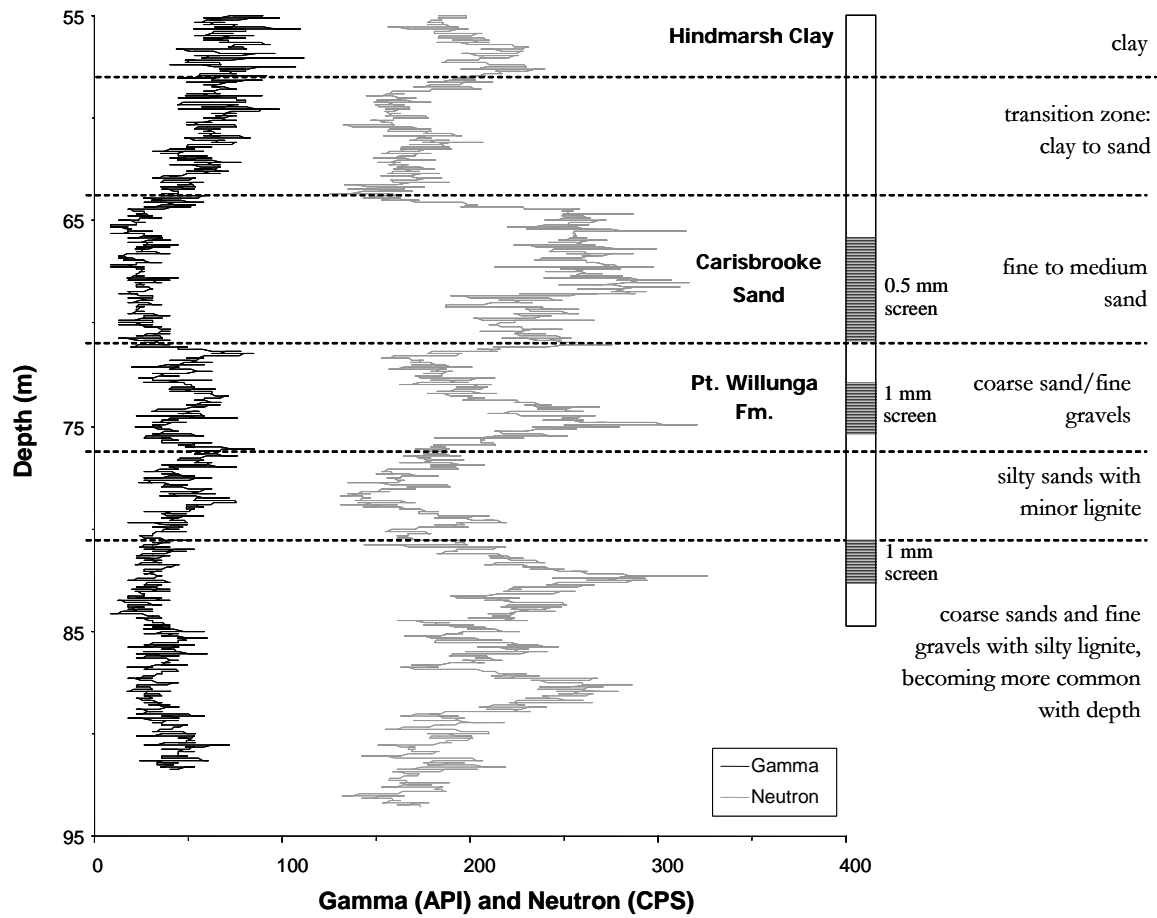
Figure 2 Geophysical logs, hydrogeology and completion arrangement of the Urrbrae ASR Well (Unit Number 18576)

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.
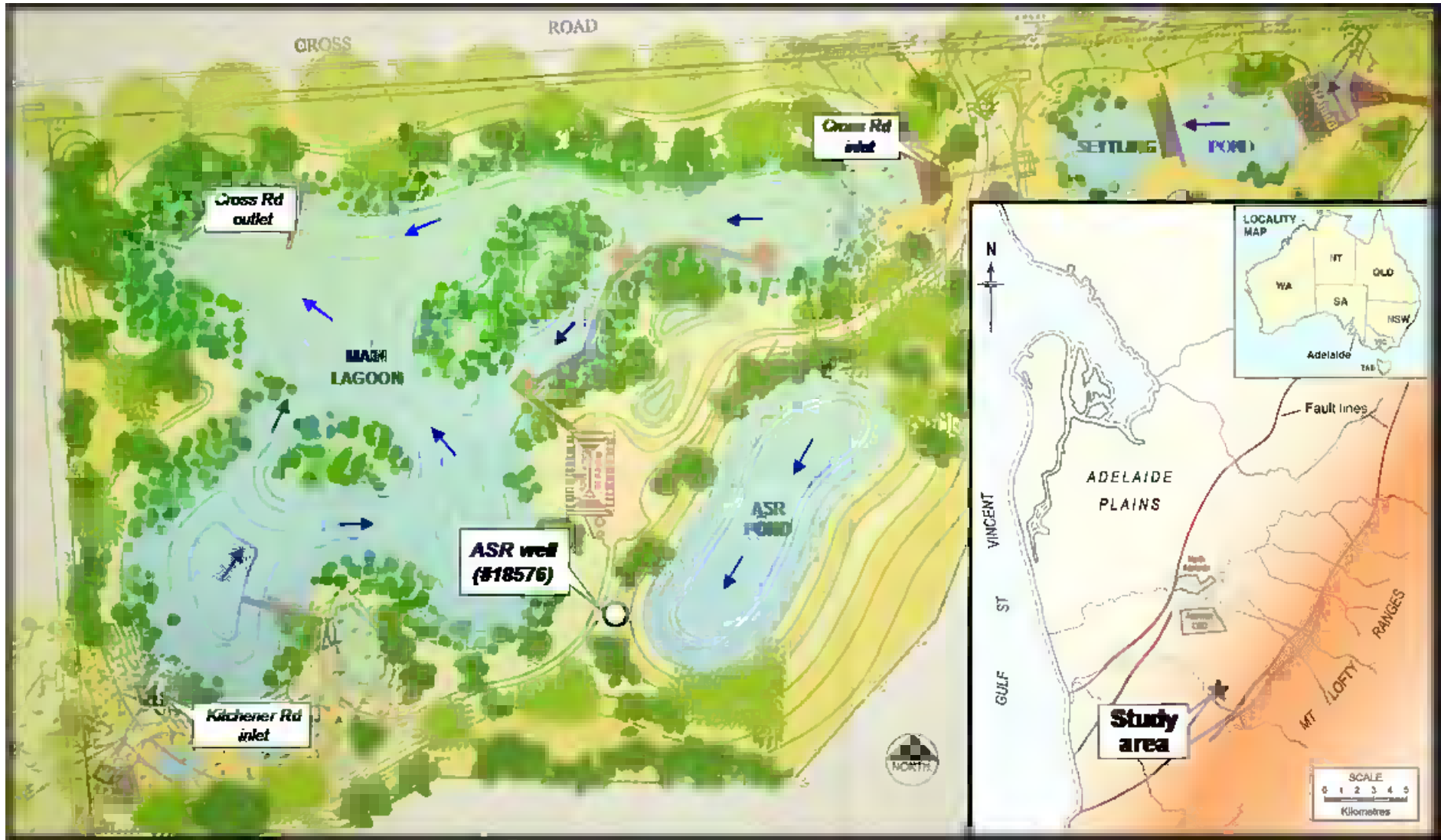
Figure 3 Site map showing location of ASR well in relation to the holding pond (*source of regional map: Bob Schuster, CSIRO*). The arrows indicate the inferred direction of flow during stormwater inflow and well injection (from Lin *et al*, 2005). Note the pre-settling ponds were established in 2003, after the conclusion of this study.

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.
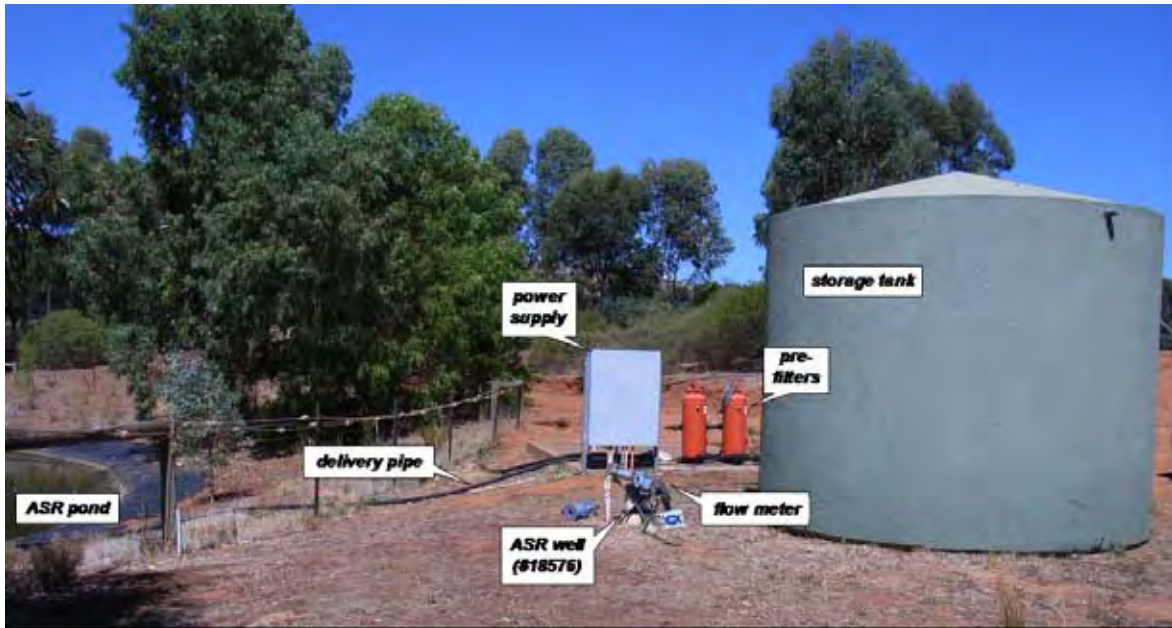
Figure 4 Study site at Urrbrae, South Australia indicating the main components of the ASR system

Catchment and wetland characteristics

The 375 hectare (Ha) catchment is comprised of two sub-catchments (Cross Road and Kitchener Street). Each sub-catchment takes in the margins of the Mount Lofty Ranges and adjacent Adelaide Plains (Figure 5). The catchment contains a mix of landuses including agriculture (mainly non-irrigated), residential, agricultural education and research facilities. The catchment contains virtually no industry and little development of commercial property (Hodson, 1999).
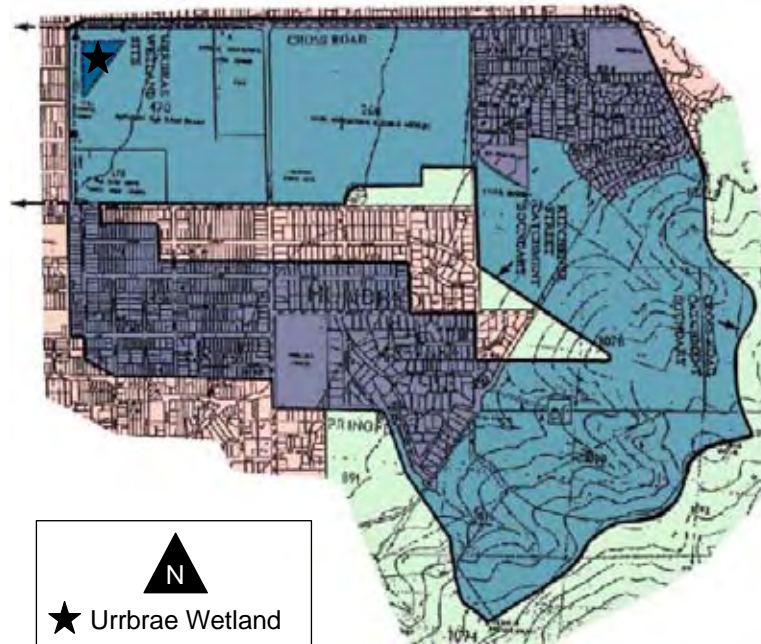
Figure 5. Surface water catchment for the Urrbrae Wetland comprising of the Cross Road and Kitchener Street sub-catchments (from Hodson, 1999)

The Urrbrae Wetland ASR site is located at the Urrbrae Agricultural High School adjacent to Cross Road, Urrbrae, South Australia. The major features of the wetland include the main lagoon and rubber-lined holding pond (Figure 3). The wetland was built in 1996 primarily to mitigate local flooding and was engineered to handle peak stormwater flows associated with a 1-in-5-year storm event (Hodson, 1999). Estimated mean annual volumetric flow through the wetland is around 350×10^3 m^3.

Water depth in the main lagoon is typically greater than 1.5 m for the majority of the year (Hodson, 1999). The bottom is clay-lined and needs to be kept full during the dry season to protect the lining from shrinkage and cracking. The bottom of the holding pond is lined with welded polythene sheeting and filled during the wet season from the main lagoon through a subsurface pipe located between the Cross Road inlet and eastern extent of the observation deck. During the wet season, flow from the main lagoon into the holding pond is minimal due to the high surface water elevation maintained in the holding pond after initial storms. During the dry season, water in the holding pond is used to replenish water lost to evaporation in the main lagoon. Source water for ASR is pumped directly from the holding pond. The anticipated direction of surface water flow during injection is indicated in Figure 3 and suggests that the injectant will be derived from stormwater entering the main lagoon via the Cross Road inlet and overflowing into the holding pond and a small component from direct rainfall. Residence time of water in the holding pond is likely to be higher than the main lagoon.

Wetland water quality

The principal gross pollutants entering the wetland are associated with the extensive vegetation cover within the catchment, which, when combined with the high runoff velocities due to the moderate to steep slopes produces significant influx of leaf and other organic debris throughout the year. The large contribution of organic matter results in elevated TOC concentrations causing periodic oxygen depletion within the wetland. Inorganic fines and colloidal matter are generally a second-order phenomenon, except during periods of building construction within the catchment.

It is recognised that there are inherent temporal variations in the quality of water in the wetland due to stormwater runoff and algal growth in the shallow, nutrient-rich water. The variability in the composition of the wetland water with respect to water quality parameters indicative of clogging are presented in Table 1. Total suspended solids (TSS) and volatile suspended solids (VSS) data reveal that most of the suspended solids in the wetland are organic in nature. Only in samples collected near the inlet-end of the main lagoon during runoff events (eg. 19 Oct. 05) are the majority of particles inorganic in nature. Table 1 shows that particulate concentrations in the wetland water are highly variable, as anticipated of urban stormwater (eg. turbidities range from 0.8 to 55 NTU). The physical clogging potential of the wetland water according to Membrane Filtration Index (MFI) data is high relative to the particulate level due to the predominantly organic nature of suspended particles in the stormwater which more easily compress and clog the filter paper pores than rigid inorganic particles (Dillon *et al,* 2001). The MFI of the detention basin is higher per unit TSS than the main lagoon due to the higher algal content.

Bacterial regrowth potential (BRP) concentrations range from 39 to 331 acetate carbon equivalent (ACE) μg/L. The assimilable organic carbon (AOC) threshold for biologically stable waters is 40 ACE μg/L (Werner and Hambsch, 1986), and the maximum permissible level for AOC in the Netherlands for injection into fine sandy aquifers is 10 μg/L (Hijnen and van der Kooij, 1992). Unfortunately AOC and BRP relate to different components of labile organic carbon and are therefore incomparable indices of nutrient bioavailability.

Electrical conductivity (EC) values of the recharge water during the winter-spring period when the greatest opportunity for injection exist are typically <300 μS/cm, and during summer-autumn period are highest at 300 to 500 μS/cm (Figure 6). This figure also shows that the temperature of the recharge water was likely to have been in the range of 15 to 20 ^{o}C.

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Table 1 Physico-chemical characteristics of the wetland water from 9 sets of analyses between March 1999 and October 2005

Date / Parameter	4 Mar. 99 [A] (MLG)	8 Mar. 99 [A] (MLG)	25 Jun.99 (MLG)	16 Jul.99 (MLG)	8 Nov.99 (HP)	8 Nov.99 (ST)	5 Apr.01 (HP)	17 Oct. 05 [B] (HP)	19 Oct. 05 [B] (MLG)
MFI (s/L^2)	170	345	389	213	323	90	123	170	207
Turbidity (NTU)	3.7	7.5	55	36	6.4	0.81	5.9	10.5	33.9
TSS (mg/L)	4	10	33	20	10	<1	-	10.6	30
d_{50} (μm)	88	195	34	15	127	-	-	130	10
VSS (mg/L)	3	10	-	-	10	<1	-	11	11
TOC (mg/L)	10.2	12.6	4.3	3.9	6.6	4.6	-	4.6	6.8
BRP (μg/L)	88	258	331	39	-	-	-	190	293

'-' = not analyzed; MLG = main lagoon; HP = holding pond; ST = storage tank; BRP = bacterial regrowth potential (acetate carbon equivalents); [A] reported in Massmann *et al,* (1999); [B] reported in Lin *et al,* (2005)
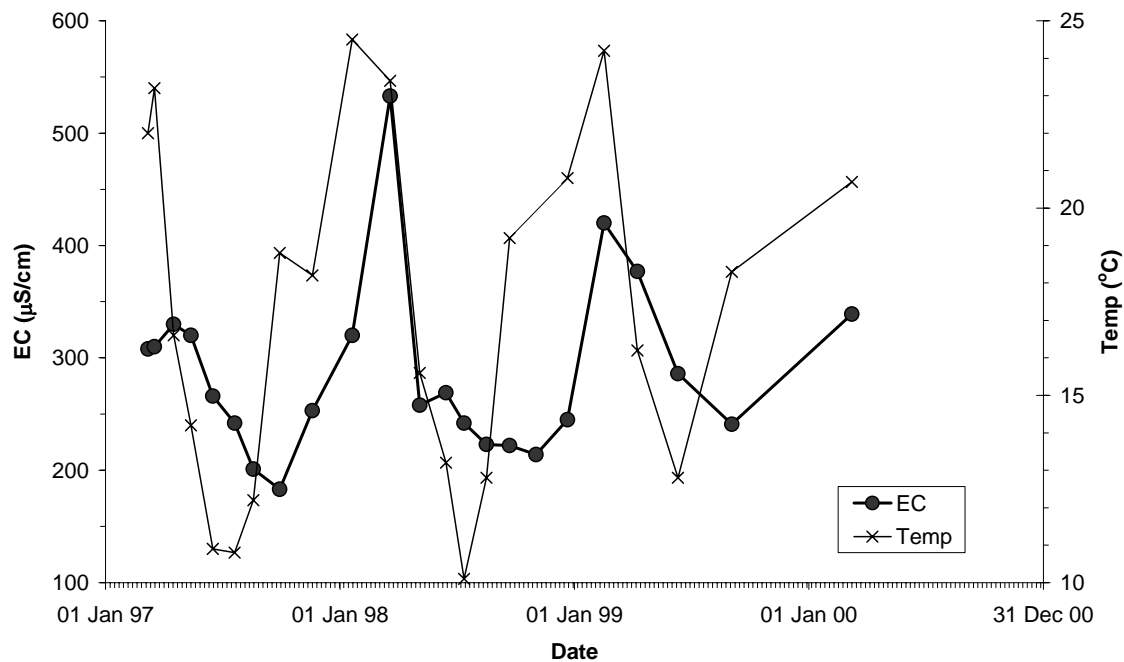
Figure 6 Electrical conductivity (EC) and temperature variations in holding pond between March 1997 and March 2000

Scanning electron microscopy

Scanning electron microscopy (SEM) images reported by Lin *et al,* (2005) reveal mostly inorganic and organic particle assemblages containing large organisms, some smaller organic remnants, diatoms, and bacteria in the main lagoon (Figure 7). Particle sizes

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range from 10 to 100 μm. Energy Dispersive X-ray (EDX) spectra indicated aluminium-silicates, iron-oxides and organics. The holding pond water contains a diverse assortment of discrete particles (mostly macro-organisms) and complex organic and inorganic particle assemblages (or flocs) bound by organic mucilage (Figure 7). Indicative particle sizes range from 50 to 300 μm. Macro-organisms included algae, diatoms, amoebas, fungi and bacteria. Minerals included clay minerals, quartz, and iron-oxides. The abundant amorphous mucilage was reflected in EDX spectra by the high C, O, P, S and K, while aluminium-silicate peaks were associated with the minerals. Macro-organisms were much more common (some algae were observed) and flocs were more uniformly coated with mucilage indicating different biological population or environmental conditions. SEM images previously reported by Massmann *et al,* (1999) show similar characteristics.
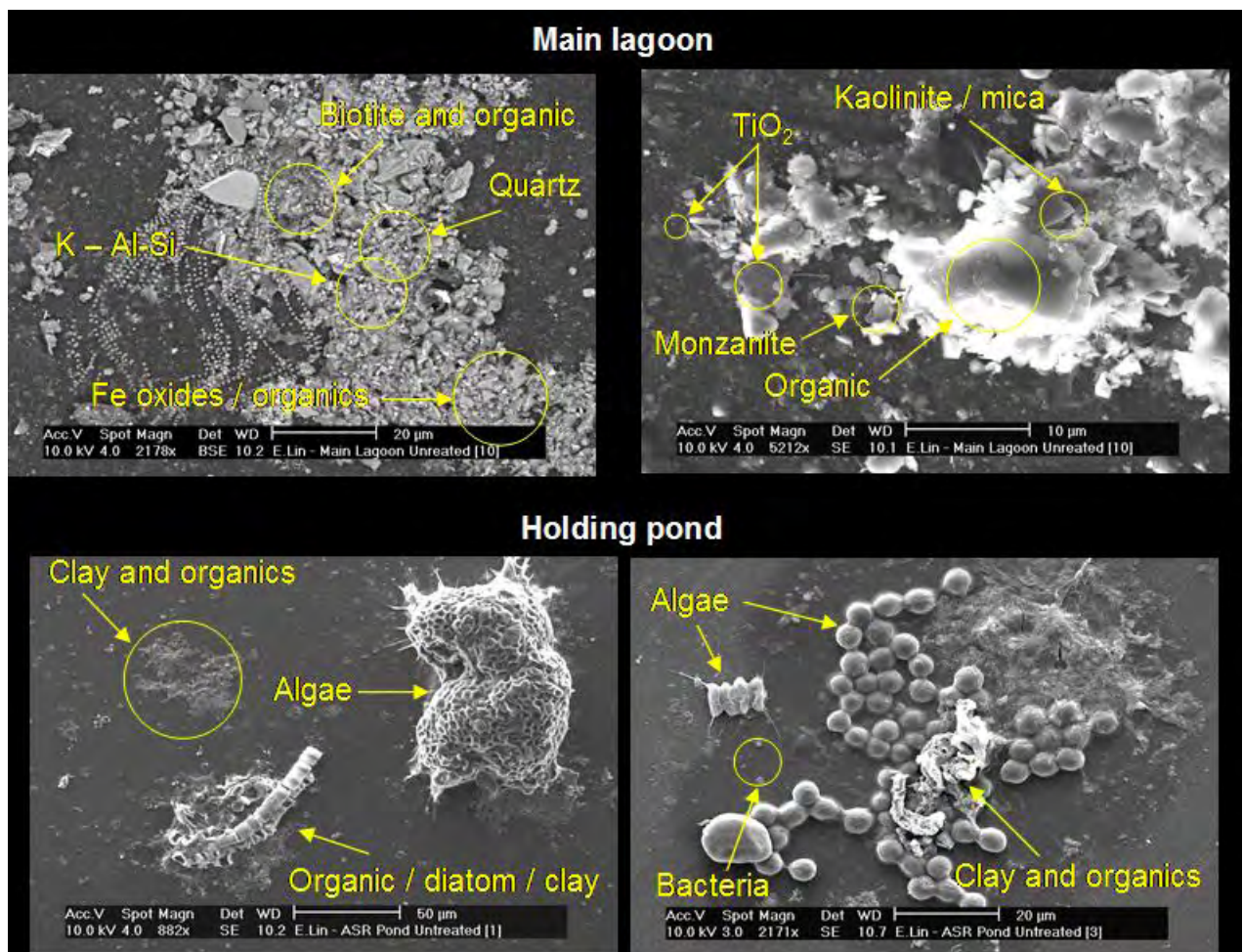
Figure 7 SEM micrographs showing typical particles in main lagoon during stormwater inflow on 19 Oct. 2005 (top row) and holding pond water from 17 Oct. 2005 (bottom row) (from Lin *et al,* 2005).

Injection phase of the trial

The trial was operated during the spring of 1999 and about 4 $\times 10^3$ m^3 was injected over a 6 week period. This amount was only one-fifth of the target volume of 20 $\times 10^3$ m^3 over winter. Initial injection rates of around 3 L/s were reduced to a final value 0.5 L/s. Injection was halted by 5 November 1999 due to the unacceptably low flow rate. The decline was noted to have occurred gradually over the injection period, although actual changes over time were not recorded. Unfortunately injection commenced approximately one month before the pump was installed in the well. Periodic backwashing of the well upon installation of the recovery system failed to stop the decline in injection rates. The small residual potentiometric head increase following injection indicated that the storage capacity of the aquifer was not a constraint. After modifying the headworks by installing the injection line, air entrainment in the injected water was eliminated, also removing this as a potential cause. In addition, there was also at least one input of engine oil from the stormwater catchment and black staining on the tank water level gauge indicate that traces of oil had breached the sand filter and entered the ASR well.

Rapid clogging occurred despite pre-treatment of the injectant by rapid sand filtration. Unfortunately injection was initiated approximately one month before the backwash/recovery pump was activated. During this period the particulate matter, which was abundant in stormwater, was subsequently found not to be substantially reduced by the rapid sand filter. Rather, the large, complex flocs evident in Figure 7 were broken-up into smaller flocs due to high shear stresses within the sand filter. Measures of particles/clogging parameters (turbidity, TSS, MFI etc) were reduced by 10-30%, but still remained high (Table 8). The sand filter, with an effective particle size of 0.95 mm and high uniformity, proved ineffective in removing an adequate proportion of particulates from the stormwater (note the uniformity coefficient, $d_{60}/d_{10} = 1.14$).

SEM imaging of the backwash water revealed that all of the particles were significantly smaller than the injected particles, with dimensions less than 5 to 10 μm (Figure 8). Lin *et al,* (2005) demonstrated that the passage of the Urrbrae Wetland water through a roughing filter pre-treatment system also reduced the effective size of particles in the treated water. The majority of particles evident in Figure 8 are not unlike those in the holding pond, although the particle assemblages are of a form that is less readily identified.

The potential causes of clogging included: suspended solids or hydrocarbons entering the well; biofilm production on the well screens and surrounding natural gravel pack; and remobilisation of drilling muds or fines from the aquifer. Chemical precipitation and gas binding by entrained or evolved gases from the injectant were eliminated. The sodium adsorption ratio of the injectant was lower than ambient groundwater and unlikely to disperse clays in the aquifer as a result of reducing groundwater salinity. The next step was to identify the most appropriate techniques for restoring the injectivity and for maintaining injection rates in the long term.

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Figure 8 SEM micrographs showing particles in backwash water from ASR well on 10 Dec. 1999

Table 2 Performance of the rapid sand filter during sampling on 8 Nov. 1999

Parameter	Pre-filter	Post-filter
MFI (s/L^2)	323	229
Turbidity (NTU)	6.4	5.2
TSS (mg/L)	10	9
d_{50} (μm)	127	104
VSS (mg/L)	10	9
TOC (mg/L)	6.6	6.3

Efforts to remediate the clogged ASR well

Over the period from December 1999 to November 2000 a series of activities involving various types of inspection methods and restoration approaches were undertaken with the aim of diagnosing the cause of the problem and remediating the ASR well. These were punctuated by a series of short, single- or multiple- step drawdown tests as a basis for assessing the change in hydraulic performance of the well. The well efficiency is defined here in terms of its specific capacity (at a specified time) as it is particularly sensitive to the well-loss component of drawdown. A summary of the main activities during the trial are given in Table 3. Details on the remediation activities are given below.

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Table 3 Inventory of major activities during the Urrbrae Wetland ASR trial

Date	Event
5 Aug. 1997	First downhole camera survey of the well
12 Aug. 1997	First pre-injection aquifer pump test (3-step, 370 mins.)
14 Jul. 1998	Second pre-injection aquifer pump test (1-step, 360 mins.)
9 Mar. 1999	Brief injection-recovery test (3-cycles, 360 mins.)
Spring, 1999	Start of injection (3 L/s)
5 Nov. 1999	Injection stopped due to >80% reduction in flow rate (4×10^3 m^3 injected)
9 Dec. 1999	First post-injection aquifer pump test (3-step, 38 mins.)
9-14 Dec. 1999	Intermittent backwashing of well
14 Dec. 1999	Second post-injection aquifer pump test (1-step, 14 mins.)
19-21 Jan. 2000	Injection of disinfection agent
1 Feb. 2000	Third post-injection aquifer pump test (3-step, 26 mins.)
8 Mar. 2000	Fourth post-injection aquifer pump test (1-step, 61 mins.)
14 Mar. 2000	Second downhole camera survey of the well
5-6 Apr. 2000	Surging of the upper screens and partial removal of sand accumulated around bottom screen
30 May 2000	Third downhole camera survey of the well
14 Jul. 2000	Downhole EM flowmeter survey of well under ambient and pumped conditions (fifth post-injection pump test, 1-step, 74 mins.)
3 Nov. 2000	Sixth post-injection aquifer pump test (3-step, 40 mins.)

Intermittent backwashing of ASR well

The first approach involved intermittent backwashing over a 5-day period (9-14 December 1999). Pumping events were scheduled on the hour for a duration of 3-5 minutes each at rates of 1.8-2.8 L/s. Aquifer pump tests were conducted before and after the backwashing to gauge the success of the approach.

As Figure 9a shows, the turbidity of the recovered water initially peaked at 3500 NTU and declined exponentially to a final value of 21 NTU after 22 m^3 had been pumped which was similar to the average injected concentration (Table 1). There was no visual evidence of oil residue in the backwash waters. Recurrent pumping and the demonstrated removal of at least some of the clogging agents from around the well failed to produce a measurable improvement in well efficiency. The specific capacity remained unchanged at 3-5 m^3/d/m, far lower than the pre-injection values of 11-13 m^3/d/m (Figure 10). The evidence seemed to suggest that only a small fraction of the most easily-dislodged clogging agents had been recovered from around the well-screens.

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Chlorination of ASR well followed by intermittent backwashing

As backwashing alone had proven ineffective, a slug of chlorine solution was introduced into the well to oxidize the organics prior to further backwashing. 34 m^3 of potable quality water was dosed with standard pool chlorine (calcium hypochlorite containing 65 % available chlorine) to an average concentration of almost 300 mg/L and injected into the well over a 2 day period (19-21 Jan. 2000). The chlorinated slug remained within the gravel pack of the aquifer for a further 6 days before 58 m^3 was pumped over a 4 day period (27-31 Jan. 2000). The aggressive character of the chlorinated water was confirmed by observed etching of the plastic coating of the pressure transducer that had been resident within the well. Dark, slimy material was clearly evident in March 2000 upon recovery of the pump column and the small-diameter UPVC access pipe for the transducer that had been placed within the ASR well in December 1999.

During pumping the initial peak in turbidity was 400 NTU and declined exponentially to reach a final value of 15 NTU (Figure 9b). Surprisingly, this peak value was almost an order of magnitude lower than the highest concentration measured in December 1999. Once again, pump testing on 1 Feb. and 8 Mar. 2000 revealed little or no improvement in the specific capacity of the well (Figure 10).

Pérez-Paricio and Carrera, (1999) notes the inadequacy of chlorine treatment in cases where the dissolved iron concentration is high owing to the high oxidizing power of the iron (levels of iron in this study were high as will later be shown). Repeated bursts of chlorination at higher concentrations than used here, followed by acidization to remove chemical precipitates often associated with the biofilm, may have been more successful (eg. Driscoll, 1986 recommends chlorine concentrations of 500-2000 mg/L).

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.
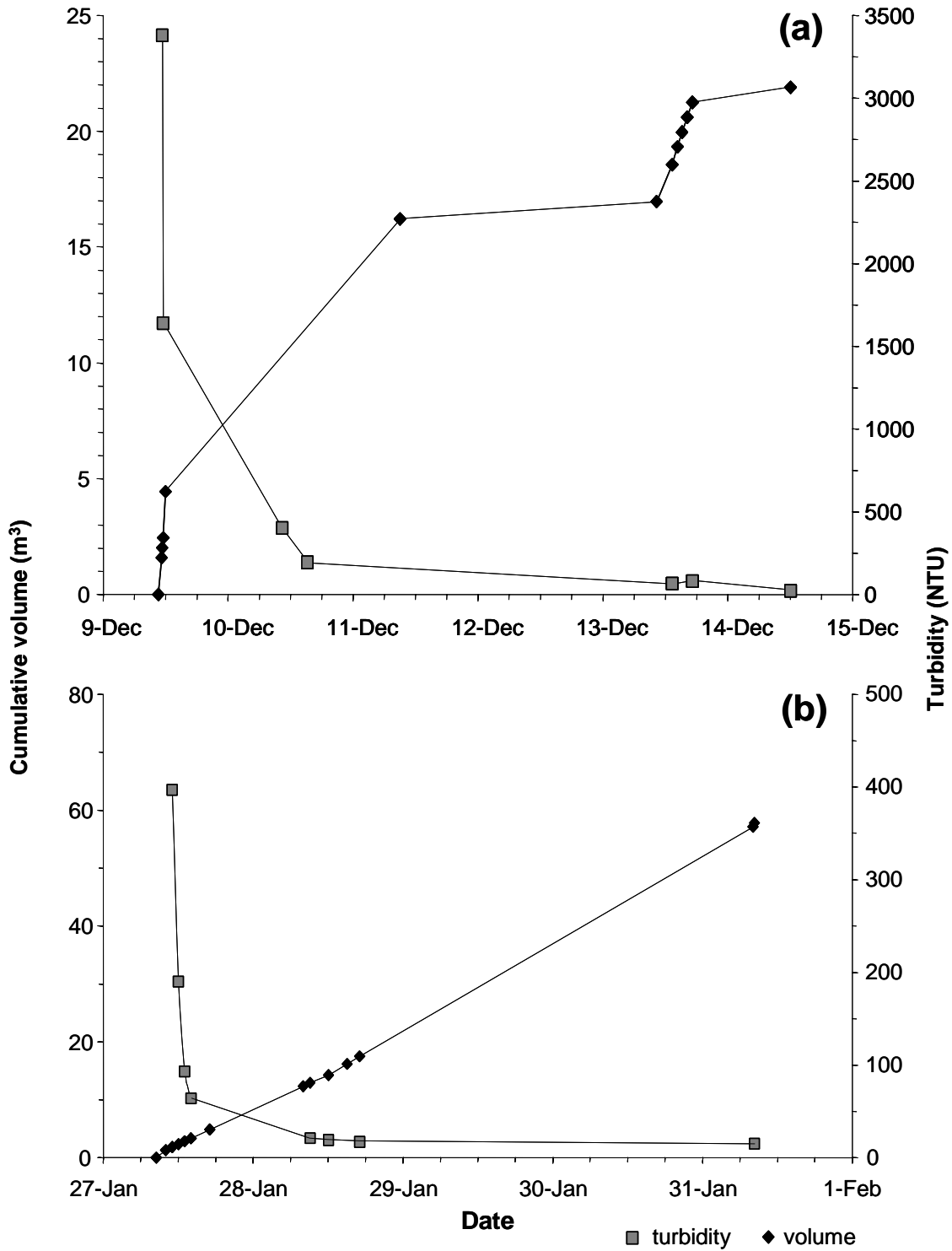
Figure 9 Changes in turbidity and the cumulative volume of water pumped during redevelopment events. The upper plot (a) is before rehabilitation in December 1999; the lower plot (b) is after well chlorination in January 2000.

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.
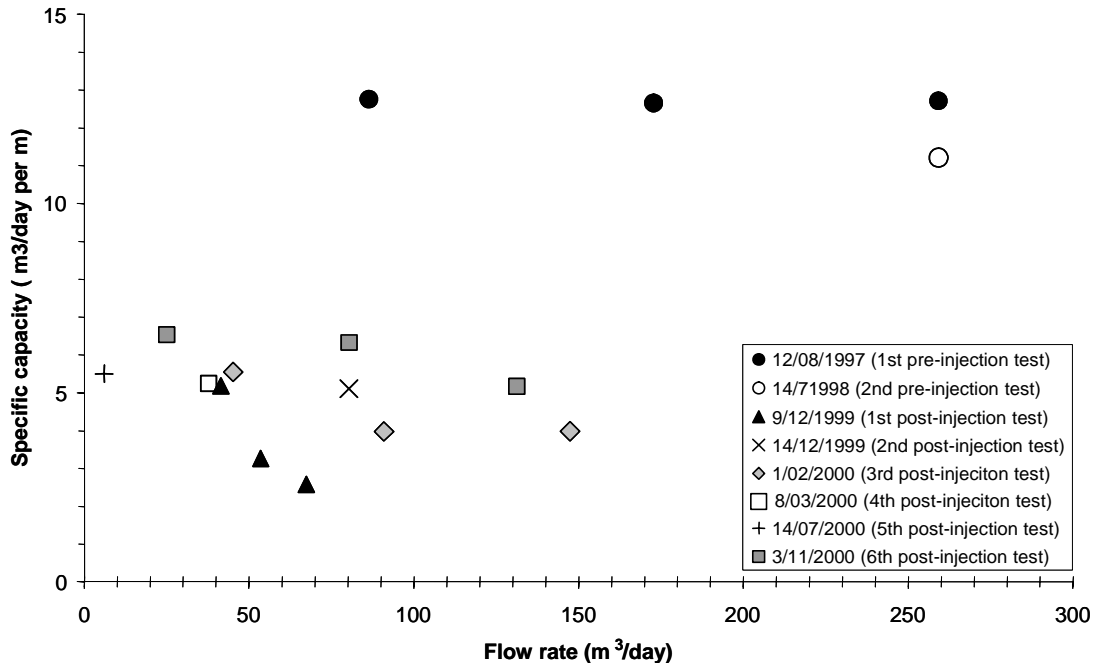
Figure 10 Plot of specific capacity (at t=10mins) versus pumping rate at different stages of the trial

Downhole camera surveys

A downhole video camera survey on 14 Mar. 2000 revealed heterogeneous discolorations on the well-screens symptomatic of persistent fouling of the screens. Such discolorations were not observed in the camera survey prior to injection (5 Aug. 1997). Rubbing of the centralizing arms of the camera on the walls of the casing and screens stripped some of the coating material that appeared to be composed of large dark coloured organic flocs, as had been seen on the pump column, and presumably the result of excessive microbial activity. There was no observed evidence of lignite protrusion through the screens.

The camera footage also showed that a metal fence post (also known as a 'star-dropper') had lodged on the casing shoe as a result of a vandalism incident at some point between August 1997 and March 2000.

The bottom of the hole was reached at a depth of 77.2 metres, or 7 m less than the actual base of the well, indicating that there had been significant in-filling of the well with sand. Although it was theoretically conceivable that the sand had entered the well via the screens, damage was noted to the rubber seal (the so called 'J-latch') set between the narrower 152 mm (6-inch) telescopic screen assembly and the wider 203 mm (8-inch). Evidence derived from flow metering (presented below) would reveal this had significantly exacerbated sand entry.

The first camera survey revealed that the screen assembly was positioned off-centre. It was considered that the long assembly had flexed under its own weight during installation when sitting on fill-material at the bottom of the hole. The misalignment between casing and screen had caused the J-latch to intrude which caused difficulty in

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lowering of the submersible pump beyond the top of the screen assembly (so as to gain additional drawdown and maximise pumping rate and encourage flow from the lowest screen).

Clearly the J-latch could have been damaged by repeated raising and lowering of the pump and/or the star-dropper incident. The detection of a 100 mm diameter plastic pipe buried under 3 m sand and recognized to be the pump shroud, reinforced the view that the J-latch had been stressed by the pump.

Bailing, airlifting and final camera survey

Attempts to remove the sediment from the base of the well through bailing and surging operations were thwarted by the pump shroud. The upper two screens were briefly surged with a rubber flange system and the bottom rebailed. Unfortunately the problem of sand ingress could not be overcome. The plan to later inject a clay dispersing agent into the well was cancelled due to the ingress.

The final video camera survey on 30 May 2000 revealed that the bailing cleared most of the sand apart from the bottom metre of screen (bottom of hole at 82.1 m). The screens were significantly cleaner than in March. The star dropper was not recovered.

Pump testing in July and November 2000 clearly showed that there was a slight improvement in well performance as compared to the situation in December 1999 situation, but still well below the initial conditions (Figure 10).

EM flowmeter survey

An electromagnetic (EM) flowmeter survey of the ASR well was conducted on 14 Jul. 2000 to determine the flow contributions from each of the three screened intervals. Positions immediately above the screens were selected and all of the flow through the cross-sectional area of the well channelled through the flowmeter using a circumferential rubber flange fitting.

The flowmeter survey was performed under pumped conditions, where the tested well was simultaneously pumped at a constant flow rate of 4.2 L/minute and the flow distribution determined after the drawdown had stabilized. Before any pumping had occurred the ambient rate of flow within the well was determined to assess the net differential flow. Details on the test procedure are given by Molz *et al,* (1994). Changes in flow rate between adjacent depths implied there was flow into or out of the well over that particular interval.

The survey revealed that the bulk of the flow contribution (58%) was derived from the perforated junction (Table 4). The remainder (42%) was derived from the screened intervals, with each interval contributing in approximate accordance with the screen length. The flowmeter data also revealed that the in-filling of the lowest screened interval had not been the cause of the substantial decline in injection rate.

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Table 4. Summary of flow meter survey results

Well zone	Percent contribution	Anticipated percent contribution [A]
Above screens [B]	58	0
Upper screen	18	52
Middle screen	11	26
Lower screen	13	22

[A] assuming uniform flow contribution per unit length of screen
[B] interval above 63.5m bgs and includes casing/screen junction

Water quality monitoring

All of the available water quality monitoring data apart from that already presented in Table 1 and Figure 6 is given in Table 5. This table provides information on ambient groundwater in the ASR well (for a limited suite of parameters), an indication of source water quality from the main lagoon (two seasons prior to injection), and groundwater from the ASR well during initial backwash redevelopment as well as prior to-, soon after- and in latter stages of- well chlorination. The following points can be drawn from the data:

- wetland water is significantly fresher than the marginally brackish ambient groundwater (by a factor of six in terms of the chloride concentration).

- marginally elevated groundwater EC with respect to the wetland water on 10 Dec. 99 and slightly higher again on 19 Jan. 01 suggest some residual ambient groundwater in backwash water, perhaps owing to incomplete flushing caused by aquifer heterogeneity exacerbated by the small volume of water injected or boundary effects from the multi-aquifer well completion.

- high iron content in the initial backwash water relative to the injectant (3-17 mg/L *cf.* 0.5 mg/L) and detection of low levels of heterotrophic iron bacteria in the groundwater is suggestive of the dissolution of iron-bearing minerals due to the injection of oxygenated injectant into a partially reduced groundwater (due to the absence of data on iron levels in ambient groundwater other mechanisms may also be possible).

- no active algal cells were observed in the backwash waters, eliminating the possibility of growth of algal species that rely on little or no light for their metabolism.

- the source water contains sufficient particulate matter to reduce pore-space of the media close to the well screens and sufficient organic matter and other key nutrients to promote biofilm production as was previously noted.

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Table 5 Composition of wetland water, ambient groundwater, and backwash water at various stages of the remediation program

Parameter	Units	Wetland (MLG) (8 Mar.97)	Ambient Groundwater (12 Aug.97)	Intermittent backwash (10 Dec.99)	Pre-chlorination (19 Jan.00)	Initial post-chlorine backwash (27 Jan.00)	Final post-chlorine backwash (31 Jan.00)
Suspended solids	mg/L			518	88	413	15
Turbidity	NTU		1.6	310	59	397	15
TDS	mg/L		1110		256	628	
Conductivity	µS/cm	330		421	458	1177	526
pH	-	6.39		7.63	6.98	7.14	7.08
Dissolved oxygen	mg/L			2.1[1]			
Alkalinity	mg/L			168	169	141	93
Bicarbonate	mg/L	46.8	511	205	206	172	114
Bromide	mg/L			0.13	0.15	0.34	0.21
Calcium	mg/L	25.2	41	44.5	44.3	55.8	31
Chloride	mg/L	63.47	383	32	40	243	91
Fluoride	mg/L			0.19	0.2	0.39	0.27
Magnesium	mg/L	6	48.2	12.3	10.3	23	9.4
Potassium	mg/L	6.2	7.1	5.3	4.3	7.6	5.5
Silica (reactive)	mg/L			8	10	8	7
Sodium	mg/L	29	317	36.2	31.7	126	60.2
Sulphate	mg/L	14.1	65.1	7.6	9.1	78.2	27.1
Arsenic (total)	mg/L						
Iron (total)	mg/L	0.5		17.1	5.5	7.45	2.91
Manganese (total)	mg/L	0.03		0.668	0.44	0.515	0.191
Phosphorus (total)	mg/L	0.1		0.443	0.472	0.272	0.285
TKN as N	mg/L			3.04	4.27	5.04	1.92
Ammonia as N	mg/L	<0.4		2.48	3.75	1.8	1.6
Nitrate + nitrite as N	mg/L	<0.4		0.026	0.014	0.039	0.032
Dissolved organic carbon	mg/L	17		8	8	17.6	4.2
Total organic carbon	mg/L			10	8	18.6	4.5
Biochemical oxygen demand	mg/L				13	22	5
Chemical oxygen demand	mg/L				26	71	25
Algae (total)	cells/mL				ND[3]	ND[3]	ND[3]
Total heterotrophic count (20°C)	cells/mL			18000	500−5000[2]	50000	515
Total coliforms	cells/50mL				0	0	0
E. coli	cells/100mL			<10			
Heterotrophic iron bacteria	cells/mL			900			
Pseudomonas spp.	cells/100mL			1800	40	D[4]	0

[1] from sampling on 14 Dec. 99; [2] value lower than expected, may be due to use of non-optimal medium and/or minor contamination of well with chlorinated water (well was purged and pumped dry after this event before sampling); [3] ND = not detected; [4] D = detected

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8 Water quality contrasts with two other ASR systems

Experience drawn from other case studies over long periods of time have shown that higher levels of pre-treatment than was provided to the recharge water at Urrbrae is required to avoid excessive well clogging problems. A review of the literature reveals that in The Netherlands, with aquifers of similar mineralogical characteristics, sites operate using source waters treated to a level such that the MFI value is $<$(3-5) s/L^2 and AOC is $<$10 μg/L, even though aquifer transmissivity can be up to two orders of magnitude higher than at Urrbrae (Table 6). While the quality of water required to avoid clogging will depend on the aquifer, it is thought that The Netherlands experience sets a target for sustainable operations in low to moderate transmissivity silicious aquifers. These parameter values are significantly lower than the values that were anticipated to have been injected at Urrbrae. The poor quality of source water is considered to be largely responsible for the failure of the Urrbrae trial.

Corresponding water quality values for the Bolivar ASR site are considerably higher than in the Netherlands (and largely within the range measured at Urrbrae) owing to the higher aquifer transmissivity and calcite content of the aquifer, which serves to offset physical and microbial clogging if injectant is undersaturated in calcite. It is interesting to note that, from a clogging viewpoint at least, the Urrbrae water may have been acceptable for injection at Bolivar. This convincingly illustrates the point that water quality criteria cannot be considered in isolation, but must also consider the nature of the receiving formation.

Table 6 The quality of recharge water at Urrbrae compared with two contrasting case studies where comprehensive investigations have shown the viability of ASR

Parameter	Urrbrae stormwater ASR [A]	Bolivar reclaimed water ASR [B]	treated River Rhine water ASR [C]
Target aquifer type:	*siliceous*	*carbonaceous*	*siliceous*
Transmissivity (m^2/day):	*6*	*150*	*80-1800*
MFI (s/L^2)	90-389	<100	<(3-5)
Turbidity (NTU)	0.8-55	<3	very low
AOC/BRP (μg/L)	39-331 (as BRP)	1000 (as BRP)	<10 (as AOC)
Total Nitrogen (mg/L)	0.9 [D]	<10	very low

[A] this study' [B] reported in Pavelic *et al*, (in prep.); [C] from Olsthoorn, (1982) and Hijnen and van der Kooij, (1992); [D] from Lin *et al*, (2005) since data in **Tab. XX.5** does not include the organic-N component

Hypotheses for failure

The reason why the ASR well performance at Urrbrae was not restored significantly by the three different mechanical techniques is interesting when one considers that these techniques have consistently been successful in a variety of other studies (eg. Olsthoorn, 1982; Pyne, 1995; Pérez-Paricio and Carrera, 1999; Pavelic *et al*, in prep.).

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Whilst we have noted that there was room for improvement in some of the approaches (eg. through the use of acid following chlorine), the literature would suggest that the outcome should have been far more successful if the deterioration of the well was due to just particle filtration and biofilm growth.

Therefore, greater emphasis must be given to the prospect that residual drilling mud had invaded the formation or that fine textured aquifer particles had been mobilized and redeposited. Clearly there is inadequate direct evidence to confirm this and the proposition is rendered likely largely by the elimination of other potential causative factors.

It was possible that on initial redevelopment of the well only a small fraction of the mud cake was removed, and injection caused blockage of the remaining unclogged parts of the formation. Segalen *et al,* (2005) offers evidence that the choice of drilling technique, the quality of the drilling, well completion and design, have a very significant effect on the performance of ASR wells in unconsolidated aquifers.

Clay release due to the injection of low salinity water can result in rapid declines in permeability in brackish aquifers that contain reactive clays minerals. Since recharge causes divalent cations to substitute with monovalent cations, preconditioning the aquifer by initial flushing with $CaCl_2$ has had some success in alleviating clay dispersion (eg. Brown and Silvey, 1973). In addition, purely physical forces can mobilize fines. The propensity of the aquifer to erode and redeposit fine particles within pore throats is dependent on the pore water velocity, and on the grain size distribution and pore geometry of the aquifer (Nakai, 2006). This issue cannot be adequately resolved since the physical and mineralogical characteristics of the target zones are largely unknown.

Summary of lessons learnt and current research

This attempt to recharge passively-treated urban stormwater via a multi-completion ASR well that targeted two confined, unconsolidated silicious aquifers at the Urrbrae Wetland site in the late 1990s, resulted in a significant decline in injection rates and the cessation of operations within the first year of operation. It was not surprising that some degree of clogging had occurred given the characteristics of the recharge water. Clogging was initially attributed to the high levels of suspended solids and bacterial growth fed by labile organic carbon and other nutrients in the wetland water. Mechanical and/or geochemical effects due to residual drilling muds or the mobilization and redepositing of aquifer fines possibly have an impact on clogging, however, this is extremely difficult to verify in practice. Further, a perforation of the well screen joint caused infilling of the screens with sand and reduced the effectiveness of procedures to unclog the ASR well.

Resolving the cause of clogging was initially considered a normal part of the ASR commissioning process. Three different attempts were made to restore the clogged ASR well. They included: repetitive backwashing; injection of chlorine disinfectant and backwashing; and bailing/surging to recover sand that had in-filled the lowest screened interval. These techniques proved to be ineffective in restoring the performance of the well. Restoration would ultimately require that the screen assembly be recovered and replaced, and the gravel pack re-established. Because the cost of retrofitting the well

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was similar to the cost of a new well, this was considerably non-viable in the absence of new funding sources.

The fundamental problem at Urrbrae was that the level of pre-treatment given to the recharge water was inadequate for the low transmissivity aquifer targeted, irrespective of the lack of success in restoring the injectivity of the ASR well. This was exacerbated by the absence of a nearby observation well and monitoring data during the injection phase of the trial. Both elements were initially intended but omitted due to budgetary constraints. We consider that a more focussed well restoration program would have ensued had this baseline information been collected. At leat one nearby monitoring well is recommended in all situations where clogging is a potential problem. A confounding problem was premature injection of water before a pump was installed to allow redevelopment. This probably resulted in filter cake compression and made subsequent redevelopment much more difficult. Infilling of the well with sand was another confounding problem.

As a result of this experience, it is concluded that fine-grained unconsolidated aquifers are unacceptable targets for operational ASR systems with wetland-treated urban stormwater until further research is conducted to ensure sustainable injection.

Several research projects have commenced to address improving the design of ASR wells (Segalen *et al*, 2005; Pavelic *et al*, 2006) and on methods of pre-treatment including the use of roughing filtration (Lin *et al*, 2005) and biofiltration (Page *et al*, in prep.). The studies on pre-treatment are aimed at removing colloidal matter and key bio-available nutrients from the recharge water. Methods have been selected for their simplicity, low cost and potentially low maintenance requirements, making them suitable for use in developing countries and in Australia for urban stormwater harvesting. This current research draws heavily from the lessons learnt from the Urrbrae ASR trial of 1997-2000.

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AWWARF Case Study, Northwest Hillsborough ASR Project, Hillsborough County, Florida

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Prepared for: ASR Systems LLC Team and the AWWA Research Foundation

April 2006

Introduction

The Northwest Hillsborough County Reclaimed water ASR project is located in southwest Florida in Hillsborough County near the City of Tampa, Florida (Ellison, 2006). The ASR project is administered by Hillsborough County Water Department and the Southwest Florida Water Management District (SWFWMD) and is specifically located at the Hillsborough Northwest Dechlorination Facility (NWDF), Hillsborough County. The initial plan for the project was to store reclaimed wastewater, that had been treated to tertiary levels, within the brackish Floridan Aquifer System (FAS). It was hoped that the ASR project would provide a dependable and cost effective irrigation supply during periods of peak demand for Hillsborough County. Currently, the Hillsborough County reclaimed water system provides water to approximately 8,000 irrigation users (NW Hillsborough Basin Board, 2004). Additional users have asked to join the system. Therefore, the proposed ASR system would store excess reclaimed water in FAS when water was available and recover the stored water for use during high demand periods. The facility was originally envisioned to be able to supply up to three months of reclaimed water supply that meets irrigation water requirements during seasonal dry periods. The ASR system was designed so that the reclaimed supply would better match the reclaimed water demand (CH2M Hill, 1998). Without the proposed ASR system it was estimated that only 60 percent of the available reclaimed water supply could be utilized (CH2M Hill, 1998).

The criteria for sizing the facilities was based upon historical demand records for the reclaimed water program. A series of cycle tests were conducted in support of the project but were not as successful as hoped. Therefore, the project has been deferred to the future or until further analysis reveals the possible reasons for project failure.

The ASR well system consists of one Class V, Group 3 injection well (TPW-1), three on site monitor wells (SZMW-1, SMW-1 and WTMW-1), and two off site monitor wells (14-D and 14-S) located within 800 feet southwest of the NWDF. The latitude and longitude coordinates of the ASR well are 28° 01' 29" North and 82° 36' 34" West. The ASR well was completed into the Suwannee Limestone of the Upper Floridan Aquifer System (UFAS) for the storage and recovery of public access reclaimed water. Cycle testing of the ASR system occurred between July 11, 2001 and May 27, 2003 (FDEP, 2006).

This case study report is based upon existing project literature and results of interviews and discussions conducted with persons involved with the project (e.g., SWFWMD, FDEP, and CH2M Hill) over a two month period in March-April 2006. Persons consulted about the project include:
- Mark McNeal, formerly of CH2M Hill (Interview pending)
- Don Ellison, SWFWMD
- Kathie Costello, FDEP
- Judy Richtar, FDEP
- Dr. Jon Arthur, FGS

The discussions focused upon the site history, operating practices, engineering data, water quality issues, operational problems, and lessons learned. Results of the literature research and interviews are discussed throughout this report and referenced as "personal communications" with persons listed above.

Site Setting - Hydrogeologic Framework

Three major aquifer systems have been identified in South Florida; the Surficial Aquifer System (SAS), the Intermediate Aquifer System (IAS) and the Floridan Aquifer System (FAS), (Southeastern Geological Society, 1986). Within the Intermediate Confining Unit is the IAS, as well as a few other aquifers of limited transmissivity and extent. The IAS is located in southwestern Florida, and consists of beds of sand, sandy limestone, limestone, and dolostone of Oligocene to Pliocene age that dip and thicken to the south and southwest. These aquifers are typically leaky artesian with moderate transmissivity and are located primarily in Polk, Sarasota, Highlands, Hardee and DeSoto Counties. The IAS includes the Tampa Member, the Arcadia Formation, and the Peace River Formation, all of the Hawthorn Group.

The FAS is the primary source of water supply for the northern counties of Florida, and is used as a source of supplemental irrigation water as far south as Martin County. The system receives direct recharge along a structural high, known as the "Ocala High", which occurs in the west-central part of the State. From this area, the FAS dips southward where it is overlain by clays and silts of the Hawthorn Group, which form a confining layer over the FAS. The potentiometric surface is also highest in the central portion of the State and decreases radially to the south, east and west (Berndt et al., 1998). Potentiometric heads in the vicinity of the ASR project ranged from 5 to 20 feet NGVD (SFWMD online data, 2006). Within the study area, heads appear to be depressed due to regional aquifer drawdown due to large water supply wells. Online SWFWMD data for three regional well clusters screened in the major aquifers support this contention. Table 1 displays water level data downloaded on April 10 and 11, 2006.

Table 1 Regional water level data (source: SWFWMD website)

Well Cluster Name	Aquifer Monitored	Water Level (NGVD 29)
Romp 97 (in north Pinellas County)	Surficial	17.15
Romp 97	FAS (Avon Park)	17.05
Romp 14-1 (NE Pinellas County)	Surficial	7.75
Romp 14-1	UFAS	6.76
Romp 66 (Hillsborough County)	Surficial	16.91
Romp 66	Tampa Limestone of IAS	16.85
Romp DV-1 (SE Hillsborough County)	Surficial	108.39
Romp DV-1	UFAS	47.06
Romp DV-1	FAS (Avon Park)	47.45

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The Suwannee Limestone member of the UFAS is interpreted to be present in the project area and continuing in a northwest-southeast trend through the northeast portion of Glades County and trending diagonally through the west and central part of Palm Beach County (Miller, 1986). The depth to the top of the Suwannee range from approximately 550 to greater than 900 feet bls. The dominant lithology of this unit in the project area is pale-orange to tan, fossiliferous, medium-grained calcarenite with moderate amounts of quartz sand and little or no phosphatic mineral grains (Reese and Memberg, 2000). At the NW Hillsborough site, the Suwannee Limestone is encountered at approximately 175 feet bls and the ASR storage zone was selected from 305 to 415 feet bls (FDEP, 2006). Fluid resistivity logging completing during ASR well completion revealed the possible presence of a significant fracture (CH2M Hill, 1998) just below the storage interval.

Lying below the Hawthorn Group and Suwannee Limestone sediments at depths of approximately 600 to 1,200 feet bls are the Eocene-aged Ocala Limestone, Avon Park Formation, and Oldsmar Formation. These formations are characterized by pale orange to brown, poorly-cemented, granular limestone, and are occasionally micritic. At the NW Hillsborough site, the Ocala Limestone begins at 445 bls and continues for several hundred feet. The Avon Park Formation lies beneath the Ocala Limestone and is typically recognized as white to cream foraminiferal limestone, with the majority of foraminifers being *Dictyconus* (Miller, 1997). The Formation is also comprised of dark brown to tan crystalline to saccharoidal dolomites. These formations are present to a depth of approximately 1,900 feet bls. Lying below the Ocala Limestone and Avon Park Formations is the Oldsmar Formation. This formation contains significant quantities of hard, yellowish brown finely crystalline dolomite. In some places it is termed the "boulder zone" due to its cavernous nature and propensity to generate cobbles and boulders during drilling operations. Anyhdrite is also found to be present in some local areas. These formations are present to depths of nearly 3,500 feet bls.

The average transmissivity of the confined Suwannee zone is 5,000 to 30,000 ft^2/day with an average storage coefficient of 1.0×10^{-4} (Bush and Johnston, 1988). At the Peace River Site in nearby Manatee County, the Tampa Limestone zone transmissivity was estimated to be approximately 4,000 ft^2/day with a storage coefficient of 8.0×10^{-5} and the deeper Avon Park may have a transmissivity of 150,000 ft^2/day and a storage coefficient of 1×10^{-3} (CH2M Hill, 1985).

Source Water Characteristics

A substantial amount of recharge water quality data was collected during the initial cycle testing of the ASR well. Data collection included many parameters such as pH, turbidity, temperature, conductivity, chlorine residual, total suspended solids (TSS), total dissolved solids (TDS), chloride, fluoride, sulfate, various metals, total trihalomethanes (TTHMs), and alkalinity (CH2M Hill, 2001). The average influent TDS, chloride, and sulfate values were 620 mg/l, 160 mg/l, and 71 mg/l, respectively. Low metal concentrations were also recorded in the recharge water including 15 mg/l of potassium, 8.8 mg/l of magnesium, 0.8 ug/l of arsenic, and 50 ug/l of iron (CH2M Hill,

2001). Continuous conductivity measurements revealed that the recharge water was somewhat variable during the cycle test # 1 activities and varied from 870 to 990 umhos/cm. The TTHM mean value was 103 ug/l over cycle 1. The recharge flow rate ranged from 600 to 1,300 gpm. The mean value over the 22-day recharge period of cycle 1 was 960 gpm or 1.4 MGD. During recharge, gradual plugging was noted by increases in wellhead pressure from 30 to 56 psi along with declining recharge rates (CH2M Hill, 2001). As TSS values of the recharge water were measured at less than 0.5 mg/l, the apparent plugging may be due to a combination of hydraulic effects, suspended solids mass loading, and possibly bacterial growth.

Water Quality

Cycle testing originally began in 2001 using well 1 completed in the Suwannee Limestone. Table 2 provides a summary of groundwater quality observed at the site at the beginning of the cycle testing program. Table 3 summarizes data from the first two ASR recharge and recovery cycles. The recovery efficiency (RE) percentage is defined by a TDS regulatory standard of 500 mg/l versus other brackish water sites that use chloride as a regulatory standard. Using a reuse irrigation standard of 1,000 mg/l TDS, the RE would be approximately 25%, although it is impossible to know the exact value due to failure of the onsite flowmeter during the cycle testing (CH2M Hill, 2001). The pilot testing was not very successful and indicated that the potential RE would be limited for some time. Subsequent cycle testing in cycle # 2 showed an improvement of RE to 28% using the alternate irrigation TDS standard. Additional cycles would be expected to improve the RE more significantly, especially if a larger freshwater buffer was left in place.

Table 2 Groundwater quality at three onsite ASR wells.

Parameter	TPW-1 (245 ft bls)	TPW-1 (340 ft bls)	TPW-1 (460 ft bls)
Alkalinity (mg/l as CaCO$_3$)	206	1266	208
Chloride (mg/l)	139	140	342
Conductivity (umhos/cm)	765	763	1284
Hardness (mg/l as CaCO$_3$)	243	287	392
Ammonia (mg/l)	0.47	0.59	0.64
Fluoride (mg/l)	0.20	0.21	0.40
Sulfate (mg/l)	14.4	24.6	56.2
Total Dissolved Solids (TDS mg/l)	507	509	1008

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Table 3 Data from initial cycle testing at NW Hillsborough ASR site

Cycle	1	2
Volume In (Mgallons)	30	90
Storage Time (days)	4	NA
Volume out (Mgallons)	29.20	NA
RE (%)	0	28
Qin (gpm)	600 to 1,300	600 to 1,300
Qout (gpm)	1,100 to 1,217	1,100 to 1,217

Data recorded during the cycle 1 recovery period indicate that upconing of more brackish native groundwater led to increased salinity concentrations at the end of the recovery period. TDS increased from 650 mg/l to 3,250 mg/l; Chloride increased from 171 mg/l to 1,616 mg/l; sulfate increased from 81 mg/l to 283 mg/l; and sodium increased from 123 mg/l to 869 mg/l. Based upon data available in Table 2, the final concentrations of these parameters are well above those measured in the recharge water or ASR storage zone, suggesting deeper salinity sources. In addition to the salinity parameters, arsenic increased from 6 ug/l to 23 ug/l at the end of the first week of recovery and then decreased slightly to 17 ug/l during the second week of testing. The arsenic concentrations were well above those recorded in the source water or the native groundwater indicating some type of water-rock reaction was ongoing. Magnesium increased from 9 mg/l to 31 mg/l, well above concentrations measured in the source water or ambient groundwater, also indicating some type of water-rock reaction.

Feasibility and Design Studies

The NW Hillsborough facility was designed and constructed in phases allowing for efficient use of limited capital. Typically, design of the wells has been based upon aquifer performance step-test data, numerical modeling, and initial pilot testing. The instrumentation and control of the whole well system is a mostly automated system design for the ASR well. In general, the facility is small in scale and is co-located with the NW Dechlorination facility. Since the facility is currently inoperative, security measures at the site are a reflection of security for the dechlorination facility including fencing and limited security monitoring.

Performance Evaluation

The NW Hillsborough ASR system posted a poor performance record. Over a 2-year cycle testing period, the ASR system stored approximately 120 million gallons of reclaimed water and recovered approximately 60 million gallons consisting of a mix of reclaimed water and native groundwater. This represents a net recovery utilization of approximately 50%. At least half of this recovered water had TDS concentrations of

greater than 1,000 mg/l meaning that it would not be very useful as an irrigation source. Therefore, a net recovery utilization of water meeting regulatory standards was approximately 25%. In addition, arsenic, magnesium, and sodium concentrations in the recovered water were elevated, further reducing its applicability for irrigation use. Sodium is especially problematic for many grasses and plants due to ion toxicity. Well clogging has been a moderate problem at the project although small decreases in specific capacity have been reversed through periodic well recovery or backflushing (CH2M Hill, 2001).

The ASR recovery water quality data supports the notion that upconing of more saline water from deeper zones of the FAS contributed to poor recovery efficiency at the site. Regional water levels reveal that artificially low heads resulting from regional pumping may also have contributed to the problem. Indeed, the water levels recorded at observation wells at the site monitoring different aquifer zones revealed that head differences are very small, indicating a well connected system that is not entirely isolated. A monitoring well was not installed below the ASR storage zone. Such a well would probably have shown that the Suwannee portion of the FAS and the Ocala portion of the FAS were in direct communication.

An economic evaluation of the system was not completed for the project but based upon the observed performance, the system economics may not have been favorable. On the other hand, storage of reclaimed water is an environmentally sustainable use of water so that even if RE values remained below 50%, the ASR program may be preferred over other surface storage options.

Problems Identified, Lessons Learned and Recommended Operational Practices

Several lessons learned can be gleaned from the NW Hillsborough ASR project. First, incomplete characterization of the hydrogeologic system can lead to incorrect assumptions regarding the confinement above and below the ASR storage zone. Second, inadequate storage buffers in the ASR storage zone will undoubtedly result in poorer recovery efficiencies. Third, upconing of more saline water from beneath the ASR storage zone can certainly result in infeasible ASR projects. Since upconing is a physical phenomenon that is dependent upon pumping rates and drawdown, reducing ASR recovery rates will reduce the magnitude of upconing, possibly resulting in improved system performance.

The NW Hillsborough project is currently inactive and may never be re-activated. The site may provide an effective scientific study site to evaluate upconing mitigation design, geochemical water-rock reactions, and operational optimization. Further studies at this site might provide further opportunities for improving ASR design and operational schemes. In addition, as the ASR technology itself matures, the re-activation of the NW Hillsborough ASR project might be feasible.

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AR Case Study
Artificial Recharge of Treated Wastewater
East Meadow, New York

Introduction

Groundwater derived from precipitation is the sole source of drinking water for residents in Nassau and Suffolk County, Long Island, New York (Figure 1). During the Long Island's early (1960s) period of development, drinking water and industrial waste water was returned to the aquifer with cesspools, septic tanks, and leach fields. With the explosive population growth in Nassau County, this practice caused widespread degradation of the chemical quality of shallow groundwater. To prevent the continuation of water quality degradation, centralized sewer systems were installed in densely populated areas. The water collected by the sewers was conveyed to wastewater treatment plants and ultimately discharge to the Atlantic Ocean. Although this process protects the groundwater quality in Nassau County, it increased the consumptive use and decreased the amount of groundwater available in the aquifer causing groundwater levels to decline and stream flow to decrease or cease.

Artificial recharge of storm water to the shallow aquifer has been successfully practiced in Nassau County since the 1930s. This artificial recharge program returns an estimated average of 60 mgd annually. Given the success of this program, the artificial recharge of treated effluent was evaluated at East Meadow, New York using both recharge basins and injection wells to determine if tertiary-treated wastewater could also be recharged to the aquifer without affecting the physical and chemical characteristics of the aquifer.
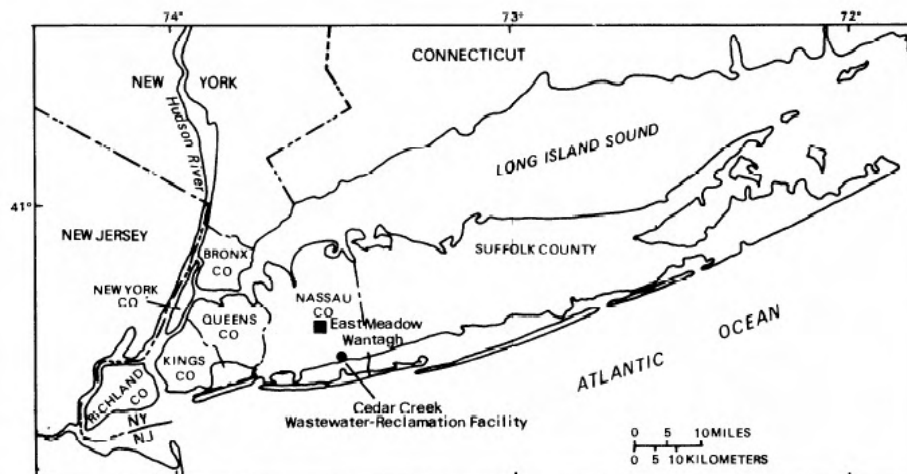

Figure 1: Location of Major Geographic Features of Long Island.

Design and Construction of Facility

Recharge Basins

The Nassau County Department of Public Works (NCDPW) augmented a site in East Meadow, New York that contained four existing recharge basins, to include seven new recharge basins, five new shallow injection wells, a process control building, and monitoring wells and instrumentation (Figure 2). Water from the Cedar Creek wasterwater treatment plant located in Wantagh, New York was pumped 6.25 miles through a 24-inch diameter pipe to 38,000 gallon reservoir at the East Meadow site. Water from the reservoir was conveyed by gravity through a 16-inch diameter pipe to the process control building and the water flow by gravity to the recharge basins and was pumped into the injection wells.
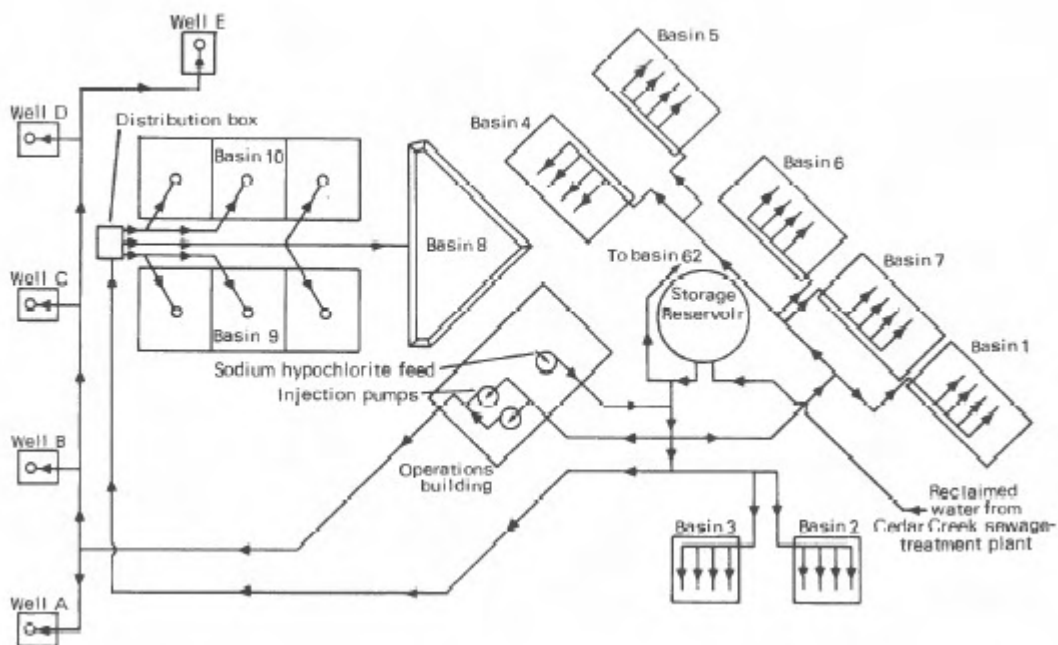
Figure 2: Location of Recharge Basins and Injection Wells.

Seven 5-foot deep recharge basins were constructed at the site each with a floor area of 5,000 ft² (50 feet x 1,000 feet). The basins are large enough to receive 0.5 million gallons of water per day. Five of the basins (Nos. 1, 4, 5, 6, and 7) were constructed with sloped walls to provide greater storage area; however, the walls were lined with an impermeable layer (Hypalon®) to make sure the water infiltrated through the basin floors. Two other basins (Nos. 2 and 3) have vertical concrete walls and contain an observation manhole in the basin floor to monitor infiltrated water. Four originally existing basins (Nos. 8, 9, 10, and 62) were available at the site in case clogging limits the infiltration capacity of the first seven basins. One basin (No. 8) is available for deep-ponding experiments. This basin is 15 feet deep and has a floor area of 3,213 ft² and a total area of 17,322 ft². This basin was used later in the study to evaluate the relationship between ponding depth and infiltration rates. Basins 9 and 10 are shallow basins that were formerly used for ponding effluent from a former secondary sewage treatment plant that was shut down in 1979. These basins

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were primarily used for the containment of water that exceeded the capacity of basins 1 through 8.   The remaining basin (No. 62) provided emergency storage in case one or more basins needs to be bypassed for maintenance. When no basins with sufficient infiltration capacity were available, flow to the recharge facility was reduced by throttling back the main pumps.

The process control building contains recharge pumps, a laboratory for water quality testing and a control room containing instrumentation that activated and monitored flow control systems, water levels, flow rates, water quality sampling, and meteorological conditions. Instrumented manholes were installed in two of the recharge basins (Nos. 2 and 3) to provide detailed monitoring of the physical and chemical condition of the water as it percolates through the unsaturated zone (Figure 3).
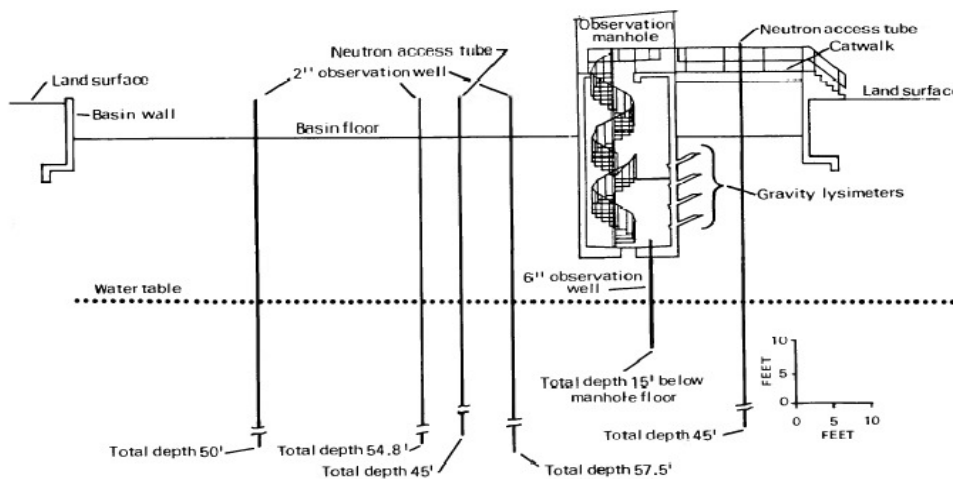
Figure 3: Cross-Section through Basin No. 3 Showing the Location of Observation Wells, Neutron Access Holes, and Lysimeters.

Water samples were collected at several depths within the manholes to study the chemical and physical effects of the unsaturated zone on the water. Four inclined lysimeters were installed through the wall of each manhole at depths 2.5, 5.3, 8.2, and 11 feet below the basin floor to capture water samples. Water samples of the ponded water were collected using a refrigerator composite sampler for laboratory analysis. Fourteen horizontal water manometers were installed through the walls of each manhole at varying depths to measure soil-moisture tension in the upper 14 feet of the unsaturated zone. Soil gas samplers were installed at varying depths to measure the oxygen content of the soil gas during recharge. Soil samples were collected from the unsaturated zone for physical and chemical analysis. Soil temperature measurements were measured by thermocouples at selected depths. Eleven galvanized steel neutron-access tubes extending 45 feet deep were installed at selected distances in and near the two basins (Nos. 2 and 3) to measure soil moisture. All of the data collected was ultimately stored on a computer or a magnetic tape unit.

Injection Wells

Artificial recharge of reclaimed water was also conducted using five 12-inch diameter injection wells that tap the basal portions of the Upper Glacial aquifer. Up to 350 gpm of reclaimed water was injected into four of the five wells. The fifth well

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remained idle and was only used if needed to maintain the 2 mgd flow rate. Three types of wells were constructed; one well was installed with a gravel packed screen, one has a natural pack screen, and one has both a gravel pack screen and a redevelopment system. The redevelopment system includes the installation of an eductor pipe and an air-line through which compressed air was injected to provide an air lift pumping during well development. The three types of well construction allowed the comparison of the effectiveness of the gravel pack well to the natural pack well with the hypothesis that even though a natural pack well is less costly the well may have an overall higher cost because the well is more prone to well clogging at the well screen and therefore requires more rehabilitation. Whereas, the gravel pack screen will have a larger effective radius and therefore provide a larger zone to distribute the clogging material. Each injection well was constructed with 65 feet of fiberglass reinforced epoxy casing and 30 feet of stainless-steel wire-wrap screen that extended from 65 to 95 feet below land surface. A sand trap or sump was attached to the bottom of the screen on four of the five wells.
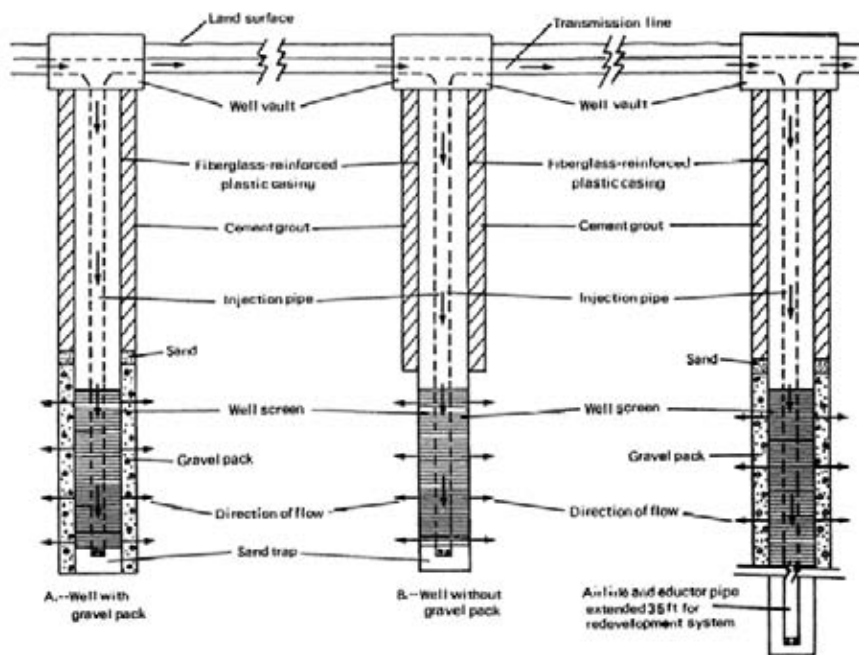
Figure 3: Three Types of Injection Well Design; Gravel Pack, Natural Pack, and Gravel Pack with Rehabilitation System.

Water was injected into the wells with a pump and the flow rate was measured with a venture flow meter. The desired flow rate was set on the flow controller, which regulates the position of the butterfly valve in the water line. The injection well was equipped with a pressure transducer that measures the water level in the well. Water was injected into the well until the water level reached 25 feet above the static level (10 lb/in²). At this point the well is shut off and redeveloped. An inline water analyzer, which allows continuous monitoring of total chlorine residual, turbidity, temperature, specific conductance, dissolved oxygen, and pH was installed inline and designed to stop the injection of water if water with excess turbidity enters the well.

Forty-seven 6-inch diameter monitoring wells were installed in well clusters at 23 locations (23 screened between 45 and 55 feet, 16 screened between 95 to 100 feet, and 8 screened between 195 to 200 feet). A 2-inch diameter monitoring well was installed at 19 sites with screens ranging from 9.3 to 64.1 feet below land surface. All 6-inch diameter monitoring wells were constructed 5 feet of stainless steel wire-wrap screen and sufficient fiberglass casing to reach land surface. Water samples were collected periodically and analyzed for inorganics, metals, and the Target Compound List (TCL) of organic compounds.

Physical Setting and Baseline Conditions

The Upper Glacial aquifer consists of 50 to 100 feet of medium to coarse quartz sand with minor amounts of interstitial silt and clay. The water table is approximately 27 feet below land surface and the hydraulic gradient is 0.0005. The horizontal and vertical hydraulic conductivity of the Upper Glacial aquifer is 390 and 160 feet per day. The specific yield of the aquifer is 0.2. Underlying the Upper Glacial Aquifer is over 800 feet of fine to medium sand comprising the Magothy aquifer.

The baseline ambient groundwater quality beneath the site was characterized by collecting over 200 groundwater samples from 47 observation wells. The groundwater samples were analyzed for inorganics, metals, and the TCL of organic parameters compounds; and pesticides. The results (Table 1) show that the ambient groundwater quality has been degraded by the cesspools, septic tanks and various industrial point sources of pollution. The results show the native groundwater contains elevated concentrations of specific conductance, nitrogen, chloride, iron and zinc, and various chlorinated volatile organic compounds. Very little to no gasoline related compounds and pesticides were detected in the native groundwater.

Tertiary-treated wastewater used during the recharge periods was provided by the Cedar Creek wastewater treatment plant located 6-miles away in Wantagh, New York. The wastewater was treated with the following processes (Figure 4).
1. Chemical Assisted Primary Treatment
2. Two Stage Biological Treatment including Nitrofication, Denitrofication, and Secondary Clarification.
3. Sand Filtration and Granular Activated Carbon (GAC).
4. Chlorination.

Water samples were collected of the treated wastewater and analyzed for inorganics, metals, and volatile organic compounds. The results show that the water contained low concentrations of inorganics, metals, and volatile organic compounds. In most cases, the water recharged to the aquifer from the Cedar Creek wastewater treatment plant was of equal or better quality than the native groundwater. The mean daily concentration of suspended solids was 2 mg/L for the recharge basins and 1 mg/L for the injection wells. The mean daily concentration of nitrogen (total) was 1.5 mg/L for recharge basin water and 0.8 mg/L for injection wells. The mean daily concentration of chlorides was 156 mg/L for the recharge basins and 146 mg/L for the injection wells. The mean daily concentration of residual chlorine was 0.4 for the recharge basins and 0.2 for the injection wells. The mean daily concentration of total and fecal coliform was 3MPN/100 ml for the recharge basins and less than 2 MPN/100 ml for the injection wells. The water used during the recharge tests met

all of the New York State Department of Conservation Effluent Limitation Guidelines as shown on Table 2.

Operation and Maintenance of Recharge Basins

Artificial recharge of reclaimed wastewater was conducted at East Meadow from October 1982 through January 1984 to evaluate the amount of groundwater mounding and the chemical effects of artificially recharging the aquifer with tertiary-treated wastewater. More than 800 million gallons of treated effluent was returned to the Upper Glacial aquifer through the recharge basins and injection wells over a 15 month period.

The recharge basins were operated two ways. The first method (constant rate mode) involved pumping a constant rate of water into each recharge basin (Nos. 1 through 7) and allowing the height of water in each basin to potentially vary. The second method of operation (constant head mode) involved maintaining a constant height of water in each basin with a potentially varied flow rate.

The computation of the infiltration rate using the constant rate method took into account the wetted area or changes in storage if ponding occurs. The infiltration rate computation for the constant head method was much simpler because the infiltration rate was equal to the inflow rate as long as the stage height remains constant.

Various Maintenance techniques were evaluated during the recharge tests. They included:

1. Alternation of wetting ad drying (application and rest).
2. Scarification
3. Mulching
4. Covering the floor with gravel to disperse the clogging material.
5. Plant Vegetation to increase permeability and porosity and create root channels.

Each of these methods was evaluated during the testing of the recharge basins. The results of the injection well tests are summarized below

1. The infiltration rate ranged from 0.5 to 2.6 feet per hour. Infiltration rates were reduced by impaired water quality (i.e., episodic periods of high turbidity water).

2. Pressure head measurements in the unsaturated zone as measured by tensiometers during infiltration shows that a clogging layer gradually formed at the bottom of the basins. The infiltration rates were increased when the clogging layer was maintained by scarifying, stripping, and allowing dry periods. Even higher infiltration rates were observed after basin floors were tilled. Allowing natural vegetation to grow on the basin floors was believed to increase the infiltration rates by creating shallow root channels.

3. Neutron geophysical logs collected beneath the basins during the infiltration tests to monitor the movement of water in the unsaturated zone show that the wetting front moved from 6 to 12 feet per hour. This was confirmed by hydrographs of the water table in underlying observation wells. Neutron logs collected at locations adjacent to basins (6, 12, and 18 feet from the basin)

during infiltration tests showed that the movement of water was essentially vertical.

4. Water samples were collected from four lysimeters (2.5, 5.3, 8.2, and 11 feet below the floor of the basin during recharge periods. The water samples were analyzed for metals, inorganics (nitrogen, carbon, and phosphorous), and low-molecular weight hydrocarbons. The results showed that aquifer matrix did not affect the water quality for most constituents; however a reduction in the concentration of phosphorous was noted until the adsorption capacity of the soil was exceeded. An increase in iron and manganese was also noted in some locations.

5. After 30 consecutive days of recharging 4 mgd in basins (2, 5, 6, and 7), the highest mound was 6.7 feet above the static water level. After 14 consecutive days of recharging 2 mgd in basins (1, 5, 6, and 7), the highest mound was 4.3 feet above the static water level.

6. The rate and duration of recharge affected the infiltration rate. After recharging for long periods of time (greater than 1.5 months), ponding increased as a result of clogging mainly from biological growth. High infiltration rates could be maintained if recharge durations were shorter and the floors of the basins were scarified.

Operation and Maintenance of Injection Wells

Injection tests were conducted between October 20th to December 11, 1983 by injecting a maximum of 340 gpm in Injection Well A, 330 gpm in Injection Well B, 250 gpm in Injection Well C, and 250 gpm in Injection Well D. Injection well E was only used sporadically during the test. The quality of the water injected into the aquifer is shown on Table 1.

The results of the injection well tests are summarized below

1. Mounding of the water table was insignificant during the injection cycles. When recharge basins and injection wells were operating simultaneously at 2 mgd each (i.e., basins and injection wells), the water table mound extended 3.5 feet above the static water level beneath the basins and 2 feet above the static water level at the injection wells.

2. Well clogging was problematic during the injection tests and was primarily due to physical and chemical clogging associated with turbidity and precipitation of iron. Water levels inside the wells rose from 10 to 25 feet during the injection of water. Backflushing of the wells during the injection cycles and pH adjustment to control the dissolution of iron was not conducted during the injection tests. Biological clogging did not appear to affect the increase of head during injection.

3. Well construction methods (gravel pack, natural pack, gravel pack with redevelopment system) did not have an affect on the rate of well clogging during injection tests.

4. Native groundwater was not degraded by the injected water. In fact, the injection tests improved the groundwater quality at the site.

Conclusions

Observations during the recharge basin tests show that the most significant factor limiting the rate of infiltration through the basin floor was the quality of the reclaimed water. Recharge water with high turbidity clogged the basin floors. Various recharge basin maintenance techniques significantly increased the infiltration rates, including scarification, stripping, alternating wet and dry periods, and tilling.

Observations during the injection well test show that high turbidity and the precipitation of iron were the two main factors that adversely impacted the injection wells causing the wells to be redeveloped. It is interesting to note that they did not backflush the wells.

The authors of these rescharge tests concluded that recharge basins were a better way to recharge the aquifer with tertiary-treated wastewater because the basins floors were more accessible and easier to rehabilitate than the inaccessible well screens.

Injected teriary-treated wastewater did not negitivey impact the native water quality. In fact, it improved the native groundwater from the degraded native groundwater that was impacted by the cesspools, septic tanks, and industrial point source contamination.

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Figure 4: Cedar Creek Wastewater Treatment Train

Schneider, B. J. and E. T. Oaksford, 1986, Design, Operation, and Monitoring of an Experimental Artificial Recharge Facility at East Meadow, Long Island, New York, USGS WRI 84-4321.

Schneider, B. J., H. Ku, and E. T. Oaksford, 1987, Hydraulic Effects of Artificial Recharge Experiments with Reclaimed Water at East Meadow, Nassau County, New York, USGS WRI 85-4323.

Table 1.--Summary of ambient water quality in upper glacial
and Magothy aquifers in study area, 1977-82

Constituent	Median	Minimum	Maximum	Number of samples
Temperature (°C)	14.0	12.0	18.5	181
Turbidity (NTU)	1.00	1.00	140	62
Specific conductance (µS/cm)	300	26.5	545	174
Oxygen, dissolved (mg/L)	3.80	.80	40.0	179
Nitrogen, total (mg/L as N)	.00	.00	20.0	162
Nitrogen, dissolved (mg/L as N)	9.10	.35	22.0	65
Nitrogen, organic total (mg/L as N)	.00	.00	1.00	142
Nitrogen, organic dissolved (mg/L as N)	.00	.00	1.00	50
Nitrogen, ammonia dissolved (mg/L as N)	.02	.01	2.40	100
Nitrogen, ammonia total (mg/L as N)	.01	.00	2.40	202
Nitrogen, NO2 + NO3 total (mg/L as N)	9.00	.03	25.0	205
Nitrogen, NO$_2$ + NO$_3$ dissolved (mg/L as N)	9.20	.08	25.0	204
Phosphorus, total (mg/L as P)	.01	.00	.75	204
Carbon, organic total (mg/L as C)	.80	.20	12.0	74
Calcium, dissolved (mg/L as Ca)	18.0	.80	55.0	205
Magnesium, dissolved (mg/L as Mg)	3.40	.30	9.30	205
Sodium, dissolved (mg/L as Na)	24.0	3.00	71.0	205
Potassium, dissolved (mg/L as K)	3.70	.20	10.0	203
Chloride, dissolved (mg/L as Cl)	28.0	1.80	100	207
Chromium, total recoverable (µg/L as Cr)	10.0	1.00	100	199
Iron, total recoverable (µg/L as Fe)	50.0	.00	18,100	207
Lead, total recoverable (µg/L as Pb)	.00	.00	450	203
Nickel, total recoverable (µg/L as Ni)	.00	.00	30.0	105
Zinc, total recoverable (µg/L as Zn)	30.0	10.0	860	172
Dichlorobromomethane total (µg/L)	.20	.00	1.00	84
Carbontetrachloride total (µg/L)	.20	.00	1.00	84
1,2-Dichloroethane total (µg/L)	.20	.00	130	53
Bromoform, total (µg/L)	.70	.00	8.00	84
Chloroform total (µg/L)	1.00	.00	33.0	84
Toluene, total (µg/L)	1.00	.00	3.00	80
Benzene, total (µg/L)	1.00	.00	30.0	80
N-butyl benzyl phthalate total (µg/L)	1.00	1.00	1.00	31
Tetrachloroethylene, total (µg/L)	5.10	.00	1,320	84
1,1-Dichloroethane, total (µg/L)	6.00	.00	300	84
1,1,1-Trichloroethane, total (µg/L)	7.95	.00	217	84
Chloroethylene, total (µg/L)	4.00	.00	500	84
2,4-Dimethylphenol, total (µg/L)	1.00	1.00	1.00	31
Di-n-butyl phthalate, total (µg/L)	1.00	1.00	5.00	31
Trichloroethylene, total (µg/L)	7.65	.00	260	84
Aldrin, total (µg/L	.00	.00	.01	175
Lindane, total (µg/L)	.00	.00	.01	175
Chlordane, total (µg/L)	.00	.00	1.00	175
DDD, total (µg/L)	.00	.00	.01	175
DDE, total (µg/L)	.00	.00	.01	174
DDT, total (µg/L)	.00	.00	.01	175
Dieldrin, total (µg/L)	.01	.00	1.40	175
Endosulfan, total (µg/L)	.00	.00	.01	173
Endrin, total (µg/L)	.00	.00	.02	175
Heptachlor, total (µg/L)	.00	.00	.01	175
Heptachlor epoxide, total (µg/L)	.00	.00	.06	175
PCB, total (µg/L)	.00	.00	.60	175
Mirex, total (µg/L)	.00	.00	.01	170
Silvex, total (µg/L)	.00	.00	.21	143
Solids, residue at 180°C dissolved (mg/L)	187	19.0	343	31
Solids, sum of constituents, dissolved (mg/L)	.00	.00	117	200
Mercury, total recoverable (µg/L as Hg)	.30	.00	.50	62

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SLAKED LIME

GRIT CHAMBERS → LIME REACTOR → PRIMARY SEDIMENTATION

RAW WASTEWATER

CARBON OXIDATION NITRIFICATION

SEDIMENTATION

TWO-STAGE BIOLOGICAL TREATMENT SYSTEM

METHANOL

DENITRIFICATION AERATION

ALUM AND POLYMER

SEDIMENTATION

EAST MEADOW ARTIFICIAL-RECHARGE PROJECT SITE

FILTRATION

6.25-MILE PIPELINE

STORAGE CLEAR WELL ← CHLORINATION ← CARBON ADSORPTION

CHLORINE

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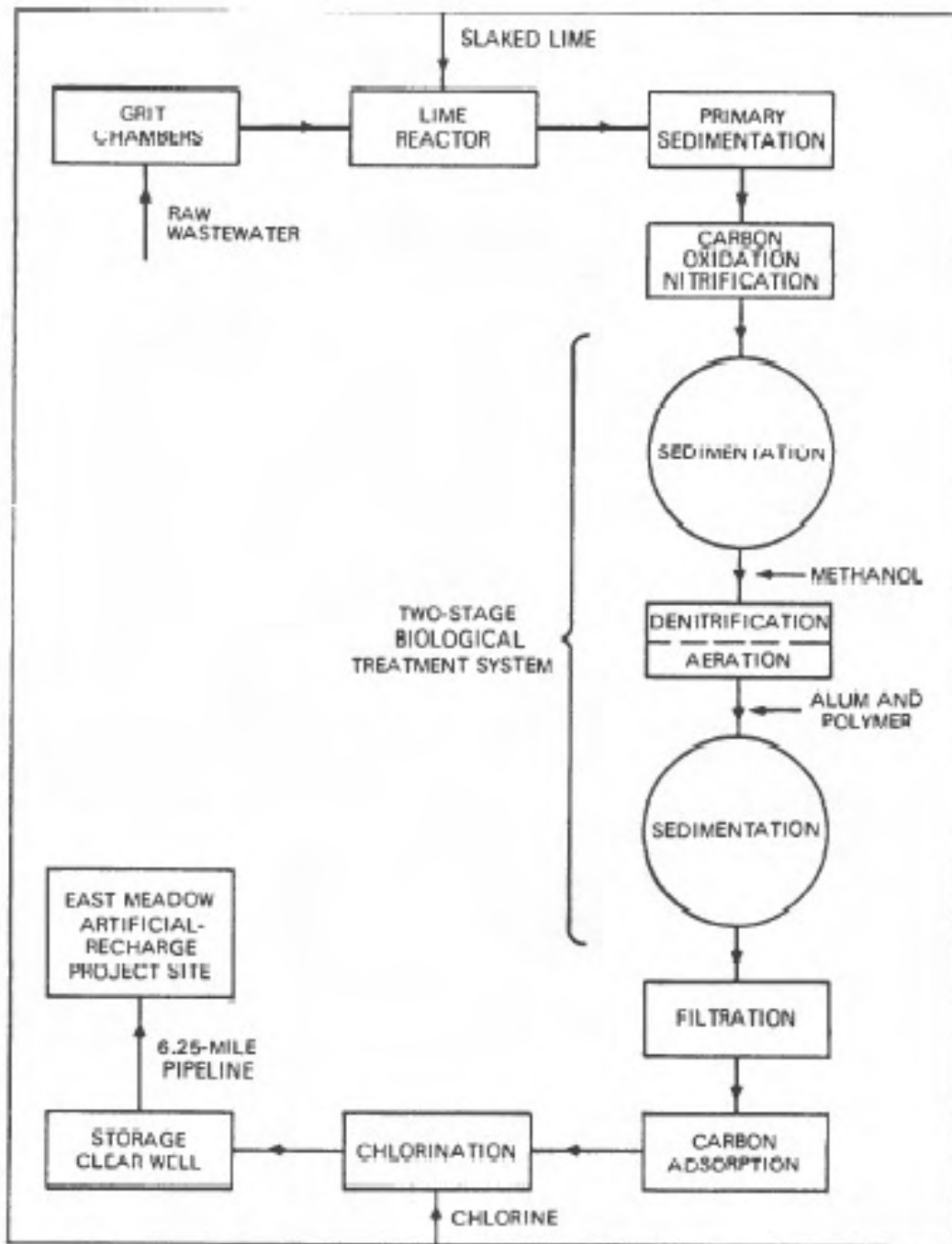
Table 4.--Concentration of selected constituents in reclaimed water
during recharge tests at basins and injection wells

Constituent	Basins[1] (median monthly values)	Recharge well[2] (median daily values)
5-day biochemical oxygen demand (mg/L)	<1	<1
Suspended solids (mg/L)	2	1
Total solids (mg/L)	528	542
Total dissolved solids (mg/L)	527	541
Nitrogen, organic total (mg/L)	.2	.2
Nitrogen, NH_4 as N (mg/L)	.7	.6
Nitrogen, NO_3 as N (mg/L)	.74	.18
Nitrogen, NO_2 as N (mg/L)	.02	<.01
Nitrogen, total (mg/L)	1.5	.8
Phosphorus, ortho as P (mg/L as P)	.3	.3
Phosphorus, total as P (mg/L as P)	.3	.4
Turbidity (NTU)	.6	.4
Cyanide (mg/L as CN)	<.01	<.01
Fluoride (mg/L as F)	.24	.23
Oil and grease (total, mg/L)	<1	<1
Chloride (mg/L as Cl)	156	146
Chemical oxygen demand (mg/L)	10	10
Hardness (mg/L)	177	180
Chlorine residual (free) (mg/L)	.4	.2
Total coliform (MPN/100 ml.)	3.	<2
Fecal coliform (MPN/100 mL)	3.	<2
pH	7.5	7.4
Antimony, µg/L as Sb	<5.0	10.0
Arsenic, µg/L as As	<5.	<5.0
Barium, µg/L as Ba	<15.0	<50.0
Beryllium, µg/L as Be	<5.0	<5.0
Cadmium, µg/L as Cd	<1.0	<1.0
Chromium, µg/L as Cr	<10.0	<10.0
Cobalt, µg/L as Co	2.0	<100.0
Copper, µg/L as Cu	14.0	<50.0
Lead, µg/L as Pb	<10.0	<10.0
Mercury, µg/L as Hg	<.5	<.5
Nickel, µg/L as Ni	<50.0	<50.
Selenium, µg/L as Se	<.5	<.5
Silver, µg/L as As	<.50	<.50
Sodium, µg/L as Na	110	100
Thallium, µg/L as Tl	<10.0	<10.0
Zinc, µg/L as Zn	50.0	50.0
Iron, total µg/L as Fe	50.0	40.0
Manganese, µg/L as Mn	<50.	<50.

[1] October 1982 through December 1983 (excluding February 1983)
[2] October 11, 1983 through December 25, 1983

[Well locations are shown in fig. 4B; dashes indicate value below detection limit]

| Constituent | Well number | | | | |
	N9202 A	N9203 B	N9204 C	N9205 D	N9206 E
pH	6.2	6.3	5.7	5.8	5.6
Specific conductance (µS/cm at 25 °C)	400	628	350	393	333
Calcium (mg/L)	29	59	a	23	23
Magnesium (mg/L)	5.7	16	a	7.1	4.0
Potassium (mg/L)	6.4	2.2	a	5.3	6.2
Sodium (mg/L)	25	80	a	33	27
Chloride (mg/L)	26	90	a	38	26
Fluoride (mg/L)	0.1	<0.1	a	<0.1	<0.1
Sulfate (mg/L)	45	85	a	39	42
Lead (ug/L)	5	12	a	7	3
Chromium (ug/L)	10	10	a	20	20
Iron (ug/L)	130	80	a	600	290
Nickel (ug/L)	5	4	a	2	3
Manganese (ug/L)	10	80	a	40	370
Zinc (ug/L)	10	20	a	10	10
Nitrogen, total as N (mg/L)	23	37	a	16	8.8
Nitrogen, total NH_4^+ as N (mg/L)	0.02	0.01	<0.01	<0.01	0.05
Nitrogen, total $NO_2 + NO_3$ as N (mg/L)	22	36	18	16	8.2
Nitrogen, total organic as N (mg/L)	0.98	--	--	--	0.55
Carbon, total organic (mg/L)	0.9	2.8	1.2	1.0	1.2
Phosphorus, total (mg/L)	0.01	0.03	0.02	0.03	<0.01
Dichlorobromethane (ug/L)	--	<1	<1	<1	<1
Tetrachloroethylene (ug/L)	--	27	<1	330	<1
1,1,1-Trichloroethane (ug/L)	26.0	25	<1	9	22
Bromoform (ug/L)	--	<1	<1	<1	<1
1,2-trans-Dichloroethylene (ug/L)	--	14	<1	170	<1

a Analysis not performed.

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Introduction

Groundwater derived from precipitation is the sole source of drinking water for residents in Nassau and Suffolk County, Long Island, New York (Figure 1). During the Long Island's early (1960s) period of development, drinking water and industrial waste water was returned to the aquifer with cesspools, septic tanks, and leach fields. With the explosive population growth in Nassau County, this practice caused widespread degradation of the chemical quality of shallow groundwater. To prevent the continuation of water quality degradation, centralized sewer systems were installed in densely populated areas. The water collected by the sewers was conveyed to wastewater treatment plants and ultimately discharge to the Atlantic Ocean. Although this process protects the groundwater quality in Nassau County, it increased the consumptive use and decreased the amount of groundwater available in the aquifer causing groundwater levels to decline and stream flow to decrease or cease. In addition, a State report (Temporary Syaye Commission on the Water Supply Needs of Southeastern New York, 1972) predicted a groundwater deficit of 93.5 to 123 mgd for Nassau County by year 2000. Because of the predicted deficit, the NCDPW and the USGS began an experimental deep-well recharge program in 1968 to determine if treated effluent from a wastewater treatment plant could be pumped into a series of barrier injection wells along the south shore of Nassau County to prevent further landward movement of the existing saltwater intrusion in the Magothy aquifer.
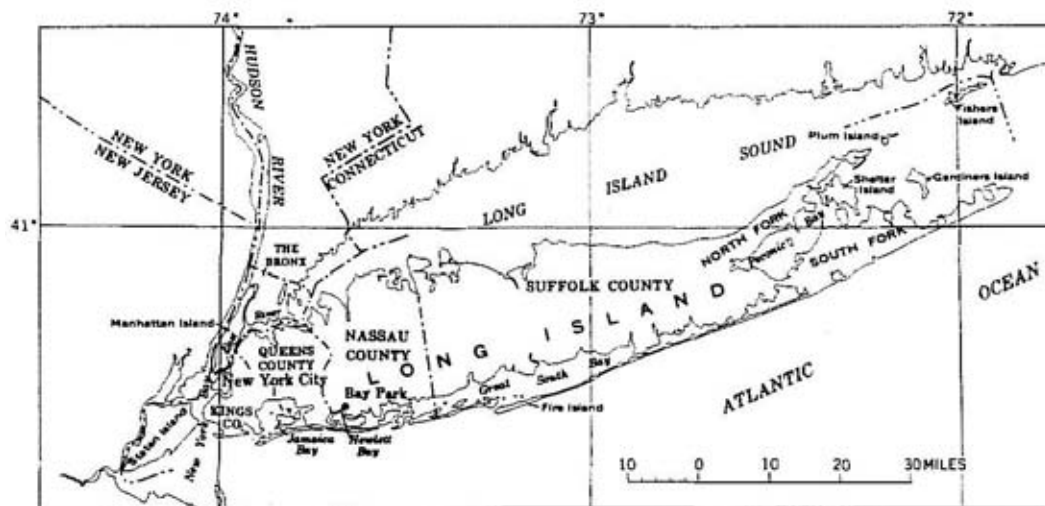

Figure 1: Location of Bay Park New York

Thirteen recharge tests using tertiary treated effluent from the Bay Park wastewater treatment plant and six tests using potable water were completed, Recharge teats ranged from 200 to 400 gpm and the longest test lasted 84.5 days.

Design and Construction of Facility

The Nassau County Department of Public Works (NCDPW) built the artificial recharge facility at the existing Bay Park Wastewater treatment plant located in Nassau County, New York. The existing wastewater treatment facilities include a 60 million gallon per day (mgd) tertiary treated wastewater treatment plant that treats the wastewater. The wastewater is treated using coagulations, sedimentation, filtration (sand and anthracite), granular activated carbon, and chlorination (figure 2). The effluent meets the most drinking water standards applicable at the time of the project.
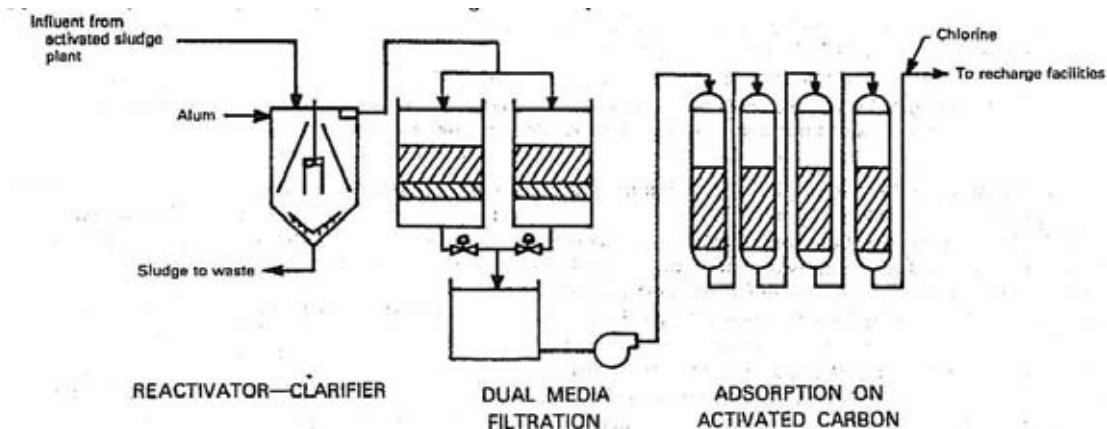
Figure 2: Location of Recharge Basins and Injection Wells.

Four Hundred twenty gallons per minute (gpm) of either potable water or tr4eated effluent water from the plant was directed to the recharge facilities. The artificial recharge facilities include a recharge well, storage tank, and building (figure 3) that includes a degassifier and pH adjustment and dechlorination equipment.

The injection well (18x16) was constructed of 62 feet slot 16-inch diameter stainless steel wire-wrap screen (418 to 480 fbg) and sufficient 18-inch diameter epoxy coated fiberglass casing to reach land surface. A 10-foot sump was added to the bottom of the screen. The 10-inch annulus between the screen and the borehole was filled with gravel pack consisting of 71 percent very coarse sand and 28 percent very fine gravel.
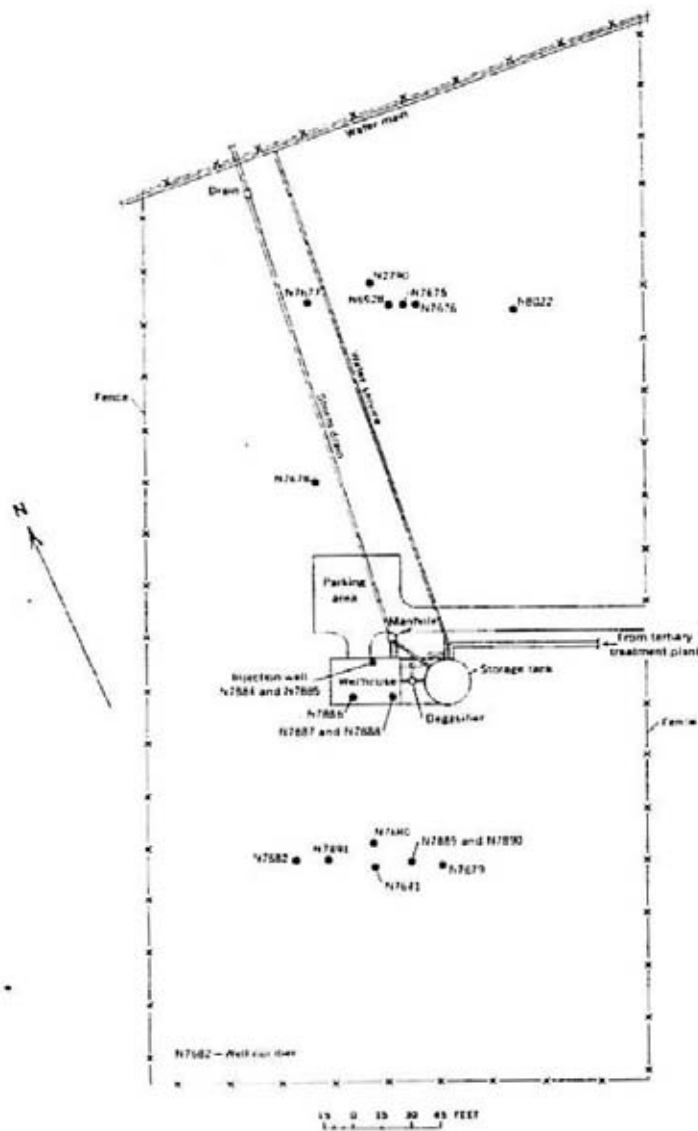
Figure 3: Artificial Recharge Site Layout at Bay Park, Nassau County, New York

Water can be injected into the injection well either through a pipe that enters the top of the well, through the pump column, or through a pipe that enters the well at 192 feet below grade. Since the static water level in the well is 5 feet below grade the use of the deep pipe prevented any cascading water entering the well. Two 1-inch diameter pipes enter the well casing at different depth to allow the collection of water level measurements. One pipe enters the well casing 3 feet below grade and the second pipe enters the casing 3 feet above the top of the screen. These pipes allow water level measurements to be collected at land surface and just above the top of the screen. Three pipes enter the well through the top of the well. One was used for the pump column. The remaining two pipes were used as access point to facilitate the collection of water samples or the lowering of instruments into the well.

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A 4-inch diameter observation well was installed in the annular space in the middle of the gravel pack. This was constructed in a similar manner as the injection well. Periodically a 2-inch diameter stainless wire-wrap screen filled with sand was lowered into this well to study potential bacterial growth. Figure 4 shows the features outside of the injection well and figure 5 shows the features inside of the injection wells
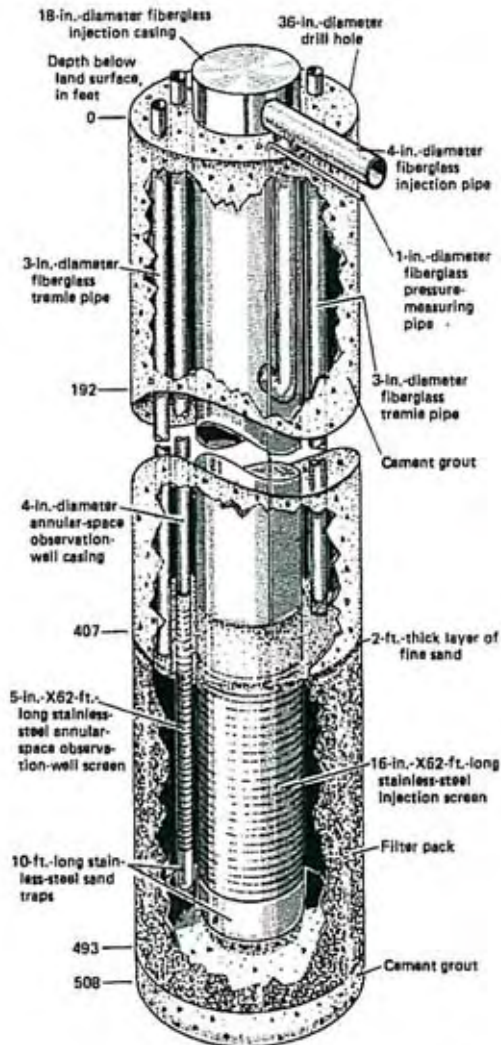

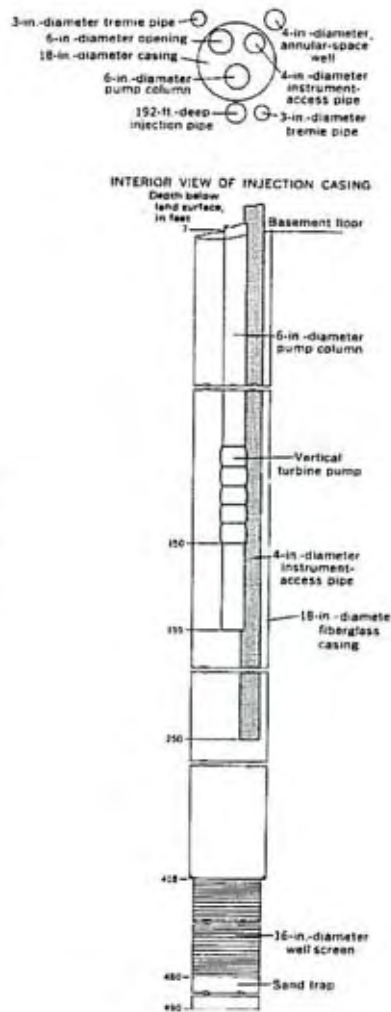Figure 4: Appurtenances Outside of Injection Well

Figure 5: Appurtenances Inside of Injection Well

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Eighteen observation wells were installed in a radial pattern away from the injection wells at distances from a few inches to 200 feet. Some observation wells are screened in the same geologic zone as the injection wells and some observation wells were installed above and below the injection zone.

The recharge facility contains instrumentation for automatic monitoring of the flow rates during injection and pumping periods. Some water quality parameters were also monitored automatically.

Table 1: Summary of Observation Wells at Pay Park Facility

Well	Screened interval (ft below land surface)	Nominal screen size (in.)	Screen material	Casing ID (in.)	Casing material	Approximate distance and direction from recharge well
N7884	418–480	16	Stainless steel[1]	17.5	Fiberglass	0 ft.
N7885	418–180	5	...do[1]	4.4	...do	Annular space.
N7886	417–480	6	...do[1]	5.4	...do	20 ft SW.
N7887	535–545	5	...do[1]	4.4	...do	20 ft S.
N7888	307–317	5	...do[1]	4.4	...do	20 ft S.
N7889	535–545	5	...do[1]	4.4	...do	100 ft SW.
N7890	452–462	5	...do[1]	4.4	...do	100 ft SW.
N7891	307–317	5	...do[1]	4.4	...do	100 ft SW.
N8022	420–480	6	Fiberglass	6.4	...do	200 ft NE.
N2790	538–560	4.5	Steel	3	PVC[2] lined steel	200 ft NE.
N6928	716–726	4	...do	6 and 2	Steel	200 ft NE.
N7676	7–10	4	...do	4	...do	200 ft NE.
N7678	65–80	4	...do	4	...do	100 ft N.
N7675	28–34	4	...do	4	...do	200 ft NE.
N7677	84–89	4	...do	4	...do	200 ft NE.
N7679	81–86	2	Stainless steel[1]	2	Fiberglass	100 ft SW.
N7680	39–44	2	...do[1]	2	...do	100 ft SW.
N7681	41–44	2	...do[1]	2	...do	100 ft SW.
N7682	9–12	2	...do[1]	2	...do	100 ft SW.

[1] Type 304.
[2] PVC is abbreviation for polyvinyl chloride.

Physical Setting and Baseline Conditions

Two distinct aquifers exist beneath Bay Park. The shallow water table aquifer, called the Upper Glacial aquifer, is approximately 42 feet deep and consists of glacially derived fine to coarse grained brown sand with minor amounts of silt and clay. Beneath the Upper Glacial aquifer is the Gardiners Clay, which is approximately 16 feet thick consisting of glacially derived blue-gray clay and silt. Beneath the Gardiners Clay is the Jameco Gravel, which is approximately 46 feet thick consisting of glacially derived very fine to coarse gravely layers. Beneath the Jameco Gravel and in direct hydraulic connection with the Jameco Gravel is the Mogothy Formation of early Cretaceous. The Magothy is approximately 490 feet thick at Bay Park and consists of gray very fine to medium grained sand and some silt. Clay layers occur throughout the aquifer. Lignite occurs as layers and as disseminated particles throughout the aquifer. Pyrite is also present throughout the aquifer, particularly associated with lignite. Accessory minerals include muscovite, pyrite-marcasite, tourmaline and in minor amounts garnet, zircon, andalusite, and sillmanite.

The injection well was screened in the Magothy aquifer, which is a semiconfined aquifer at this location. Several aquifer tests were conducted at this facility before the beginning of the recharge tests. The results show that the horizontal and

vertical hydraulic conductivity was 940 gpd/ft^2 (126 feet/day) and 12.5 gpd/ft^2 (1.67 feet/day), respectively. The Storativity of the aquifer was calculated to be 0.00015. because of the sites close proximity to the Atlantic Ocean and bays, the water levels at the site showed a tidal oscillation that ranged from 0.067 to 0.098 feet for those wells that are over 300 feet deep. The tidal responses in the observation wells seemed to lag 3 to 7 minutes behind the bay.
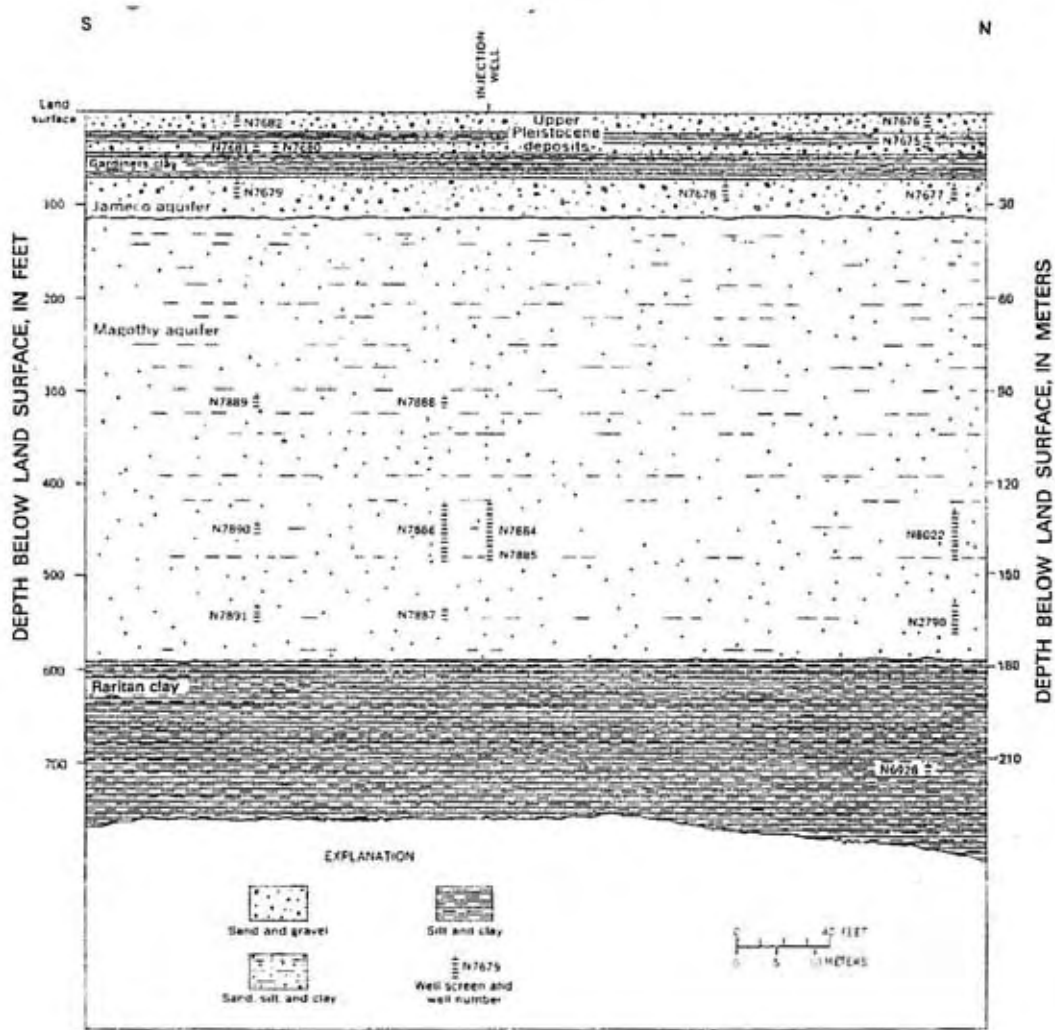
Figure 6: Geology at Bay Park

Water quality samples were collected from the injection well and observation wells and analyzed to determine the baseline water quality conditions in the Magothy aquifer. The results on Table 1 show that the native groundwater is anaerobic with a Eh ranging from -0.03 to -0.1. the pH ranges from 5.2 to 5.7 and the average temperature is 15 degrees centigrade. The concentration of iron ranges from 140 to 300 µg/L and the concentration of sulfate ranges from 3.6 to 0.4. the groundwater contains nitrogen (total) at concentrations ranging from 01. to 04. and phosphate ranging from 0.01 to 0.02.

Water samples were collected of the treated effluent and analyzed to determine the baseline conditions of the source water. The results on Table 1 show that the treated effluent contains an average concentration of dissolved oxygen of 4.6 and a temperature of 14.6. Turbidity of the water is 0.4 and the pH is 6.1. The concentration of iron is 410 µg/L and the concentration of sulfate is 160 µg/L. the water also contains nitrogen (total) at 0.6 mg/L and phosphate at 0.17 mg/L.

Table 2: Characterization of Native and 'Reclaimed' or Tertiary Wastwater

[All data in milligrams per liter unless otherwise indicated; N.D., not determined]

| | Native water[1] | | | Reclaimed water[4] | |
| | Range and(or) mean value | | | | Standard deviation |
	Minimum	Mean	Maximum	Mean	
Silica (SiO₂)	7.2	....	7.4	13	1
Iron (Fe), total,[2] (µg/l)	140	....	300	410	260
Iron (Fe), dissolved,[2] (µg/l)	140	....	300	230	180
Manganese (Mn), total, (µg/l)	....	30	....	75	31
Calcium (Ca)	.4	....	1.4	20	3
Magnesium (Mg)	....	.2	....	6.4	2.1
Sodium (Na)	3.7	....	3.9	86	12
Potassium (K)	.4	....	.6	13	1
Bicarbonate[3] (HCO₃)	4.5	....	7.5	59	19
Sulfate (SO₄)	3.6	....	4.0	160	20
Chlorine, total residual[4]	....	ᐧ0	....	2.1	.5
Chloride (Cl)	....	3.9	....	99	23
Nitrate (NO₃), total	....	0	....	.4	.2
Nitrite (NO₂), total	....	ᐧ0	....	.3	.1
Nitrogen, ammonia, total as NH₄	.8	....	ᐧ.9	30	2
Nitrogen, total organic as N	.1	....	ᐧ.4	.6	.4
Phosphorus (PO₄), total	.01	....	.02	.17	.24
Dissolved solids, residue at 180°C	....	N.D.	....	398	68
Dissolved solids, calculated (sum of dissolved constituents)	22	....	25	463	48
Specific conductance,[3] (micromhos/cm at 25°C)	28	....	32	786	81
pH[3]	5.22	....	5.72	ᐧ6.1	.2
Water temperature,[3] °C	....	15	....	17.5	2.0
Turbidity[3] as SiO₂	....	ᐧ0	....	.4	.3
Oxidation reduction potential[3] (Eh, in volts)	-.03	....	-.10	N.D.	....
Dissolved oxygen[3] (DO)	....	ᐧ0	....	4.5	1.6
Chemical oxygen demand (COD) 0.025N K₂Cr₂O₇	....	ᐧ0	....	9	5
Detergents (MBAS)	0	....	ᐧ.01	.07	.08

[1] After Verschudi and others (1974), for samples taken from the 20-, 100-, and 200-ft (6.1-, 30-, and 61-m) observation wells before any recharge, unless otherwise stated.
[2] Based on the weighted average of at least 33 daily or weekly composite samples.
[3] Measured or determined on site.
[4] Data from RW10 (reclaimed-water test 10).
[5] Mean calculated after converting pH values from logarithmic form to exponential form

Operation and Maintenance of Recharge Basins

Fourteen injection tests were conducted from 1968 to 1973 with durations from 2 to 84.5 days. Recharge rates varied from 200 to 400 gpm. Six of the recharge tests were conducted with potable water and 13 tests were conducted with treated wastewater. The details of each test are provided in Table 2.

Source: *Design, Operation, and Maintenance for Sustainable Underground Storage Facilities* by Herman Bouwer et al. ©2008 AwwaRF.

Table 3: Summary of Injections

[CW, city water; RW, reclaimed water]

Test No.	Date	Approximate length (days)	Recharge rate (gal/min)	Gallons recharged (thousands)	Special treatment of water
		City water			
CW1	..... Feb. 27–29, 1968	2	190	570	Degassed, dosed with H₂SO₄ and Na₂SO₃.
CW2	..... Mar. 12–14, 1968	2	385	1,118	Dosed with H₂SO₄ and Na₂SO₃.
CW3	..... Mar. 25–27, 1968	3	185	840	Degassed.
CW4	..... Apr. 16–19, 1968	3	390	1,655	None.
CW5	..... Feb. 17–19, 1970	2	400	1,200	Dosed with H₃PO₄.
CW6	..... Sept. 13–17, 1971	4	395	2,281	None.
		Reclaimed water			
RW1	..... Oct. 8–10, 1968	2	200	616	Degassed.
RW2	..... Dec. 10–12, 1968	2	350	1,063	Degassed.
RW3	..... Feb. 25–27, 1969	2	350	1,066	None.
RW4	..... May 6–16, 1969	10	360	5,247	None.
RW5	..... Sept. 15–25, 1969	10	350	5,117	Degassed.
RW6	..... Mar. 17–19, 1970	2	340	1,133	None.
RW7	..... Apr. 14–May 17, 1970	33	200 to 350	13,970	None.
RW8	..... Nov. 30–Dec. 9, 1970	10	340	4,930	Unchlorinated.
RW9	..... Apr. 21–May 7, 1971	10	355	5,156	None.
RW10	..... Nov. 1–11, 1971	10	350	5,180	Bypassed carbon adsorbers.
RW10A	... Jan. 10–14, 1972	4	350	2,000	Unchlorinated.
RW11	.... Mar. 6–16, 1972	10	350	5,124	Dosed with Na₂S₂O₃ • 5H₂O.
RW12	.... June 5–15, 1972	10	355	5,050	Dosed with NaOH.
RW13	.... Oct. 24, 1972–May 11, 1973	84.5	355	41,695	None.

Concentration of Iron

The results show that the highest concentration of iron (3.05 mg/L) in the recovered water occurred when the wastewater bypassed the carbon filters and the Chemical Oxygen Demand was high. The second highest concentration of iron (2.88 mg/L) in the recovered water occurred when the wastewater received no additional treatment other than at the Bay park wastewater treatment plant. The third highest concentration of iron (2.6 mg/L) occurred when the water was dechlorinated. pH adjusted water contained the lowest concentrations of iron 1.37 mg/L other than a brief injection of untreated water. Therefore controlling the dissolution of iron was best achieved by increasing the pH of the water to 8. however, numerous discussions with the operators of the Bay park facility (verbal communication Henry Ku) revealed that controlling pH adjustment was problematic and the operators felt that the concentration of iron could have been better controlled if an automated method of pH adjustment was used.

Table 4: Summary of Injections

Recharge test	Treatment	Maximum iron concentration (mg/l)
RW10, part I	Carbon filters bypassed; high COD	3.05
RW10, part II	Unchlorinated	1.20
RW11	Dechlorinated using sodium thiosulfate	2.60
RW12	Raised pH to 8 using sodium hydroxide	1.37
RW13	Normal treatment (see section "Bay Park sewage-treatment plant and demonstration tertiary-treatment plant")	2.88
CW6	...... do	No change from native water.

Well Clogging

During the recharge test, the injection well and selected portions of the aquifer developed various degrees of clogging. This resulted in excess head buildup in the injection well. The maximum reduction of specific capacity from 23.5 to 2.5 gpm/ft occurred while injecting treated wastewater. The well clogging was caused primarily by suspended solids in the injected water. Biological growth was not believed to play a significant role in well clogging. The specific capacity of the injection decreased during a number of the injection tests; however, the specific capacity was partially restored by pumping and surging the well. The specific capacity was also partially restored using chemicals such as sodium hypochlorite, hydrochloric acid, and bacterial ammonium. In addition, the specific capacity was partially restored when the well remained idle after injections and pumping and surging was more successful after several week idle periods.

Microbial Activity

The results showed that maintaining a residual chlorination of 2.5 mg/L suppressed microbial growth to the point where it did not contribute significantly to head buildup during injection. The results also showed that as the concentration of chlorine dissipated between injections microbial growth ensued. In an injection cycle where the source water contained substantial total coliform, fecal coliform, and fecal streptococcal densities, no fecal coliform or fecal streptococcal bacteria and only total coliform bacteria was detected in water samples collected 20 feet from the injection well.

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