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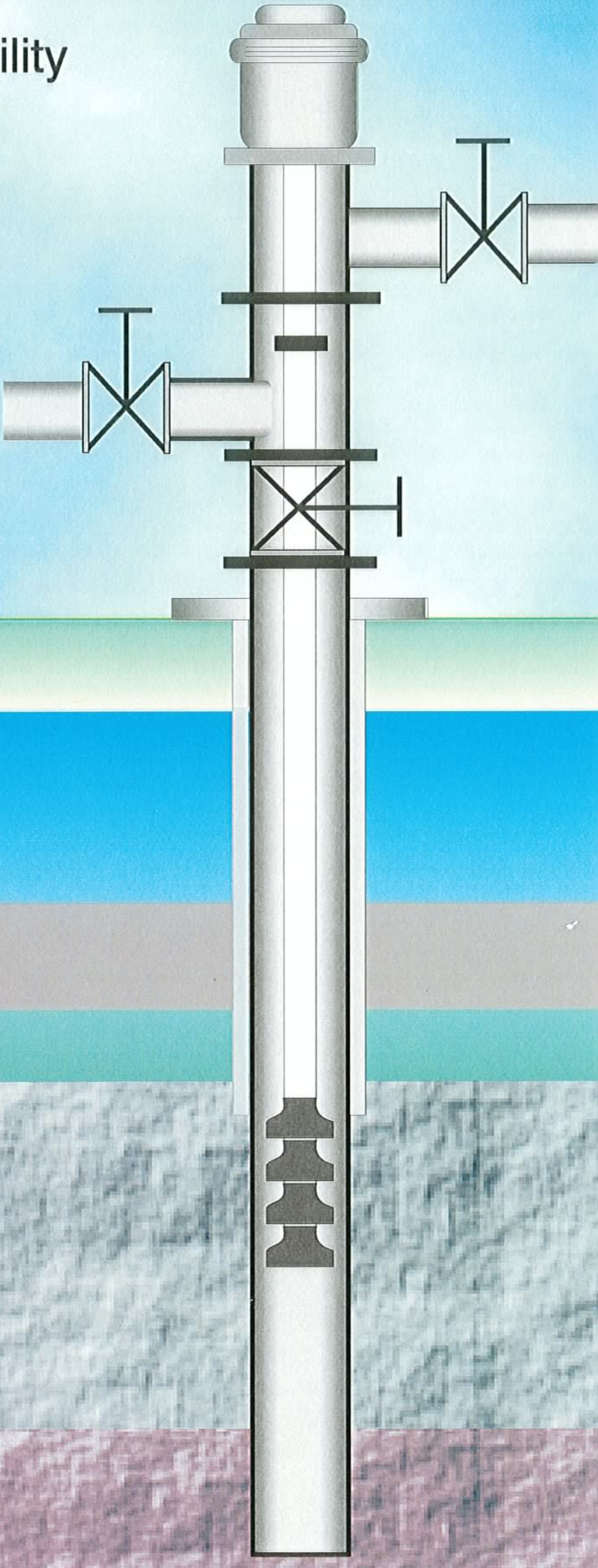
Corkscrew Water Treatment Facility
Aquifer Storage and Recovery
Pilot Project
Final Report

July 1997



LCRWSA

Lee County Regional
Water Supply Authority

CORKSCREW WATER TREATMENT FACILITY
AQUIFER STORAGE AND RECOVERY
PILOT PROJECT

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> Lee County Utilities (LCU), Lee County Department of Natural Resource Management (LCDNRM), Florida Department of Environmental Protection (DEP), South Florida Water Management District (SFWMD), Lee County Health Department (LCHD), Lee County Environmental Laboratory (LCEL), ST Environmental (STE), and the Lee County Regional Water Supply Authority (LCRWSA).

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We would also like to recognize the Authority Board Members who have provided guidance and input during the pilot project. The current Board Members are:

Commissioner Ray Judah, Chairman	Lee County
Mr. Archie Grant	Lee County Adhoc Representative
Councilman Bob Anderson	City of Fort Myers
Mr. Emmett P. Waite, Jr.	City of Fort Myers
Mr. J.W. French	Lee County
Mr. James Garner	Attorney
Mr. James Yaeger	Attorney

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1.0 EXECUTIVE SUMMARY

This report summarizes the findings and recommendations of a four phase aquifer storage and recovery (ASR) pilot project conducted at the Lee County Utilities - Corkscrew water treatment plant by Camp Dresser & McKee, Inc., and ViroGroup, Inc. The project was conducted through a cooperative agreement between the Lee County Regional Water Supply Authority and the South Florida Water Management District, with funding provided by Lee County Utilities and the District.

The purpose of the project is to evaluate the use of ASR as an alternate method of water supply at the facility, with a primary goal of maximizing the water supply and treatment capacity of the Corkscrew wellfield and water treatment plant. Some of the important findings and recommendations of the project are summarized below.

1. Results of the pilot ASR program indicate that aquifer storage and recovery can be successfully accomplished in the area surrounding the Corkscrew water treatment facility by utilizing Zone I of the mid-Hawthorn aquifer as the storage zone. Pilot system performance data suggest that facilities can be installed that would increase the existing ASR capacity to at least three million gallons per day (mgd) and provide a seasonal recovery capacity of approximately 270 million gallons of stored water.

2. Construction of a 3.0 mgd ASR system would save utility customers capital and operating costs when compared to the development of conventional water supply and treatment options. Capital costs for providing 3.0 mgd of additional water supply are estimated at $2.8 million for ASR, and from $5.6 to $7.8 million for conventional water supply and treatment alternatives, or a savings of $2.8 to $5.0 million using ASR. Estimated operating costs for conventional treatment range from $0.77 to $1.35 per thousand gallons, compared with $0.22 per thousand gallons of water supplied by the ASR system.

3. ASR recovery efficiency is expected to approach 100% due to the favorable storage characteristics of the aquifer and the excellent ambient groundwater quality of the storage zone in this area.

4. An ASR system designed to operate at the 3 mgd average capacity should consist of 10 ASR wells. A linear wellfield aligned in a roughly north-south direction adjacent to Alico Road is the recommended conceptual design. Hydraulic modeling suggests that an appropriate well spacing is 1,500 feet for the five wells nearest the Corkscrew water treatment plant, and 1,000 feet for the remaining five wells. The final well placement should consider land availability, and the use of existing County owned property along Alico Road. An alternative alignment would be to site five wells along Alico Road and five wells along Corkscrew Road.

5. An ASR system consisting of 10 wells at the recommended spacing could be operated to withdraw water at rates of approximately 4.6 mgd to meet emergency demands for a period of at least two weeks.

6. Investigation results indicate that the storage zone transmissivity decreases in the eastern portion of the ASR test site, and suggests the presence of a restricted flow boundary further east. The anticipated formation pressure response to this bounded aquifer condition and corresponding increase in injection pressure during recharge is the primary reason for recommending the well spacings as proposed.

7. It may be possible to reduce the spacing between wells based on information obtained during the wellfield design and construction phases. It is currently recommended that single well pumping tests be conducted at all new ASR well sites, and that aquifer performance tests be conducted at two sites located at opposite ends of the wellfield during the initial phase of construction. The

hydraulic model should be updated with the new information collected and used to help finalize locations for the remaining ASR wells in the wellfield.

8. It is expected that the ASR wells will require periodic back-flushing and mild acid stimulation during the recharge period to clear the wells of deposits accumulated on the walls of the wellbore. Wells should be back-flushed by pumping for one to two hours at approximately two week intervals during the recharge period. Pilot project results indicate that mild acid stimulation will be required to remove mineral precipitates. The frequency of this treatment will likely vary among wells, but can be expected to be needed once or twice during the recharge period.

9. One of the goals of the ASR system is to maximize the water supply of the Corkscrew wellfield and the treatment capacity of the water treatment plant. Increasing the permitted rainy season wellfield withdrawal from the surficial aquifer will optimize this existing water source. An alternative to increasing wellfield withdrawals would be to identify another treated water source to contribute additional water for ASR storage. Review of the Corkscrew water treatment plant processes may be necessary to determine if the treatment plant can be re-rated to treat the additional raw water supply.

2.0 INTRODUCTION

The public water supply systems in Lee County have a permitted treatment capacity of 71 million gallons per day (mgd), and currently supply greater than 53 mgd of treated water during peak periods. By the year 2030, the total peak day demand is expected to reach over 150 mgd (LCRWSA Water Supply Master Plan, 1993). The utilities will have to double in size to meet this future demand.

Water managers recognize the growing demand for potable (drinkable) water to meet the urban needs. They also recognize that the water needs of the community and those of the environment must be balanced. New tools are being developed to assist community planners in managing their water supplies. Aquifer storage and recovery (ASR) is one of these tools.

ASR involves water storage in an isolated aquifer during periods of excess supply, which then allows recovery of the stored water during periods of peak demand. The plan for ASR at the project site is to store treated water from the Department of Lee County Utilities (LCU) Corkscrew Water Treatment Plant (WTP) in the mid-Hawthorn aquifer during the summer rainy season. The water will be recovered during the winter dry season, when demand is greatest. The recovered water will be disinfected prior to its distribution to the public.

Application of ASR will allow the current facility to maximize water treatment and delivery capacity, and to operate in a more efficient manner by utilizing excess capacity during low demand periods. The peak capacity of the facility during high demand periods will be increased by ASR supply. Use of ASR will also enable the County to better manage the Corkscrew wellfield by shifting a substantial portion of its dry season aquifer withdrawals to periods when precipitation rates are normally high, and the potential for environmental impact is low.

The entire Lee County area has been identified by the South Florida Water Management District (SFWMD) as a Reduced Threshold Area under the SFWMD's rules for general water use permits. Reduced Threshold Areas have been identified based on low resource availability, the potential for undesirable saline water movement into groundwater or surface water bodies, or the lack of water availability to meet projected needs of the region. The development of pilot projects to study the use of ASR as an alternate water supply option to help meet future demands in the County was recommended in the Lee County Regional Water Supply Authority Water Supply Master Plan (Camp, Dresser & McKee et al, 1993), and in the Lower West Coast Water Supply Plan (SFWMD, 1994).

The Corkscrew ASR pilot project is jointly funded by Lee County and the SFWMD. The cooperative agreement with SFWMD was signed on August 30, 1993. Amendment 1 to the agreement, which was signed by the District's Governing Board on April 11, 1994, changed the work startup date from November 1, 1993 to July 15, 1994 and revised the expiration date to December 31, 1996. Amendment 2, which was approved on January 16, 1997, extended the expiration date to July 30, 1997. This extension in the project duration enabled the project team to conduct additional aquifer hydraulic tests, extend the duration of the injection and recovery phases of the cyclic tests, and to evaluate well maintenance procedures to reduce water injection pressures.

2.1 Project Scope of Services

The ASR pilot project was divided into four work phases, which are described below:

Phase I Exploratory Well Drilling and Testing Program: This phase included design and construction of an 8-inch diameter exploratory well, geophysical logging, water sampling, pump tests and collection of geologic data. The data collected during construction of the exploratory well was used in Phase II to design the ASR pilot well. Field investigations and recommendations from the Phase I work were summarized in the Phase I Completion Report for Lee County Authority Test Well #LM-3982 (ViroGroup, Inc., 1994).

Phase II Preparation of Well Specifications and Drawings, and Well Construction: During this phase, the well specifications and drawings were prepared and all required permits obtained for construction of the ASR pilot well and two monitor wells. The exploratory well was also converted into a third monitor well. Selection of a well contractor and construction of the wells was completed. Geophysical logging of the ASR pilot well was conducted and this information was reviewed and correlated with other well data. Construction and testing of the ASR pilot well was conducted as recommended in the Phase I Completion Report.

Phase III Aquifer Performance Tests and ASR Cyclic Testing: During this phase, aquifer performance tests were conducted to evaluate the hydraulic characteristics of the ASR storage zone. Three ASR cyclic tests were conducted during the pilot testing program. Each test included three phases: injection, storage and recovery. Data collected from the ASR well and three monitor wells during the cyclic tests included: injection and recovery flow rates, well pressures, water levels and water quality data.

Phase IV Data Compilation, Wellfield Modeling and Final Report: The fourth and final phase of the project included the compilation and analysis of data, wellfield hydraulic and solute transport modeling, and final project report preparation. This final report documents the information obtained during the pilot testing program, provides a conceptual ASR system design and presents recommendations concerning the construction of a full scale ASR system.

2.2 Background Information

The Lee County Regional Water Supply Authority Water Supply Master Plan (Master Plan) recommended implementation of a pilot project to evaluate the use of ASR to help meet future water demands in Lee County. Operational ASR systems would benefit the region by:

- Extending the supply capacity of existing wellfields and treatment facilities and deferring capital expenditures.
- Reducing the size of the proposed SR 82 regional water treatment facility required to meet peak demands.
- Reducing the peak day water supply requirements for future wellfields.
- Providing a significant volume of emergency water supply.

The Master Plan recommended development of ASR facilities based on the significant benefits this technology can provide. It concluded that construction of ASR systems would reduce the capital expenditures required to meet the project year 2030 needs in Lee County by $15,000,000. Every million gallons of ASR supply made available reduces the cost to meet the projected water demand by approximately $1,500,000.

The Master Plan narrowed the list of potential ASR facilities in Lee County to two sites considered most favorable, based on a review of the regional hydrogeology and other factors. The two top-ranked sites were the LCU - Corkscrew WTP, and the Bonita Springs - West Terry wellfield. Exploratory drilling was recommended to further evaluate the site-specific hydrogeologic characteristics of each site. The pilot ASR project was to be constructed at the site with the most favorable test results.

The exploratory drilling program was reduced to a single test site. The Corkscrew WTP site was selected for this pilot project based on regional hydrogeologic data, the availability of water supply facilities necessary to conduct the test, and Lee County's willingness to fund the local share of the project cost. The test site location is shown on Figures 2-1 and Figures 2-2.

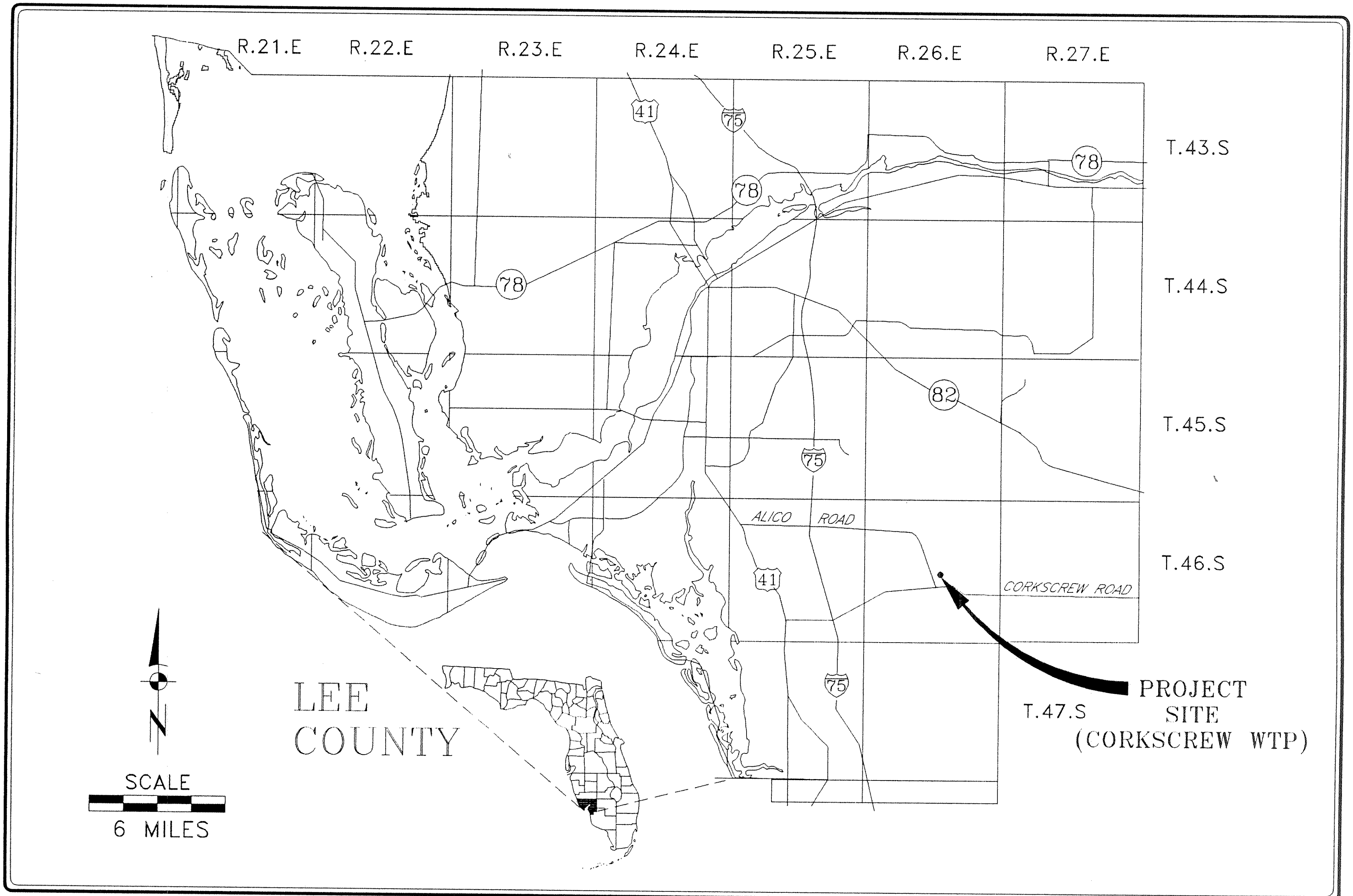

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FIGURE 2-1
MAP SHOWING PROJECT SITE LOCATION

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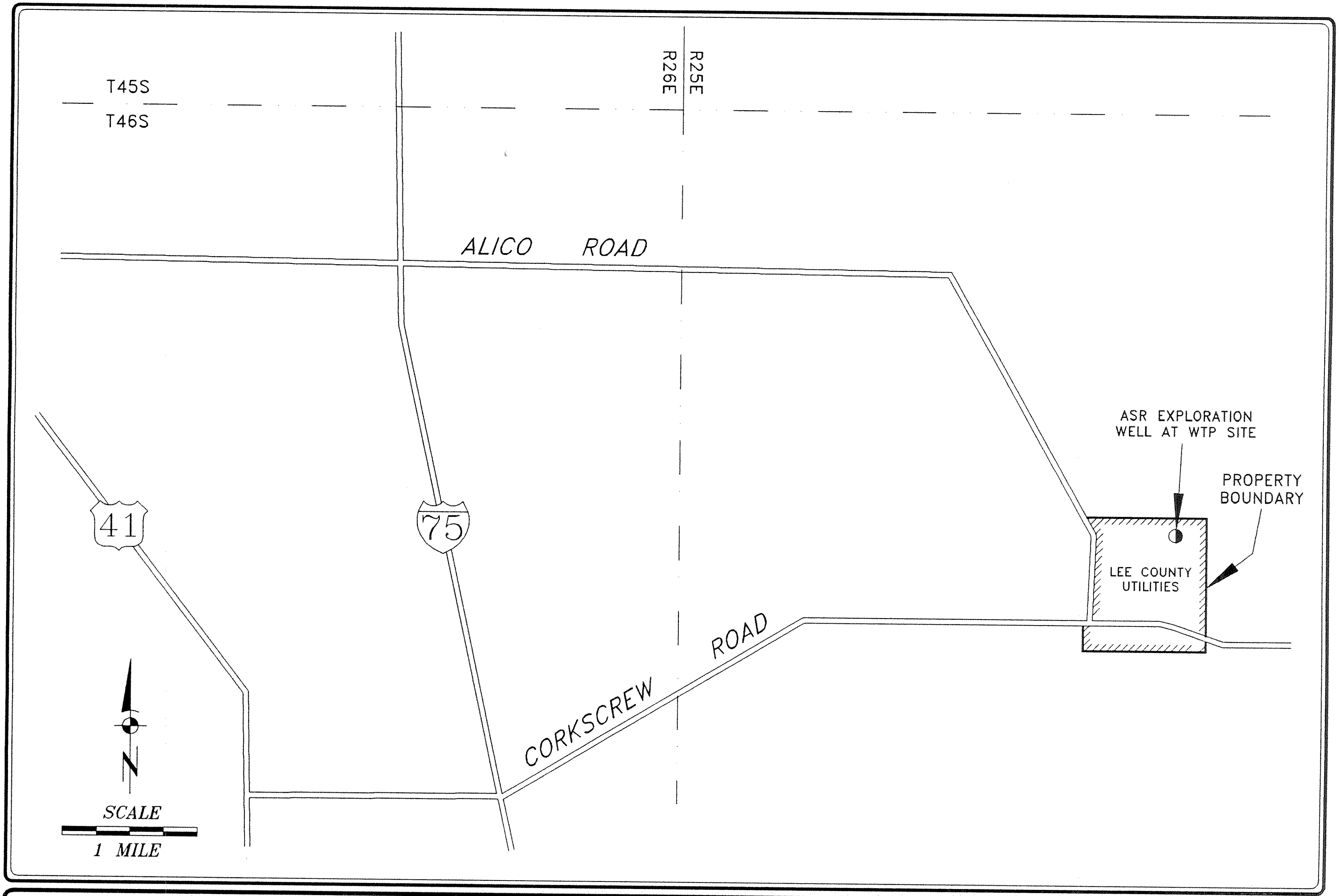

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FIGURE 2-2
MAP SHOWING LOCATION OF ASR EXPLORATION WELL

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3.0 HYDROGEOLOGY

3.1 Aquifer Designations and Description

The following summary of the hydrogeology of the project area is based on lithologic data collected during construction and testing of the ASR exploratory well and other wells installed for the pilot ASR project. Lithologic logs for these well borings are provided in Appendix A. The on-site data was supplemented with information provided in the Water Supply Master Plan and other sources of regional geologic data, as referenced below.

The principal aquifer systems in the area, in descending order, are the surficial aquifer system, the intermediate aquifer system, and the Floridan aquifer system. Feedwater for the Corkscrew WTP is currently supplied by six production wells that tap the Sandstone aquifer of the intermediate aquifer system, and 17 production wells completed in the surficial aquifer system. The ASR storage zone occurs within limestones of the intermediate aquifer system present about 120 feet below the base of the Sandstone aquifer. A hydrostratigraphic column for the area is illustrated on Figure 3-1. A west to east geologic cross-section that transects the test site is illustrated on Figure 3-2. The cross-section location is illustrated on Figure 3-3.

The surficial aquifer system (surficial aquifer) consists of a heterogeneous group of undifferentiated Pliocene to recent age sediments, and underlying Pliocene age limestones of the Tamiami Formation. The undifferentiated sediments in the upper portion of the aquifer include quartz sand, sandstone, shell beds, and shelly limestone. A unit of sandy clay, ranging from 0 to 15 feet thick, within the undifferentiated section retards vertical flow between the surficial sands and deeper sediments in the aquifer. The undifferentiated strata are about 35 feet thick near the ASR test site, but thicken to about 60 feet in the southeastern portion of the Corkscrew wellfield.

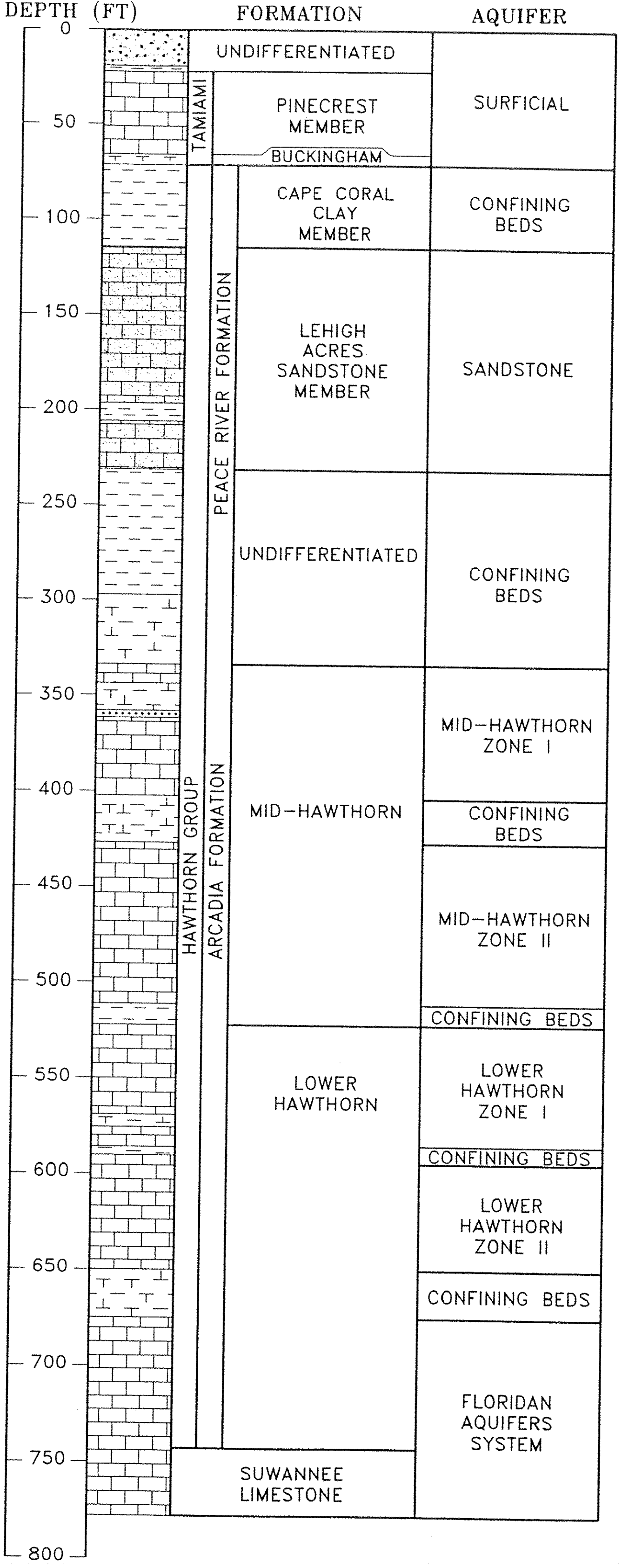

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FIGURE 3-1
GENERALIZED HYDROSTRATIGRAPHIC COLUMN
FOR THE ASR TEST SITE

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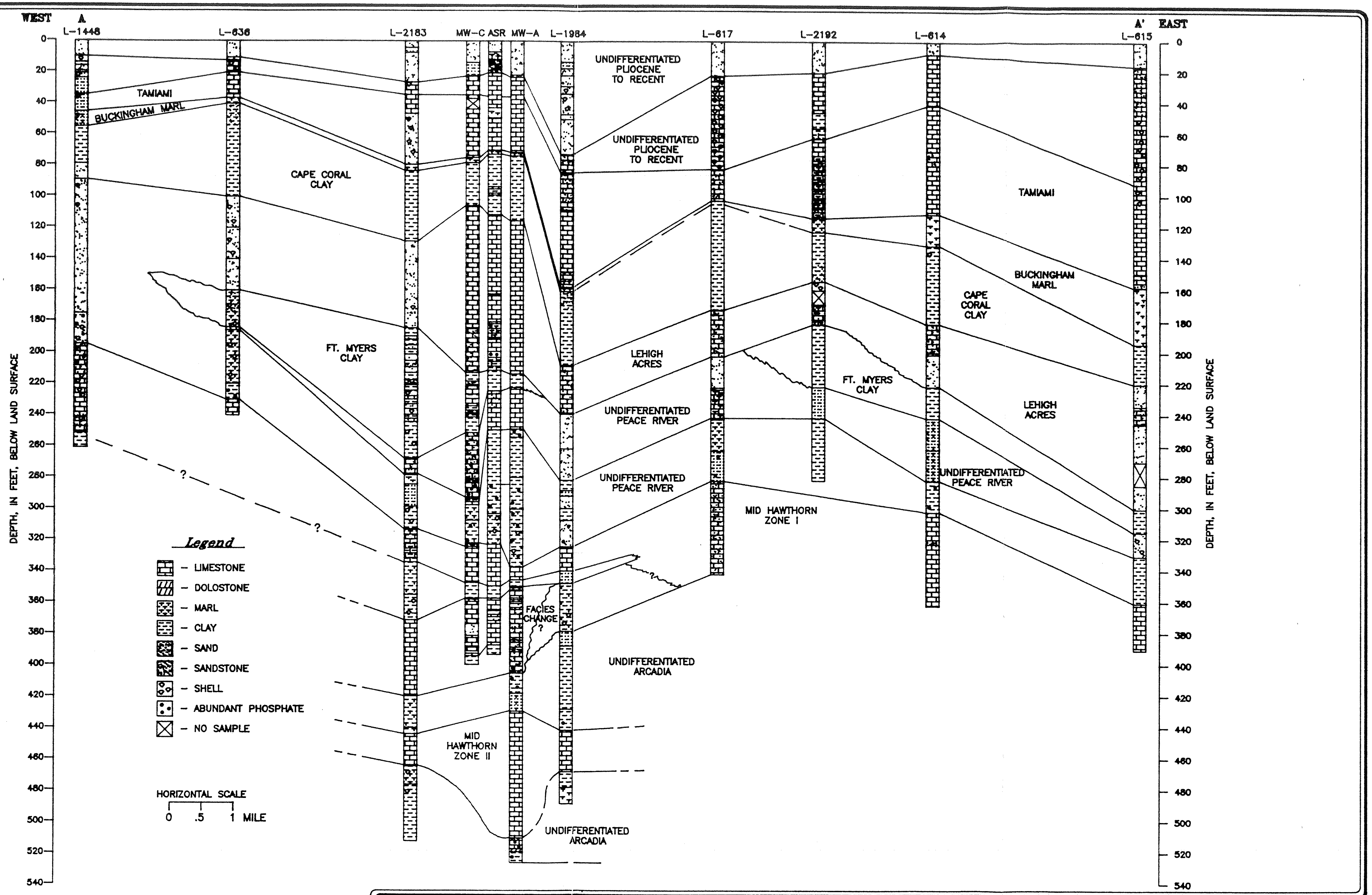

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FIGURE 3–2
WEST TO EAST GEOLOGIC CROSS–SECTION A–A'

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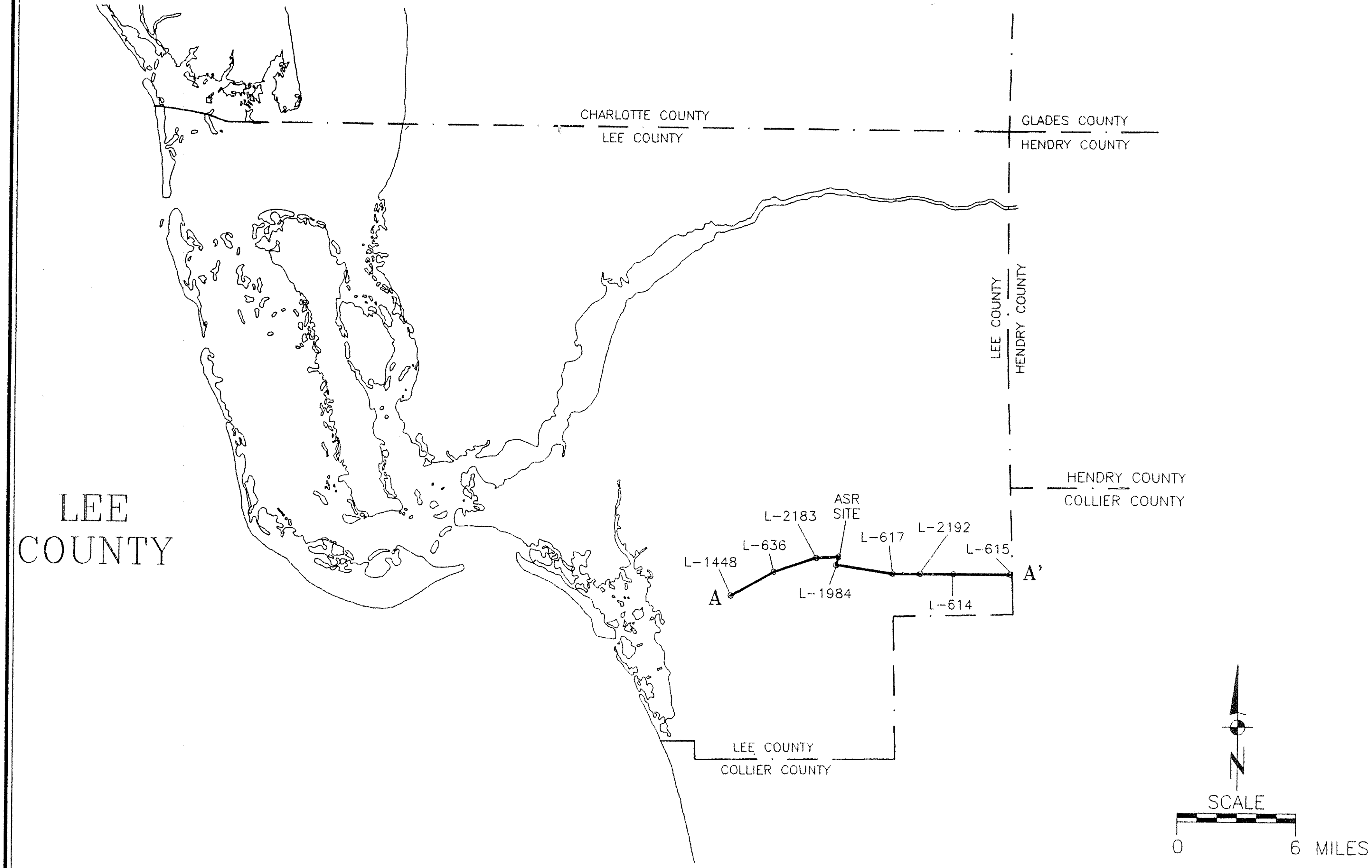

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FIGURE 3-3
CROSS SECTION LOCATION MAP

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The Tamiami Formation, which consists mainly of gray, moldic limestone in the area, forms the principal surficial aquifer production zone for the Corkscrew wellfield. Like the overlying undifferentiated sediments, the Tamiami Formation carbonates thicken from about 35 to 80 feet in a northwest to southeast direction across wellfield.

The basal sediments of the Tamiami Formation consist of 5 to 10 feet of gray marl, characteristic of the Buckingham marl member of the formation. This marl, and underlying dolomitic clays present in the upper portion of the Hawthorn Group, form the confining beds that separate the surficial aquifer from the uppermost aquifer in intermediate aquifer system. The total thickness of the upper Hawthorn confining beds ranges from 30 to 60 feet.

The intermediate aquifer system is a heterogeneous sequence of aquifers and confining beds that occur within Miocene age sediments of the Hawthorn Group. The Hawthorn Group in South Florida is divided into a predominantly clastic upper unit, termed the Peace River Formation, and a lower, predominantly carbonate unit, termed the Arcadia Formation (Scott, 1988)

The Sandstone aquifer is the uppermost aquifer of the intermediate aquifer system. It occurs within a heterogeneous group of Peace River Formation sediments that include calcareous sandstone, poorly indurated quartz sand, sandy limestone, and shell beds. The top of the Sandstone aquifer deepens from about 110 feet near the ASR test site, to over 200 feet in the southeastern portion of the Corkscrew wellfield. The base of the aquifer also deepens from 200 to 280 feet across the same area.

Data collected during construction of the ASR exploratory well indicate that five distinct water yielding zones are present in the Hawthorn Group below the Sandstone aquifer. The upper four of these zones are considered to represent subordinate aquifers of the intermediate aquifer system. The basal water yielding zone of the Hawthorn Group is in direct contact with the Suwannee Limestone Formation, and is therefore grouped within the Floridan aquifer system.

The ASR storage zone, referred to as mid-Hawthorn aquifer Zone I (Hawthorn Zone I), is the uppermost aquifer in the Hawthorn Group below the Sandstone aquifer. About 120 feet of marl, clay and sandy limestone separate the Sandstone aquifer from Hawthorn Zone I. Geologic data interpretation indicates that Hawthorn Zone I occurs within the uppermost sediments of the Arcadia Formation. The strongest support for this conclusion is that the unit directly underlies a phosphate rich "rubble zone", which is characteristic of the base of the Peace River Formation throughout the region.

Hawthorn Zone I at the ASR test site extends from about 330 to 395 feet below land surface. The phosphatic limestone that forms the aquifer is typically gray, with good to moderate porosity. Quartz sand content of the limestone ranges from absent to 30%. Lenses of quartz sand up to several feet thick are interbedded with the limestone at some well sites. A unit of moderately cohesive lime mud and clay, about 10 feet thick, occurs near the center of the aquifer. Geologic data suggest that the clayey unit thickens to the west of the test site, and may subdivide Hawthorn Zone I into two distinct carbonate units.

A USGS observation well (L-1984) is located about 4,500 feet southeast of the ASR test site. Lithologic data for L-1984 suggest that in this area, sediments underlying the clayey unit in Hawthorn Zone I consist of marl, rather than limestone. This facies change, if correctly identified, could signify a substantial decrease in transmissivity directly east of the test site. Further east, Hawthorn Zone I limestones are reported to be at least 60 feet thick, as indicated on Figure 3-2.

An isopach map for Hawthorn Zone I is illustrated on Figure 3-4. This figure was prepared using information compiled for the Water Supply Master Plan and more

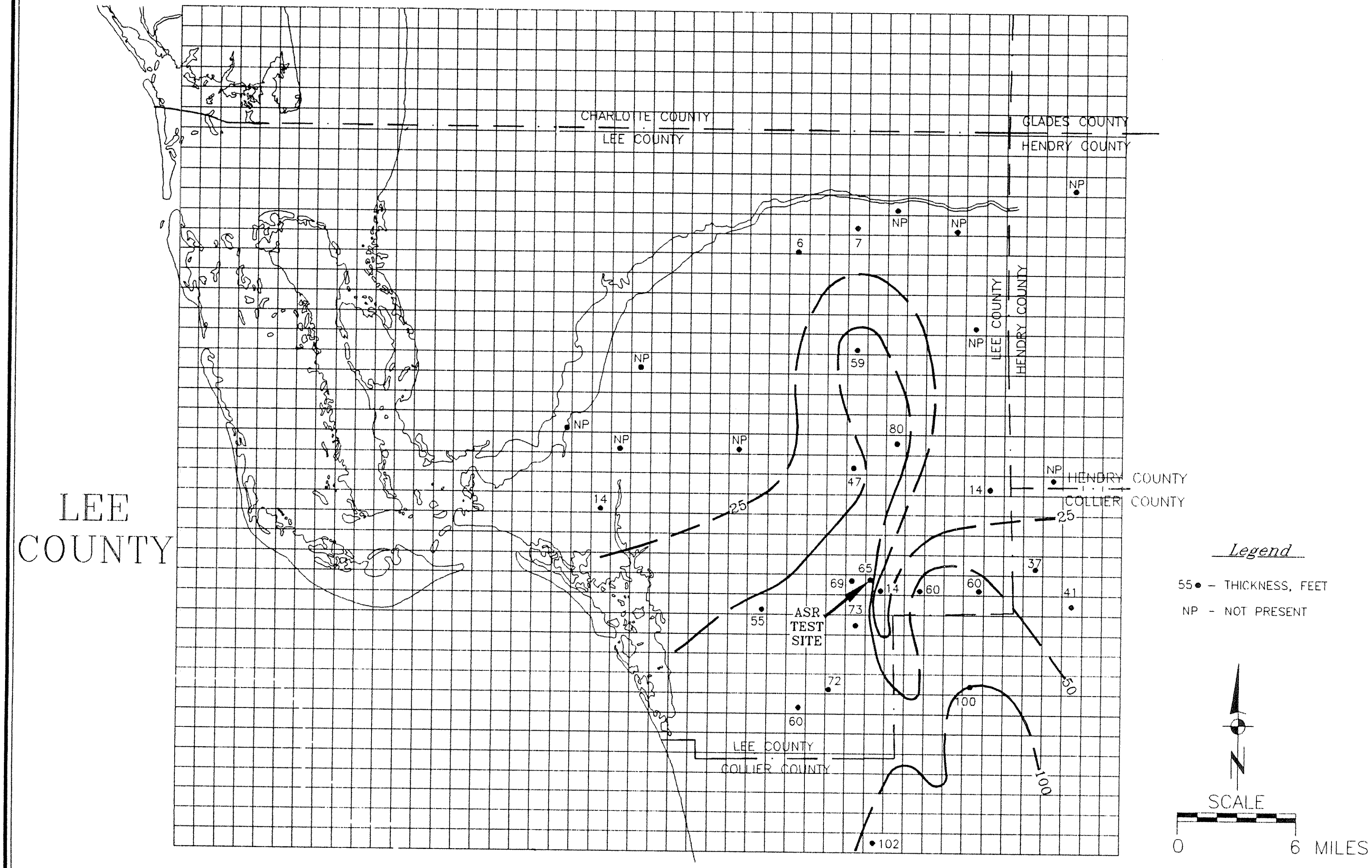

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FIGURE 3-4
ISOPACH MAP FOR ASR STORAGE ZONE

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recent data. Thinning of the storage zone east of the test site is inferred from the data for L-1984. Thinning, and the ultimate pinchout of Hawthorn Zone I in the northwestern portion of Lee County is based on the conservative interpretation that the storage zone is stratigraphically separate from the mid-Hawthorn carbonates used for domestic self-supply in the cities of Fort Myers and Cape Coral. It is more likely that a considerable degree of hydraulic connection actually exists between these units.

About 30 feet of lime mud and clay separates Hawthorn Zone I from the underlying mid-Hawthorn aquifer Zone II (Hawthorn Zone II). The limestones that form Hawthorn Zone II are 60 to 80 feet thick, and lithologically similar to Hawthorn Zone I, except that porosity in Zone II tends to be lower.

Three other water bearing units in the Arcadia Formation were identified during construction of the ASR exploratory well, as illustrated on Figure 3-1. These units, in descending order, are informally referred to as lower Hawthorn Zone I, lower Hawthorn Zone II, and the basal lower Hawthorn aquifers. The rocks that form each of the three units are described in the exploratory well completion report as micritic to biomicritic limestones with variable porosity. The basal Hawthorn limestones directly overly the Suwannee Limestone Formation, and are therefore included in the Floridan aquifer system.

The top of the Suwannee Limestone Formation was penetrated at a depth of 744 feet in the ASR exploration well. The unit is described as slightly dolomitic, calcarenite with good moldic and intergranular porosity.

3.2 Test Well Construction

Drilling operations for the pilot ASR project commenced on July 28, 1994, when construction of the ASR exploratory well was initiated. The purpose of the exploratory well was to characterize the stratigraphy and water quality at the test site, and to provide initial estimates of the hydraulic properties for potential storage zones. Test drilling was terminated at a depth of 778 feet, in the upper portion of the Suwannee Limestone Formation of the Floridan aquifer system. Data collected during construction of the exploratory well were used to select the ASR storage zone, and to design the test production well and monitor wells for the project.

Hawthorn Zone I of the intermediate aquifer system was selected as the primary storage zone based on favorable geologic characteristics and preliminary estimates of hydraulic properties. Ambient water quality in the aquifer was found to be unexpectedly good, with chloride ion and total dissolved solids (TDS) concentrations in the range of 50 and 360 milligrams/liter (mg/l). The option of storing water in both Hawthorn Zone I and an in underlying aquifer of the Arcadia Formation (Hawthorn Zone II) was also under consideration. Additional testing of Hawthorn Zone II was conducted during construction of the ASR monitor wells, as part of the Phase II work scope. Storage in Hawthorn Zone II was eliminated from consideration, because the transmissivity of this aquifer was found to be unacceptably low.

Three monitor wells and one test production well (ASR well) were constructed for the project. Well locations are illustrated on Figure 3-5. Schematic construction details are depicted on Figure 3-6. The ASR well and monitor well MW-C fully penetrate the storage zone. The ASR exploratory well was converted to a monitor well (MW-A) by backplugging a portion of the open hole section. The well is now completed in the storage zone only, but was originally open to Hawthorn Zones I and II while combined storage in both zones was being considered. The two storage zone monitor wells (MW-A and MW-C) form an east-west trending line with the test production well located between them. Monitor well MW-B fully penetrates Hawthorn Zone II, and is located about 240 feet south of MW-C. An existing

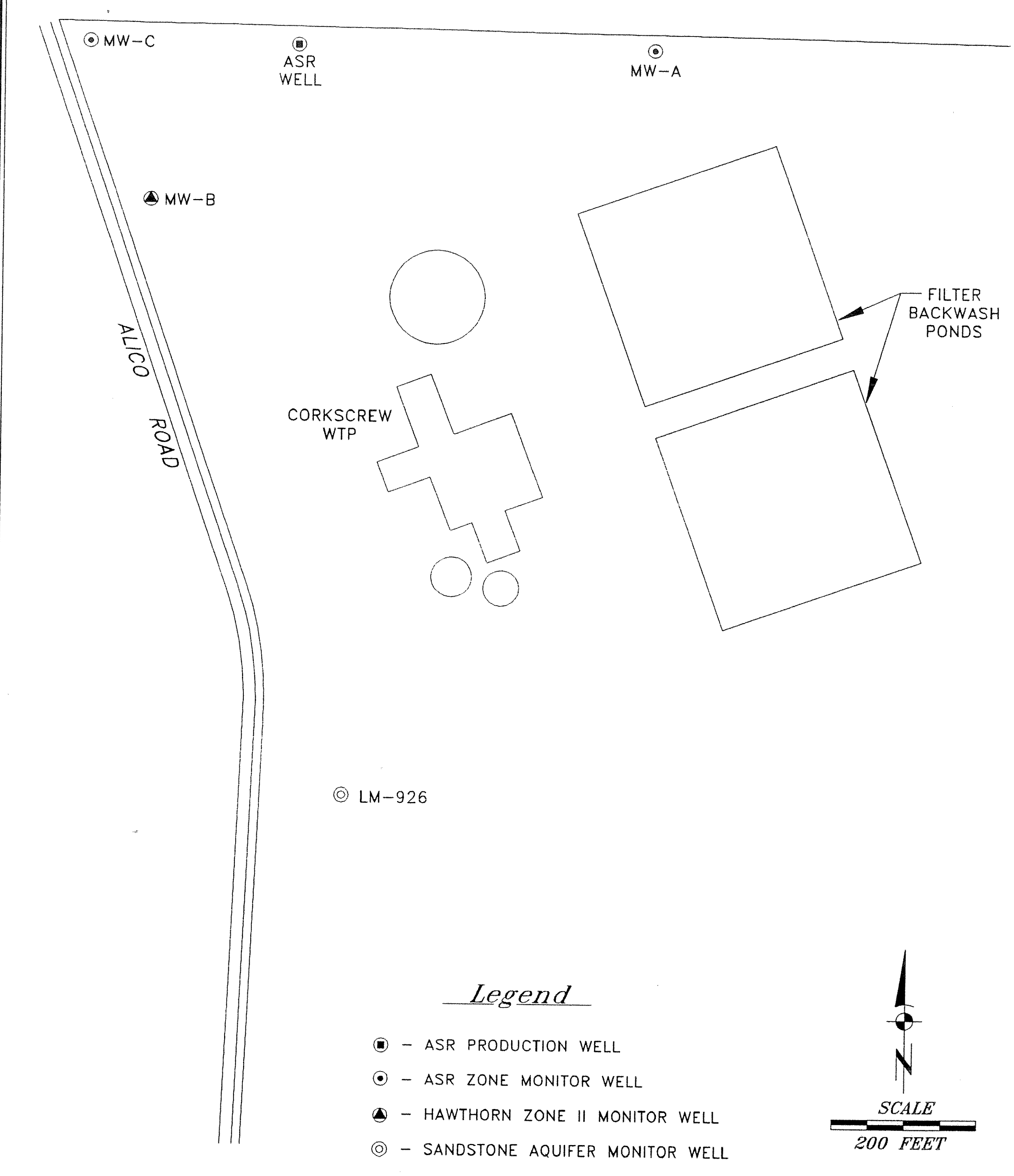

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FIGURE 3-5
ASR TEST SITE LOCATION MAP

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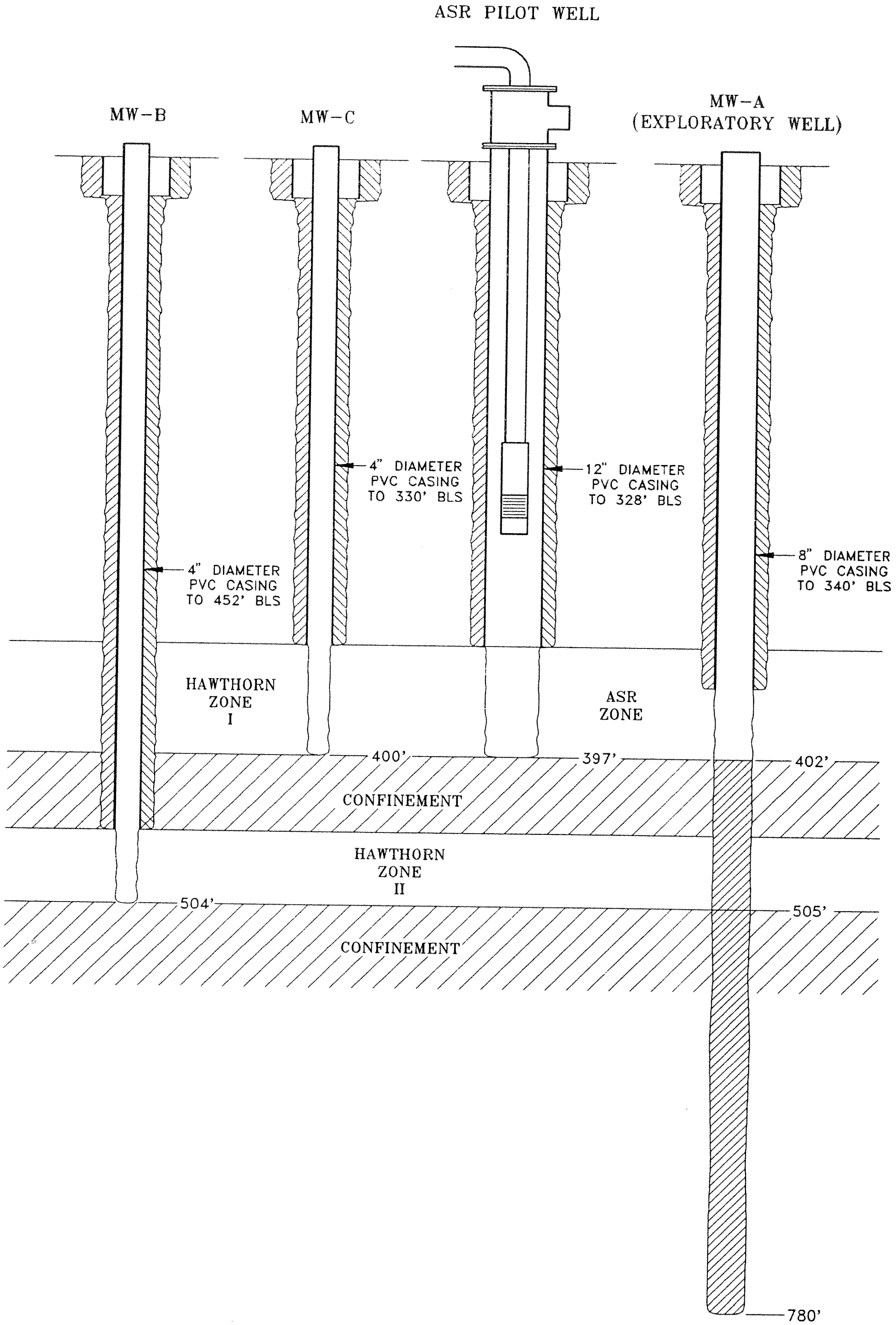

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FIGURE 3-6
SCHEMATIC DIAGRAM OF PILOT ASR AND MONITOR WELL CONSTRUCTION DETAILS

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monitor well at the Corkscrew wellfield (LM-926) was used to monitor conditions in the Sandstone aquifer during the cyclic tests.

Preparations for pilot testing were completed by installing a submersible pump (Grundfos, Model 375S250-4) in the ASR well at a depth of 146 feet below grade. Electrical service was supplied to the site to operate the well pump and ancillary equipment needed for the tests.

3.3 Native Groundwater Quality

Water samples for chloride ion analyses were collected from the reverse air discharge during construction of the ASR exploratory well. Reported chloride concentrations in the storage zone, Hawthorn Zone II, and lower Hawthorn Zone 1 were all less than 50 mg/l. Salinity in the lower portion of the intermediate aquifer system increased slightly, with reported chloride concentrations ranging from 100 to 140 mg/l.

Four sets of groundwater samples were collected from the ASR well discharge during an aquifer performance test conducted in September 1995 (refer to Section 3.4). The samples were analyzed for a comprehensive list of chemical parameters, to help characterize water quality in the production zone. These data are summarized in Table 3-1. Copies of the laboratory analytical reports are provided in Appendix B.

Storage zone monitor well MW-A was sampled on April 4, 1997 for analysis of all inorganic, primary and secondary drinking water standard parameters, per Chapter 62-550 of the Florida Administrative Code (FAC). Results of these analyses are summarized in Table 3-2. The applicable drinking water standard for each analyte is also listed on Table 3-2 for reference.

TABLE 3-1

NATIVE GROUNDWATER QUALITY SUMMARY
AQUIFER PERFORMANCE TEST I (APT) SAMPLES

ANALYTE	UNIT	SAMPLE APT-1 SEPT. 6, 1995	SAMPLE APT-2 SEPT. 7, 1995	SAMPLE APT-3 SEPT. 8, 1995	SAMPLE APT-4 SEPT. 9, 1995
ALKALINITY, TOTAL	mg/l	218	224	222	219
ALKALINITY, BICARBONATE	mg/l	218	224	221	219
ALKALINITY, CARBONATE	mg/l	1	0	0	1
HARDNESS, TOTAL	mg/l	140	140	130	154
HARDNESS, NONCARBONATE	mg/l	0	0	0	0
HARDNESS, CARBONATE	mg/l	140	140	130	154
HARDNESS, CALCIUM	mg/l	110	106	114	122
CALCIUM	mg/l	17.1	16.1	16.2	15.8
MANGANESE	mg/l	<0.01	<0.01	<0.01	0.01
IRON	mg/l	0.03	<0.03	<0.03	<0.03
SILICA	mg/l	16	16	38	32
SULFATE	mg/l	8	11	9	18
SULFITE	mg/l	<2	<2	<2	<2
CHLORIDE	mg/l	41	42	44	39
FLUORIDE	mg/l	1.3	1.3	1.4	1.4
PHOSPHORUS, ORTHO	mg/l	0.076	<0.001	<0.001	0.022
TDS	mg/l	312	331	335	336
SPEC. CONDUCTANCE	umhos/cm	584	600	NA	NA
COLOR	CU	14	15	15	13
pH	pH units	7.58	7.05	7.39	7.58
pHs	Units	7.401	7.385	7.384	7.371
RYZNAR INDEX	Units	7.222	7.72	7.378	7.162
LANGELIER INDEX	LU	0.179	-0.335	0.006	0.209
GROSS ALPHA	pCi/l	2.6	1.6	2.7	2.1
THMs, TOTAL	ug/l	<0.5	<0.5	<0.5	<0.5
COLIFORM BACTERIA	colonies/100 ml	None Detected	None Detected	NA	NA

NA - NOT ANALYZED

TABLE 3-2

NATIVE GROUNDWATER CHARACTERIZATION
MONITOR WELL MW-A
SAMPLES COLLECTED APRIL 4, 1997

ANALYTE	UNIT	REPORTED CONCENTRATION	MAXIMUM CONCENTRATION *	APPLICABLE STANDARD
Antimony	ug/L	<3	6	Primary
Arsenic	ug/L	<1	50	Primary
Barium	mg/L	0.02	2	Primary
Beryllium	ug/L	<0.2	4	Primary
Cyanide	mg/L	<0.005	0.2	Primary
Fluoride	mg/L	2.45	4	Primary
Lead	ug/L	115, 114 **	15	Primary
Mercury	ug/L	<0.2	2	Primary
Nitrate	mg/L	<0.01	10	Primary
Nitrite	mg/L	<0.001	1	Primary
Nitrate + Nitrite	mg/L	<0.01	10	Primary
Sodium	mg/L	98.5	160	Primary
Thallium	ug/L	<1	2	Primary
Cadmium	ug/L	<0.1	5	Primary
Chromium	ug/L	<1	100	Primary
Nickel	ug/L	<1	100	Primary
Selenium	ug/L	<2	50	Primary
Aluminum	mg/L	0.01	0.20	Secondary
Chloride	mg/L	42	250	Secondary
Copper	mg/L	<0.01	1	Secondary
Fluoride	mg/L	2.45	2.00	Secondary
Iron	mg/L	0.08	0.30	Secondary
Manganese	mg/L	0.01	0.05	Secondary
Silver	mg/L	<0.01	0.10	Secondary
Sulfate	mg/L	34	250	Secondary
Zinc	mg/L	<0.01	5	Secondary
Color	CU	8	15	Secondary
Odor	TON	<1	3	Secondary
pH	pH Units	8.5	6.5-8.5	Secondary
TDS	mg/L	348	500	Secondary
Surfactants	mg MBAS/L	0.03	0.50	Secondary

* DRINKING WATER STANDARD PER FAC 62-550

** RESAMPLED 4/22/97

Laboratory analytical results confirm that the ambient groundwater quality of the storage zone at the test site is unexpectedly good, when compared with other data available for the mid-Hawthorn aquifer. Reported chloride ion and TDS concentrations at the test site average 50 and 360 mg/l, respectively, while total alkalinity and total hardness concentrations average 230 and 145 mg/l.

The reported concentrations of all but two analytes in the sample collected from MW-A are significantly less than the applicable primary or secondary drinking water standard criteria. Lead was reported above the primary standard of 15 micrograms/liter (ug/l), and fluoride slightly exceeded the secondary standard of 2 mg/l, but was below the primary standard of 4 mg/l. The unexpectedly high concentration of lead (115 ug/l) reported for MW-A is believed to be a result of lead introduced during drilling operations (pipe thread grease), and is not considered to be indicative of conditions in the aquifer. As described in Section 4.2, reported lead concentrations in the recovered water samples collected during the pilot test were 2 ug/l or less.

Additional information regarding the native groundwater quality of the storage zone and adjacent aquifers is provided in Section 4.3.

3.4 Aquifer Testing

Aquifer Test Procedures

Initial estimates of potential storage zone transmissivity were made from specific capacity and step-drawdown test data collected during construction of the ASR exploratory well, as described in the completion report. Short-term pumping tests were also conducted at monitor wells MW-B and MW-C during Phase II of the project. Results of these preliminary aquifer tests are summarized in Table 3-3. Test results yielded transmissivity estimates for the storage zone ranging from

TABLE 3-3

AQUIFER PARAMETER ESTIMATES
PROJECT PHASES I AND II

Aquifer/Zone	**Interval (ft) below land surface**	**Net Thickness (ft)**	**Est. Transmissivity (gpd/ft)**	**Estimated Horizontal Hydraulic Conductivity (gpd/ft^2)**
ASR Exploration Well Data				
Mid Hawthorn Zone I	336-404	61	10,000	170
Mid Hawthorn Zone II	428-515	85	4,000	45
Lower Hawthorn Zone I	524-589	60	4,000	50
Suwannee*	Partial Penetration (34 of an estimated 240 feet)	--	96,000	--
Monitor Wells MW-B and MW-C Data				
Mid Hawthorn Zone I	330-400	70	20,000-27,000	285-385
Mid Hawthorn Zone II	452-504	52	900-1,200	17-23

*Also includes basal portion of Lower Hawthorn (693 to 744 feet) with which it is in contact

10,000 to 27,000 gpd/ft, and a value of about 1,000 gpd/ft for underlying Hawthorn Zone II.

Two aquifer performance tests (APTs) were conducted after the ASR well and monitor wells had been constructed to provide more accurate estimates of storage zone hydraulic properties. The first APT (APT 1) was conducted in September 1995. Monitor well MW-A was completed in both Hawthorn Zones I and II at that time. A second APT (APT 2) was conducted in June 1996, after MW-A had been backplugged to the base of Hawthorn Zone I, to evaluate aquifer characteristics in the eastern portion of the test site.

Detailed test procedures and results for APT 1 were reported in a Technical Memorandum issued by the project team on January 9, 1996. A copy of that memorandum is provided in Appendix C. The test was performed by pumping the ASR well at a constant rate of 400 gpm for a period of 115.5 hours (4.8 days). Water levels in monitor wells MW-B and MW-C were continuously monitored using a data logger and pressure transducers. Continuous water level data was also collected at MW-A using a Steven Type F water level recorder. Periodic measurements of drawdown in the ASR well, and confirmatory water levels in the monitor wells were made using an electronic water level indicator. Water samples from the pump discharge were collected at predetermined intervals for field and laboratory analyses (refer to Section 3.3). Recovery data was collected for up to seven days after completing the withdrawal portion of the test, using the same instrumentation described above.

APT 2 was initiated on June 4, 1996, at the start of the Cycle 2 recovery phase. The ASR well was pumped at an average rate of 415 gpm for 120 hours (5.0 days). Some fluctuation in the discharge rate occurred during the test. Recorded withdrawal rates ranged from 400 to 430 gpm. Water levels in monitor wells MW-A and MW-C were continuously monitored using a data logger and pressure transducers. The test was terminated prematurely on June 9, 1996 when a staff member at the Corkscrew WTP deactivated the pump. This occurred during a weekend, without prior notification. Therefore, no useful recovery data was recorded.

Aquifer Test Analysis Results

Hawthorn Zone I is a semi-confined aquifer separated from overlying and underlying aquifers by low permeability sediments. Water removed from a semi-confined aquifer is in response to a decline in the reservoir pressure. As the pressure in the aquifer is reduced, water is contributed by a release from aquifer storage, and by leakage through the confining beds from adjacent aquifers (source beds). During the early stages of pumping, water released from storage in the confining beds may also contribute to the total withdrawal.

There are generally three hydraulic coefficients computed in the evaluation of a semi-confined aquifer. Transmissivity (T) is used to quantify the rate at which water is transmitted through a unit width of the aquifer under a unit hydraulic gradient. The storage coefficient (S) is defined as the volume of water an aquifer releases from or takes into storage, per unit surface area of aquifer, per unit change in hydraulic head normal to that surface. The leakance coefficient (L) is used to quantify the rate at which water will leak into the pumped aquifer through the adjacent confining beds. Leakance is equal to the vertical hydraulic conductivity of the confining bed divided by the confining bed thickness.

Time and drawdown data in tabular form for APT 1 are included in the January 9, 1996 Technical Memorandum. The raw data for APT 2 are provided in Appendix C. Results of all on-site APT data analyses are summarized in Table 3-4.

TABLE 3-4

AQUIFER PERFORMANCE TEST ANAYSIS SUMMARY

	TRANSMISSIVITY (T) (GPD/FT)	STORAGE COEFFICIENT (Sc)	LEAKANCE (L) (GPD/FT^3)
APT 1, Theis, Drawdown, MW-C	25,500	7.7 x 10^{-5}	1.2 x 10^{-4}
APT 1, Cooper-Jacob, Drawdown, MW-C	25,150	6.6 x 10^{-5}	ND
APT 1, Theis, Recovery, MW-C	25,900	ND	ND
APT 2, Theis, Drawdown, MW-C	23,800	5.7 x 10^{-5}	ND
APT 2, Cooper-Jacob, Drawdown, MW-C	25,500	4.9 x 10^{-5}	ND
APT 2, Theis, Drawdown, MW-A	13,200	2.3 x 10^{-4}	ND
APT 2, Cooper-Jacob, Drawdown, MW-A	14,200	1.7 x 10^{-4}	ND
Average, All Analyses MW-C MW-A	25,200 13,700	6.2 x 10^{-5} 2.0 x 10^{-4}	1.2 x 10^{-4} gpd/ft

A preliminary analysis of the drawdown data was made using the Jacob straight line method (Cooper and Jacob, 1946; Jacob, 1950). This method yields approximate values of transmissivity and storage coefficient, and serves as a useful check on the validity of other test evaluation methods.

The primary drawdown data analyses was made by matching a logarithmic plot of the time and drawdown data to one of a family of type curves published by Cooper (1963). This curve matching approach is used to solve the non-steady state flow equations for leaky confined aquifers proposed by Hantush and Jacob (1955). The aquitard storage contribution is considered negligible in this solution (Kruseman and de Ridder, 1994). The equations used in the test analyses are included in Appendix C.

Semi-log plots of drawdown versus time for both APTs that illustrate selected Jacob solution variables are depicted on Figures C-1 through C-3, Appendix C. Calculated transmissivity values, based on the drawdown data collected from MW-C, range from 25,150 to 25,500 gpd/ft. Data analysis for MW-A yields a considerably lower transmissivity estimate of 14,200 gpd/ft.

Logarithmic plots of the time and drawdown data showing selected type curves and solution match points for all tests are illustrated on Figures C-4 through C-6, Appendix C. Transmissivity estimates developed from the curve matching solution are similar to those calculated by the straight line method; with values ranging from 25,500 to 25,900 gpd/ft at MW-C, and about one-half that value at MW-A (13,200 gpd/ft).

Storage coefficient estimates derived from both the curve matching and straight line solutions methods for MW-C are similar, with calculated values ranging from 4.9×10^{-5} to 7.7×10^{-5}. Estimated values for MW-A are higher, ranging from 1.7×10^{-4} to 2.3×10^{-4}.

During APT 1, drawdown at monitor well MW-C began to depart onto a leakance curve about 2,100 minutes into the test. At that time, approximately 3.5 feet of drawdown had been recorded at monitor well MW-B, completed in Hawthorn Zone II. Curve matching analysis of the data indicates a leakance value of about 1.2 X 10^{-4} gpd/ft^3 for the ASR storage zone at the test site. The site geology, and water level data collected during the APTs and cyclic tests, suggest that most leakage occurs between the storage zone and Hawthorn Zone II.

Drawdown at MW-C during APT 2, dropped below the Theis curve from 150 to 3,800 minutes into the test. This is apparently due to a 15 gpm decrease in the pumping rate. The second departure below Theis, that occurred about 7,000 minutes into the test, is believed to be in response to vertical leakage. Unfortunately, the test ended prematurely, so the data recorded was insufficient to estimate the leakance coefficient.

Drawdown at MW-A began to depart above the Theis curve approximately 500 minutes into APT 2. This trend continued until the end of the test. Data evaluation suggests that the change in slope of the drawdown trend line (refer to Figure C-7, Appendix C) reflects the cone of depression intersecting an aquifer boundary. The theoretical distance to the boundary, based on the analysis illustrated on Figure C-7 (Todd, 1976), is about 3,800 feet. Drawdown at MW-C during APT 1 temporarily departed above the Theis curve at about the same test duration as the response in MW-A during APT 2 (refer to Figure C-1, Appendix C). This response at MW-C supports the boundary concept. If the boundary interpretation is correct, the leakance coefficient at MW-C is actually greater than that calculated, since the leakance and boundary effects are opposing.

The recovery data from MW-C during APT 1 were evaluated using the straight line analysis method proposed by Theis (1935). In this method, residual drawdown is plotted versus the total test time divided by the recovery time. Transmissivity is estimated from the slope of the best fit line to the data, as illustrated on Figure C-8, Appendix C. The calculated transmissivity of 25,900 gpd/ft is in good agreement with other results for MW-C.

4.0 ASR CYCLIC TESTING

4.1 General Cyclic Testing Procedures

Specific Condition 7 of the Class V injection permit issued by the Florida Department of Environmental Protection (FDEP) for the pilot ASR project required submittal and FDEP approval of an operational plan prior to conducting long-term system testing. This plan was submitted by the project team on October 13, 1995, and approved by the FDEP in a letter dated October 18, 1995. Copies of the plan and the FDEP approval letter are provided in Appendix D.

The operational plan specified the number, expected duration, and general procedures for the ASR cyclic tests to be performed. Guidelines were specified regarding the type and frequency of field and laboratory analytical data that would be collected to monitor system performance and to track the influence of ASR operations on the storage zone and adjacent aquifers. The plan was structured into three test cycles; an initial short-duration test designed to quickly identify operational problems, followed by two longer-term tests with storage and recovery volumes representative of anticipated ASR wellfield operational conditions. Each test cycle consisted of an injection phase, a storage phase, and a recovery phase. The procedures implemented during the cyclic tests closely followed those proposed in the plan, with minor modifications made to respond to field conditions.

Cycle 1: Cycle test #1 (Cycle 1) was conducted between October 25 and November 14, 1995. The test consisted of 7 days of injection, 1 day of storage, and 10 days of recovery. The recovery volume of nearly three million gallons exceeded the injection volume of just over two million gallons by about 50% to help define the mixing characteristics of the injected and native fluids.

Cycle 2: The injection phase of Cycle test #2 (Cycle 2) was conducted between February 14 and April 30, 1996. During that 76 day period, 31.3 million gallons (mg) of water was injected into the storage aquifer at a rate of approximately 300 gallons/minute (gpm). The injected water volume for the test was designed to produce a stored water "bubble" that extended beyond monitor well MW-C, to assist in evaluating storage zone transport processes and their rates. The injection volume proved to be more than adequate for this purpose.

The recovery phase of Cycle 2 was initiated on June 4, 1996 after 35 days of storage. Recovery was suspended between July 15 and September 13, 1996, while an alternate means of water discharge was implemented. Prior to July 15, 1996, recovered water was discharged to one of the filter backwash ponds at the WTP. The plant backpumps water from the ponds into the raw water treatment system. The ponds experience increased turbidity and color levels during the spring and summer months. The alternate discharge was implemented to minimize the amount of backpumping required.

The recovery phase of Cycle 2 was completed on October 4, 1996. The total volume of water recovered was 22.8 mg, or approximately 73% of the injected volume. The residual water maintained in storage was intended to begin forming an injected water buffer in the aquifer.

Cycle 3: Cycle test #3 (Cycle 3) was initiated on October 7, 1996, three days after completing the Cycle 2 test. The basic approach followed in Cycle 3 was similar to that of Cycle 2. A volume of 26.1 mg of water was injected into the storage zone at a rate of 300 gpm, during a 63-day period. Recovery was initiated on January 9, 1997 after 30 days of storage. The recovered water volume, of 19.8 mg, was again established at 75% of the injected volume, to enlarge the buffer maintained in storage. The recovery phase of Cycle 3 was completed on February 12, 1997.

Cycle 3 injection began with a residual storage volume of 8.5 mg of water remaining in the aquifer following Cycle 2. Therefore, by evaluating the recovery data for Cycle 3, the project team was able to identify changes in recovered water quality during a repetitive ASR cycle with some buffer already developed. Cycle 3 was also used to further evaluate the effectiveness of the ASR well maintenance procedures described in Section 4.4.

4.2 ASR Well Water Quality Trends

Water samples were collected from designated ports installed on the ASR well piping during all phases of the cyclic tests, in accordance with the testing plan. These samples were analyzed by a combination of field and laboratory methods to monitor injected, stored, and recovered water quality trends. These data were used to evaluate the suitability of the recovered water for public supply after disinfection. The pilot test results were also used to help evaluate ASR recovery efficiency and the mixing characteristics of the injected and native fluids. All laboratory analyses for the project were performed by Lee County Environmental Laboratory, Fort Myers, Florida. Copies of the laboratory analytical reports are provided in Appendix B.

Water quality data for all test cycles are summarized in Tables 4-1 and 4-2, Appendix E. Laboratory analytical results for a sample of the recovered water collected at the conclusion of Cycle 3 on April 4, 1997, which was analyzed for all primary and secondary drinking water standard parameters and unregulated contaminants are summarized in Table 4-3. The applicable standard for each analyte is also listed on Table 4-3 for reference. The reported concentrations of all inorganic, primary and secondary drinking water standard parameters were all well below applicable standards. None of the organic contaminants analyzed were detected.

TABLE 4-3

RECOVERED WATER CHARACTERIZATION
SAMPLES COLLECTED APRIL 4, 1997

ANALYTE	UNIT	REPORTED CONCENTRATION	MAXIMUM CONCENTRATION *	APPLICABLE STANDARD
Antimony	ug/L	<3	6	Primary
Arsenic	ug/L	9	50	Primary
Asbestos	Mf/L	<0.19	7	Primary
Barium	mg/L	0.01	2	Primary
Beryllium	ug/L	<0.2	4	Primary
Cyanide	mg/L	<0.005	0.2	Primary
Fluoride	mg/L	0.57	4	Primary
Lead	ug/L	<1, 2 **	15	Primary
Mercury	ug/L	<0.2	2	Primary
Nitrate	mg/L	<0.01	10	Primary
Nitrite	mg/L	<0.001	1	Primary
Nitrate + Nitrite	mg/L	<0.01	10	Primary
Sodium	mg/L	40.1	160	Primary
Thallium	ug/L	<1	2	Primary
Cadmium	ug/L	<0.1	5	Primary
Chromium	ug/L	<1	100	Primary
Nickel	ug/L	<1	100	Primary
Selenium	ug/L	<2	50	Primary
Aluminum	mg/L	0.02	0.20	Secondary
Chloride	mg/L	42	250	Secondary
Copper	mg/L	<0.01	1	Secondary
Fluoride	mg/L	0.57	2.00	Secondary
Iron	mg/L	0.20	0.30	Secondary
Manganese	mg/L	<0.01	0.05	Secondary
Silver	mg/L	<0.01	0.10	Secondary
Sulfate	mg/L	30	250	Secondary
Zinc	mg/L	0.02	5	Secondary
Color	CU	15	15	Secondary
Odor	TON	<1	3	Secondary
pH	pH Units	7.3	6.5-8.5	Secondary
TDS	mg/L	220	500	Secondary
Surfactants	mg MBAS/L	0.05	0.50	Secondary

* DRINKING WATER STANDARD PER FAC 62-550

** RESAMPLED 4/22/97

NOTE: ALL ORGANIC CONTAMINANTS REPORTED BELOW DETECTION LIMIT

Analytical results indicate that aquifer storage at the test site can supply a high quality water for public use. Reported alkalinity, hardness, and TDS concentrations for the recovered water samples collected during the two longer-term recovery cycles range from 61 to 147 milligrams/liter (mg/l), 104 to 138 mg/l, and 176 to 294 mg/l, respectively. Reported chloride ion concentrations range from 44 to 80 mg/l, with an average value of 56 mg/l for all recovery cycles. In general, the chemical characteristics of the water recovered during Cycle 3 were more like the injectate than during earlier cycles. This freshening trend with time is apparently due to the combined influence of the stored water buffer developed during Cycle 2 and to a decrease in salinity of the injected water during Cycle 3.

Phosphate, which is typically enriched in radionuclides, is common in Hawthorn Group sediments. Gross alpha activity of the injected and recovered fluids was monitored during the cyclic tests to test for evidence of radionuclide dissolution during aquifer storage. Reported gross alpha activity of the injected water was consistently non-detectable (typically less than 1.2 picocuries/liter (pCi/l)). Activity of the recovered water samples increased only slightly, with reported values ranging from less than 1.6 to 3.2 pCi/l, compared with the primary drinking water standard of 15 pCi/l. A decrease in activity during later test cycles is indicated from the data. The average reported gross alpha activity for the native groundwater in the storage zone is 2.7 pCi/l. These results indicate that radionuclide dissolution during storage was not appreciable under test conditions.

Color and turbidity are secondary drinking water standard parameters that potentially could increase during storage. Water quality data collected during the pilot tests indicate that levels of both parameters in the recovered water samples were slightly lower than injected water values. The average color levels of the injected and recovered waters were 11.4 and 10.5 color units, respectively. Reported turbidity levels for the injected water samples range from 0.17 to 1.1

nephelometric turbidity units (NTU), while recovered water values range from 0.14 to 0.77 NTU.

Chloride ion concentration trends are commonly used in other ASR projects to evaluate ASR recovery efficiency and to track the movement of the stored water bubble in the subsurface. Chloride concentrations of both the native and injected waters at the project site average about 50 mg/l, therefore this parameter cannot be used to differentiate the two fluids. Of the major ions analyzed, alkalinity, hardness, and TDS concentrations of the native and injected waters show the most separation.

Graphs depicting alkalinity, hardness, and TDS concentrations in samples collected from the ASR well versus cumulative stored water volume are illustrated on Figures 4-1 through 4-3. The water quality data shown on the graphs include a combination of injection, storage and recovery phase results for all three ASR test cycles. Analyte concentrations for graph segments where cumulative storage volumes are increasing (upward on graph) represent injected water quality data. Stored water quality data correlate with graph segments where the cumulative storage volume is horizontal, and thus not changing. Similarly, recovered water quality data points lie above or below graph segments where the storage volume is decreasing.

Water quality data for Cycles 2 and 3 indicate that alkalinity, hardness, and TDS concentrations increase during aquifer storage by about 30, 60, and 50 mg/l, respectively. A 10 mg/l increase in sulfate is also indicated from the data. Alkalinity and TDS concentrations are similar to stored water values early in the recovery phases, then slowly increase with time toward native water values. The hardness data display a slightly different trend. Total hardness concentrations reach maximum values during storage and early recovery times, decrease during intermediate recovery periods, then increase slowly with time through the remainder of the recovery phases.

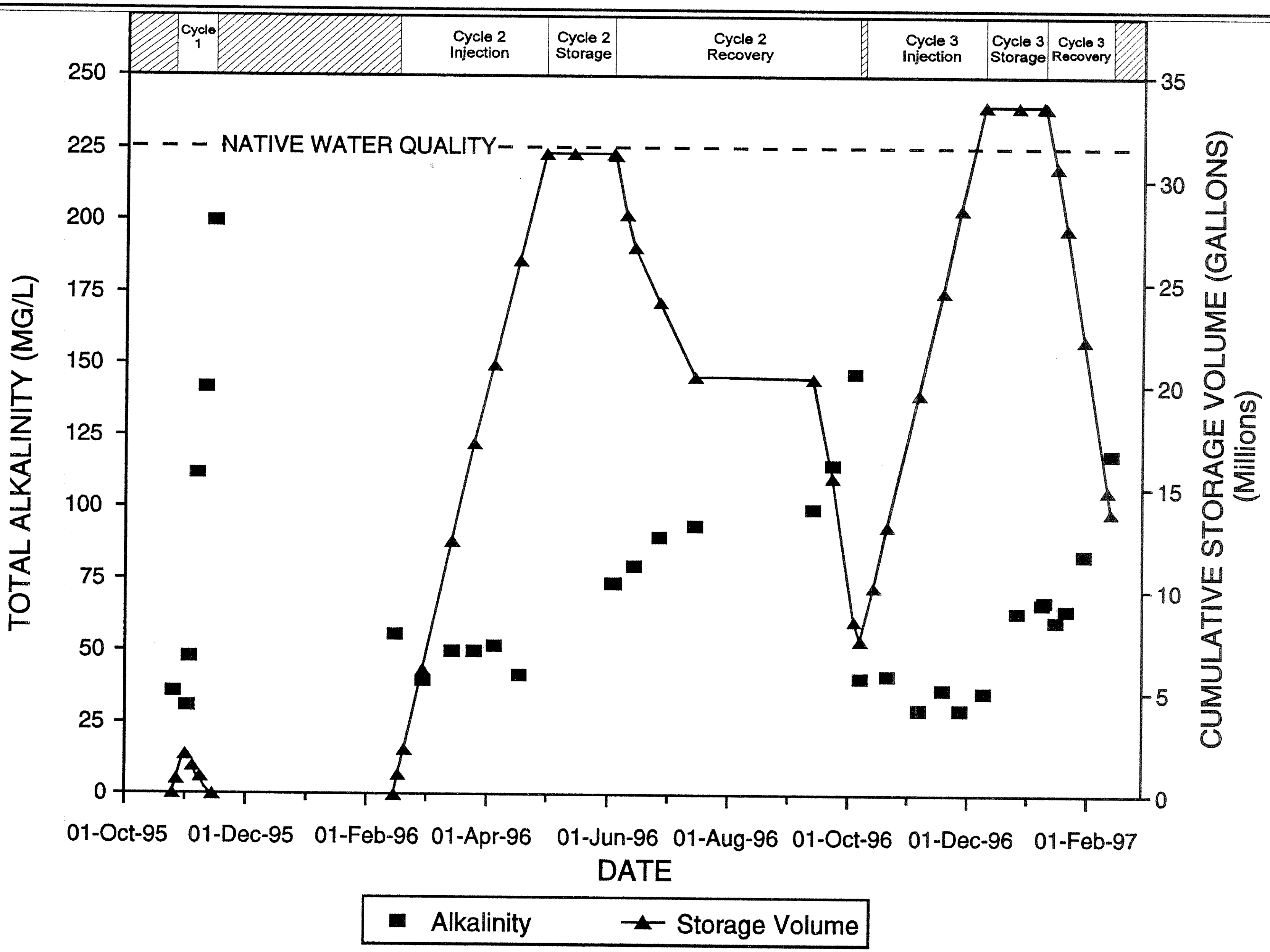

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FIGURE 4-1
GRAPH OF TOTAL ALKALINITY VERSUS CULULATIVE STORAGE VOLUME
ASR WELL DATA

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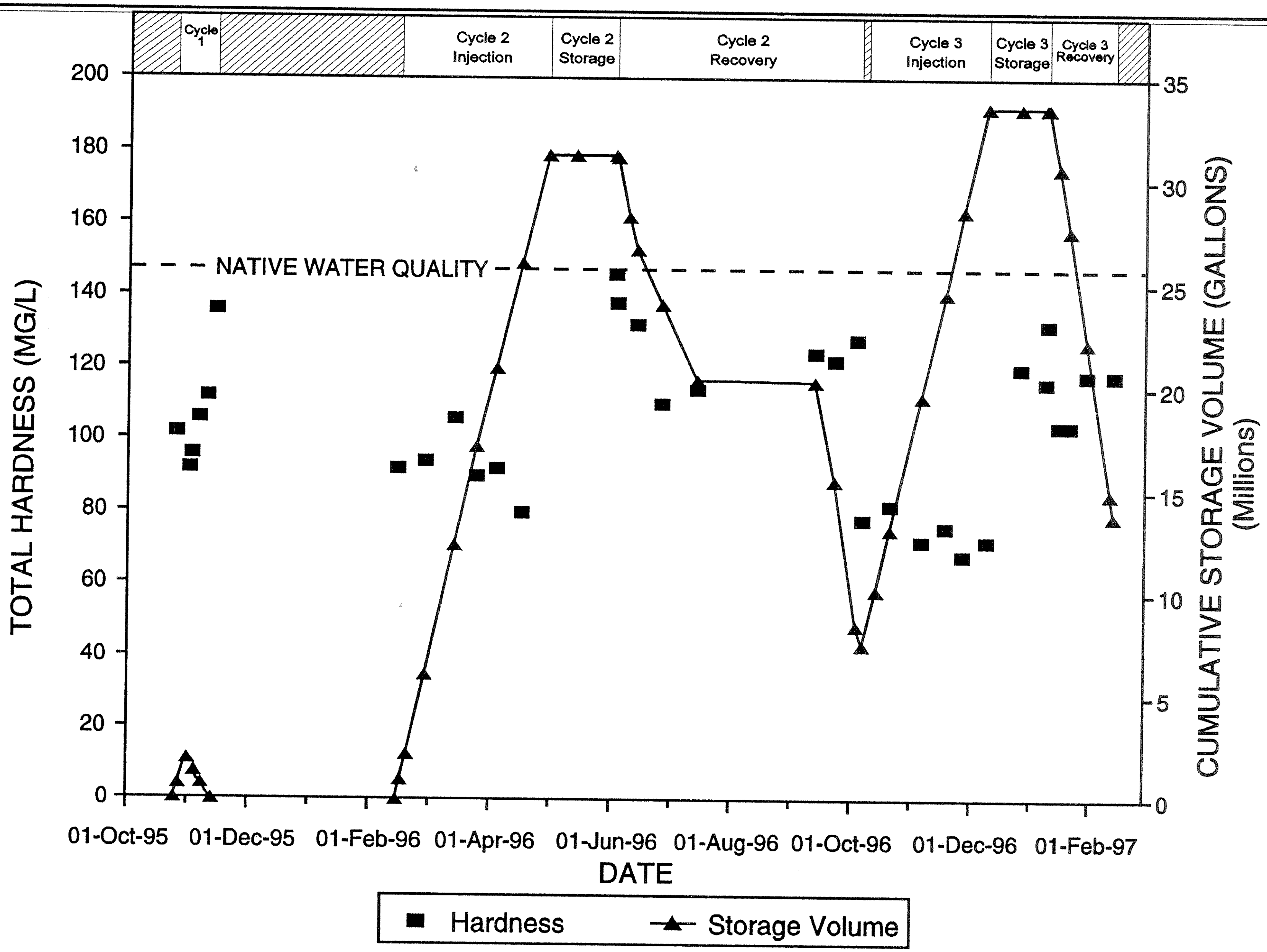

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FIGURE 4-2
GRAPH OF TOTAL HARDNESS VERSUS CUMULATIVE STORAGE VOLUME
ASR WELL DATA

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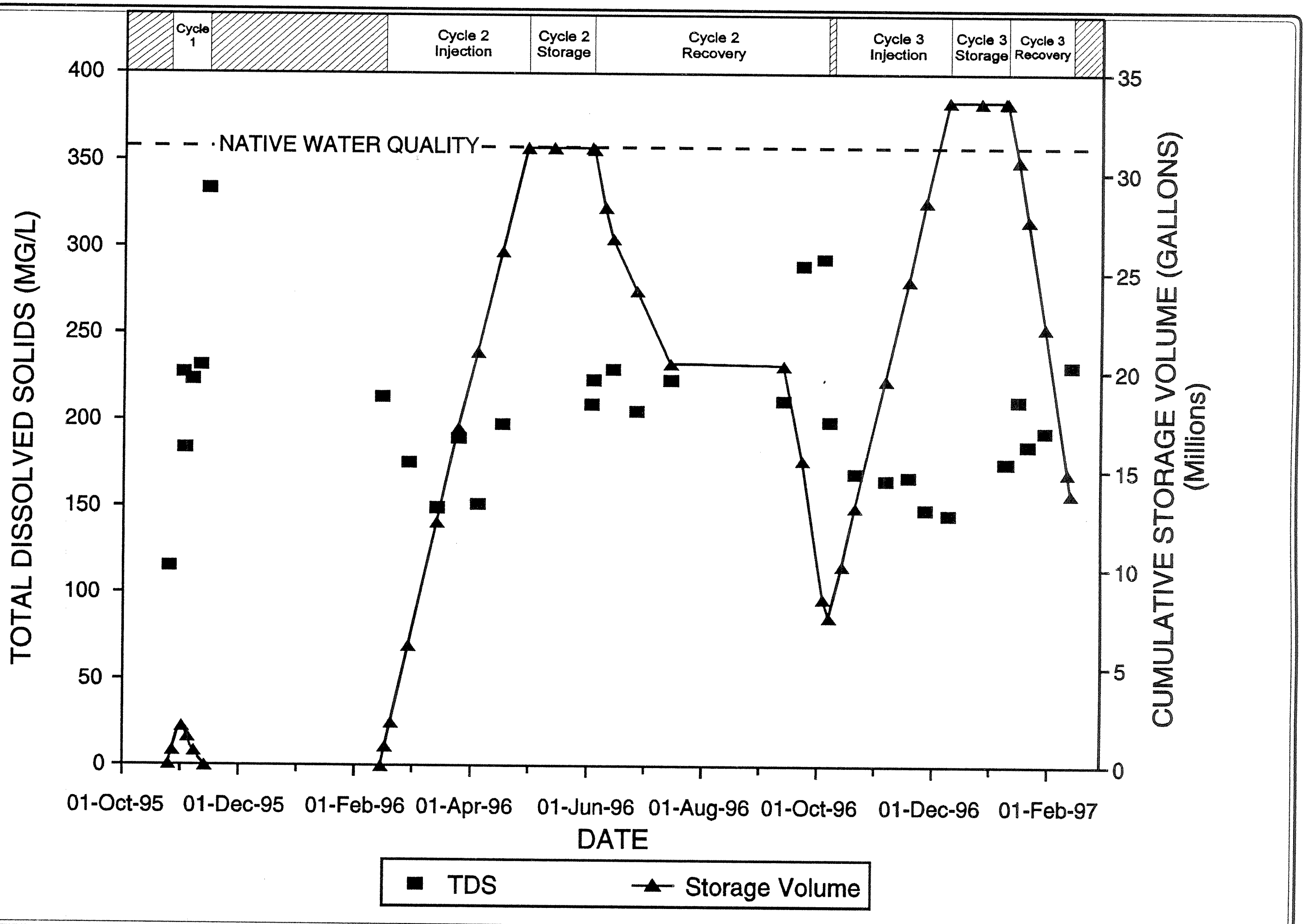

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FIGURE 4-3
GRAPH OF TOTAL DISSOLVED SOLIDS VERSUS CUMULATIVE STORAGE VOLUME
ASR WELL DATA

CDM *ViroGroup*

As discussed in Appendix F, the increase in alkalinity, hardness, sulfate, and TDS concentrations during storage is apparently related to the dissolution of mineral phases in the aquifer media. A review of the water quality data suggests that the injected water becomes saturated with respect to $CaCO_3$ during storage, and that increases in TDS are directly attributable to the addition of calcium and carbonate ions in solution.

Changes in ionic concentrations during recovery correlate directly with mixing between the $CaCO_3$ saturated injected fluid and native groundwater in the aquifer. Figures 4-4 and 4-5 show that mixing can be used to account for the observed concentration changes in alkalinity, sulfate ion, and TDS. Changes in hardness concentrations are not completely explained by this simple chemical mixing model. Additional data will be required to characterize the behavior of this constituent.

In order to assess ASR mixing characteristics, recovered water alkalinity data were corrected for equilibrium dissolution during storage. The corrected data were then used to calculate the fraction of injected and native fluids in the recovered water during the recovery phases of the cyclic tests. The solute transport model was calibrated to these data, as described in Section 5.2. After calculating how the injected water fraction changed with time, a water budget was developed to estimate the volume of injected water actually recovered during each test cycle. A graph depicting recovery trends for all test cycles is illustrated on Figure 4-6.

The x-axis of the graph represents the volumetric total recovered to total injected water ratio for each cycle. At a value of 1.0 on the x-axis, the recovered and injected water volumes for the cycle are equal. The graph for Cycle 1 extends to 1.5 along the x-axis, because 50% more water was recovered than injected. Similarly, data points for Cycles 2 and 3 extend to 0.75 on the x-axis, reflecting the fact that a 25% residual storage volume was maintained in the aquifer during each of these cycles. The y-axis of the graph represents the volumetric ratio between

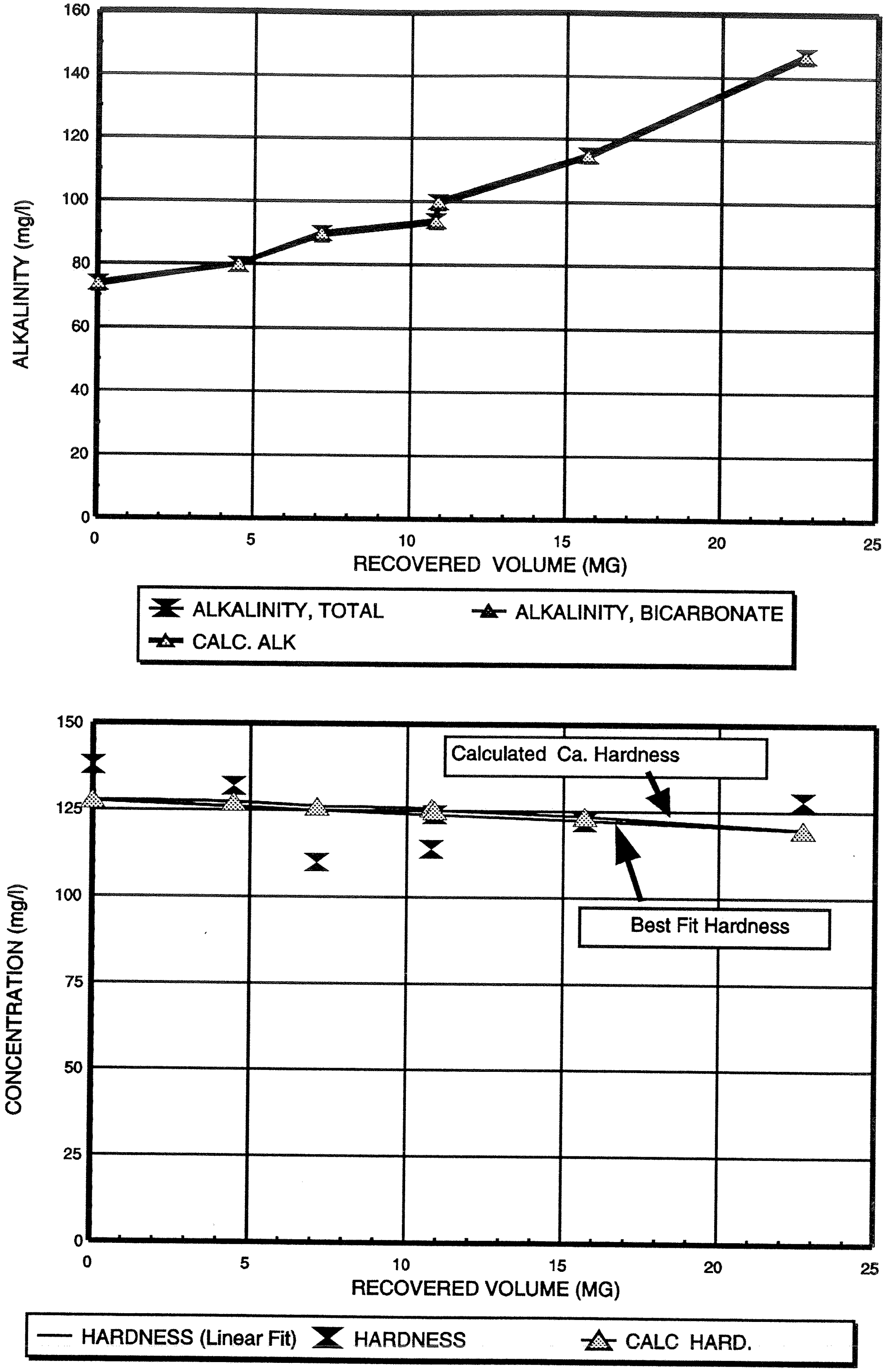

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FIGURE 4-4
COMPARISION BETWEEN RECOVERED AND CALCULATED CONCENTRATIONS FOR ALKALINITY AND HARDNESS WITH RECOVERED VOLUME (CYCLE 2)

CDM ViroGroup

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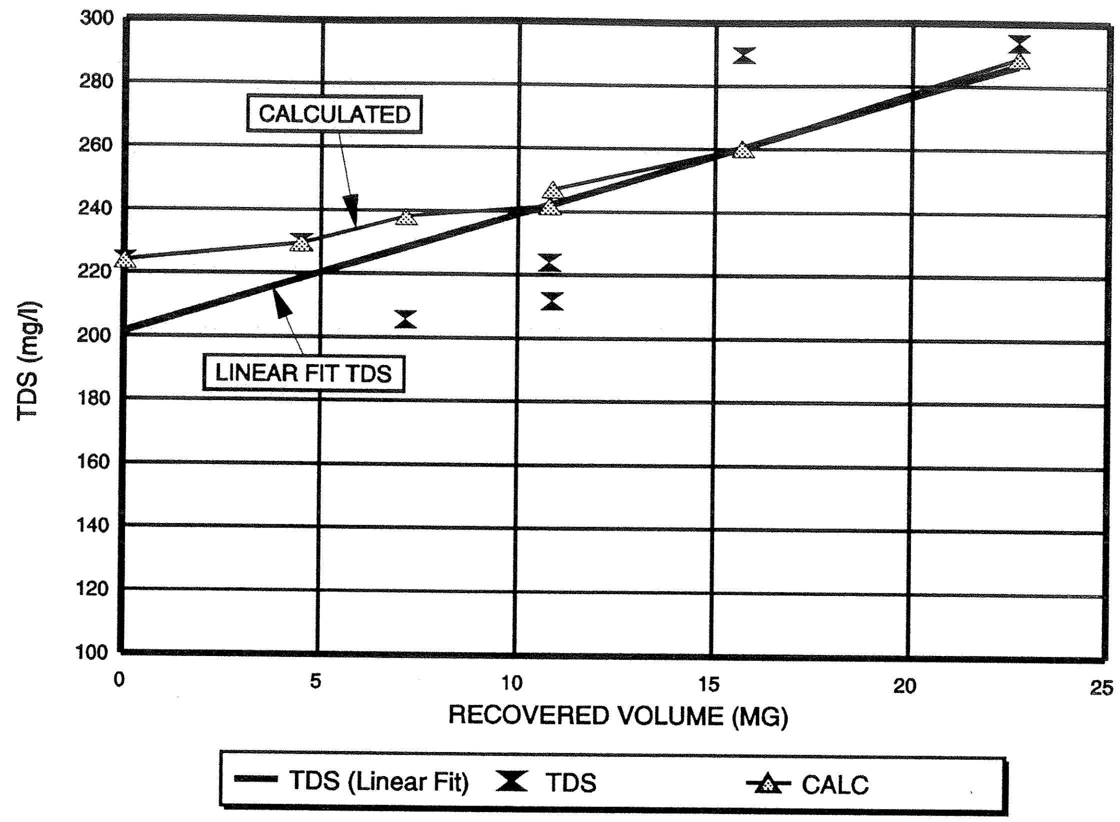

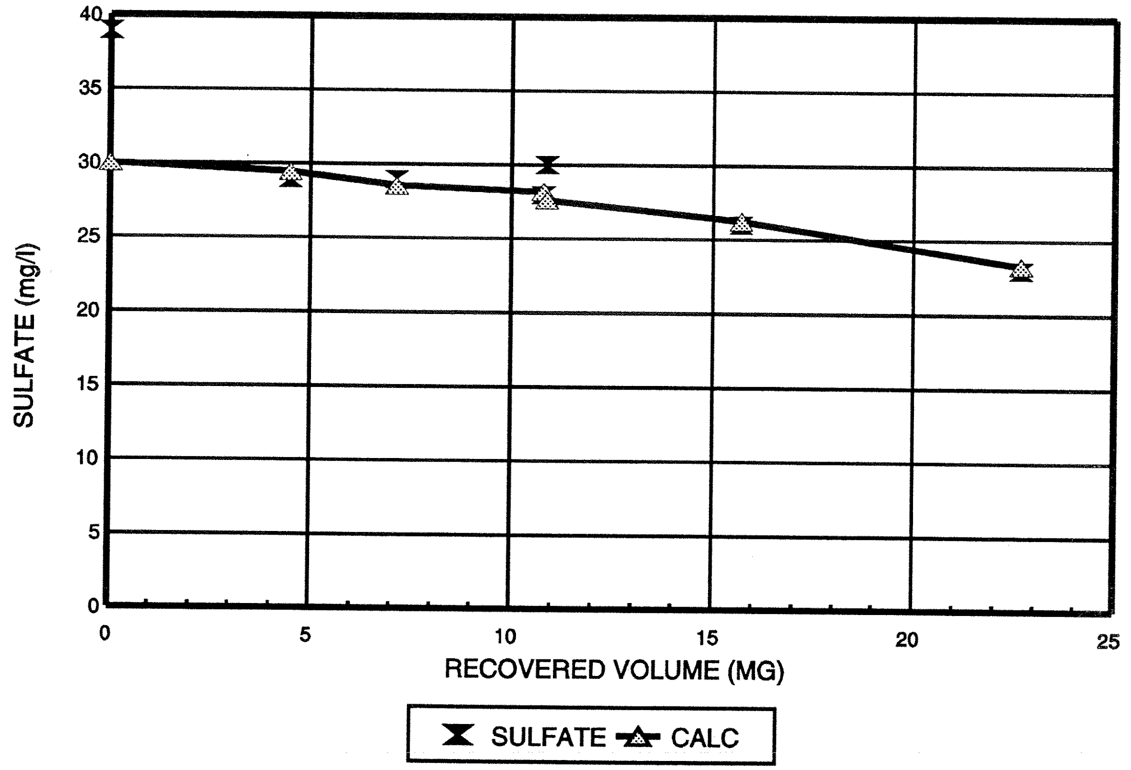

Lee County Regional Water Supply Authority	FIGURE 4-5 COMPARISION BETWEEN RECOVERED AND CALCULATED CONCENTRATIONS FOR TDS AND SULFATE WITH RECOVERED VOLUME (CYCLE 2).	CDM ViroGroup

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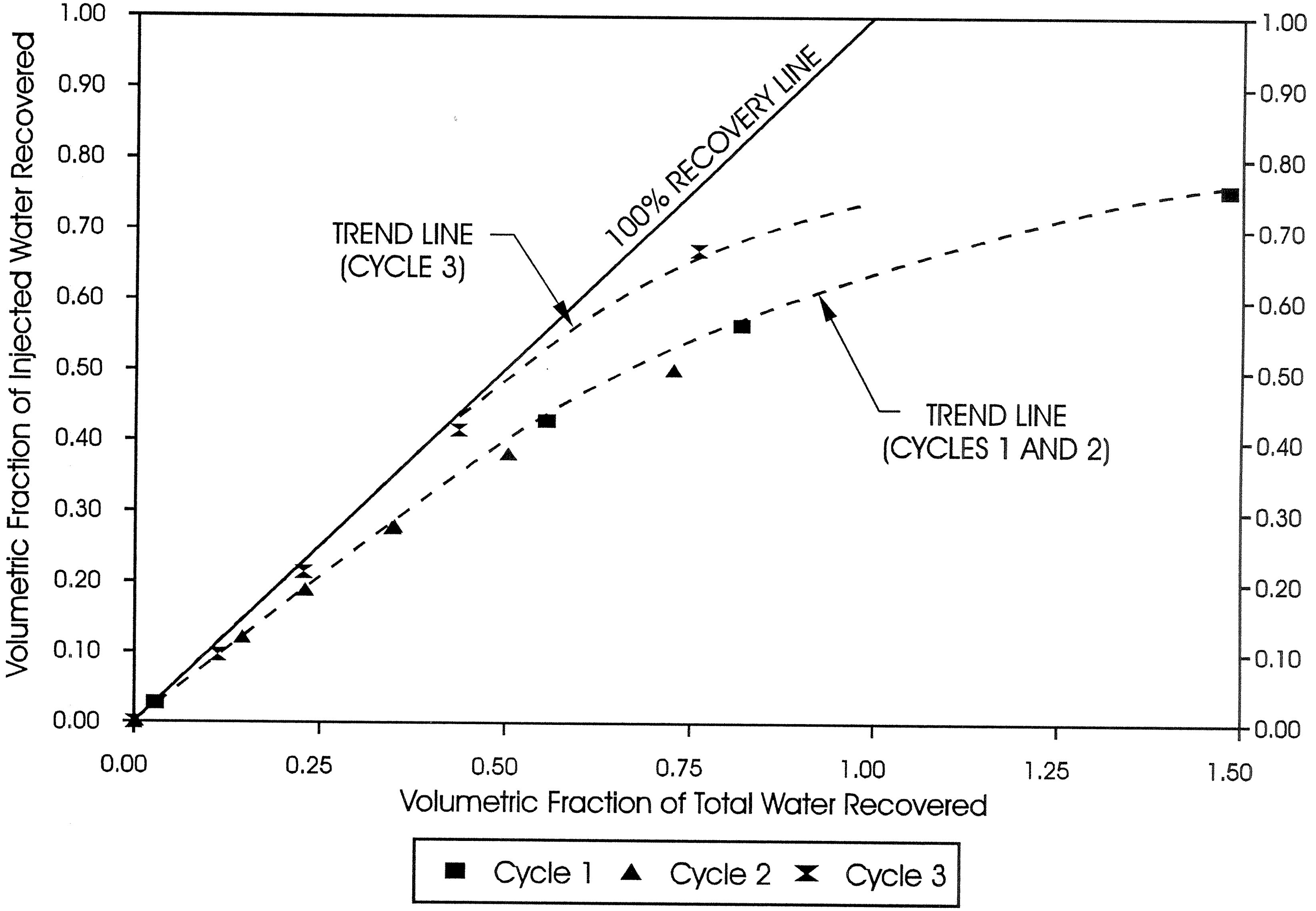

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FIGURE 4-6
INJECTED / NATIVE WATER MIXING CHARACTERISTICS AT ASR WELL,
BASED ON ALKALINITY TRENDS

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injected and total water recovered. For a test cycle in which no mixing occurs (100% recovery), the data would fall along a line that trends 45 degrees from the origin of the graph, as illustrated on Figure 4-6.

The data for Cycles 1 and 2 are similar enough to be described by a single trend line. This is reasonable, because the initial reservoir conditions for Cycles 1 and 2 were similar (no residual storage volume in the aquifer at the start of cycle). Data evaluation for these test cycles suggest that about 63% of the injected water was recovered when the total injected and recovered water volumes were equal. The remainder of the water recovered was native groundwater. This unrecovered volume of stored water during Cycle 2 added to the 8.5 mg of water intentionally left in the aquifer to help develop a stored water buffer.

The beneficial influence of this buffer zone is evident in the data for Cycle 3. Projection of the trend line for this cycle to a point where it intersects the x-axis at 1.0, yields an injected to total recovered water ratio of 75%, or 12% more than indicated for Cycle 2. The recovered water fraction for Cycle 3 follows the 100% recovery curve for the first two-thirds of the test. Data for Cycle 3 suggest that recovery yields will continue to improve during subsequent ASR cycles as the buffer zone further develops.

It is important to point out that the water budget evaluation described above is not directly related to ASR recovery efficiency. Recovery efficiency, by definition, is the ratio of "recoverable" to injected water. Stored water is recoverable until the concentrations of selected chemical parameters, normally TDS or chloride ion, exceed predetermined values, and the water is no longer suitable for its intended use. The native groundwater in the storage zone is of high quality, and generally meets drinking water standards for all analytes tested. Therefore, 100% recovery efficiency could be achieved, even with substantial mixing between the injected and native fluids.

Trends in trihalomethane (THM) concentrations were also used to evaluate injected and native water mixing characteristics during the cyclic tests. Reported THM concentrations in the injected water samples range from 15 to 58 micrograms/liter (ug/l), while recovered water values range from less than 1 to 43 ug/l. As demonstrated in Figure 4-7, THM concentrations in the recovered water for Cycle 2 decrease at a rate that is substantially faster than would be expected due to mixing, indicating that the THMs were attenuated during aquifer storage. In contrast, THMs near the wellbore were apparently not attenuated significantly. This is based on the observation that THM concentrations in samples collected during the early portion of all recovery phases are similar to those reported for the later stages of injection.

The shape of the recovery curve for the THMs suggests that the mechanism responsible for the reduction in concentration is best associated with the volume of formation contacted (adsorption), but it could also be argued that contact time between the THMs and the formation (chemical or biological reaction) could account for the observed attenuation. Future ASR cycles will provide additional information to help identify the mechanism responsible for the attenuation of these compounds.

4.3 Monitor Well Water Quality Trends

Water samples for field and laboratory analyses were collected from the three monitor wells constructed for the pilot ASR project, and from an existing monitor well (LM-926) completed in the Sandstone aquifer. Well locations are illustrated on Figure 3-1. These data were used to identify any changes in water quality in the ASR zone and adjacent aquifers, and to help track the movement of the injected water with time. Copies of the laboratory analytical reports are included in Appendix B. Water quality data for the monitor wells are summarized in Tables 4-3 and 4-4, Appendix E.

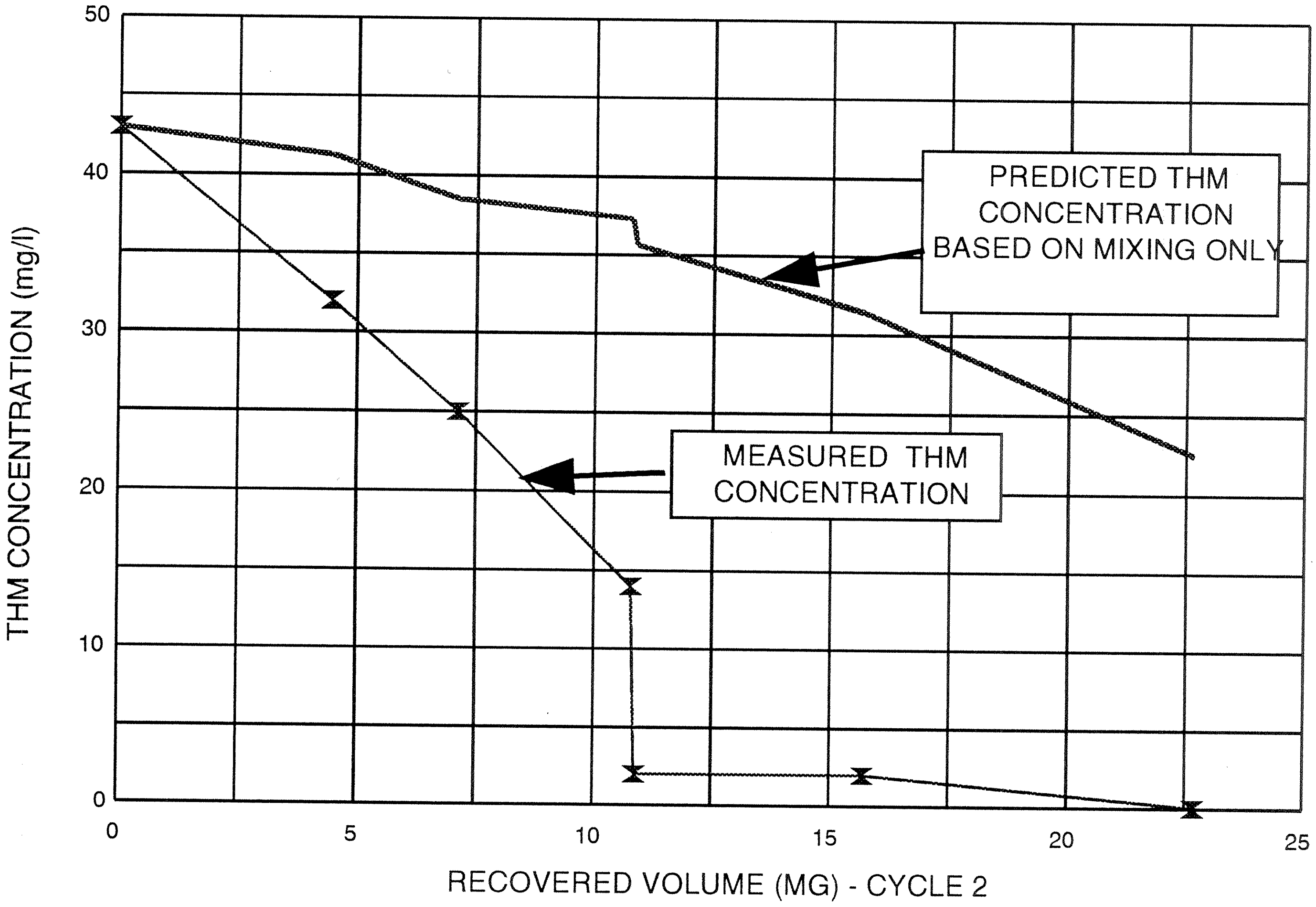

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FIGURE 4-7
COMPARISON BETWEEN THM CONCENTRATIONS PREDICTED BY MIXING AND THE MEASURED THM CONCENTRATIONS DURING CYCLE 2 RECOVERY

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Graphs depicting alkalinity, hardness, TDS, and THM concentrations versus time at all monitor wells are illustrated on Figures 4-8 through 4-11. The test cycle number and phase are also shown at top of each figure for reference. The two storage zone monitor wells, MW-A and MW-C, are located at distances of 497 and 294 feet from the ASR well. Water quality data for MW-A vary somewhat with time, but display no trends at all that correlate with ASR operations.

Concentrations of all four constituents at MW-C exhibit variations that clearly correlate with ASR operational events. During the injection phases of Cycles 2 and 3, alkalinity, hardness and TDS concentrations all decrease with time toward injected water values. A reversal of this trend occurs during recovery, as water quality parameters increase back toward native water conditions. These data are interpreted as evidence for a dispersed front of stored water intersecting the monitor well during injection, then retreating back toward the ASR well during recovery. A general freshening of groundwater at MW-C with time is evident from the data. This freshening trend is attributed to a progressive increase in the residual volume of injected water, as the buffer zone expands outward from the ASR well.

Trihalomethanes, being disinfection byproducts, are present in the injected water, but were not detected in the native groundwater of the storage zone prior to conducting the cyclic tests (refer to Tables 3-1 and 4-4). The arrival of THMs at MW-C during the injection phases of Cycles 2 and 3 provides convincing evidence of direct contact with injected water. Trihalomethanes were first detected at MW-C on Day 15 (6.1 mg injected) of the Cycle 2 injection phase, and on Day 40 (17.0 mg injected) of Cycle 3. Reported THM concentrations at MW-C decrease considerably during both storage phases, which supports the conclusion that these compounds are attenuated during storage, as discussed in Section 4.2.

Alkalinity data, corrected for equilibrium dissolution during storage, were used to evaluate the injected and native water mixing characteristic at MW-C during the

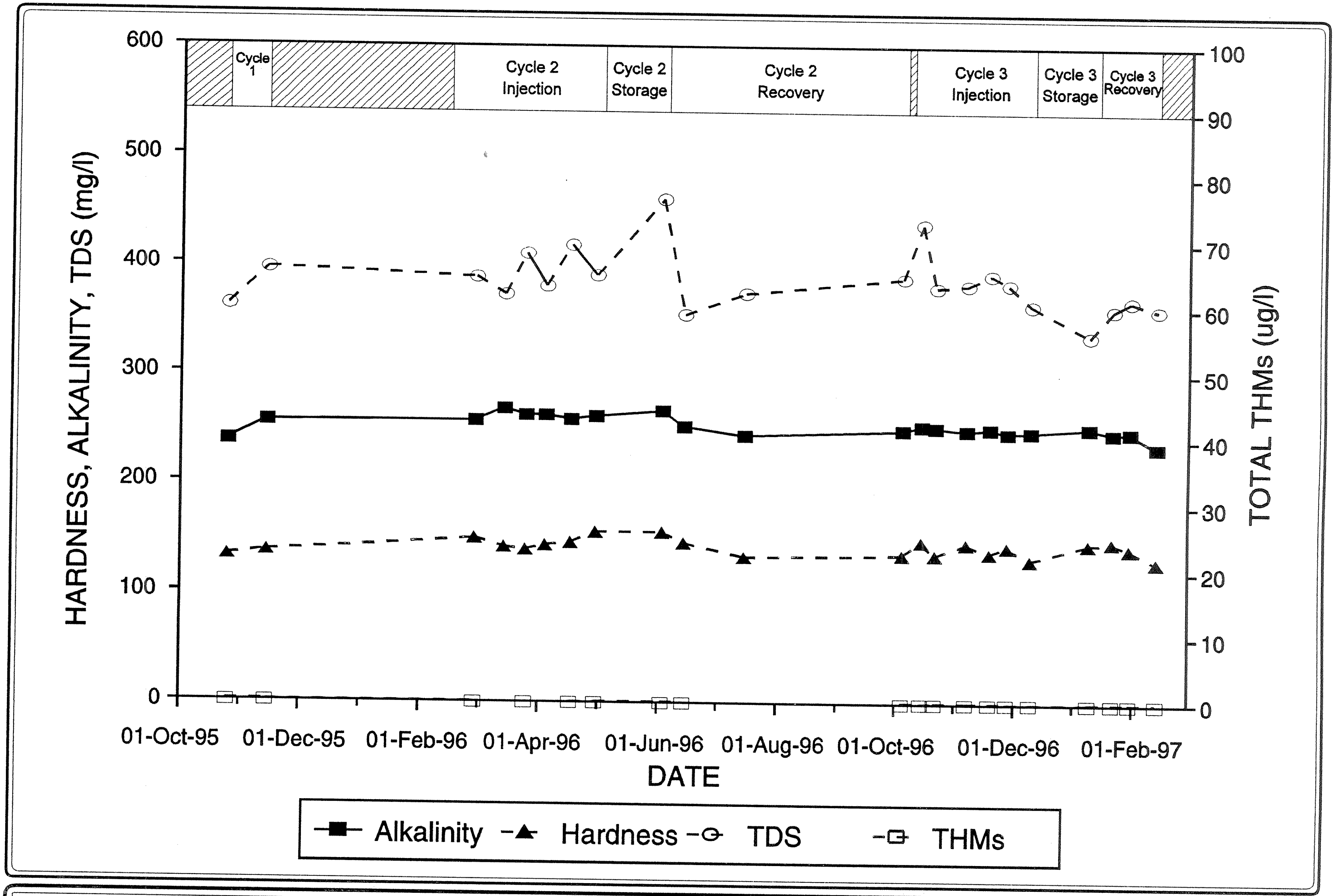

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FIGURE 4-8
WATER QUALITY TRENDS IN ASR ZONE MONITOR WELL MW-A

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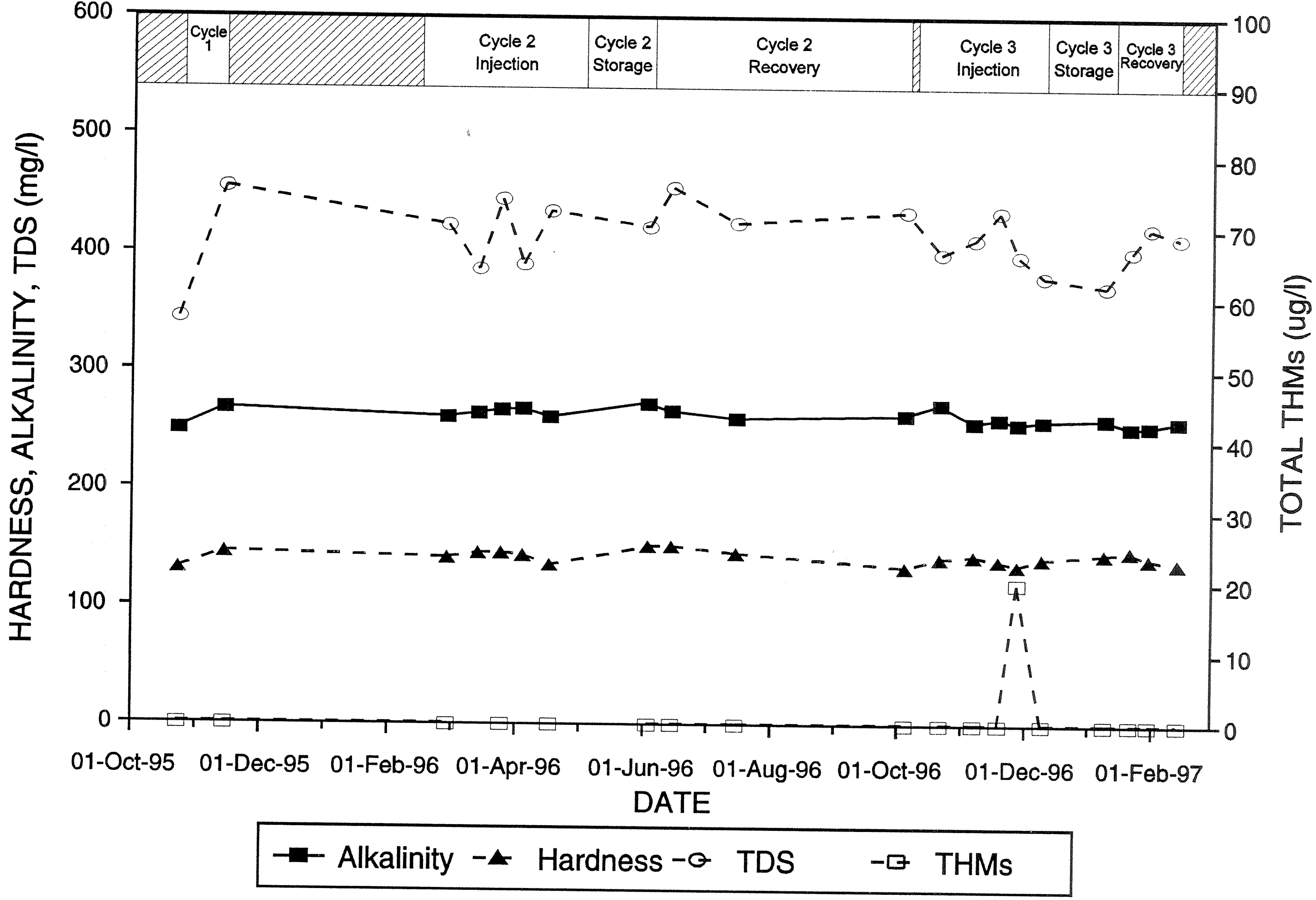

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FIGURE 4-9
WATER QUALITY TRENDS IN HAWTHORN ZONE II MONITOR WELL MW-B

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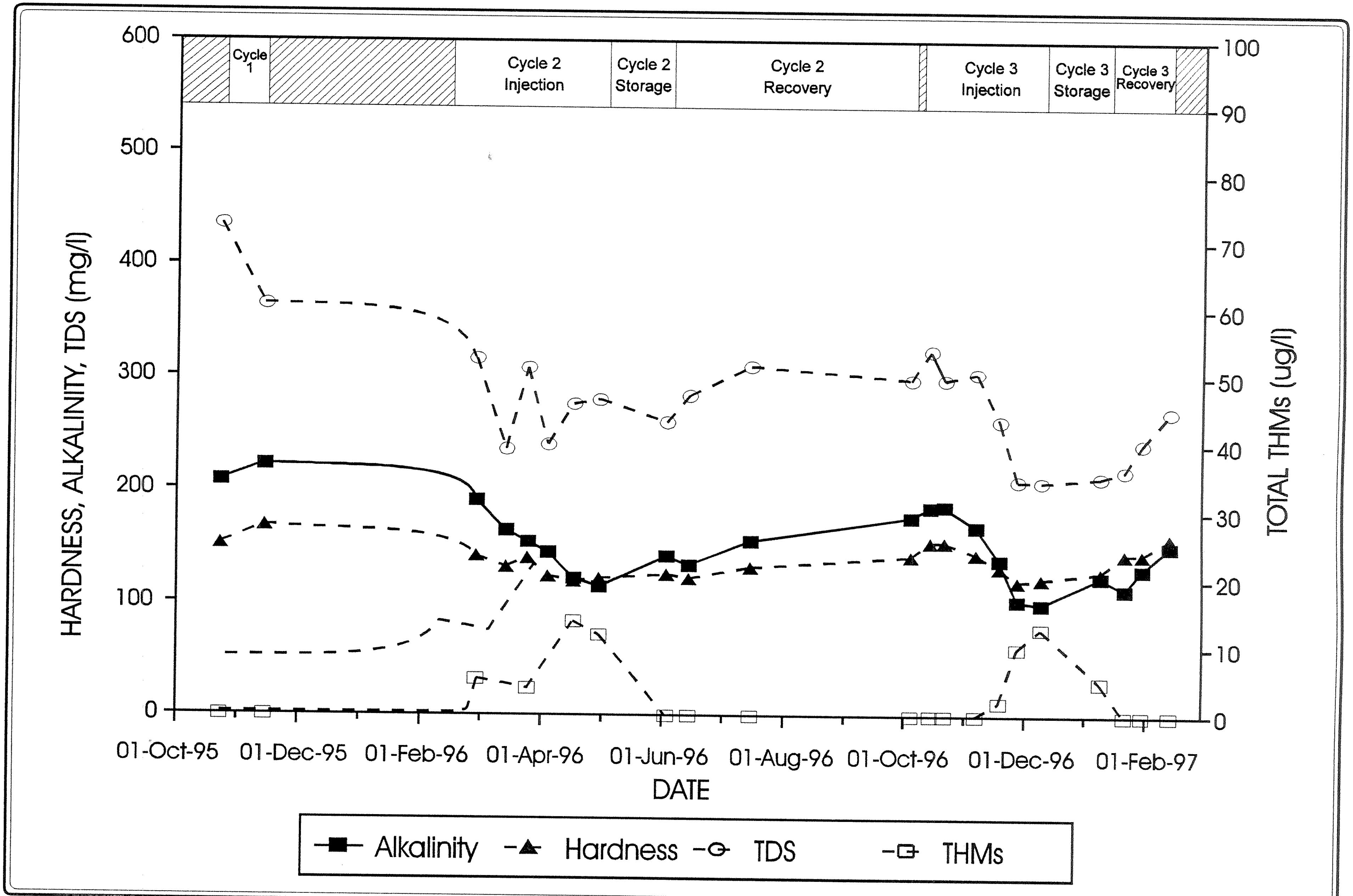

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FIGURE 4-10
WATER QUALITY TRENDS IN ASR ZONE MONITOR WELL MW-C

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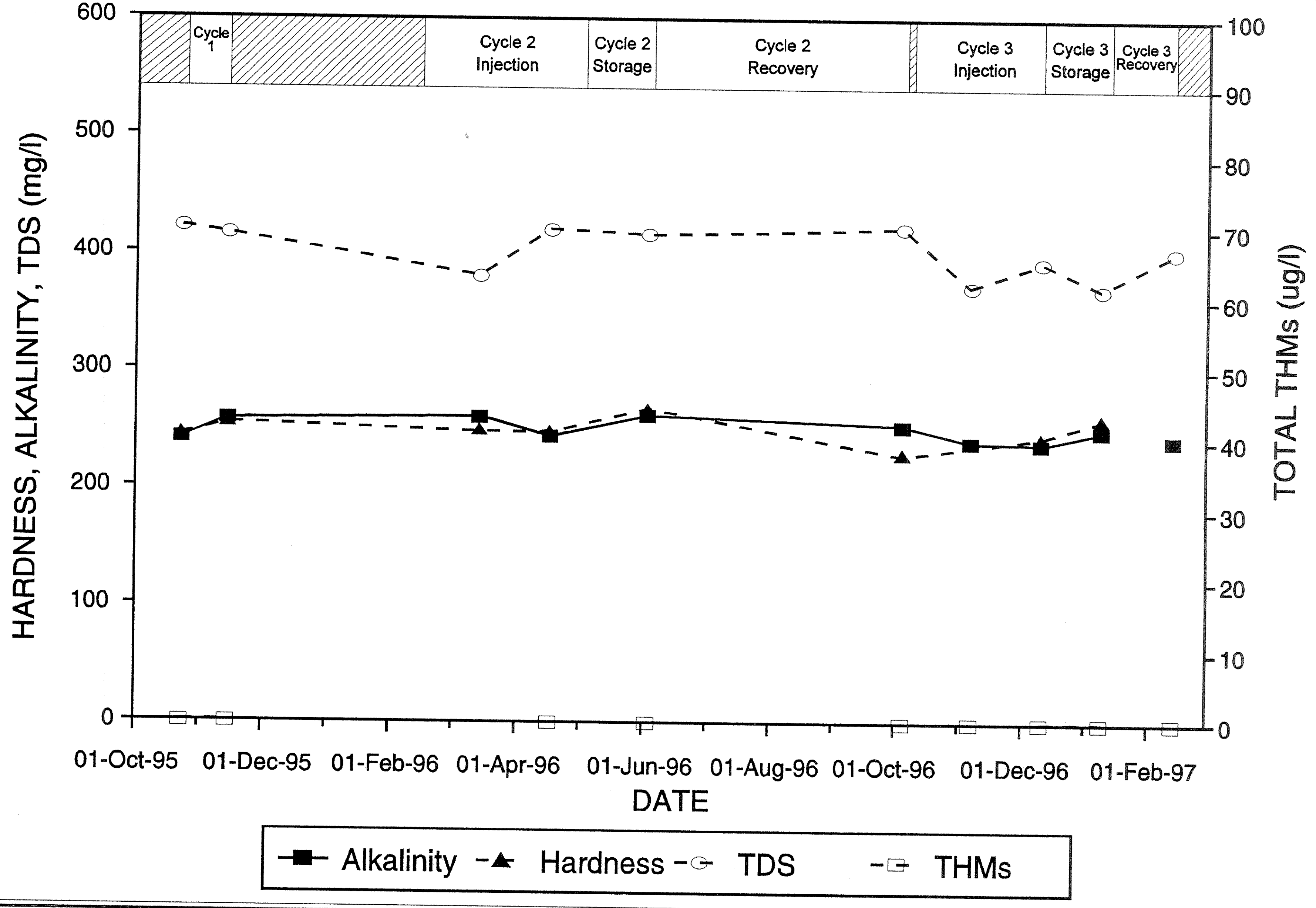

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FIGURE 4-11
WATER QUALITY TRENDS IN SANDSTONE AQUIFER MONITOR WELL LM-926

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injection phases of Cycles 2 and 3. A graph depicting calculated injected to native water ratios versus injected water volume for each test cycle is illustrated on Figure 4-12. Both data sets have been fitted to a similar type of exponential growth curve.

Cycle 2 alkalinity data evaluation suggests that a significant fraction of injection water (20%) had reached MW-C no later than 6.0 mg into the injection phase. This conclusion is supported by the THM data referenced above. The calculated fraction of injected water at the end of Cycle 2 is 70%.

Data evaluation for Cycle 3 suggests that the cycle began with a 25% residual injected water fraction remaining in storage at MW-C. No additional influx of injected water is indicated until the volume of water injected into the ASR well had reached about 12.0 mg, which suggest a somewhat longer travel time than indicated for Cycle 2. The THM data tend to support this interpretation. Trihalomethanes were not detected in the Day 30 sample (12.0 mg injected) collected from MW-C, but were detected in the Day 40 sample (17.0 mg injected). Alkalinity data for the later portion of Cycle 3 follow a similar trend as Cycle 2, reaching a final injected water fraction of about 80%.

The additional lag time before chemical response was observed at MW-C during Cycle 3 injection may be an artifact caused by variability in the sample analytical results or the sampling frequency. Alternatively, it may reflect an actual difference in the groundwater transport rate during the early portion of the injection phase. Regardless of the cause, the difference in response at MW-C during Cycle 3 ends one-third of the way into the injection phase. Thereafter, water quality data for Cycles 2 and 3 follow similar trends. A net increase in the injected water fraction at MW-C during Cycle 3 is indicated from the data.

Monitor well MW-B is completed in Hawthorn Zone II, which underlies the ASR storage zone. It is located 240 feet south of MW-C, at about the same radial

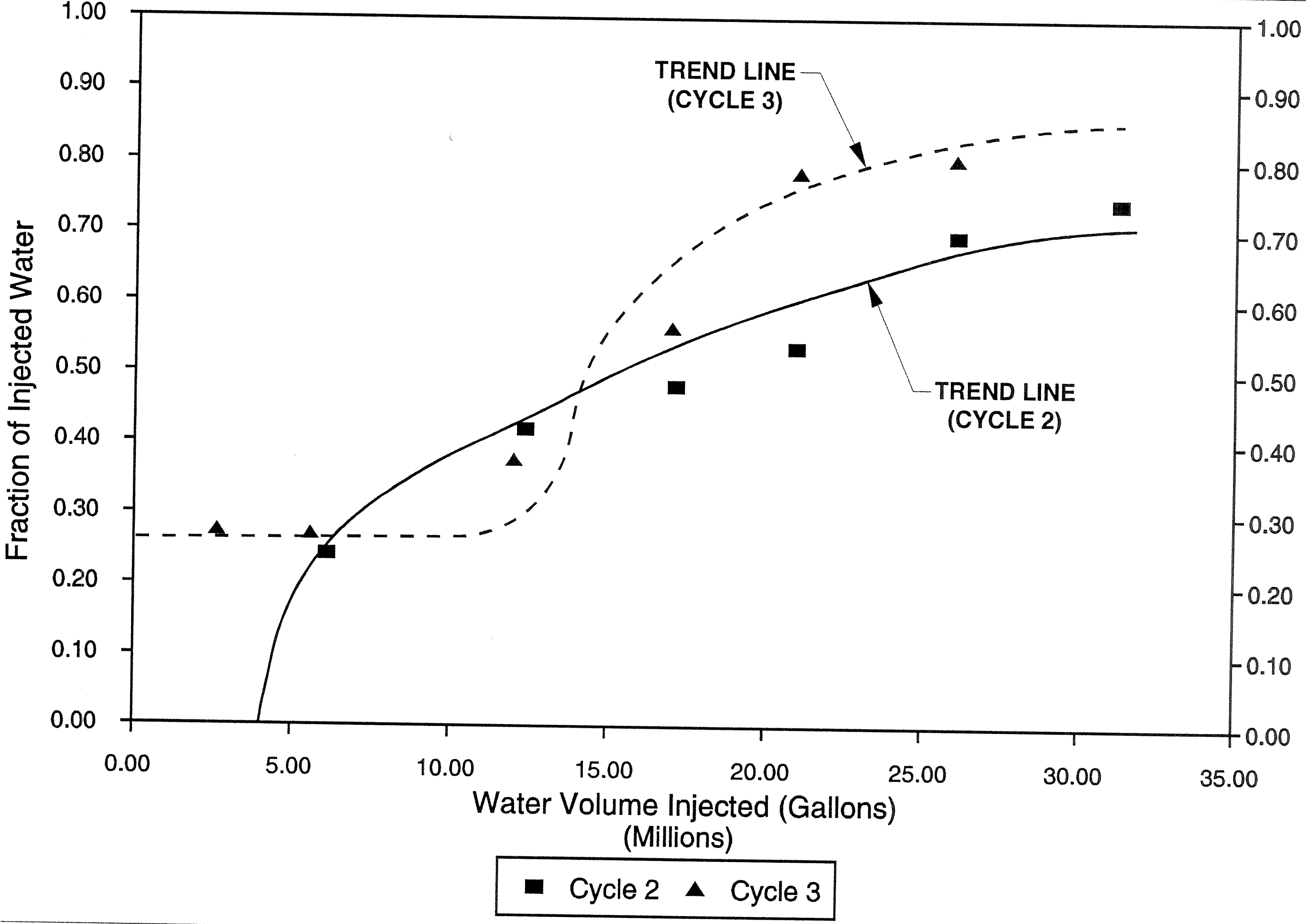

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FIGURE 4-12
ALKALINITY MIXING TRENDS AT ASR ZONE MONITOR WELL MW-C

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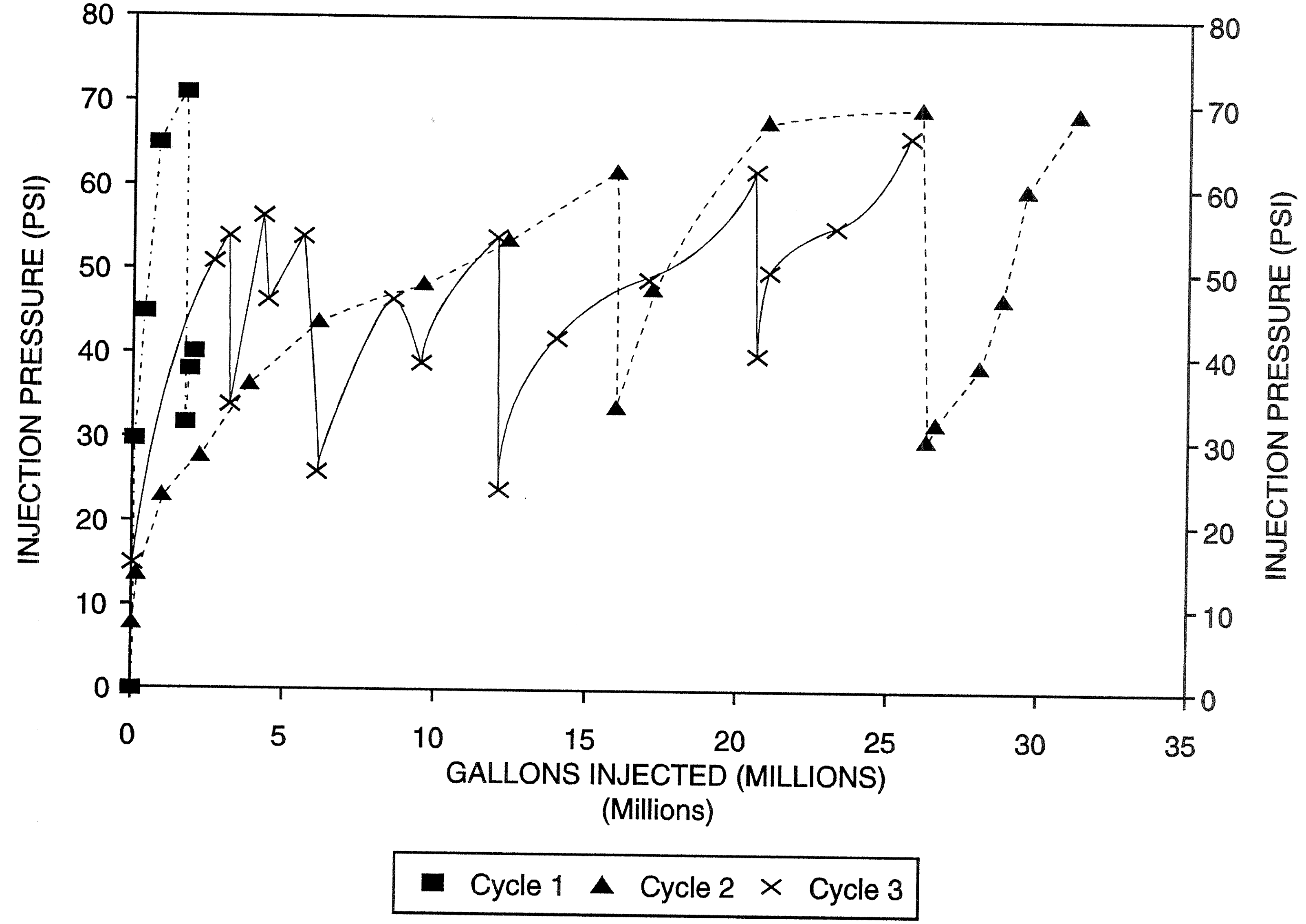
distance from the ASR well (298 feet). Water quality parameters at MW-B remained stable throughout the cyclic tests, suggesting that there was no significant exchange of water between the storage zone and Hawthorn Zone II in the area of the well. Trihalomethanes were reported at 20 ug/l in one sample collected from MW-B during Cycle 3. The data point is anomalous when compared with other analytical results, and is likely an error arising from a mislabeled sample container.

Water quality data for Sandstone aquifer monitor well LM-926 show no influence from ASR operations.

4.4 Injection Pressure Response

A graph showing injection pressure trends versus injected water volume for all ASR test cycles is illustrated on Figure 4-13. A rapid pressure increase occurred during Cycle 1. By Day 5 of the test, the ASR well injection pressure had reached the line pressure (71 psi), and the injection rate had decreased by 45% due to the additional head loss. The formation pressure response accounted for 15% of the total pressure rise, indicating that most of the increase was caused by a loss of well efficiency, presumably due to some plugging process. The project team immediately began an investigation to identify the cause of the rapid injection pressure buildup.

The rate of pressure increase was linear for a constant injection rate, suggesting that the response was due to plugging of the formation adjacent to the well by suspended solids in the injected water. Other plugging mechanisms are characterized by rates that change with time. However, a wide range of potential plugging processes were investigated including: permeability reduction by the entrainment of oxygen or air in the aquifer media, mineralization resulting from blending of the injected and native fluids, and permeability loss caused by entrapment of residual drill cuttings in the formation adjacent to the well.



Lee County Regional Water Supply Authority	FIGURE 4-13 GRAPH OF INJECTION PRESSURE VERSUS WATER VOLUME INJECTED, ALL ASR TEST CYCLES	CDM ViroGroup

The investigative methods included batch experiments in which blends of native and injected fluids were mixed and tested for increases in solids, filter tests, X-ray diffraction analyses of solids recovered from the well, and a literature review. The pipe line from the plant to the ASR well was also flushed again.

Results of the investigation were summarized in a Technical Memorandum issued by the project team on January 8, 1996. A copy of the memorandum is included in Appendix D. The plugging process was not clearly identified, but the drill cutting entrapment theory seemed to be the most reasonable explanation at the time. Plugging by suspended solids did not appear to be an important factor, because reported total suspended solids concentrations in the injected water were low (less than 0.35 mg/l). As described below, there is now evidence that the plugging, at least in part, is caused by iron deposition at the wellbore during injection.

The project team implemented a maintenance program as an operational response to plugging, while more data was gathered during subsequent cyclic tests to identify the process responsible. The maintenance plan included a combination of mild acid stimulation and periodic redevelopment of the ASR well. Acid treatment involved pumping a large volume of injected water, adjusted to a pH of 3.0 by the slow addition of hydrochloric acid (HCl), into the ASR well over a few day period. This approach has several advantages over the more traditional high concentration treatment method; it provides a more continuous chemical reaction time, no reject water disposal is required, the pump can remain in the well, and the process can be incorporated into a routine injection sequence without delaying the project.

A mild acid-stimulation of the ASR well was conducted in January 1996. Approximately 110 gallons of HCl mixed in nearly one million gallons of water was injected into the well. About 65% of the injected water volume was then recovered and discharged to the WTP filter backwash ponds, while samples were collected periodically for laboratory analyses. Analytical results indicate that only a slight increase in chloride ion (5-10 mg/l) and total hardness (10-20 mg/l) occurred in the area of the ASR well. Post-treatment test results show that the specific capacity of the well increased by 60% compared with post-construction values, and that specific injectivity increased by 400%. A more detailed summary of the treatment process was reported in a Technical Memorandum issued by the project team on February 21, 1996. A copy of this memorandum is included in Appendix D.

Cycle 2 injection directly followed the first ASR well stimulation procedure. The rate of injection pressure increase during Cycle 2 was much lower than that of Cycle 1, reflecting the effectiveness of the acid-treatment process. More importantly, the Cycle 2 injection rate was maintained at 300 gpm throughout the test, as planned.

As an additional maintenance approach, the ASR well was redeveloped twice during Cycle 2, at about five week intervals. This redevelopment, which was conducted over a two to three hour period, involved alternating cycles of intensive pumping and resting the well. The two sharp injection pressure drops that occurred during Cycle 2 (refer to Figure 4-13) correspond with these back-flushing events. The fact that specific injectivity increases after redevelopment indicates that the plugging process is reversible.

A second mild acid stimulation of the ASR well was conducted 10 days into Cycle 3, in response to injection pressure increases. The basic procedures followed were the same as those used earlier, except that the treatment was incorporated into the injection phase without delaying the cyclic test. Bi-monthly back-flushing of the ASR well was also conducted during Cycle 3 as a routine maintenance procedure. These well maintenance procedures proved to be effective, and like Cycle 2, the injection rate during Cycle 3 was maintained at 300 gpm. The injection pressure trend for Cycle 3, after acid stimulation, was similar to that of Cycle 2.

The WTP uses SHAN-NO-CORR polyphosphate stabilizer as a corrosion inhibitor (refer to Section 6-2). There was some concern that mineral precipitation caused by reactions between the stabilizer and the native water, or the aquifer media, was contributing to the plugging process. To test this, stabilizer use during Cycle 2 was suspended until the final two weeks of the injection phase. Polyphosphate treatment was resumed immediately after the second redevelopment of the ASR well had been conducted. As indicated on Figure 4-13, resumed use of the stabilizer did not seem to affect the injection pressure trend.

Several lines of evidence from data collected during the pilot project suggest that ASR well plugging, at least in part, is caused by iron precipitation at the wellbore during injection. Redevelopment of the ASR well reverses the plugging process, suggesting that the plugging materials are largely removed from the wellbore when it is back-flushed. The initial water recovered from the ASR well during redevelopment typically contains abundant reddish-colored suspended solids that, by appearance, contain ferric iron. The amount of solids diminishes rapidly with additional development time (refer to February 21, 1996 Technical Memorandum).

A one gallon water sample was collected during redevelopment of the ASR well on April 17, 1996. Solids in the sample were separated by filtration, dissolved in acid, and analyzed for a list of cations by atomic adsorption. The laboratory reported that the major cations present were iron (47%) and calcium (46%). Other cations detected in order of decreasing concentration were magnesium, aluminum, sodium, potassium, strontium, and manganese. The abundance of calcium can be explained by the presence of fine limestone particles in the recovered water. The iron is believed to be flushed from the wellbore were it accumulates during injection. A copy of the laboratory analytical report is included in Appendix B.

Water quality data collected during the cyclic tests provide another line of evidence suggesting that iron plays a significant role in the plugging process. Recovered water samples for all test cycles contain less iron than the injected water. The magnitude of the deficit broadens as the iron concentration in the injected water increases.

Average iron concentrations in the injected and recovered waters during Cycle 2 were 0.09 and 0.06 mg/l (refer to Table 4-2, Appendix E), reflecting a slight (33%) iron reduction during recovery. During Cycle 3, the average iron concentration in the injected water increased to 0.18 mg/l, while the recovered water average decreased to 0.04 mg/l; indicating a loss of nearly 80% of the iron contained in the injected water. Iron deposition at a rate of 0.14 mg/l, integrated over the entire injection phase of the cycle, would accumulate more than 30 pounds of iron at the wellbore. The total mass available for plugging would be greater, since the iron would probably be bound to an ion such as hydroxide, or grouped within a larger complex. Based on these findings, it is recommended that iron sequestering, with an agent such as citric acid, be investigated on an experimental basis in future ASR operations at the site.

4.5 Monitor Well Water Level Response

Water level data for the four monitor wells at the test site were collected in accordance with the testing plan. These data are summarized in Table 4-3, Appendix E. Water levels in all wells were below casing tops under static conditions, and during the recovery phases of all cyclic tests. Monitor well MW-B developed positive heads of up to a few feet above casing level at times during injection phases, however, levels in the other monitor wells remained below the casings.

A graph of water level variations in the monitor wells versus time for all test cycles is illustrated on Figure 4-14. Positive values denote mounding during injection, while negative values reflect drawdown during recovery phases. These data indicate that

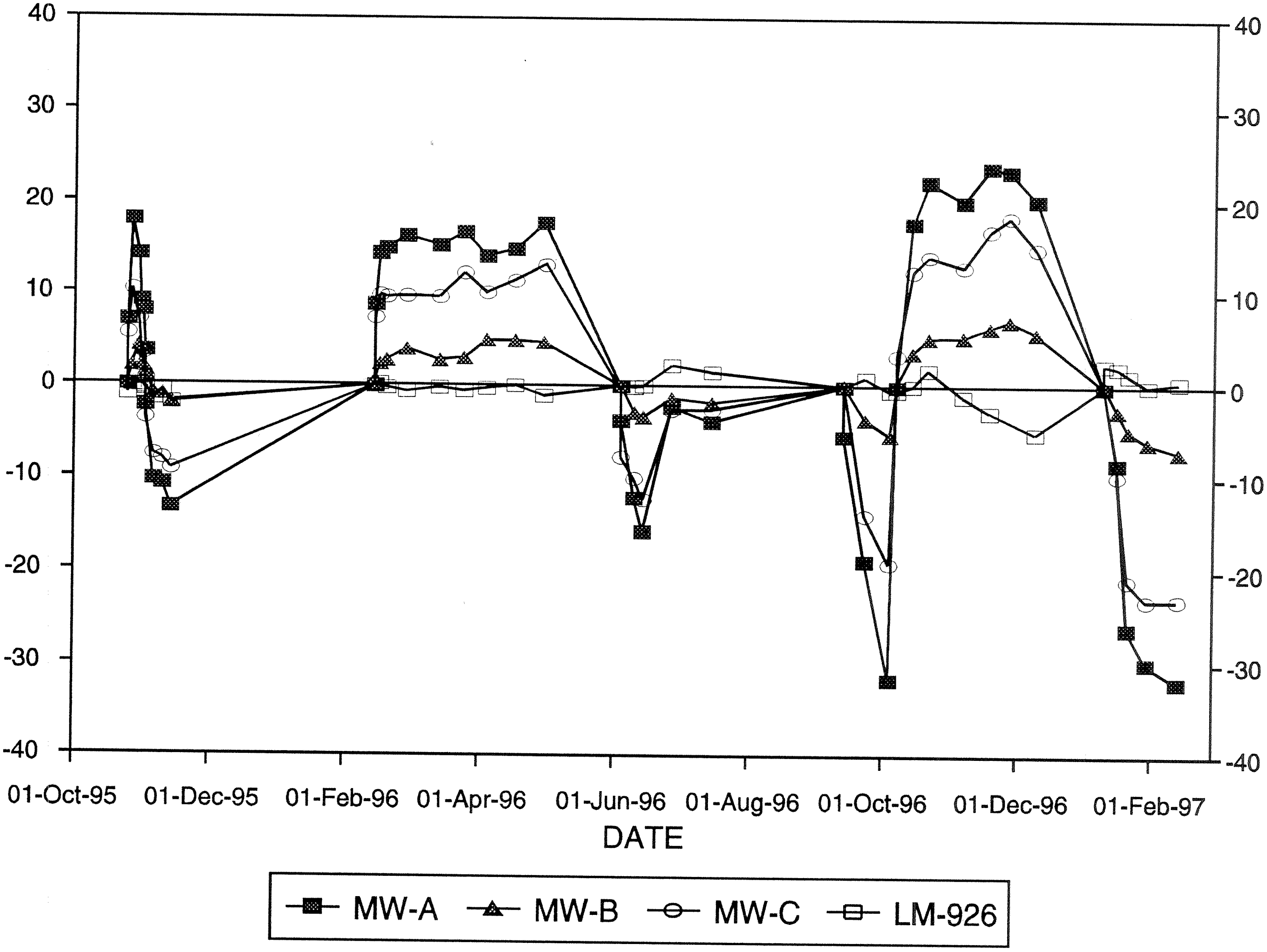


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FIGURE 4-14
MONITOR WELL DRAWDOWN VERSUS TIME

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the cone of depression and mounding surrounding the ASR well is asymmetric, with more drawdown to the east, than the west. Drawdown in monitor well MW-A, located 497 feet east of the ASR well, is typically 1.2 to 1.4 times greater than the drawdown in monitor well MW-C, which is also completed in the storage zone 294 feet west of the ASR well. These data suggest that the storage aquifer is heterogeneous.

Hydraulic heads in monitor well MW-B (Hawthorn Zone II) display a clear response to ASR operations. Measured drawdown values (positive and negative) at MW-B average about 30% of those recorded at MW-C, which is located at nearly the same radial distance from the ASR well. A similar data comparison indicates that the response at MW-B is about 20% of that at MW-A, which is located about 200 feet further from the ASR well. Pressure response to pumping of adjacent aquifers is typical of the intermediate aquifer system, even where hydraulic connection between zones is not significant. Field data and modeling results described in Sections 5.1 and 5.2 indicate that leakage of stored water between aquifers during ASR operations is minimal.

Water levels in Sandstone aquifer monitor well LM-926, located about 1,100 feet south of the ASR well, remained relatively stable during the cyclic tests. There is no evidence of a hydraulic response in LM-926 from ASR operations.

5.0 HYDRAULIC AND SOLUTE TRANSPORT MODELING

5.1 Hydraulic Modeling

Model Description and Calibration

The ASR plan is to inject water at the line pressure of approximately 70 psi. The primary ASR system design criterion is limiting formation pressure response during injection, because of injection pressure buildup concerns. Other factors being equal, injection can be maintained at desired rates for longer periods of time before ASR well maintenance is required, when formation pressurization is minimized. For a given set of hydraulic conditions, formation pressure response during ASR operations (mounding or drawdown) is best controlled by adjusting the spacing between production wells, and by favorably configuring the wellfield with respect to aquifer boundaries and hydraulic properties.

Hydraulic modeling was used to predict formation pressure response for various ASR well configuration options, as an aid to developing a conceptual wellfield design. This modeling was performed using the U.S. Geological Survey three-dimensional, finite difference model, MODFLOW (McDonald and Harbaugh, 1988). Models at two different scales were utilized. A small-scale model, with a relatively fine grid, was first used to define the hydraulic characteristics of ASR test site. A larger scale model, with broader cell size and a core area that covers the entire ASR wellfield, as envisioned, was then prepared using the small-scale model results as a guide. Both models were developed from on-site hydraulic data. Model sensitivity to variations in aquifer parameters and boundary conditions was evaluated.

The basic structure of both the small-scale and large-scale hydraulic models is similar. Both models contain three transmissive layers. In descending order, these layers represent the Sandstone aquifer, Hawthorn Zone I, and Hawthorn Zone II.

Cells in model Layers 1 and 3 are specified with constant heads of 2 and 22 feet, respectively, following field data. A combination of active cells, inactive cells, and constant head cells are used to simulate the ASR storage zone (model Layer 2). Vertical conductance terms (Vcont) calculated from APT analyses were used to represent the confining beds between conductive model layers.

Both models were first run under steady state conditions. A homogeneous Layer 2 transmissivity of 25,200 gpd/ft was applied in the steady state simulations. The initial steady state run of the small-scale model was performed with the APT calculated total leakance value (1.6 E^{-5}/day) split equally above and below the storage zone. Leakance (Vcont) was then redistributed between layers, without changing the total leakance term, until the final heads in Layer 2 were about 10.6 feet. The large-scale model uses the same leakance distribution as the smaller model. Layer 2 heads produced from the final steady state model runs were used as initial heads in subsequent transient simulations.

Cyclic test and APT results, described previously in this report, indicate that the cone of depression and mounding surrounding the ASR well at the test site is asymmetric with more drawdown and mounding to the east, than the west. Data analysis indicates that this hydraulic response is due to a reduction of transmissivity in the eastern portion of test site. The presence of a restricted flow boundary about 4,000 feet east of monitor well MW-A is also suggested from the data.

The small-scale MODFLOW model, with a 1,050 by 1,100 foot core area and a cell spacing of 50 feet, was developed to better define the hydraulic characteristics of the test site. The model core encompasses the existing ASR well and the storage zone monitor wells. A figure showing the model core and other grid information is provided in Appendix G. Cells outside of the model core progressively increase in size by a factor of 1.5. The perimeter of the model extends about 5.7 miles beyond the core area.

A number of transient simulations were run on the small-scale grid with variations made to transmissivity, leakance, and storage properties. Stress period lengths and production well stress rates were modified to simulate APT and cyclic test conditions, thus calibrating the model to field data. These results indicate that the most reasonable means of calibrating the model is to apply a no-flow boundary condition along the eastern edge of the model grid, at a distance of about 100 feet east of monitor well MW-A. This interpretation places the boundary much closer than is suggested from the APT analysis.

Figure G-1, Appendix G illustrates drawdown contours for a small-scale model run that closely matches APT 2 field data.

A copy of the model output file is also included in Appendix G. As indicated from Figure G-1, the asymmetric cone of depression predicted by this model run is within 0.6 feet of drawdown values measured during the APT. Model agreement with field data for the longer-term cyclic test simulations is similar. The best calibration to field data is produced when Layer 2 is simulated as a wedge-shaped aquifer with transmissivity that decreases across the model grid, from about 50,000 gpd/ft west of the test site to zero at the eastern no-flow boundary.

The data collected for the pilot ASR project are the only useful information available within 15 miles of the ASR test site to characterize the hydraulic properties of the storage zone. The goal of the small-scale modeling is to provide an adequate prediction of formation pressure response over a much larger area where site-specific hydraulic data are not available. Therefore the large-scale hydraulic model used in the wellfield design process was developed with more conservative estimates of storage zone transmissivity than those indicated from the small-scale model results. The approach used in developing the large-scale model is described below.

The grid for the large-scale model is illustrated on Figure 5-1. The 6,000 by 15,000 foot core area of the model consists of 5,760 cells with a constant cell size of 125 feet, as illustrated on Figure 5-2. The entire eastern perimeter of the model terminates one cell (125 feet) east of monitor well MW-A. All Layer 2 cells at the eastern edge of the model are active, ensuring that no hydraulic flow occurs across this boundary. This approach conservatively assumes that the inferred no-flow boundary east of the ASR test site is continuous in a north-south direction throughout the entire area modeled. The remaining perimeter cells in model Layer 2 are specified with a constant head of 10.6 feet. Any regional aquifer hydraulic gradients are neglected, since potentiometric surface maps for the storage zone are not available.

Calibration of the small-scale model suggests that the transmissivity of the storage zone increases to over 50,000 gpd/ft west of the ASR test site. This almost doubles the highest value calculated from on-site APT data. As a conservative approach, storage zone transmissivity for the large-scale model simulations is limited to the APT calculated value of 25,200 gpd/ft. A constant transmissivity of 25,200 gpd/ft is used in all Layer 2 cells west of the existing ASR well. Transmissivity in Layer 2 cells located east of the ASR well decreases in a west to east direction until the no-flow boundary is reached.

A transient run of the large-scale model was made to simulate the ASR well pumping stresses experienced during APT 2. The model overestimates drawdown at the ASR zone monitor wells by about 5 feet compared with field data and the best fit small-scale model results, due to the lower transmissivity estimates used in the large-scale model. A run of the small-scale model, made after lowering transmissivity to large-scale model conditions, yielded drawdown values similar to those produced by the large-scale model. These results confirm that the response of both the large and small-scale models is similar. Figures G-2 and G-3, Appendix G show predicted drawdown values for both of these model runs.

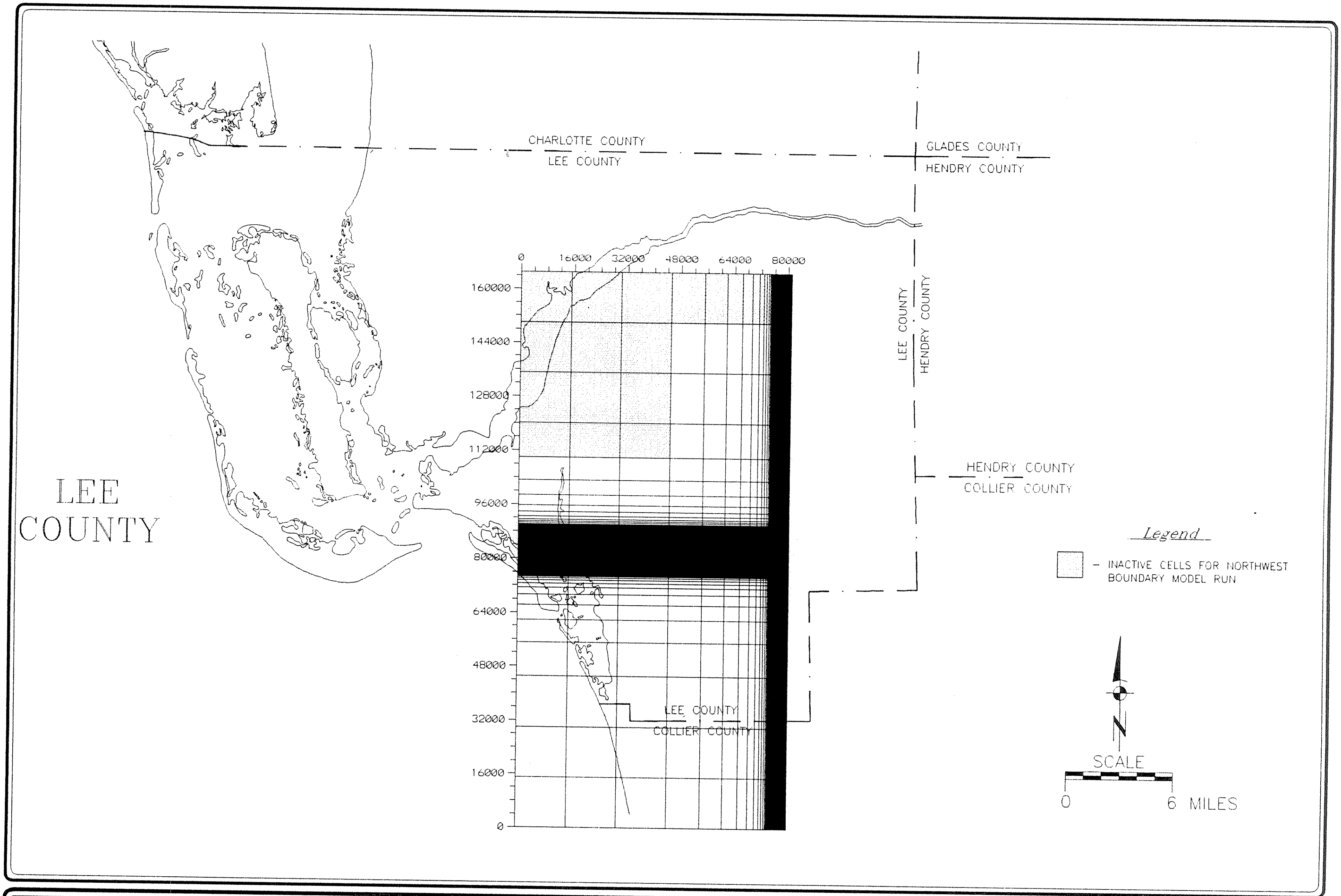

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FIGURE 5-1
LARGE SCALE HYDRAULIC MODEL GRID

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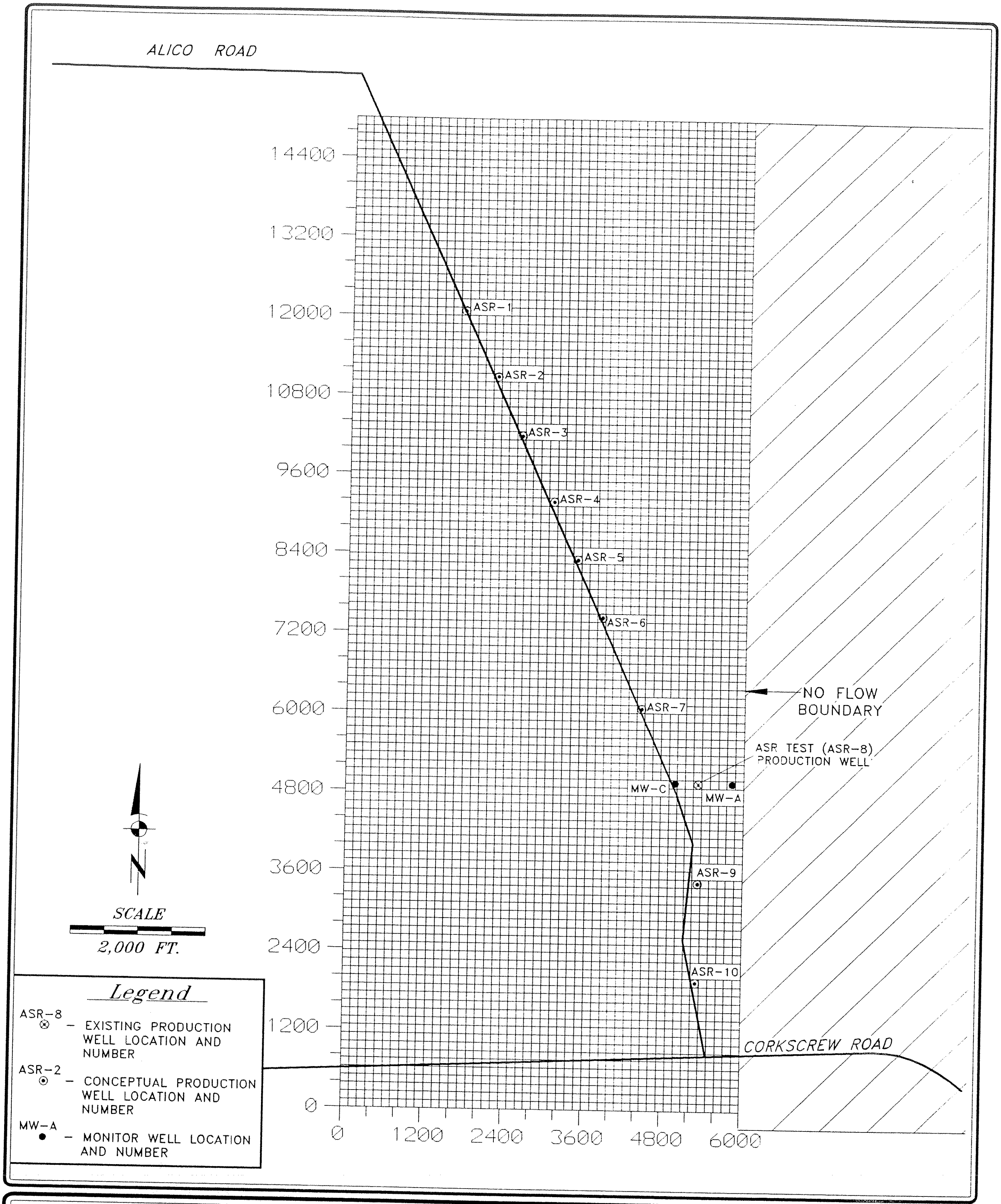

Lee County Regional Water Supply Authority	FIGURE 5-2 LARGE-SCALE HYDRAULIC MODEL CORE AREA WITH CONCEPTUAL ASR WELLFIELD CONFIGURATION	CDM ViroGroup

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ASR Injection Simulations

The operational plan for the ASR wellfield involves the recovery of 270 million gallons (mg) of stored water during a four month dry season period. Based on the water quality of the storage zone and recovery data from the cyclic tests, 100% recovery efficiency may be achieved. As a safety factor, the wellfield is designed to store 300 mg of water during a four month rainy season to account for some potential loss of recovery efficiency. Operational data from the cyclic tests suggest that a seasonal storage volume of about 30 mg per ASR well is a reasonable design value. Therefore, an ASR wellfield consisting of ten production wells supplying the total seasonal storage volume of 300 mg was considered in the conceptual wellfield design process.

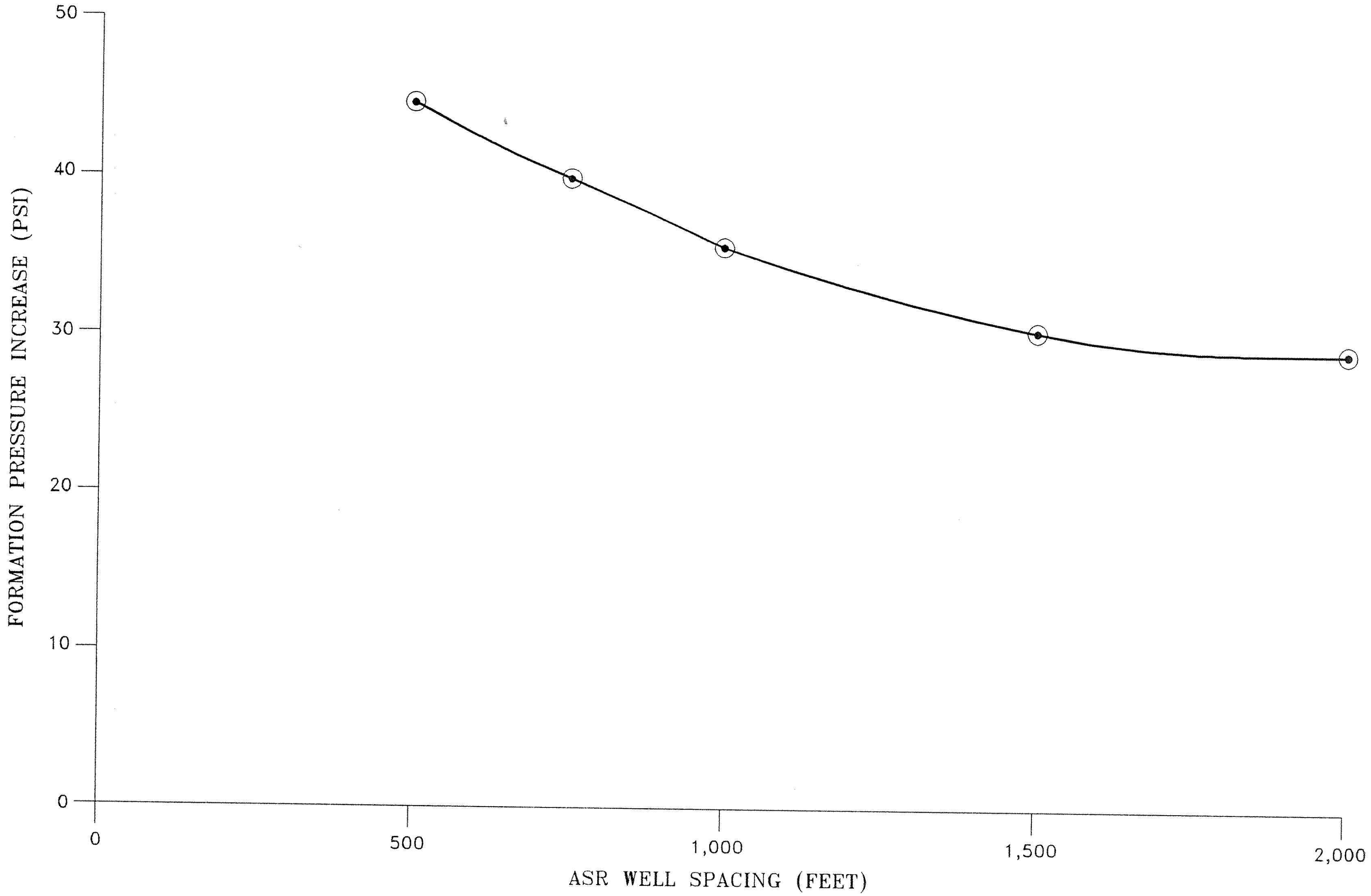
Potential ASR well locations considered are limited to property currently owned by Lee County at the Corkscrew wellfield, and easements that parallel adjacent roadways. Geologic factors (i.e. transmissivity reduction to the east, and the inferred no-flow boundary) further limit potential well locations. To comply with these constraints, two conceptual wellfield configurations were developed; a linear wellfield oriented north-south along Alico Road, and an L-shaped wellfield following both Alico and Corkscrew roads (refer to Figure 5-2). Both of these wellfield options were evaluated with the aid of hydraulic modeling.

One element of the conceptual wellfield design is to select the ASR well spacing. Pressure buildup during injection is more of an operational concern than pressure falloff during recovery, because of wellbore plugging during injection. Water recovery back-flushes the well and removes solids from the wellbore, which tends to restore the well capacity. Therefore, under normal operating conditions, the minimum required spacing between production wells is directly related to pressure response during injection.

The first step in selecting the ASR well spacing was to perform a series of injection simulations with the large-scale MODFLOW model. The distance between ASR wells was varied between 500 and 2,000 feet in these simulations. In each model run, ten ASR wells were placed a constant distance apart in a roughly north-south alignment along Alico Road. The total seasonal storage volume of 300 mg was simulated using a constant injection rate of approximately 175 gpm (33,420 ft^3/day) per well for a 120 day stress period. The flow equations were solved in five time steps using the Preconditioned Conjugate Gradient (PCG2) solution package.

Formation pressure response versus well spacing is illustrated on Figure 5-3. The pressure data shown on the graph represent the maximum amount of mounding predicted by the model (converted from feet of water to psi), adjusted for the two correction factors described later in this section. The maximum pressure buildup for all simulations occurred at the existing ASR well (ASR-8 on Figure 5-2), which reflects the wells proximity to the inferred no-flow boundary. Results of this evaluation indicate that formation pressure at the injection wells decreases as a logarithmic function of well spacing. Increasing the well spacing from 500 to 1,500 feet provides a beneficial reduction in formation pressure response. Broadening the distance between wells to 2,000 feet has minimal influence (1.0 psi change). Based on these results, a 1,500 foot spacing for ASR wells in the southern portion of the wellfield was selected for the conceptual wellfield design. Figure G-4, Appendix G shows predicted formation pressure buildup for the constant-spaced, 1,500 foot model simulation.

Wells in the northern portion of the linear alignment are further from the no-flow boundary, and experience less interference drawdown than the more centralized wells. A model run was made to evaluate the effect of reducing the spacing between the five northern wells to 1,000 feet, while maintaining a 1,500 foot spacing for the remaining wells. Predicted mounding after 120 days of pumping is illustrated



NOTE: FORMATION PRESSURE INCREASE VERSUS WELL SPACING FOR 10 ASR WELLS WITH AVERAGE CONTINUOUS PUMPING RATE OF 175 GPM PER WELL FOR 120 DAYS

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FIGURE 5-3
MAXIMUM FORMATION PRESSURE

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on Figure 5-4. A comparison of these results with the 1,500 foot, constant-spaced model shows that reducing the well spacing in the northern portion of the wellfield by one-third only increases the maximum formation pressure response by 1.5 psi. Based on these results, an ASR wellfield with a variable well spacing of 1,000 and 1,500 feet was selected for the conceptual wellfield design, as illustrated on Figure 5-2. The MODFLOW output file for the conceptual design model is included in Appendix G.

In the case of the L-shaped wellfield, wells along Corkscrew Road are aligned perpendicular to the no-flow boundary at the eastern edge of the model grid. This increases mounding compared with scenarios in which wells are spaced the same distance apart and aligned parallel to the boundary. Figure G-5, Appendix G shows predicted pressure buildup for an L-shaped wellfield with a constant well spacing of 1,500 feet and the same stresses used in the other injection simulations. The maximum mounding for the L-shaped configuration is about 5 feet greater than that of the linear wellfield with the same well spacing. As discussed previously in this section, minimizing mounding during injection is advantageous during ASR operations.

ASR Emergency Withdrawal Simulations

In an emergency such as a hurricane, it is possible that the water treatment plant could be inoperable for a sustained period of time, leaving the ASR system as the only source of treated water for public supply. Under these conditions, the demand for recovered water could far exceed normal withdrawal rates. The specific capacity of the pilot ASR well is about twice its specific injectivity, suggesting that the ASR wellfield could recover stored water at rates considerably higher than normal injection rates to help meet emergency needs.

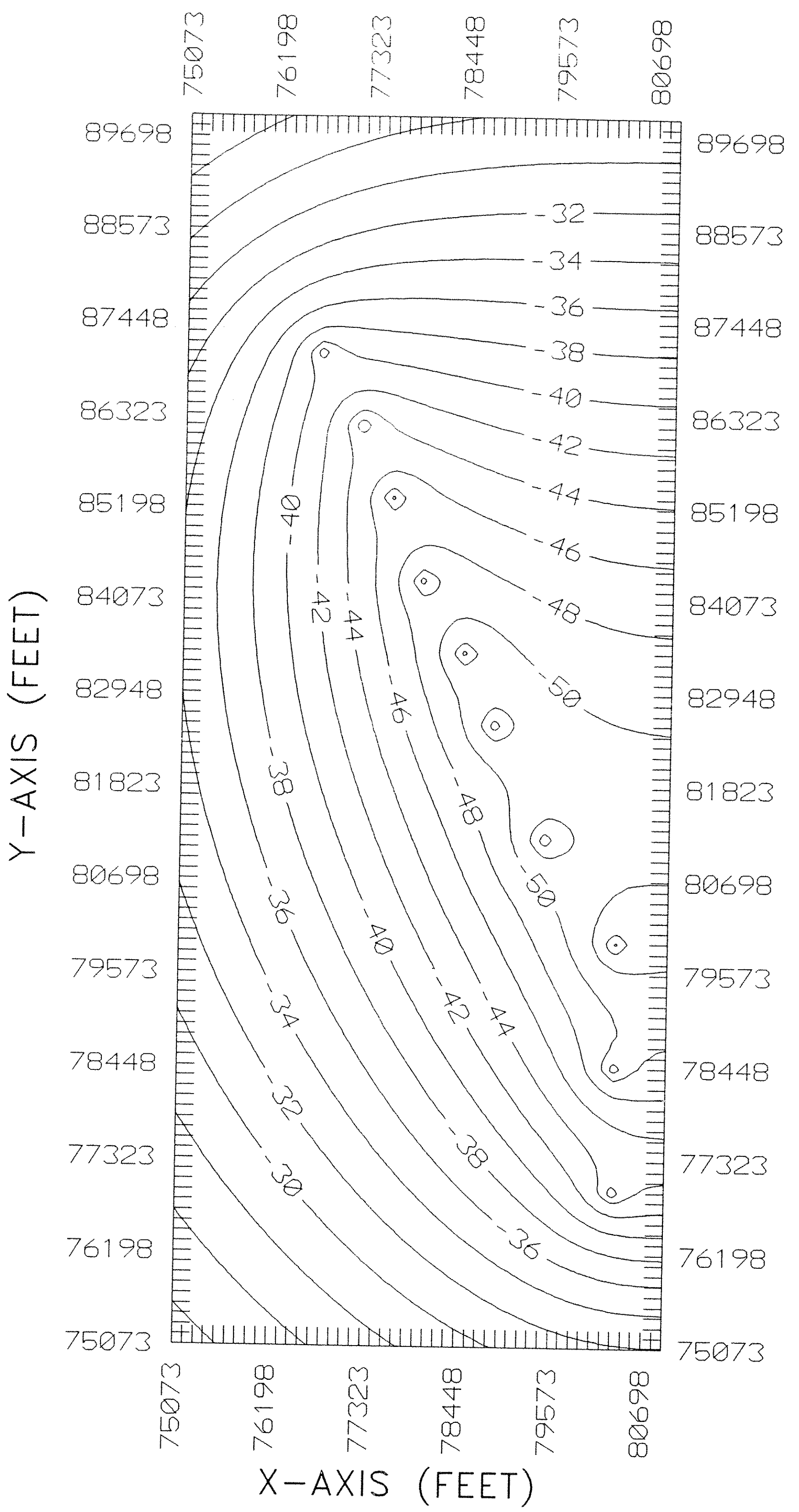

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FIGURE 5-4
MODFLOW MODEL-PREDICTED DRAWDOWN, CONSTANT AVERAGE INJECTION OF 2.5 MGD AFTER 120 DAYS

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A series of simulations with the large-scale MODFLOW model was performed to help estimate the maximum recovery rate that could be expected for different drawdown scenarios. The variable-spaced, linear wellfield selected as for the conceptual wellfield design was used in this evaluation. Recovery rates for the ten ASR wells in each simulation were held constant for a 14 day stress period. Multiple simulations were run at different recovery rates ranging from 275 to 375 gpm per well.

Drawdown data predicted by the model were corrected for production well friction losses, using well efficiency values calculated from the pilot ASR well step-drawdown test results. Drawdown at production well ASR-6 (refer to Figure 5-2) was considered in the evaluation. This well is located in the center of the wellfield and exhibits slightly higher than average drawdown compared with the other ASR wells simulated. Calculated drawdown at ASR-6 for the model simulations run are summarized in Table 5-1. Results of this analysis indicate that recovery of 4.6 mgd of stored water for a 14-day period could be conducted with an average drawdown of 145 feet in the ASR wells. Predicted wellfield drawdown for the 4.6 mgd pumping scenario is illustrated on Figure 5-5.

Model Checks and Sensitivity Analysis

The finite-difference solution of the groundwater flow equations used by MODFLOW provides an average drawdown value for an entire model cell. As a result, formation drawdown at production wells is underestimated, unless the cell size is extremely small. In contrast, drawdown calculated by analytical methods is not affected by cell size.

A check on the MODFLOW results was conducted by preparing an analytical model representative of the 1,500 foot constant-spaced, linear wellfield alignment used in one of the injection simulations described above. The no-flow boundary at the

TABLE 5-1

EMERGENCY WITHDRAWAL ANALYSIS

WITHDRAWAL RATE (Q) (gpm)*	FORMATION DRAWDOWN ALL WELLS (FEET)	FORMATION DRAWDOWN TYPICAL ASR WELL (FEET)	FORMATION PLUS WELL LOSS (FEET)	INTERFERENCE DRAWDOWN AT TYPICAL ASR WELL (FEET)	TOTAL DRAWDOWN (FEET)	TOTAL YIELD, ALL WELLS (MGD)
275	91	39	66	52	118	3.96
300	102	42	72	60	132	4.32
325	108	45	81	63	144	4.68
350	115	48	89	67	156	5.04
375	127	51	98	76	174	5.40

*Reflects constant per well withdrawal for 14 day period.

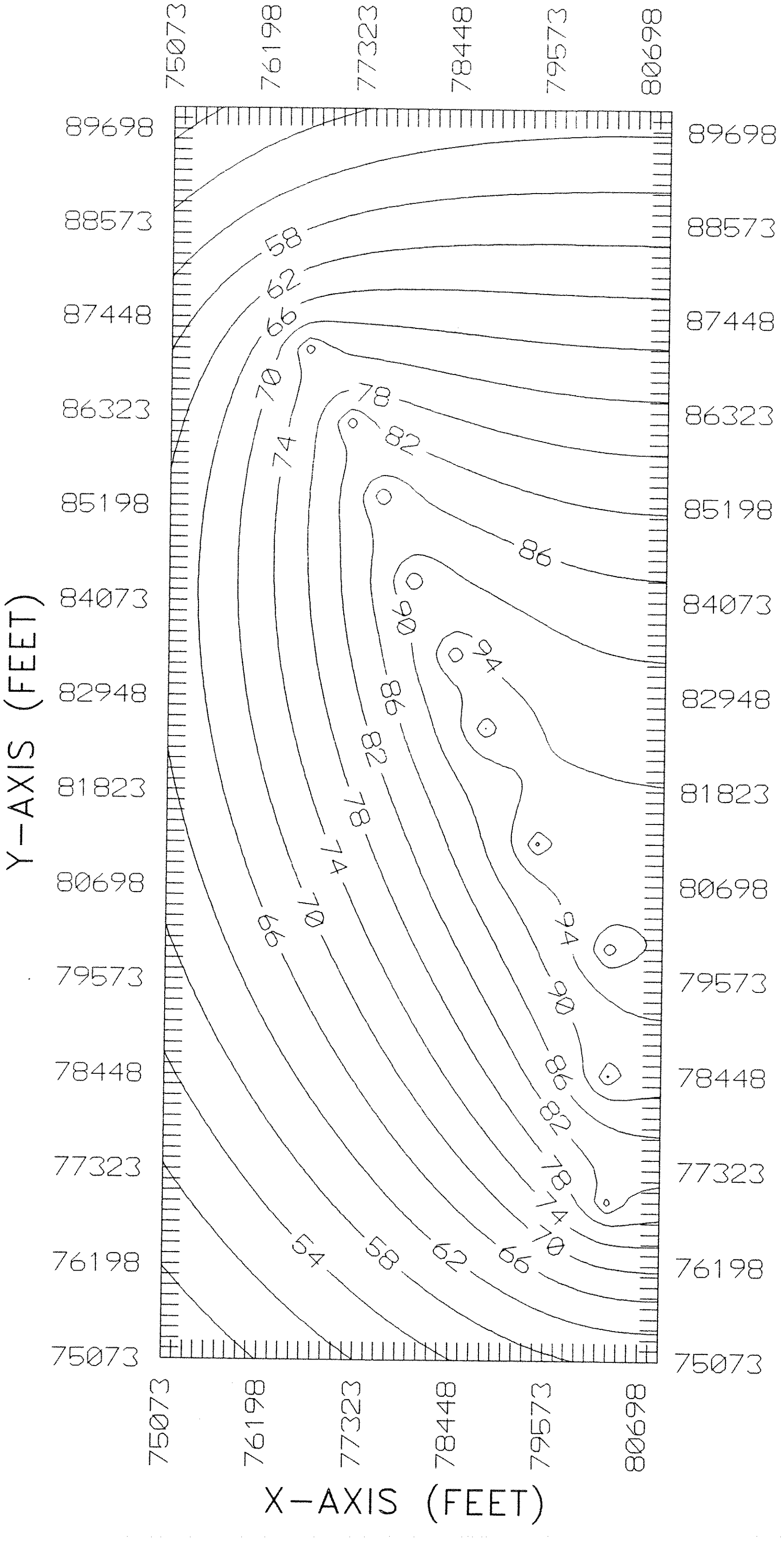

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FIGURE 5-5
MODFLOW MODEL-PREDICTED DRAWDOWN, EMERGENCY
WITHDRAWAL OF 4.7 MGD FOR 14 DAYS

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eastern edge of the MODFLOW model was simulated analytically by a series of image wells located on the opposite side of the boundary. ASR wells in the analytical model were placed at model nodes to compute the highest possible drawdown. The maximum drawdown predicted by both the analytical and MODFLOW models occurs at well ASR-8. Calculated maximum drawdown values for the two models are 60.3 and 53.6 feet, respectively, or a difference of about 7 feet. The formation pressure data reported on Figure 5-3 and Table 5-1 include 7 feet of additional drawdown as a correction factor to account for the MODFLOW solution approach.

A second variable in the MODFLOW model is introduced by assigning constant heads in model Layers 1 and 3, and making related adjustments to the model. Calculations provided in Appendix G suggest that the modeling approach used accurately represents recovery simulations, but may cause the model to overestimate vertical leakage by about 40% for injection simulations. The difference is mainly related to a larger potential error in the gradient between Layers 2 and 3 applied by the model during injection runs. Testing of the storage zone shows that it is only slightly "leaky" and exhibits good characteristics for water storage at the test site. However, integrated over the hundreds of square miles covered by the large-scale hydraulic model, vertical leakage does become a factor in formation pressure response.

Sensitivity of the large-scale model to leakance variation was evaluated using the conceptual wellfield design model described previously. Results indicate that decreasing leakance by 50% throughout the model increases drawdown by about 15% near the production wells. Figure G-6, Appendix G illustrates predicted drawdown for the modified model run. Based on this evaluation, formation pressure data reported on Figure 5-3 include a 1.15 multiplication factor, as a conservative modeling approach.

The lack of data to adequately characterize the hydraulic properties of the storage zone on a regional scale, and the heterogeneity of the system combine to introduce some uncertainty in the modeling process. Therefore, two approaches were used to evaluate model sensitivity to changes in transmissivity and boundary conditions.

In one case, the no-flow boundary at the eastern edge of the model was eliminated by reproducing the cell arrangement west of the model core on the east side of the model. This added 14 more columns to the large-scale MODFLOW model. Model runs were then performed with transmissivity in Layer 2 held constant at both the low and high values calculated from the on-site APT data. Figures G-7 and G-8, Appendix G illustrate predicted drawdown contours for these two model runs. Removal of the eastern no-flow boundary and simulating the aquifer as a homogeneous system with a transmissivity of 25,200 gpd/ft decreases the average drawdown in the core area by about 35%. Execution of the same model with a transmissivity of 13,700 gpd/ft yields similar drawdown values as the conceptual design model, except that the area of maximum drawdown shifts northward to the center of the wellfield.

The final sensitivity analysis of the MODFLOW model was performed to evaluate the effect on the conceptual design model caused by adding a no-flow boundary in the northern and northwestern portions of the model grid. This change in the model design might be warranted if the lateral extent of the storage zone is similar to that depicted on the isopach map illustrated on Figure 3-4. The no-flow boundary condition was simulated by converting the applicable active and constant head cells in Layer 2 to inactive cells, as illustrated on Figure 5-1. Figure G-9, Appendix G shows predicted drawdown for this model run. This change in model design did not significantly influence the drawdown predicted by the model.

5.2 Solute Transport Modeling

In general, the successful recovery of high quality water in ASR projects can be associated with the difference in quality between the injected and native waters and a number of aquifer properties such as matrix versus fracture conductivity, porosity, formation dispersivity, vertical confinement, interval thickness, and interval homogeneity. Although the quality of the native water at the ASR test site is sufficiently high to minimize concerns about recovered water quality, there is some justification for estimating the formation dispersivity at each future ASR well site.

In general terms, formation dispersivity is the major term used to account for the amount of mixing that will occur between the native and injected waters as the ASR wells undergo repeated cycling. Thus, the evaluation of formation dispersivity for each ASR well will lead to better predictions of the overall quality of the recovered water. This information can ultimately be used to enhance storage and recovery operations by allocating storage volumes based on individual well recovery performance. Formation dispersivity provides the numerical insight for direct well performance comparisons.

Since dispersion relates to the mixing between native and injected waters, it must be calculated in a manner that reflects the actual mixing of these waters. This task is best accomplished by matching actual recovered water quality data to model-predicted results. Model simulations for this work are compared with the alkalinity data collected during pilot test Cycles 2 and 3, as discussed in Section 4.3

The solute transport simulator used in this work is the Sandia Waste Isolation Flow and Transport computer program SWIFT III/486) developed for the U.S. Geological Survey (Reeves et. al. 1984). This particular computer program has been used extensively by the hazardous waste injection industry as the tool for estimating solute transport within the subsurface for Class I Injection Wells. This computer program is extremely versatile and has been used to address a number of different types of solute transport issues.

All model runs were performed by using a central in time and central in space solution to the numeric problem. This solution approach was used to eliminate numerical dispersion. Solution overshoot and undershoot problems due to convergence were reduced to less than 2% by appropriate choices in cell size and time steps. The input information for the SWIFT simulations and a summary table of output data for these runs are provided in Appendix H.

For this work, the subsurface was represented using a three layer model. Individual layer parameters as well as the position of the no flow boundary are the same as those used in the small-scale hydraulic model. For the solute transport simulations, a constant transmissivity of 25,200 gpd/ft was applied in all Layer 2 cells. Model injection, storage, and recovery periods were based on the results obtained during Cycles 2 and 3.

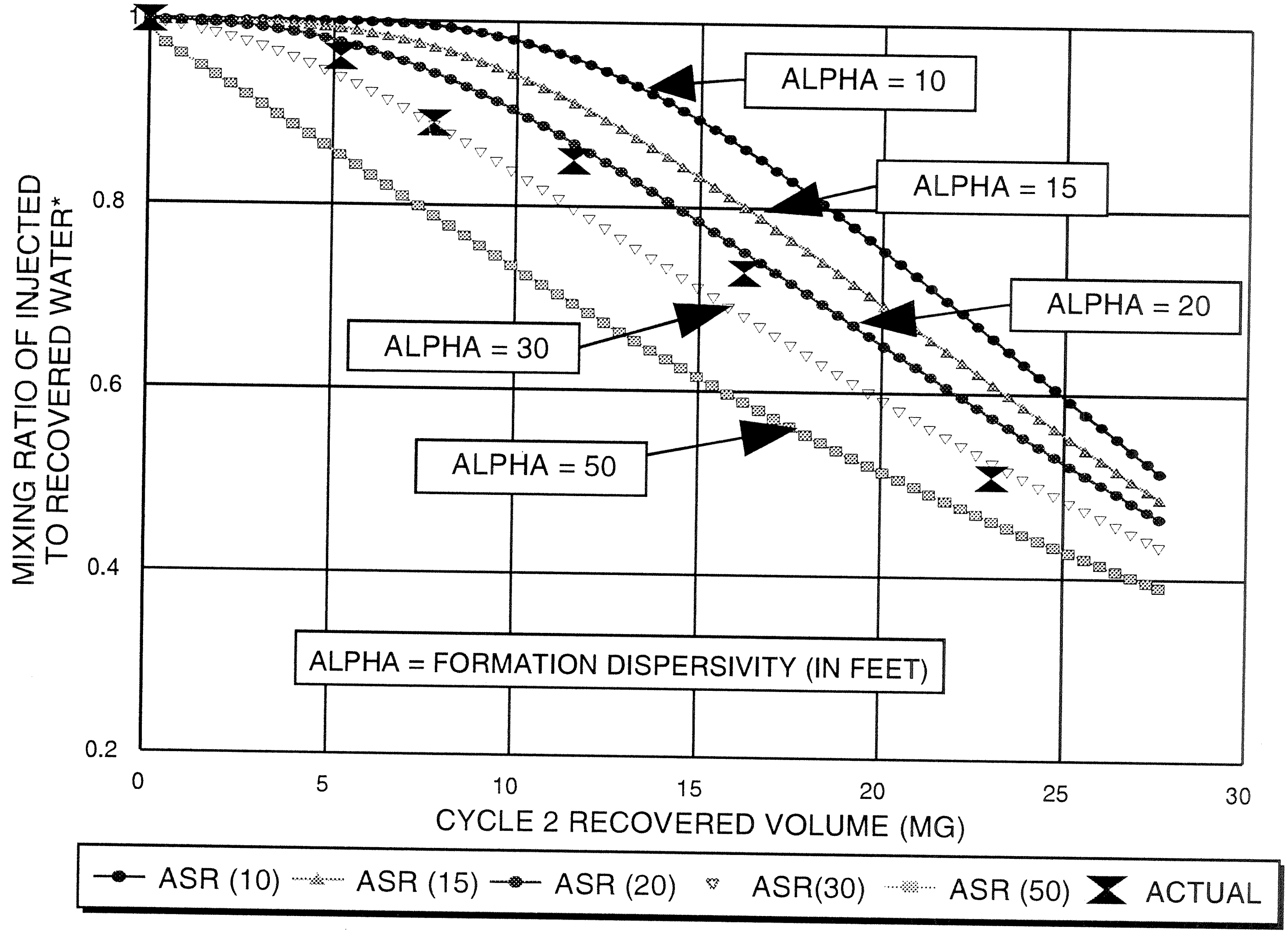
The storage zone was simulated as a flat surface under static conditions, since potentiometric surface maps for the aquifer are not available. Therefore, any solute transport during storage due to regional hydraulic gradients was not considered in the simulations. Published water level data for the mid-Hawthorn aquifer can be used to estimate the regional flow component, as described below. However, considering the fact that there is a 10 foot head difference between mid-Hawthorn aquifer Zones I and II at the ASR test site, use of regional water level data for the composite mid-Hawthorn aquifer to estimate hydraulic gradients and flow rates in the storage zone must be viewed as approximations only.

Data included in a potentiometric surface map for the mid-Hawthorn aquifer by Montgomery (1988) suggest a local hydraulic gradient in the range of $1.6 \text{ X } 10^{-4}$ toward the northwest. Based on a hydraulic conductivity of 51.8 ft/day and a porosity of 0.25, the estimated groundwater flow velocity would be 0.03 ft/day. If these values are correct, the stored water bubble would only be displaced about four feet during the course of a 120 day storage period. Water level data for the storage zone should be collected as new wells are constructed to confirm or adjust these hydraulic gradient and flow rate estimates. The updated information should be considered in the development of future transport simulations for the design and construction phase of the project.

Figure 5-6 compares the results of five individual model runs using different values of dispersivity with the actual alkalinity data obtained during the recovery phase of Cycle 2. As is evident in Figure 5-6, the actual data, assuming a porosity of 0.25, are bracketed by formation dispersion coefficients of 20 feet and 30 feet, respectively. Based on this information, a dispersivity of 25 feet and a porosity of 0.25 is currently assigned to the local rock matrix surrounding the ASR test well.

Once the near well dispersivity was calculated, data from both Cycle 2 injection and recovery were utilized to estimate the dispersivity associated with fluid movement near monitor well C (refer to Figure 5-7).

Analysis of the data for monitor well MW-C indicate that at this well, which lies 294 feet from the ASR well, the overall dispersivity increases to a value of 50 or higher based on the alkalinity data for the recovery phase of Cycle 2. The increase in dispersivity with distance was anticipated based on the available literature (de Marsily, 1986). However, the difference in apparent dispersivities at MW-C for injection and recovery phases was not anticipated, and as yet, remains to be explained. The impact of increasing dispersivity as a function of distance from the ASR well indicates that the current plan to use multiple, relatively low volume wells may be far more appropriate than using fewer high volume wells. Reasonable agreement of the solute transport model results to actual field data was achieved with the same transmissivity, leakance and boundary conditions used in the



* A VALUE OF 0.75 INDICATES WATER SAMPLE CONTAINS 75% INJECTED WATER AND 25% NATIVE WATER

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FIGURE 5-6
ACTUAL AND MODELED ALKALINITY CONCENTRATIONS
ASR WELL

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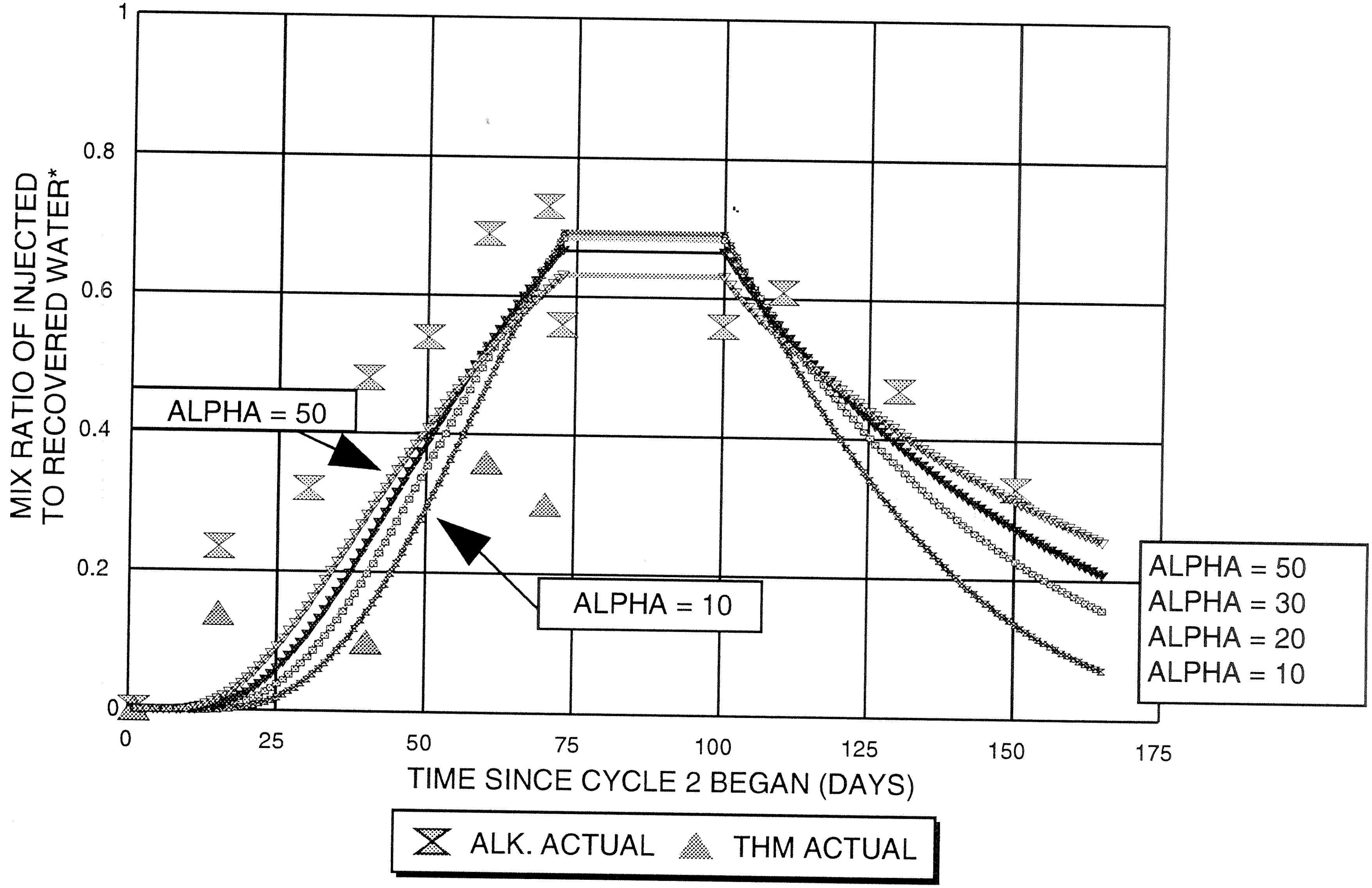

* A VALUE OF 0.75 INDICATES THE RECOVERED WATER CONTAINED 75% INJECTED WATER AND 25% NATIVE WATER

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FIGURE 5-7
ACTUAL AND MODELED ALKALINITY CONCENTRATIONS
MONITOR WELL MW-C

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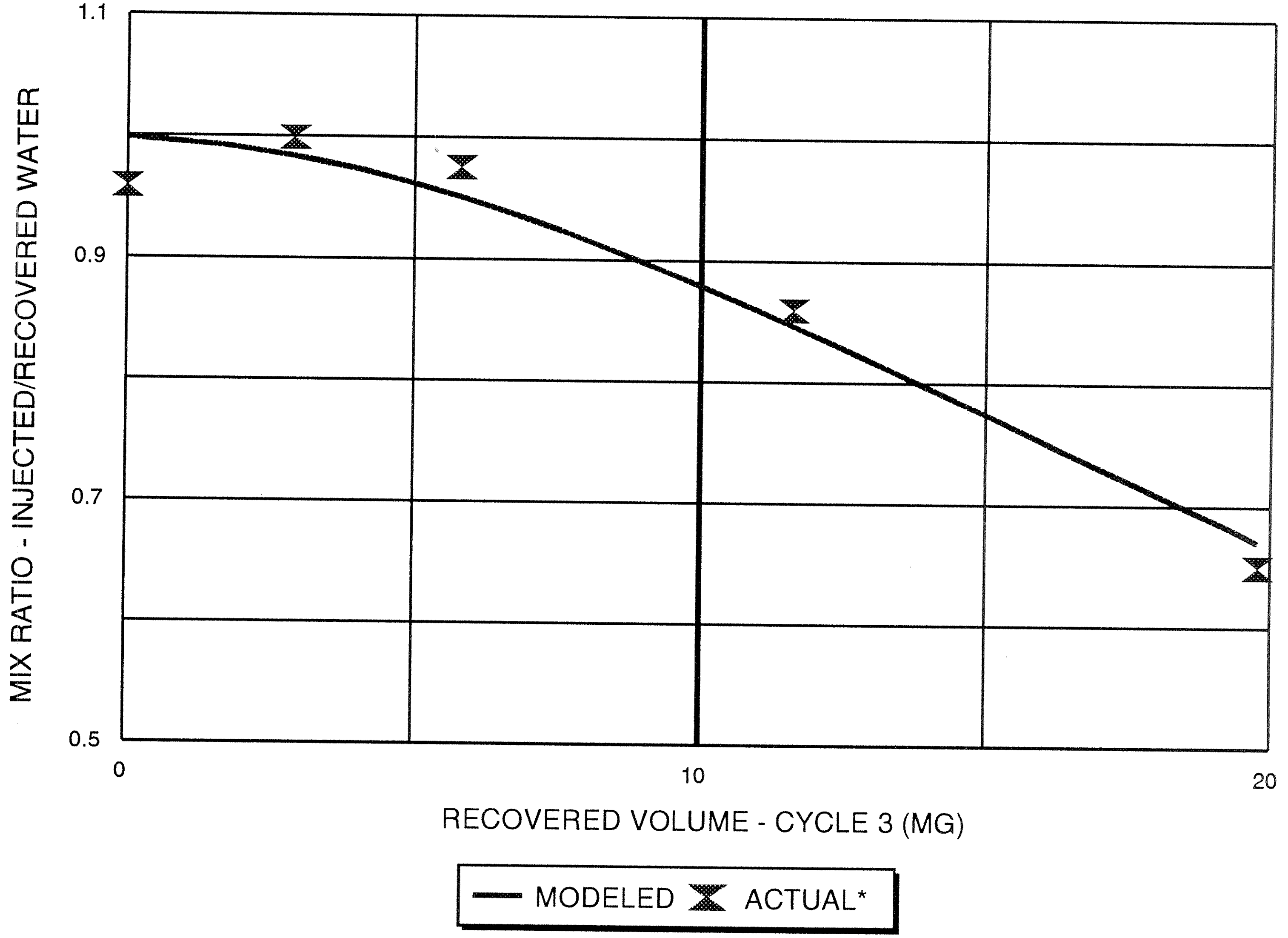
hydraulic model. Therefore, the solute transport results support the basic hydraulic model design.

As a final evaluation, the actual recovery data for Cycle 3 was compared against the modeled recovery using a porosity of 0.25 and a formation dispersivity of 25 feet. The ratio of injected to recovered water for this simulation is based on an alkalinity concentration, at saturation, of 61 mg/l. Figure 5-8 presents the comparison which shows an excellent fit between the actual data and the modeled data. These results reinforce the conclusions developed for Cycle 2.



* ACTUAL CONCENTRATIONS BASED ON THE PREMISE THAT RECOVERED WATER CANNOT EXCEED 100% INJECTED WATER

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FIGURE 5-8
COMPARISON BETWEEN RATIO OF INJECTED WATER TO RECOVERED WATER AS MODELED AND AS CALCULATED FROM ALKALINITY DATA FOR CYCLE 3

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6.0 ASR SYSTEM DEVELOPMENT

6.1 Preliminary Wellfield Configuration and Development Plan

The ASR wellfield design criteria was based on limiting formation pressure responses during injection and recovery of the water supply. The ASR system is also being developed with the capacity to provide water supply for a sustained period of time during emergency operations, such as a hurricane or wellfield contamination. Based on hydraulic modeling results, the linear alignment of wells along Alico Road is the recommended preliminary wellfield configuration. However, additional subsurface testing to the west of the Corkscrew WTP along Corkscrew Road, in land currently owned by Lee County, is recommended as part of the ASR development plan to further evaluate ASR system expansion in this area. Construction activities to develop the ASR system should be conducted in stages, with additional hydraulic testing performed at key points to determine the most advantageous ASR well configuration at the site.

The conceptual wellfield configuration is shown on Figure 6-1 for a 3.0 mgd ASR system. This figure also shows well sites acquired by the county during a recent land exchange, and alternate or future sites for construction of the ASR wells. Six to seven additional well sites would be required to expand the ASR system to 5.0 mgd.

The preliminary wellfield configuration recommends the first two wells running either northerly or southerly be spaced at 1,500-foot intervals, and the remaining wells be spaced at 1,000-foot intervals. Well site No. 1 was recently acquired by the county and is approximate 2,000 feet north of the Corkscrew treatment plant, with well sites No. 2 and No. 3 spaced at approximately 1,500-feet intervals. ASR wells were located at these sites to minimize land acquisition requirements. Spacing of the ASR wells were continued on 1,000-foot intervals after well site No. 3. The linear

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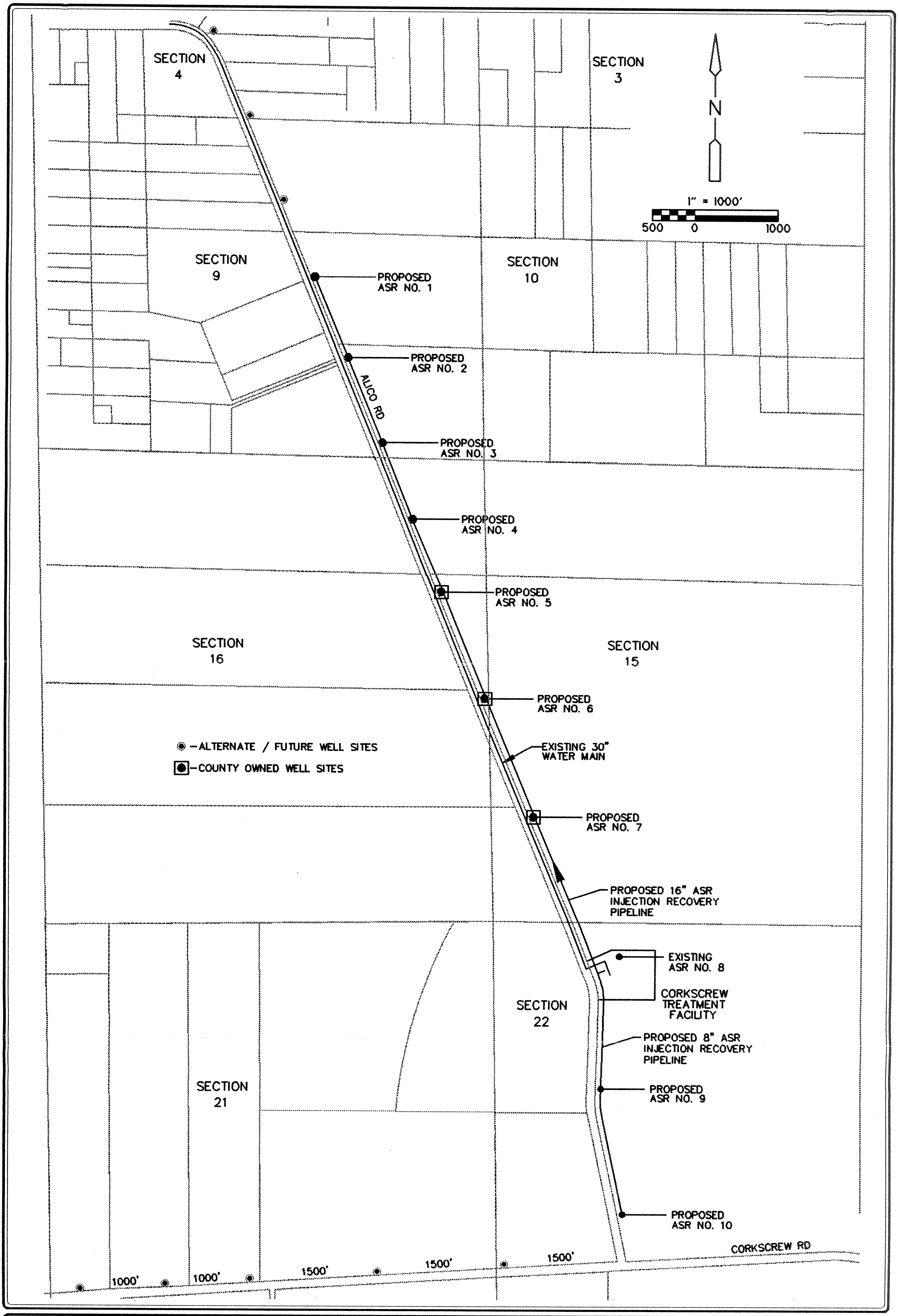

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CORKSCREW WATER TREATMENT FACILITY
ASR WELL PILOT PROGRAM
ASR WELL LOCATION MAP

FIGURE 6-1

alignment of wells on Alico Road will be in an area of approximately 11,000 linear feet with two wells spaced at 1,500-foot intervals, south of the plant, and seven wells located north of the Corkscrew treatment plant as described above. The existing pilot well will be converted to a functional ASR well.

In addition to the preliminary wellfield layout an alternate wellfield arrangement was evaluated which included siting 5 wells along Corkscrew Road. This arrangement showed slightly higher pressure response during hydraulic modeling, however, it may prove to be a suitable alignment after additional testing.

6.2 Fluid Compatibility Analysis

A series of water quality tests were conducted to evaluate fluid compatibility when the native water is mixed with the injected water. Precipitation of insoluble products may occur as a result of the water mixing. In addition, a geochemical analysis of the water quality data was conducted to review saturation of minerals when the injected water is mixed with native water.

Analysis of suspended solids and turbidity was conducted for several blends of native and injected water. Two samples were prepared for each blend: one sample was allowed to set for 24 hours, the other sample was allowed to incubate in the dark for one week. Results from the suspended solids testing is represented on Table 6-1. Results indicated that no increase in suspended solids or turbidity occur (over background levels) in the samples tested.

In addition to the fluid mixing test an analyses of the geochemical properties of the native and injection water and the resultant mixture of the two was conducted. Geochemical analysis of the water quality data indicate that all waters are similar chemically, given the chemical analyses conducted and the assumptions described below.

TABLE 6-1

ASR PILOT PROJECT CORKSCREW WATER TREATMENT PLANT
WATER BLENDING ANALYSIS

	Suspended Solids (mg/l)	Turbidity (NTU)
Native Water	Not detected	0.21
	Not detected	0.18
Injected Water		
Sample 1[2]	0.15	0.40
Sample 2[3]	0.18	0.37
10: 80[1]		
Sample 1	0.17	0.36
Sample 2	0.34	0.38
40: 60		
Sample 1	0.15	0.35
Sample 2	0.10	0.30
60: 40		
Sample 1	0.14	0.26
Sample 2	0.05	0.28
80: 20		
Sample 1	Not detected	0.24
Sample 2	Not detected	0.24

[1] % Native water/% Injected water ratio.

[2] Sample 1 was incubated for 24 hours.

[3] Sample 2 was incubated for 7 days.

- The finish product water is considerably more dilute than the native groundwater. This should result in dilution of the groundwater upon injection of (mixing with) the finish product water. Minerals at, near, or above saturation in the native groundwater would not tend to precipitate upon mixing with the finish product water due to the dilution effect.

- For purposes of geochemical modeling, these parameters were estimated as follows: K was assumed to 0.1 mg/L; Mg was calculated using Ca concentration and total hardness (total hardness = 2.5 (Ca) + 4.1 (Mg); Na was then calculated based on the assumption of perfect charge balance.

Polyphosphate Addition to Treated Water Before Storage

A corrosion inhibitor is added to the finish water produced at the Corkscrew water treatment plant. Sodium zinc metaphosphate (zinc polyphosphate) distributed by SHAN-NO-CORR Chemical Corporation is currently being used at the plant.

The chemical is added to reduce corrosion of lead and copper material. Copper pipe and lead joints has been used in residential and commercial plumbing for many years. The addition of a corrosion inhibitor is a common practice in the water treatment industry and is necessary to provide water quality at the tap that meets lead and copper regulatory requirements.

The addition of polyphosphate with water containing iron or calcium can result in the precipitation of iron phosphate and/or calcium phosphate. The native groundwater in the ASR injection zone contains both calcium and iron in sufficient quantities to react with the polyphosphate and form a precipitate. However, the concentration of iron and calcium in the native groundwater should not result in a significant amount of precipitation to cause a problem. The potential for precipitation of iron or calcium phosphate should be minimized as the ASR system storage bubble is developed and the mixing front moves away from the well bore.

Clogging of the injection zone during the first week of ASR operation due to precipitation of the polyphosphate corrosion inhibitor is not considered the likely source of the observed pressure increases. However, design of the long-term ASR system should relocate the chemical addition of polyphosphate to a point after the ASR supply location.

6.3 Blended Water Quality

Mass balances were used to calculate the water quality of combined finished water and recovered ASR water. Water quality estimates were made for three blends of Corkscrew finished water (6, 8 and 10 mgd), combined with three ASR recovery flows (1, 2 and 3 mgd). Blended water quality was estimated for the following parameters: alkalinity, total hardness, calcium hardness and color. The parameters were selected because the concentrations in the ASR recovery water were higher or lower than concentrations typically found in the treated water produced by the Corkscrew WTP. ASR recovery water quality is based on water quality test results from Cycle 3 and the Corkscrew finished water quality goals established by Lee County. Established water quality goals for Corkscrew WTP finished water are provided below:

PARAMETER	WATER QUALITY GOALS FOR THE WTP
Alkalinity, ppm	67
Total Hardness, ppm	86
Calcium Hardness, ppm	76
Color, color units	6

Table 4-2, Appendix B provides the ASR recovery water quality for Cycle 3. Figures in Appendix I provide the blended water qualities for the operating scenarios described above.

As these figures indicate, the highest alkalinity concentrations encountered in the recovery water occurred in the later days (Day 20 and Day 30). The maximum blended water alkalinity occurs when the Corkscrew WTP finished water flow is lowest, 6 mgd. The highest calculated alkalinity concentration ranges from 69 to 84 mg/l on recovery Day 30, using 1 to 3 mgd of recovered water blended with 6 mgd finished water flow from the Corkscrew WTP.

The highest total hardness concentrations are encountered in the initial recovery day (Day 0). The highest total hardness occurs when blended with the Corkscrew WTP finished water flow at 6 mgd. The maximum calculated total hardness of the blended water ranges from 92 to 101 mg/l on recovery Day 0, with a 6 mgd finished water flow from the Corkscrew WTP. During later recovery days, total hardness concentration stabilized in the range of 91 to 97 mg/l when 1 to 3 mgd of recovered water is blended with a 6 mgd Corkscrew WTP flow.

The highest calcium hardness concentrations are in the initial recovery day (Day 0). The maximum calcium hardness occurs when the Corkscrew WTP finished water flow is at 6 mgd and is blended with 1 to 3 mgd of recovered water. The maximum calcium hardness concentration ranges from 81 to 89 mg/l on recovery Day 0. Calcium hardness concentrations stabilize in the range of 79 to 83 mg/l during the later recovery days.

The highest color occurs during the initial recovery day (Day 0), when the recovered water is blended with the Corkscrew WTP operating at 6 mgd. The highest calculated color value ranges from 7.1 to 8.7 on recovery Day 0 with a 6 mgd finished water flow from the Corkscrew WTP. During later recovery days, color stabilize between 6.6 to 7.3 when blended with a 6 mgd Corkscrew WTP flow during the later stages of recovery.

The results of blending of ASR recovery water with finished water from the Corkscrew WTP is a net increase in alkalinity, total hardness, and calcium hardness and a reduction in color. The expected range of these concentrations is presented in Table 6-2.

6.4 Chemical Addition Requirements

Disinfection of the recovered water is necessary to meet the drinking water standards established in Chapter 62-550, F.A.C., and to provide a chlorine residual in the distribution system. Disinfection at the Corkscrew WTP is accomplished through the addition of chlorine and ammonia to produce chloramines. Chlorine and ammonia are added to the filtered water supply at the header between the filters and the filtered water pump station. The filtered water pump station transfers the water from the filters to the 5 mg storage tank. The water supply is stored in the existing 5 mg storage tank to provide additional contact time for disinfection. A schematic diagram of this process is provided in Section 6.5

Water recovered from the ASR wells will be blended with treated water from the plant prior to storage in the 5 mg tank. The chlorine and ammonia chemical feed systems should be modified to provide a set-point chlorine residual to maintain adequate disinfection of the blended water supply.

In addition to modifications of the disinfection system, relocation of the polyphosphate injection point is recommended. During injection of water into the ASR wells, the polyphosphate corrosion stabilizer should be added after the ASR injection water connection to the existing 30-inch water transmission main. Relocating the chemical addition point at the flow meter facility on the 30-inch

TABLE 6-2 SUMMARY OF ANTICIPATED BLENDED WATER QUALITY
CORKSCREW WTP - ASR PILOT PROJECT

CONSISTENT	BLENDED WATER[1] CONCENTRATION	DRINKING WATER STANDARDS
Hardness	85-105 mg/l	250 mg/l
Alkalinity	60-90 mg/l	Not Applicable
TDS	200-300 mg/l	500 mg/l
Iron	0.03-0.10 mg/l	0.3 mg/l
Manganese	<0.001	0.5 mg/l
Sulfate	20-30 mg/l	250 mg/l
THM (Total)	0.01-0.06 mg/l	0.10 mg/l
Gross Alpha	1-4 pCi/l	15 pCi/l
Color	6-8 color units	15 color units
pH	7-8	6.5-8.5

Notes

[1] Blended water concentrations is based on a recovered water flow rate between 1 to 3 mgd, mixing with a flow rate of 5 to 10 mgd from the Corkscrew WTP meeting established water quality goals.

transmission main would accomplish this system modification. Based on discussions with Lee County Health Department, relocating the polyphosphate injection location will not require relocating the water quality sampling point for the plant.

Mild acid stimulation of the ASR wells will be periodically required to maintain injection pressures below operating pressure of the county's 30-inch water transmission main. Acid injection can be accomplished through manual addition at each well site or an automated system located at the Corkscrew WTP. Acid stimulation will require the preparation of the solution and injection at a predetermined volume. Providing a centralized acid injection system at the Corkscrew WTP provides several advantages including: minimizing the handling and transportation of chemicals, minimizing risks to operations personnel, and reducing manpower requirements. The automated acid injection system will include a 6,000 gallon solution tank, chemical feed pump and controls, separate 1-1/2 to 2-inch chemical line to the ASR wells. To provide for operator safety, emergency eyewash and shower should be provided both at the acid solution batching area and on the service truck for the ASR system. Design and permitting of the acid injection system would have to meet applicable requirements of the Florida Department of Environmental Protection.

The use of a sequestering agent in the injected water, such as citric acid, may inhibit iron precipitation at the face of the wellbore, and reduce the rate of pressure increase during water injection. The benefit of sequestering agent addition should be further investigated during the preliminary design of any ASR system expansion.

6.5 ASR System Operations and Maintenance

The ASR system operations has three distinct phases: 1) injection phase, 2) storage phase and 3) recovery phase. In addition, the ASR system will be available for emergency operations year-round. Table 6-3 presents the approximate operating schedule for the ASR system.

Injection Phase

Water injection should occur during the rainy season when supply is more abundant and water demands are lower. Table 6-3 presents the average monthly flow factor the Corkscrew WTP for the last three years of operations. These factors represent the average flow for the month, divided by the annual average flow. The rainy season and lower flows generally occur from May through September. Injection of water into the ASR systems will typically need to be accomplished during this period. A daily flow rate ranging from 1 to 3 mgd will be required to provide 270 million gallons of stored water in the ASR system. During the initial years of operation, additional water supply will need to be injected into the ASR wells to develop the storage zone. An additional 20-30 million gallons should be injected during the first 3 to 5 years of operation. The average injection rate at each ASR well should be approximately 175-180 gpm.

One of the goals of the ASR system is to maximize the water supply and treatment capacity of the Corkscrew wellfield and water treatment plant. Increasing the permitted withdrawal from the surficial aquifer during the rainy season will allow optimal use of the existing wellfield. Table 6-3 lists the average volume of water available for ASR storage during water years 1994 through 1996, compiled on a monthly basis. An increase in the surficial aquifer allocation to a maximum day withdrawal of 11.5 to 12.0 mgd during the injection periods, and a 290 to 320 million gallon increase in the annual groundwater allocation would provide a sufficient volume of water to fill the ASR system.

TABLE 6-3 CORKSCREW WTP - ASR PILOT PROJECT MONTHLY WATER PRODUCTION FLOW FACTORS FOR THE CORKSCREW WTP, AND TYPICAL ASR OPERATING SCHEDULE

MONTH	OPERATING SCHEDULE	MONTHLY FLOW FACTOR[1]	AVAILABLE WATER FOR ASR STORAGE[2] (mg)
January	Storage	0.99	
February	Storage	1.04	
March	Recovery	1.10	
April	Recovery	1.07	
May	Storage	1.01	
June	Injection	0.94	18
July	Injection	0.89	34
August	Injection	0.88	38
September	Injection	0.90	30
October	Storage	1.03	
November	Recovery	1.11	
December	Storage	1.03	
	TOTAL STORAGE:		120

[1] Monthly flow factors = $\frac{\text{Monthly Flow}}{\text{Annual Average Flow}}$
The monthly flow factors were averaged for 1994, 1995 and 1996.

[2] Assuming Average Annual Flow of 10 mgd
Available Water = (10 mgd - 10 (Monthly Flow Factor)) * days in month

Storage Phase

The storage duration of the ASR water supply will vary depending on the system demands and rainfall patterns. Based on the flow factor established in Table 6-3, storage will typically occur during May, December, January and portions of October and February. Recovery of stored water might occur during these months to enable the treatment plant to operate at a steady treatment rate during peak demand periods, or in response to an emergency condition at the WTP.

Recovery Phase

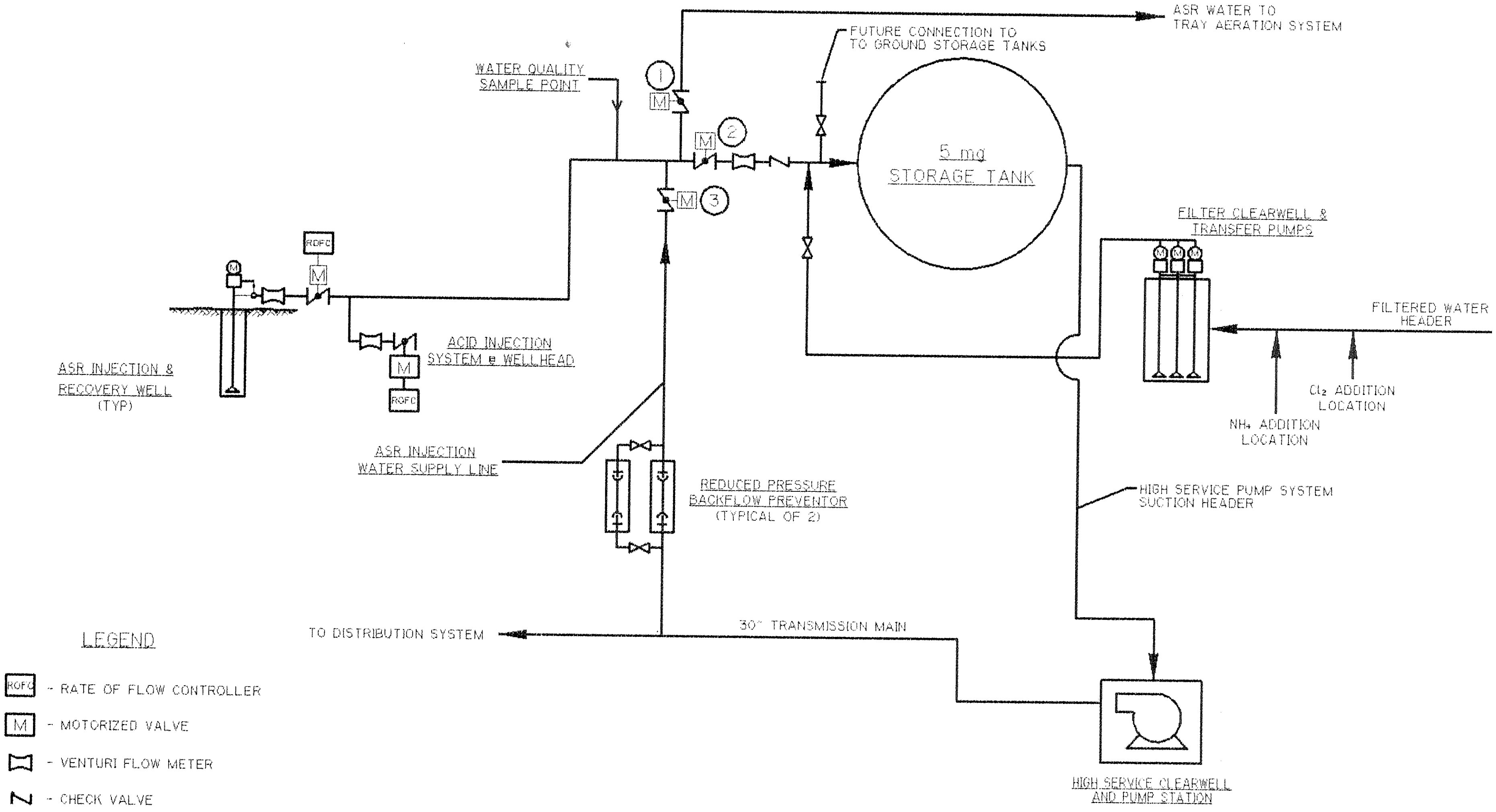
Recovery of stored water will occur predominantly during November, April and March and during peak demand periods in December, January, and February. The ASR system is also being designed to allow recovery during emergency operations. The ASR system will be designed to allow recovery at a maximum rate of 325 gpm from each well (total capacity 4.6 mgd). The maximum recovery will depend on the specific capacity of each well and will be evaluated during ASR system construction. The total recovery goal for the system is 270 mg. This equates to an average day flow of 3.0 for a 90-day period.

ASR System Process Diagram

Figure 6-2 presents the ASR process diagram. Water supply for injection will be provided from the 30-inch water transmission main at line pressure.

Two 8-inch reduced pressure back flow preventers (one is existing) will be provided at the interconnect with the 30-inch water transmission main.

During the injection cycle, water will be conveyed to the ASR wells through the common ASR system water transmission main. Motorized valves (valves 1 and 2)



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CORKSCREW WATER TREATMENT FACILITY
ASR WELL PILOT PROGRAM PROCESS DIAGRAM
FIGURE 6-2

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located on the supply main to the 5 mg storage tank and to the tray aerator will be closed during the injection cycle. The rate of flow to each well will be regulated by a modulating butterfly valve. The flow rate to each well will be determine during well construction and testing and should range between 150 and 250 gpm.

During the storage cycle all motorized valves (valves 1, 2 and 3) will be closed. When water recovery is requested, the plant operators will select the number of wells necessary to meet the desired recovery rate. Recovery from each well will be approximately 325 gpm. Prior to initial recovery, each well will be back pumped to flush deposits that accumulated during the injection cycle. Back-flushing of the wells should be accomplished once each year just prior to recovery period. All water recovered during the flush cycle will discharged to the front of the plant for treatment. Back-flushing of each well should be conducted for one to two hours. A water quality sampling port will be provided on the ASR water transmission main. The motorized valve on the ASR water supply line and to the tray aerator (valves 1 and 3) will be closed during the recovery cycle. The ASR supply will be blended with the plant's finished water in the 5 mg storage tank. Blending the water supply at this location will allow adequate mixing and provided a consistent water quality to the utility customers. Additional disinfection agent will be added to the blended water supply in the high service pump suction line, as necessary for distribution system chlorine residual.

Maintenance of the ASR system wells will require periodic injection of a low concentration acid solution. The acid will be provided to open the pores around the well borehole. Mixing of a low concentration acid solution will be conducted at the treatment plant. A separate acid solution transmission line and flow control will be provided to each ASR well. The operator at the treatment plant will select the wells to be treated and a predetermined volume of the low concentration acid will be injected into the well. Mild acid stimulation of the wells needs to be conducted each year prior to the injection phase. Periodic back-flushing of the wells for a short period of time will also likely be required during the injection period. The need for back-flushing will depend on individual well performance, and may be conducted bi-weekly. The ASR system will conserve all water flushed from the wells by returning the water to the head of the plant for treatment and use in the water system.

6.6 Project Schedule and Construction Cost Estimate

Schedule

The preliminary schedule for completing design, receiving bids, and awarding a contract for the ASR system is as follows:

3 months	Preliminary and Final Design
3 months	Permitting
7-9 months	Construction of Facilities

Once the Notice to Proceed has been issued, the Contractor will be required to complete construction within 7-9 months. The project can be completed within one year. Although construction of the ASR system can be accomplished within one year, operational testing and regulatory approval to operate the system may require additional time.

Capital Costs

Table 6-4 presents the probable cost of construction for the ASR system. The preliminary probably cost of construction is approximately \$2.7 million for the 3 mgd ASR system. The construction cost also reflects the cost to size common components that would be used for a 5 mgd system. The common components sized for additional capacity could be used and funded by other members of the authority.

TABLE 6-4 CORKSCREW WTP - ASR PILOT PROJECT
ESTIMATE OF PROBABLE CONSTRUCTION COSTS
3.0 MGD ASR SYSTEM

Item	Construction Costs
12-inch ASR Wells (9 wells)	$540,000
4-inch Monitor Wells	$ 80,000
Well Pump Tests	$ 30,000
Recovery Injection Piping and Valves	$640,000
Wellhead Piping and Pumps	$300,000
Acid Injection System	$ 36,000
Acid Feed Piping	$ 66,000
Chlorine Feed Modification	$ 30,000
Relocate Corrosion Stabilizer Addition	$ 10,000
Subtotal	$1,732,000
Electrical/Telemetry System	$200,000
Miscellaneous Structure/Concrete @5%	$ 50,000
Instrumentation @ 5%	$ 50,000
Subtotal	$2,032,000
Contingency @ 15%	$305,000
Engineering/Construction Services @ 20%	$406,000
Total Project Construction Cost	**$2,743,000**

Note: Construction costs do not include cost for well sites legal or administration costs.

Operating Costs

Operating costs for the ASR will include the additional pressure required to inject and recover the water supply, losses due to system efficiency and well maintenance. Injection and recovery pressure will be approximately 65 psi and 80 psi, respectively. Table 6-5 presents the annual operating costs per 1000 gallons for the ASR system. These costs are based on injection of 270 million gallons annually and recovery of 270 million gallons. As the storage bubble is developed in the aquifer, injection and recovery should be approximately the same. During the initial 3 to 4 years of operations, an additional 30 million gallons will need to be injected into the ASR system to establish the storage zones. The annual operating cost to treat and inject this water is approximately $22,000. This estimate is based on treatment cost of $0.60 per 1000 gallons for lime softening water treatment and the pressure to inject the water.

Cost/Benefit Analysis

Operations of the ASR system will benefit the Lee County Utilities customer by optimizing the use of existing Corkscrew water supply and treatment facilities. The savings to the utility customer can be estimated by comparing the costs required to expand the Corkscrew WTP and wellfield to provide an additional 3.0 mgd of treated water. Modifying the consumptive use permit for additional water supply at the Corkscrew wellfield would possibly require tapping the Floridan aquifer or acquiring additional well sites to allow increased withdrawals from the Sandstone or surficial aquifers. The water quality of the Floridan aquifer would require treatment by low pressure reverse osmosis treatment, while lime softening treatment is adequate for treating additional water supplied from the surficial or Sandstone aquifers. Table 6-6 presents both capital and operating considerations for ASR and conventional water supply and treatment options. Table 6-7 presents the estimated capital and operating costs for each of water supply and treatment options.

TABLE 6-5

CORKSCREW WTP - ASR PILOT PROJECT
ASR SYSTEM OPERATING COST
3.0 MGD ASR SYSTEM

COST COMPONENT	ANNUAL OPERATING COSTS (DOLLARS)	COST PER 1000 GALLONS (DOLLARS)
Injection Pressure Energy	$16,000	0.060
Recovery Pressure Energy	$17,200	0.063
Well Maintenance Costs	$24,000	0.086
TOTAL MAINTENANCE COST:	$57,200	$0.22

Note: Total costs based on operations of a 270 million gallon annual storage recovery and 300 million gallons injected. Energy costs based on $0.08 per kw/hour.

TABLE 6-6

COMPARISON OF SYSTEM REQUIREMENTS ASR SYSTEM AND CONVENTIONAL WATER SUPPLY AND TREATMENT

	CAPITAL COSTS	OPERATIONS COSTS
ASR	-ASR wells -injection/recovery pipeline -Well maintenance system -instrumentation & controls -chemical feed system	-injection/recovery pressure -rechlorination -distribution system pressure -system maintenance -water losses and back flushing
Conventional Water Supply and Treatment	-additional water supply wells -treatment processes -chemical feed system -instrumentation & controls -disposal system	-distribution system pressure -treatment costs -maintenance costs -chemical feed system

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TABLE 6-7

CORKSCREW WTP ASR PILOT PROJECT CAPITAL AND OPERATING COST ANALYSIS FOR ASR AND CONVENTIONAL WATER TREATMENT SYSTEM

SYSTEM/COMPONENT	CAPITAL COSTS (DOLLARS)	OPERATING COSTS (DOLLARS/1000 GALLONS)
ASR System		
Wells & Appurtenances	1,300,000	0.12
Pipeline and Appurtenances	1,400,000	0.10
Storage Zone Establishment	66,000	
TOTAL COSTS:	$2,766,000	$0.22
Conventional Water Treatment System/Reverse Osmosis		
Wellfield	580,000	0.30
Treatment Plant	4,500,000	1.00
Reject Disposal Well	2,800,000	0.05
TOTAL COSTS:	$7,880,000	$1.35
Conventional Water Treatment System/Lime Softening		
Wellfield	1,400,000	0.18
Treatment Plant	3,900,000	0.55
Backwash Recovery Ponds	300,000	0.04
TOTAL COSTS:	$5,600,000	$0.77

The capital cost for constructing the ASR system, including the cost for establishing the storage bubble, is approximately $2.8 million; and the operating cost is approximately $0.22 per 1,000 gallons of water supplied. The capital cost for supplying and treating additional groundwater using either lime softening or reverse osmosis treatment is between $5.60 and $7.88 million. Operating costs for these treatment options range from $0.77 to $1.35 per 1,000 gallons. The cost savings to Lee County customers for constructing the 3.0 mgd ASR system will range from $2.8 to $5 million, and an operating cost savings of $0.77 to $1.13 per 1,000 gallons of water recovered from the ASR system will be realized. An additional benefit of constructing the ASR system is the emergency storage and supply it will provide. This supply can be readily recovered using the potable generators and can provide a maximum capacity of 4.6 mgd for distribution to utility customers.

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