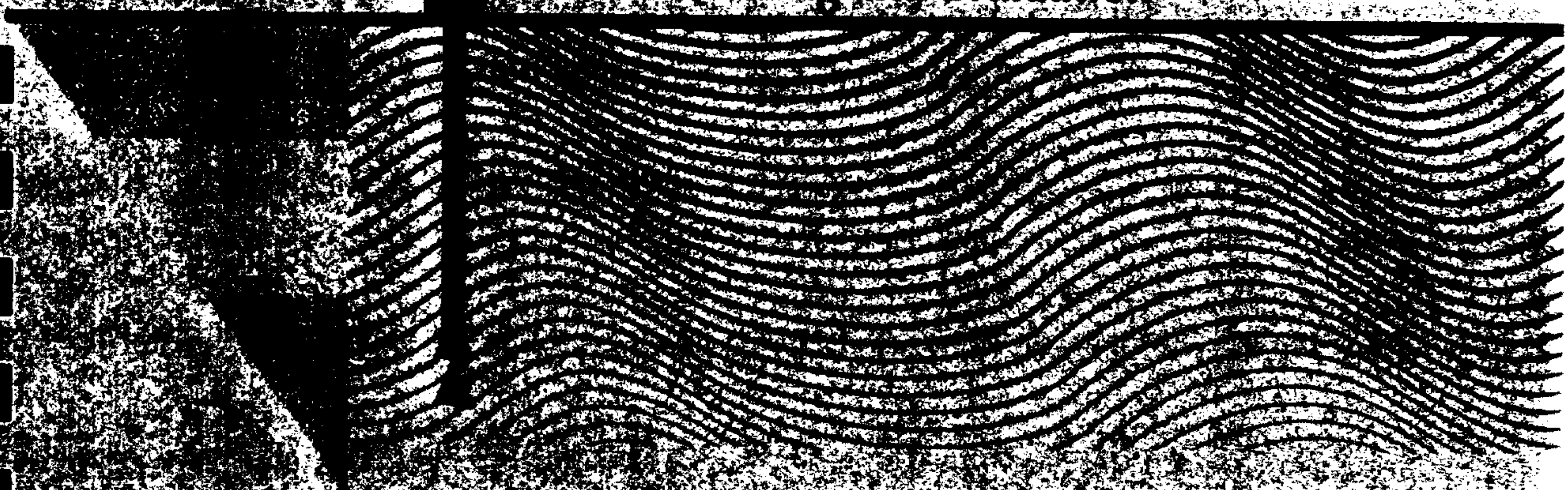
Aquifer Storage Recovery Feasibility Investigation

Phase IIB: Full-Scale Testing and Evaluation



Prepared for

Upper Guadalupe River Authority
Kerrville, Texas

CH2M HILL

AQUIFER STORAGE RECOVERY FEASIBILITY INVESTIGATION

PHASE IIB: FULL-SCALE TESTING AND EVALUATION

Prepared for:

UPPER GUADALUPE RIVER AUTHORITY
KERRVILLE, TEXAS

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PHASE IIB ASR FEASIBILITY STUDY
FULL-SCALE TESTING AND EVALUATION

TABLE OF CONTENTS

TABLE OF CONTENTS (continued)

LIST OF TABLES

LIST OF FIGURES

EXECUTIVE SUMMARY

EXECUTIVE SUMMARY

This report summarizes work performed by CH2M HILL for the Upper Guadalupe River Authority (UGRA) in 1990 and 1991 as Phase IIB of ongoing studies to determine the feasibility of aquifer storage recovery (ASR) in Kerrville. Because the test results for ASR are positive, this phase also included development of an implementation plan.

Phase I was a preliminary feasibility assessment (completed in 1988); Phase IIA consisted of drilling, geochemical modeling, and other detailed analyses designed to confirm the feasibility of ASR with more certainty (completed in 1989).

Phase IIB--covered in this report--included construction and testing of a full-scale ASR system, as well as additional study of water supply and demand, groundwater modeling, and surface water modeling. The implementation plan focuses on a strategy for operating the water treatment plant, permitting, and rehabilitation of City of Kerrville wells to assure adequate capacity for withdrawal when stored supplies are needed.

The Phase IIB Study has concluded that ASR has the potential to provide the following benefits to UGRA and the residents of Kerrville:

- Provide an assured water supply during peak-use and drought periods for the next 20 years
- Help restore water levels in the Hosston-Sligo Formation of the Trinity Aquifer
- Protect stored water from evaporative losses and contamination
- Make use of currently underutilized treatment plant capacity
- Provide a secure water supply for use in emergency situations (e.g., flooding or temporary contamination of river water)
- Reduce demand on Guadalupe River flow during dry summer months
- Prevent the environmental impact of a surface water reservoir
- Be easy to operate

This report recommends implementing the ASR program outlined in Section 7 instead of proceeding with plans to expand the UGRA's surface water treatment plant and to construct a surface water reservoir on the Guadalupe River. The ASR program represents a major savings over the plant expansion/surface reservoir alternative, in addition to providing the benefits listed above.

What is ASR?

Using aquifer storage recovery (ASR), raw water is treated when excess treatment plant capacity is available--during the winter--and stored underground in an aquifer. When water demand rises to near treatment plant capacity or when surface water supplies are not available, the stored water is pumped back out of the aquifer through the same wells used for recharge, disinfected, and put directly into the water distribution system. Thus, when needed, stored supplies of treated surface water supplement the surface water flowing daily through the treatment plant.

For ASR to be applicable, an aquifer must have certain characteristics, which have been the focus of the studies conducted. It must:

- Be able to hold the treated water in a relatively confined area so that it does to move out of reach, even during prolonged storage periods

- Be able to receive water and yield it up quickly enough to enable injection and pumping adequate to meet demand

- Be chemically compatible with the stored surface water so that "plugging" and other undesirable effects do not occur

ASR is a relatively new technology. There are about a dozen projects now in operation in the United States, and several more in the test phase. The Kerrville project serves as the first working demonstration of this technology's potential in Texas.

How Much Water is Needed?

Water needs for the City of Kerrville are expected to increase by 58% over the next 20 years. Total demand is projected to be 5,550 acre-feet per year (ac-ft/yr) by the year 2015 and 5,850 ac-ft/year by the year 2040. In addition, major seasonal variations in demand and the potential for a drought that might severely but temporarily limit surface water supplies mean that the timing of supply is also a key factor.

The study team recognizes that demand beyond the year 2015 is difficult to predict. We therefore recommend that in 2015 probable future demands be re-evaluated on the basis of the year 2010 census and per capita usage and trends up to 2015. This allows adequate time to expand supply or treatment capacity to meet the projected 2040 demands.

Although the Kerrville area can draw on supplies from both the Guadalupe River and the Hosston-Sligo Formation of the Trinity Group Aquifer, currently allowable supplies from these two sources will not be adequate to meet peak or drought-period needs beyond the year 2000 without additional surface water diversion rights.

Under normal weather conditions, in the year 2015 about 5,550 ac-ft/yr will be required, and about 90% of this amount must come from surface water supplies. With natural recharge only, the aquifer can provide about 560 ac-ft/yr (2 mgd safe yield during summer only, or 0.5 mgd year round). For short periods, the aquifer is capable of supplying 8 to 10 mgd if the City of Kerrville's well field is operated at design capacity.

Maximum groundwater demand could average as high as 7.6 mgd during August of a drought year under the study's worst-case scenario. Thus, the shortfall between the 2-mgd summertime safe yield and the 7.6 mgd demand will be required.

Is The Existing Treatment Plant Large Enough?

Using the current water supply scheme, the plant would need to be expanded beyond its present 5-mgd capacity. By instituting an ASR system and providing additional water rights from the Guadalupe River, this capacity expansion would not be needed until the year 2015 or beyond.

This is the case because ASR enables the UGRA to put idle capacity to use during "wet" periods and to store this water for later use. If the treatment plant operated at a sustained rate of 4.5 mgd year round, a total of 5,040 ac-ft/yr could be supplied from this source. Added to the 560 ac-ft/yr available from native groundwater, total supplies would equal approximately 5,600 ac-ft/yr and will be adequate to meet the projected 2015 demands of 5,550 ac-ft/yr.

Although the capacity of the existing plant is adequate for 2015 demands, little additional uncommitted capacity is available. Additional treatment capacity will be required for future demands.

Is The Hosston-Sligo Suitable For ASR?

The formation's permeability, transmissivity, and geochemistry were found to be consistent with successful ASR application. Full-scale testing of ASR in this aquifer has confirmed its suitability. The native water and the surface water are chemically compatible, and there was essentially no change in the quality of the water stored during the test cycles of recharge and recovery.

The 1-mgd ASR well installed as part of this study and Kerrville Well No. 5 (the retrofitting of which is currently under construction by the City of Kerrville) will have a combined recharge capacity of 1.58 mgd. As stated above, up to about 10 mgd of water could be pumped from the City of Kerrville well field if it were operated at design capacity.

What About Water Rights?

Two issues are involved: the need for adequate diversion rights for surface water and the ability to protect for the City of Kerrville's use treated surface water stored underground.

Surface Water Diversion Rights

The UGRA now holds a permit to divert 3,603 ac-ft/yr, with a maximum diversion rate of 6.2 mgd. As noted above, this amount would need to be increased to at least 5,040 ac-ft/yr to operate the treatment plant at a sustained rate of 4.5 mgd and thereby provide sufficient supplies through the year 2015. By increasing the diversion right to 5,600 ac-ft/yr, projected demand to the year 2040 could be met. This study recommends applying to the Texas Water Commission for diversion rights of 5,600 ac-ft/yr.

Protecting Water Stored Underground

Since Texas water law allows the "right of capture" to groundwater, the study team considered the need to reserve to the UGRA the right to withdraw surface water stored underground.

Under the proposed ASR program, the study found that most of the stored water would remain under the land on which the City of Kerrville's well field and the UGRA Riverside treatment plant are located. To ensure that nearby pumping does not "mine" stored water, the City of Kerrville has ordinances limiting well construction within city limits. We recommend seeking county-wide authority to regulate pumping.

In November 1991, Kerrville voters approved formation of the Headwaters Underground Water District. The District has a key role to play in monitoring groundwater levels and safeguarding ASR supplies.

How Much Water Will Need to be Stored?

Groundwater and surface water modeling and testing showed that adequate drought and peak-period supplies will be available if the aquifer water level is maintained at about 1,500 feet mean sea level (msl) steady-state elevation. This steady-state level is believed to be the minimum natural level in the aquifer during the winter months when pumping is not occurring.

After the summer peak pumping is over, the ASR system would be operated to fill the aquifer to the 1,500 ft msl level when excess surface water supplies and treatment capacity are available. How much water will be required will vary depending on the amount of rainfall in a given year, the amount of pumping, and the water level at the summer's end.

Even during drought years, there will be months during which recharge can occur. This aquifer level would be sufficient to meet demand during a drought similar to that of the 1950's without having to construct new wells. The ability to pump enough water from the aquifer does, however, require that these wells be operating at their design capacity and adjustments to pump elevations, conditions that do not now exist.

What is Required to Implement an ASR System?

Implementation of the proposed ASR program entails both obtaining permits and making prudent institutional arrangements, and assuring that adequate well field capacity is functional. These activities are covered below as the Groundwater Management Program and the Well Field Rehabilitation Program.

The well construction carried out as part of the studies performed to confirm the applicability of ASR have provided an initial 1-mgd ASR well (Well R-1), and the City of Kerrville is currently converting its Well No. 5 to a ASR well. No additional recharge capacity will be necessary.

Groundwater Management Program

Four main elements comprise this aspect of the program:

- Submit application to the Texas Water Commission for a permit to divert up to 5,600 ac-ft/yr from the Guadalupe River
- Obtain Class V permits to operate Wells R-1 and No. 5 as ASR wells; an application to this effect was submitted to the Texas Water Commission in September 1991
- Put the Headwaters Underground Water District into operation
- Implement an Aquifer Management Plan to maintain the aquifer level at 1,500 ft msl; the groundwater model developed as part of this study could be modified to assist with this aspect of the program

Well Field Rehabilitation Program

Over a five-year period as much as $1 million will be needed to ensure that the City of Kerrville's well field is fully operational at design capacity. This will require the following action:

- Complete the conversion of Well No. 5 to an ASR well
- Conduct a Needs Assessment to determine specific repairs for each well and a schedule of implementation
- Carry out the rehabilitation program identified through the Needs Assessment

How This Report is Organized

The first five chapters of this report cover the elements that laid the foundation for the modeling and predictions that comprise the results of the Phase IIB study:

- **Section 1** introduces the study assignment and study objectives. It also provides an overview of existing conditions.
- **Section 2** discusses water demand versus water supply, including how the study team arrived at the projected demands used throughout the study. The seasonal nature of supply and demand and special considerations related to drought conditions are also covered.
- **Section 3** describes how the full-scale prototype ASR well was designed and built, and how the study team set requirements for this well. This section also discusses the installation of monitoring wells used to gather data as part of this project.
- **Section 4** gives a detailed account of the test cycles that the study team conducted using the prototype ASR well described in Section 3. Treated surface water was injected into the Hosston-Sligo Formation and recovered to record data on the actual performance of the ASR well and recharge/recovery water quality.
- **Section 5** explains how the three-dimensional groundwater model was constructed and its features. Key model assumptions, the sources of critical data for model development, and how the model was calibrated are covered.

- **Section 6** draws together the study findings on supply and demand, aquifer and water characteristics, and the groundwater and surface water modeling done to predict groundwater and surface water availability. It reports on simulations with and without an ASR system during a drought and under normal weather conditions in the years 2015 and 2040.

- **Section 7** outlines the recommended implementation plan and summarizes the implications of the study's findings.

- **Section 8** presents a list of references used in this report.

Section 1
INTRODUCTION

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INTRODUCTION

Beginning in 1987, the Upper Guadalupe River Authority (UGRA) has conducted engineering studies to develop a cost-effective system to meet future water demands for the City of Kerrville. Water demands are now being met through a combination of surface water and groundwater supplies. However, because groundwater is limited and surface water is not always available, an alternative supply will be required to meet the increasing water needs for the future.

This report culminates studies investigating the feasibility of aquifer storage recovery (ASR) as a long-term water supply alternative to meet projected water needs. Previous ASR studies include a Phase I Feasibility Investigation (CH2M HILL, 1988) and a Phase IIA Monitoring Well Construction (CH2M HILL, 1989). Table 1-1 presents a brief summary of these studies.

1.1 EXISTING CONDITIONS

The City of Kerrville uses two water sources: treated Guadalupe River water from the UGRA's 5-mgd Riverside Water Treatment Plant, and groundwater from the City-operated wellfield. Groundwater comes from the Hosston-Sligo formation of the Trinity Group aquifer, a limestone formation approximately 500 feet below ground surface.

Surface water typically meets normal water demands and is augmented with well water during the peak demand months of summer. Prior to 1981, when the UGRA plant went into operation, City water wells were the sole source of supply. But as population and water demands grew, groundwater levels dropped significantly. Currently, well pumping is limited to an annual average of 0.5 mgd to avoid further lowering of the area's groundwater table.

In 1989, water diversions from the Guadalupe River reached permitted limits, triggering the need for an amendment to the UGRA's diversion permit issued by Texas Water Commission (TWC). In 1991, an application to amend the existing water diversion permit was submitted to the TWC. If approved, the amended permit will allow increased diversion from the Guadalupe River, but will limit diversion to periods when there is adequate flow in the river. The required minimum flow or flow-through will be determined during the permitting process, but water quality impact river studies conducted by Espey, Huston & Associates (EH&A) suggest the flow-through restrictions to range between 4.7 and 30 cubic feet per second (cfs).

A restriction will greatly affect when, at what rate, and volume of water diverted from the river. Although a lower restriction increases the period when water is available,

Table 1-1
Summary of ASR Feasibility Investigation

Activity	Status	Description
Phase I - Preliminary Assessment	Completed April 1988	A desk-top engineering study evaluating the applicability of ASR for the UGRA. It reviewed existing hydrogeological data, presented project water demands, and assessed legal and permitting issues. It concluded that favorable conditions exist for ASR and recommended construction of a prototype ASR test facility.
Phase IIA - Construction of Monitoring Well PZ-1	Completed December 1989	A detailed study and evaluation to refine basic geologic and hydrogeologic features of the aquifer. Work involved construction of a monitoring well in the Hosston-Sligo (the ASR storage zone), water quality testing and geochemical modeling, pumping tests, and well logging. These results confirmed the conclusion of the Phase I report and a full-scale system was recommended.
Phase II-B - Full-Scale Testing and Evaluation	This study	Construction and testing of a full-scale ASR system.
Phase III - ASR Implementation	Future	Integration and operation of ASR.

historical data suggests that during a drought there will be periods when no diversion will be permissible, no matter what limits are imposed. Under these conditions, stored water will have to be the source of supply.

Other studies (EH&A, 1981; EH&A, 1988) conducted by the UGRA have developed off-channel surface water storage alternatives to meet both flow-through requirements and projected water demands. While this approach is feasible, it requires a major front-end investment amounting to more than $28 million. As an alternative, the UGRA explored the use of ASR to provide low-cost storage and extend the usefulness of the existing water treatment plant.

1.2 OVERVIEW OF ASR

ASR is a water supply program in which treated drinking water is stored underground in a suitable aquifer by recharge wells during "wet" months, and then recovered in the same wells during "dry" months to meet peak water demands which exceed the capacity of the surface water treatment facilities. This cyclic storage and recovery process is illustrated in Figure 1-1. No further treatment of the recovered water is required other than disinfection.

This concept offers the UGRA several key features:

- ASR wells meet increasing peak demands without the need to increase water treatment plant capacity
- "Bank account" of stored water for future emergencies is created by leaving more water in the ground than is recovered
- Groundwater levels are restored
- Stored water is protected from evaporative losses and contamination

1.3 OBJECTIVES OF THIS STUDY

The overall objective of the Phase IIB Study was to test and evaluate a full-scale ASR system and--if ASR showed promise--develop a water management plan to meet projected water needs for Kerrville through the year 2040. Work consisted of the following elements:

- Design and construct a full-scale ASR test facility located at the UGRA WTP
- Assist the UGRA in acquiring appropriate permits from the TWC and Texas Department of Health

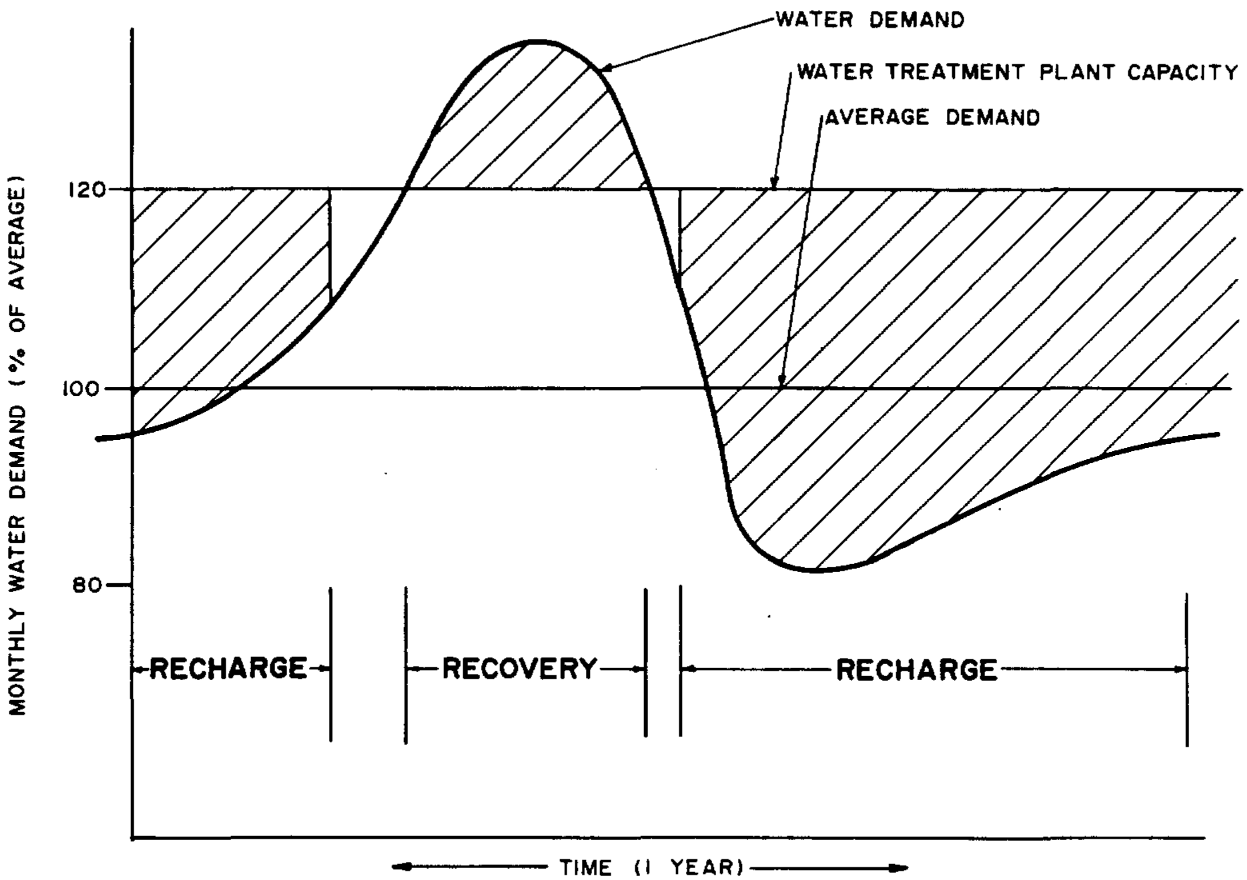

Figure 1-1
Typical ASR Operating Schedule

- Review and update as required previous water demand projections for the City of Kerrville through the year 2040

- Conduct cyclic recharge and recovery test of the ASR system and establish its operational characteristics

- Develop a groundwater model, evaluate the effect of ASR operation in the Kerrville area, and assess storage and delivery potential

- Develop a water management plan to meet projected water needs using ASR technology to augment projected surface and groundwater supplies

Section 2
WATER DEMAND VERSUS WATER SUPPLY

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WATER DEMAND VERSUS WATER SUPPLY

The need for a water management plan is driven by the relationship between water demand and supply in a given area. Where a sole source of supply (either groundwater or surface water) is available to meet the projected demands, the water management plan becomes an implementation plan for capital improvements needed to get the water from the source to the users. Where multiple sources exist, the water management plan must consider the supply limitations of each source along with the costs for implementing each supply alternative.

Water treatment facilities are usually planned and designed to meet the peak day demand and distribution system components are usually designed to handle peak-hour demands. In a system such as Kerrville's--where surface water is the primary supply but groundwater is also available--surface water treatment facilities can be designed to meet base seasonal demands with wells used to meet short-term peak demands. The Phase I--Preliminary Assessment (CH2M HILL, 1988) concluded that peak monthly demand controls the design of surface water treatment facilities in Kerrville when conjunctive use of both ground and surface water is implemented.

Water treatment plant capacity can approximate the average annual demand with some reserve capacity for maintenance and emergencies when ASR wells are used to store and recover water on an annual cycle. A surplus or "bank account" of stored water can be created when the surface water treatment plant capacity is greater than the average annual demand. The stored water would be used to meet future drought demands.

Three conditions of water demand versus supply were considered for Kerrville:

- Average annual demand versus supply
- Seasonal peak demand versus supply
- Drought demand versus supply

A 50-year planning period was used, beginning with 1990, for all demand projections. This allowed use of 1990 census data and updated TWDB (1991, draft) water demand projections. These new projections varied significantly from the demand projections adopted in the Phase I--Preliminary Assessment (CH2M HILL, 1988) and are presented along with the Phase I projections where appropriate.

2.1 AVERAGE ANNUAL DEMAND VERSUS SUPPLY

Average annual demands in Kerrville were compared to average annual supplies available from surface water and groundwater. A review was also performed of the method used for determining demands and a comparison of present projections to those adopted in the Phase I effort was made. Using the new demand projections, present infrastructure appears to be capable of meeting the City of Kerrville demands through 2020, and might meet the 2040 demands. However, authority to divert additional water from the Guadalupe River will need to be obtained from the TWC.

2.1.1 Average Annual Demands

Projected population and per capita demand expressed in gallons per capita per day (gpcd) were used to project average annual demands. Multiplying the projected population by the per capita demand gives the annual demand.

Population Projections

A wide variety of methods are used by planners to project future population, but the projections most commonly adopted by agencies in Texas and by the Alamo Area Council of Governments (which includes Kerr County) are the projections prepared by the TWDB. The 1991 TWDB high-series projections through the year 2040 have been adopted for this project (Table 2-1).

Table 2-1
Kerrville Population Projections
TWDB, 1991

Year	Low Series	High Series
1990	17,384	17,384
2000	22,678	24,044
2010	25,511	27,528
2020	26,990	29,092
2030	27,375	30,531
2040	26,733	31,275

The TWDB uses a cohort-survival model that projects births, deaths, and net migration. Their high-series forecast reflects the higher levels of migration experienced during the rapid economic expansion of the last 20 years, and their low series projection uses the lower levels of migration experienced on the average during the previous 30-year period. The low series indicates a population decline from 2030 to 2040.

This prediction method was substantially affected by the results of the 1990 census. Previous projections (TWDB, 1988) vary from a high series for 2030 of 34,828 to a low series of 33,104 (Figure 2-1). The 1991 projections vary from a high series for 2040 of 31,275 to a low series for 2040 of 26,733 (Figure 2-1). Note that the new high-series projection for 2040 is lower than the old low-series projection for 2030.

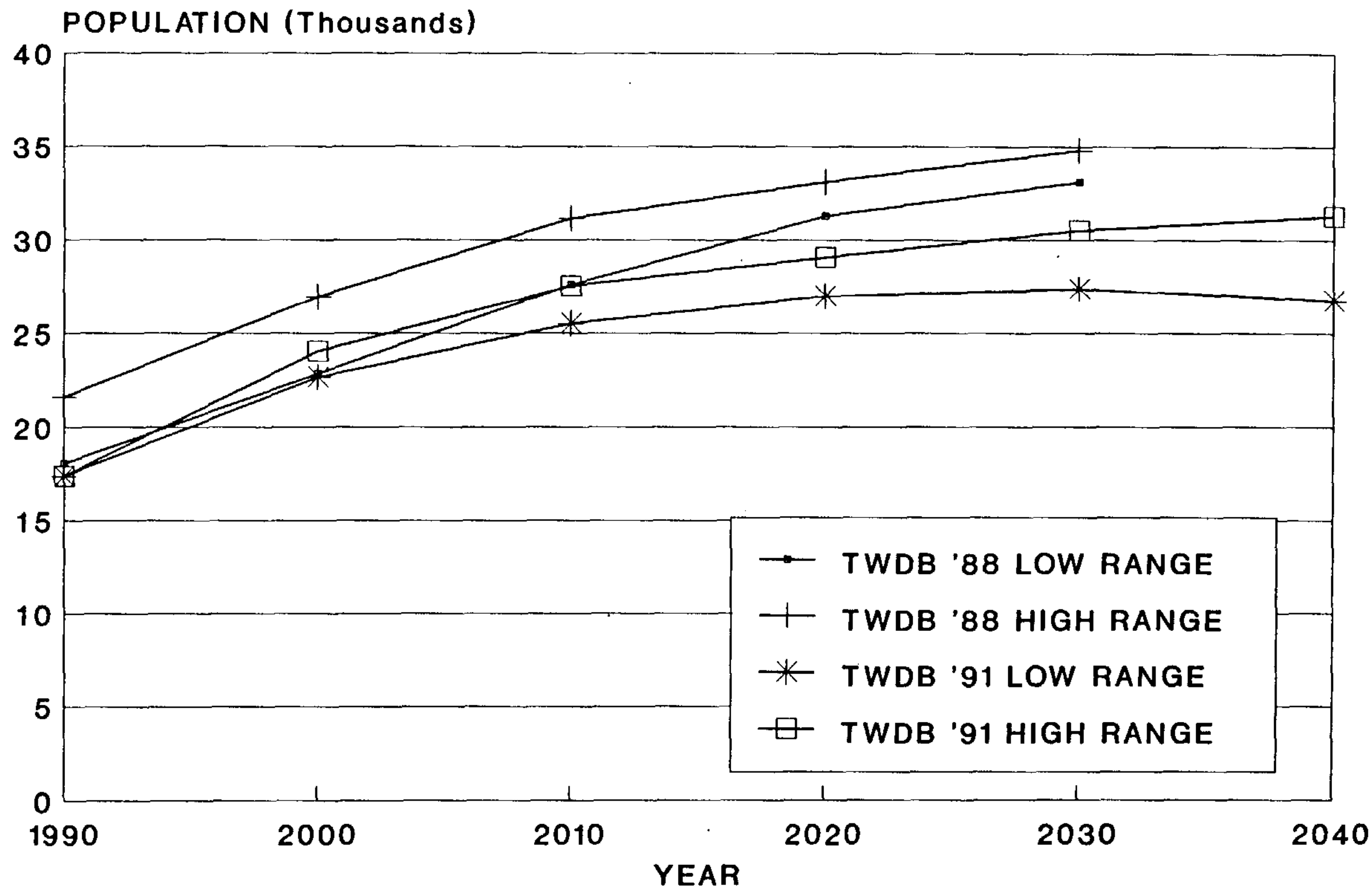


Figure 2-1
City of Kerrville
Population Projections

The population projections for Kerr County are given in Figure 2-2 and indicate a lesser long-term change after inclusion of the 1990 census data. The proportion of the Kerr County population that is located in Kerrville changed from 53 percent in 1980 to approximately 48 percent in 1990, indicating that population growth has shifted to areas outside of Kerrville. This may account for the substantial change in the City of Kerrville projections when Kerr County projections did not change appreciably.

The high-series projections for the City of Kerrville have been adopted for planning purposes herein. The population served by Kerrville's water system is expected to increase internally as well as through extension of service to some high-growth areas just outside of the City. This would bring some of the County population growth into the City's service area.

Per Capita Consumption

The per capita demands for Kerrville were evaluated on the basis of historical consumption and compared to the TWDB categories for water use. The TWDB has adopted four categories for water use:

1. Average per capita water use
2. High per capita water use
3. Average per capita water use with conservation
4. High per capita water use with conservation

The fourth category, high per capita water use with conservation, was adopted for this study.

The average water use for Kerrville customers was estimated to be approximately 183 gpcd as shown in Table 2-2. This is based on total average daily water use including residential, commercial, and industrial uses.

Table 2-2
Historical Per Capita Consumption

Year	Population	Total Demand (ac-ft/yr)	Per Capita Demand (gpcd)
1980	15,276	3,274	191
1985	18,488	3,694	178
1990	17,384	3,515	181
		Average	**183**

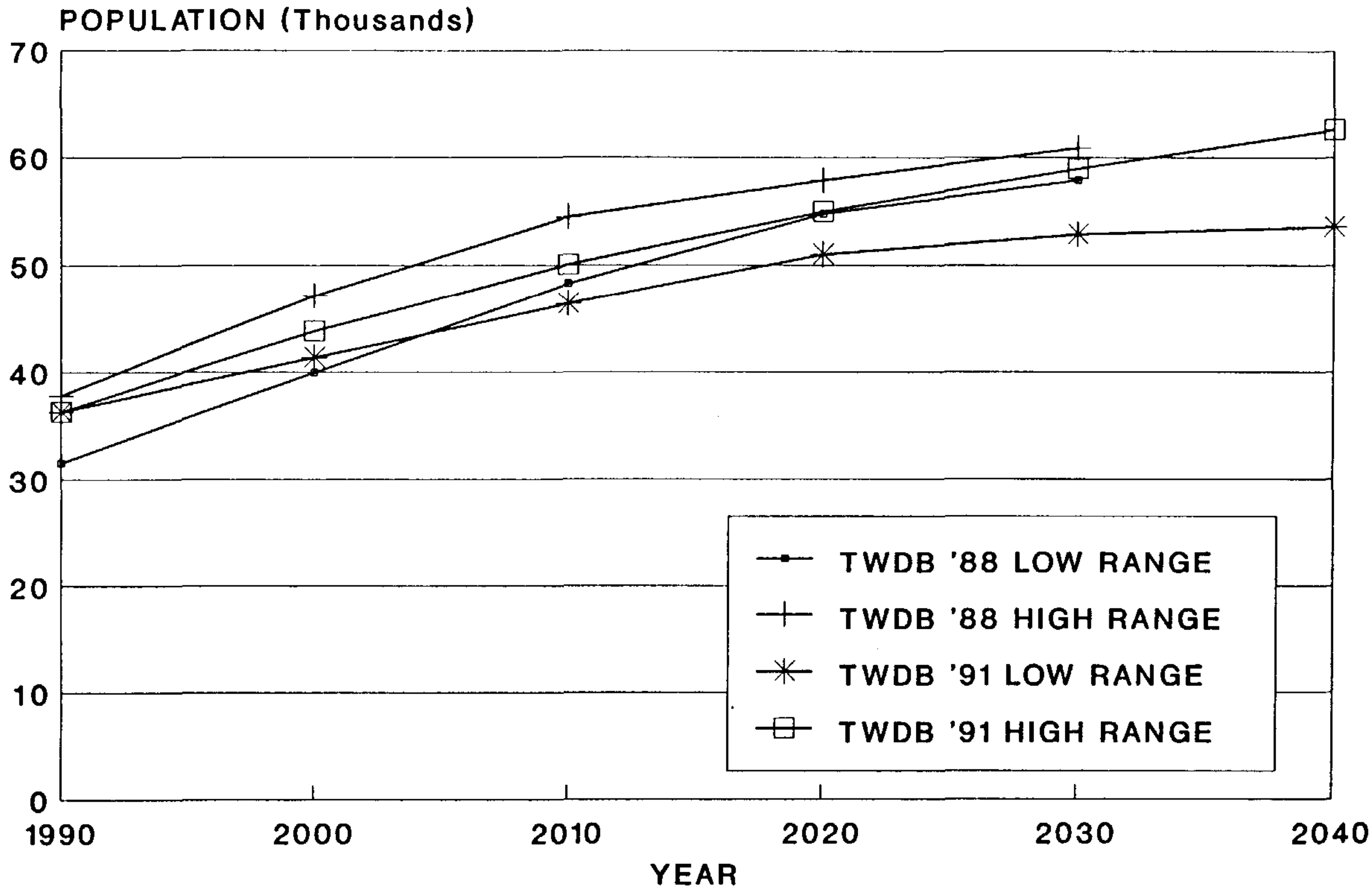


Figure 2-2
Kerr County
Population Projections

CH2M HILL

The demand for the four water use categories developed by the TWDB appears in Table 2-3. The two categories "without conservation" remain constant through 2040, and the two categories "with conservation" are gradually reduced to reflect TWDB goals regarding water conservation savings.

Table 2-3
TWDB Per Capita Water Use
1991 Projections (gpcd)

Year	(1) Average	(2) High	(3) Average/ Conservation	(4) High/ Conservation
1980-1990 (actual)	183	183	183	**183**
2000	174	197	165	**188**
2010	174	197	158	**179**
2020	174	197	151	**171**
2030	174	197	149	**169**
2040	174	197	146	**166**

The TWDB per capita demands presented in Category 4 are most similar to the present Kerrville demands, and the 2040 demand of 166 gpcd represents about a 10-percent reduction from the 1980-90 average Kerrville demand of 183 gpcd.

Projected Demands

The projected annual demands for Kerrville have changed significantly since completion of the 1990 census and inclusion of this information in the TWDB water demand projections. Water demands adopted in the Phase I report are presented in relation to the newly adopted demands in Figure 2-3.

The total water demand for Kerrville is computed by multiplying the adopted population projection by the adopted per capita demand. Using the TWDB projections, a matrix is developed with 2 population projections multiplied by 4 demand categories (Table 2-4).

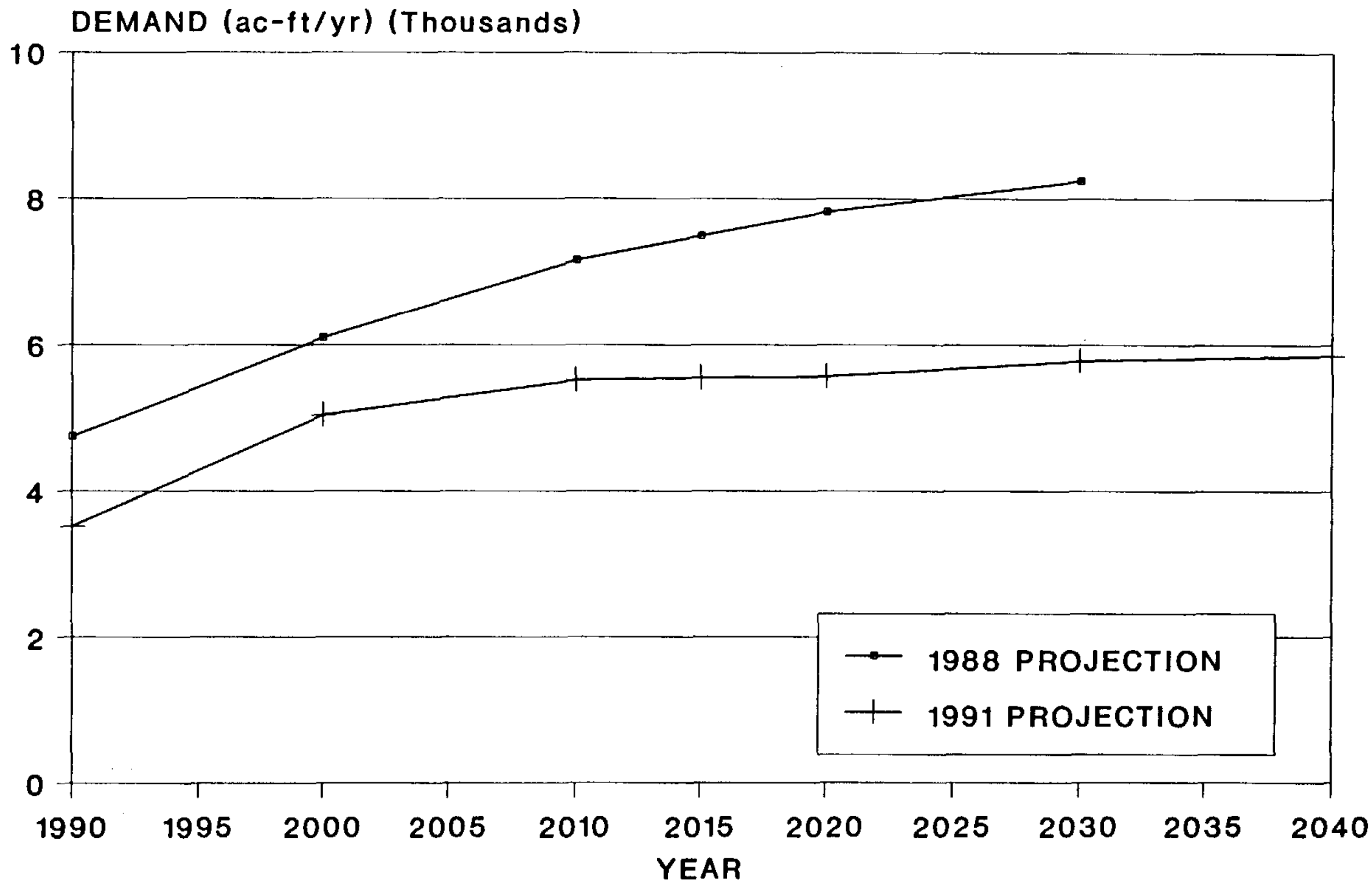


Figure 2-3
City of Kerrville
Average Annual Water Demand

CH2M HILL

Table 2-4
TWDB Water Demand Matrix

Population Projections	Per Capita Water Use Categories (1)	(2)	(3)	(4)
High-Series	H1	H2	H3	**H4**
Low-Series	L1	L2	L3	L4

The total water demand projections adopted for use in this project are based on the high-series population projections multiplied by Category 4 (high per capita use with conservation) demand indicated by Block H4 in the matrix. The adopted demands are given in Table 2-5.

Table 2-5
Kerrville Water Demand

Year	1988 Projection (ac-ft/yr)	1991 Projection (ac-ft/yr)
1990	4,745	3,515
2000	6,094	5,036
2010	7,165	5,520
2015	7,492	5,550
2020	7,820	5,572
2030	8,246	5,780
2040	--	5,850

Note that demands predicted by H4 are approximately in the middle of the range between the highest demand projections (H2) and the lowest demand projection (L3) as shown in Figure 2-4.

2.1.2 Average Annual Supplies

The City of Kerrville has two sources of supply:

- The Guadalupe River
- The Lower Trinity Aquifer

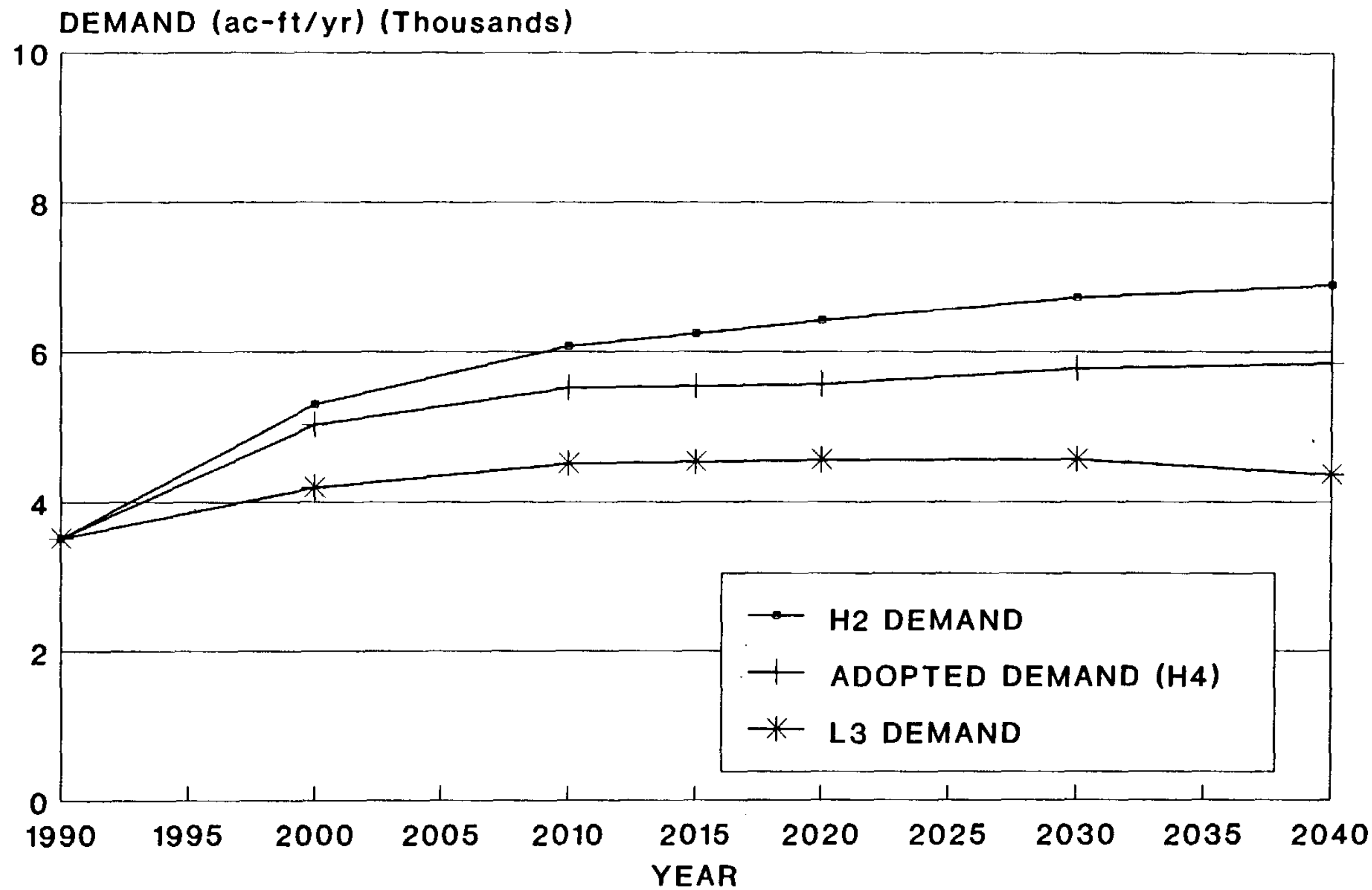


Figure 2-4
Range of 1991
Water Demand Projections

CH2M HILL

Guadalupe River water is currently supplied to the City of Kerrville from the UGRA's water treatment plant, which has a design capacity of 5.0 mgd. The capacity for sustained seasonal operation was noted in the Phase I report (CH2M HILL, 1988) as 4.5 mgd (5,040 ac-ft/yr). The plant was designed for expansion. The existing 5-mgd facility could be expanded in 5-mgd increments to a maximum capacity of 20 mgd.

Water from the Lower Trinity Aquifer is supplied by City of Kerrville-owned wells. The maximum sustained "safe yield" of the Lower Trinity aquifer is estimated at 560 ac-ft/yr (CH2M HILL, 1988). Therefore, the total sustained capacity of the present water sources is estimated at 5,600 ac-ft/yr (5,040 ac-ft/yr + 560 ac-ft/yr = 5,600 ac-ft).

Diversions from the river by the UGRA under Texas Water Commission Permit No. 3505 are presently limited to 3,603 ac-ft/yr (3.22 mgd) with a maximum diversion rate of 9.7 cfs (6.2 mgd). The permitted annual diversion would need to be increased to 5,600 ac-ft/yr in order to operate the existing plant at a sustained capacity of 5.0 mgd.

The Lower Trinity aquifer and presently installed City wells are capable of delivering water at a rate much higher than the "safe yield" of the aquifer. To meet peak-day demands and accommodate emergencies, such as water treatment plant shutdowns, the wells are estimated to be able to deliver about 8 to 10 mgd. This is based on extrapolation of values presented in a report by William F. Guyton and Associates (1973), discussion with the City of Kerrville, and a review of existing well logs. A detailed description of the existing well system capacity is presented in Section 6.

2.1.3 Comparison of Annual Demands and Supplies

When the updated demand projections shown in Figure 2-5 are compared to the total sustained capacity of present water supply infrastructure (5,040 ac-ft/yr surface water and 540 ac-ft/yr groundwater, for a total of 5,600 ac-ft/yr), the 2020 demand of 5,572 ac-ft/yr can just be met. This would indicate that no additional treatment capacity is needed until 2020.

A planning horizon of 2015 is recommended for this study because of the time required for developing additional treatment facilities and the potential changes in long-term (i.e. 2040) demands. Under normal conditions, construction of new water treatment facilities can take 2 to 3 years from preliminary design through final design, construction, and startup. This would require a water treatment plant project to be initiated in about 2017. The need for additional facilities should be reevaluated prior to this time (in about 2015) for the following reasons:

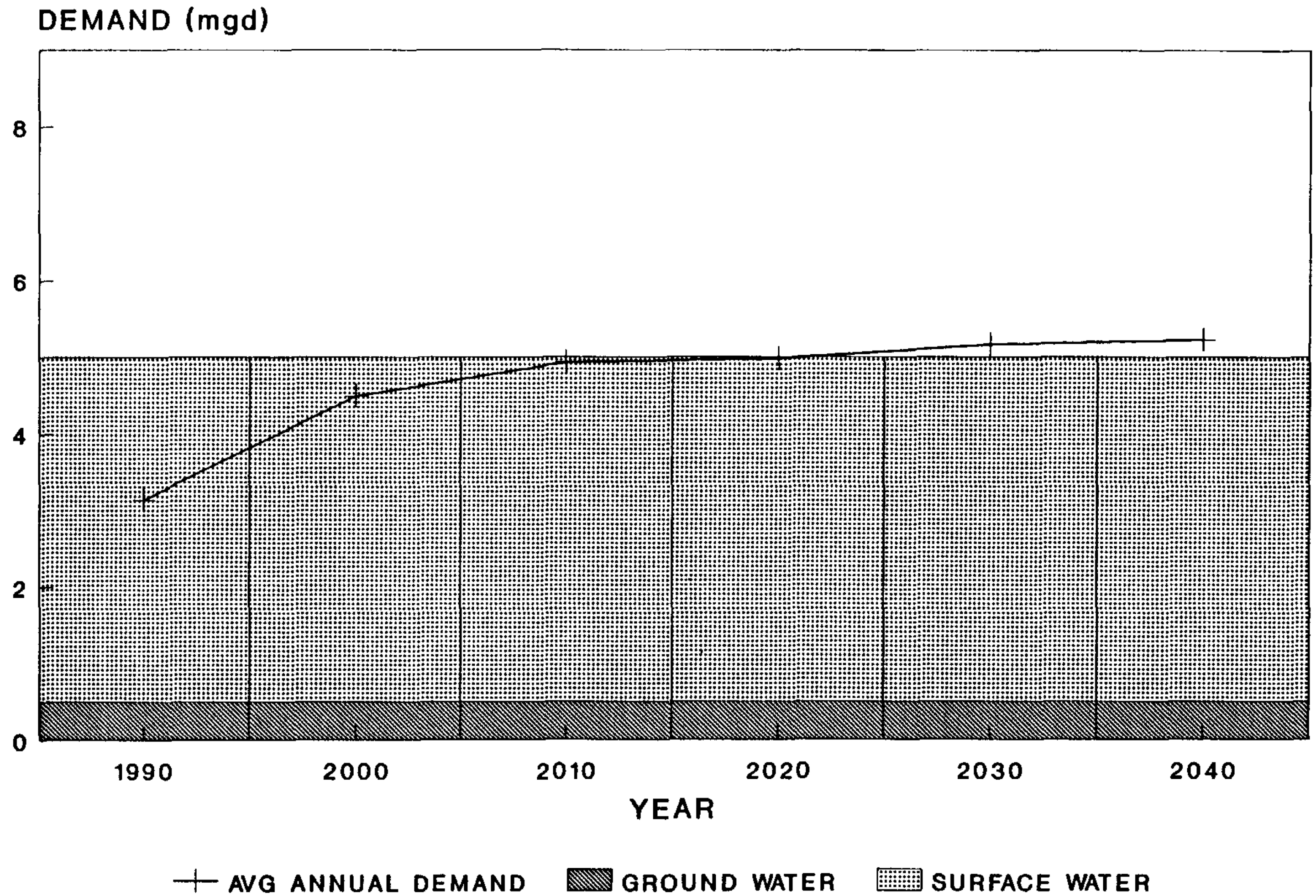


Figure 2-5
Average Demand vs. Present Sources

- Demand projections are uncertain and provide better estimates in the short-term than in the long-term. The year 2015 is more than 20 years in the future--a sufficient period for potentially significant changes to occur.

- Census data for 2000 and 2010 will be available to adjust demand projections to then-current trends.

- The existing plant will be about 35 years old in 2015 and may need major rehabilitation or improvements to meet changing water treatment requirements.

If modifications were made such that the existing UGRA water treatment plant could achieve a maximum sustained capacity for seasonal operation of 5.0 mgd, the total water available to meet Kerrville demands (5,600 ac-ft/yr from the plant plus 560 ac-ft/yr for groundwater = 6,160 ac-ft/yr) would exceed the projected 2040 demand of 5,850 ac-ft/yr. It is surprising that only 0.5 mgd (560 ac-ft/yr) would be sufficient to extend the capability to meet demands for more than 20 years (from 2020 to beyond 2040). This is because the present demand projections indicate a very slow increase in the distant years, which results from low population growth rate assumptions coupled with conservation goals. This situation may not occur. As stated previously, 2015 would be a good point to reevaluate demands and the capabilities of the existing water supply facilities, unless changing conditions warrant an earlier review.

2.2 SEASONAL DEMAND VERSUS SUPPLY

The study team compared seasonal water demands in Kerrville to available water supplies. Seasonal demand was determined from historical data and was related to annual demand. Water supplies available to satisfy peak demand were considered to include surface water, groundwater, and the potential role of ASR.

2.2.1 SEASONAL DEMANDS

Seasonal demands for Kerrville have been defined as peak-month demand because of groundwater availability. Peak-month demand was determined by evaluating the historical monthly demands relative to average annual demands.

In the Phase 1 effort, an analysis was performed to evaluate average monthly demand as a percentage of annual demand for a 10-year period. The demand ratios are shown in Figure 2-6 and range from a low of about 73 percent in December, January, and February to a high of 153 percent in August. In contrast, maximum-day demands equal 220 percent of annual average demand (CH2M HILL, 1988). Monthly ratios are presented below:

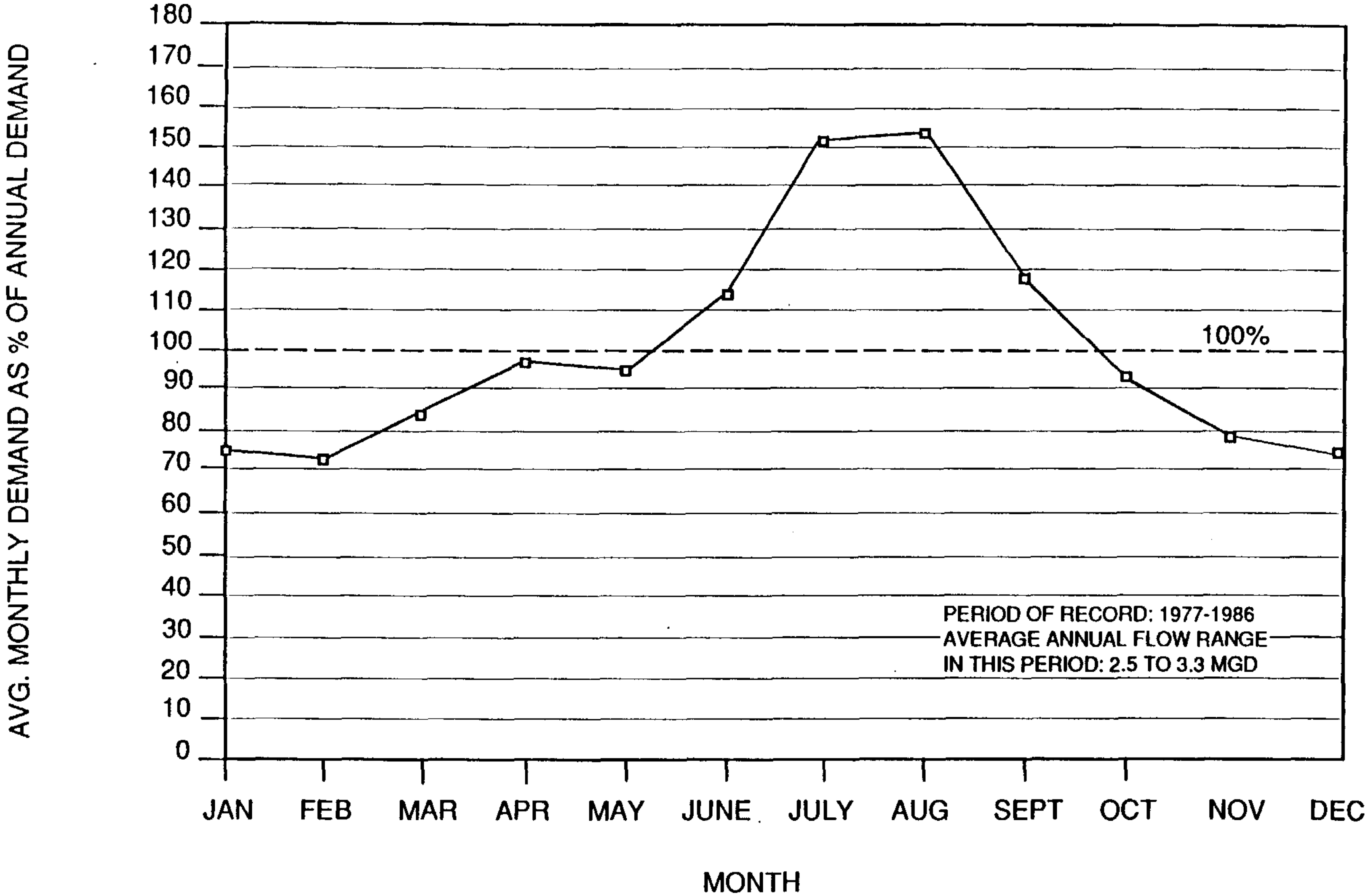


Figure 2-6
Typical Monthly Demand Variation

CH2M HILL

Month	Percentage of Average Annual Demand
January	73
February	73
March	83
April	97
May	95
June	114
July	151
August	153
September	119
October	93
November	78
December	73

Multiplying 0.73 by the projected average annual demand gives the projected minimum monthly demand. Multiplying 1.53 by the projected average annual demand gives the projected maximum monthly demand (Figure 2-7).

2.2.2 Seasonal Supplies

As stated in the discussion of annual supplies, the UGRA water treatment plant has a peak capacity of 5 mgd which could be sustained for short periods, but its practical maximum capacity for sustained seasonal operation is 4.5 mgd.

Kerrville wells can produce water from the Lower Trinity at a maximum capacity of 8 to 10 mgd for short periods, but long-term pumping at these rates could damage the aquifer's recharge and yield characteristics. Sustained pumping from the Lower Trinity should be limited to a maximum rate of approximately 2.0 mgd during July and August in order to achieve a safe yield target of about 560 ac-ft/yr (CH2M HILL, 1988).

2.2.3 Comparison of Seasonal Demands and Supplies

The total water available to meet seasonal peak demands without ASR is estimated at 7.0 mgd (5.0 mgd from surface water and 2.0 mgd from groundwater). This would

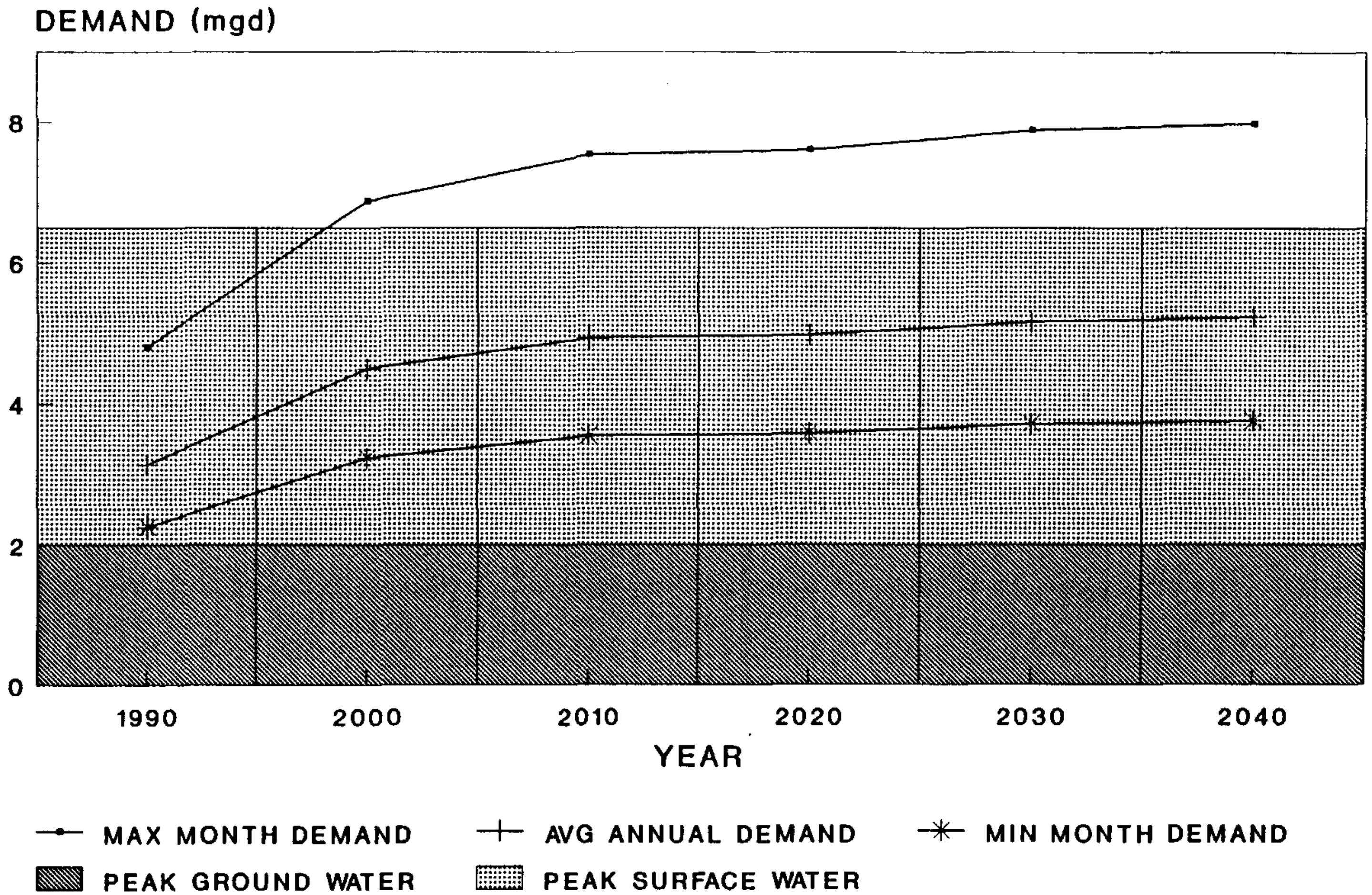


Figure 2-7
Seasonal Demand vs. Supply
Without ASR

CH2M HILL

satisfy peak demands until about 2000 or 2001 (Figure 2-7). At that time, the surface water treatment plant could be expanded, ASR wells could be used, or a combination of both could be implemented to increase the peak availability of surface water.

If the UGRA water treatment plant operates at 4.5 mgd during the minimum demand of 3.63 mgd in December, January, and February, then 0.87 mgd is available for storage. When summer demand is highest (7.6 mgd), the water treatment plant can supply 4.5 mgd and Kerrville's well field can supply 2.0 mgd. This results in a shortfall of 1.1 mgd (7.6 mgd - 4.5 mgd - 2.0 mgd = 1.1 mgd) which could be supplied from stored water. The aquifer's capability to respond to this type of recharge and withdrawal was evaluated using a mathematical groundwater model and is presented in Section 6.

2.3 DROUGHT DEMAND VERSUS SUPPLY

Drought is an extreme condition that cannot easily be evaluated on the basis of return frequency (i.e. 10-year or 100-year drought) as can rainfall or storm events. The potential effect of drought conditions is therefore usually evaluated by considering the "drought of record". For Kerrville and the Guadalupe River, the drought of record occurred during the 1950s and is the basis for evaluating ASR's potential to augment existing supplies to meet drought demands.

Drought demand and drought supply evaluation were accomplished by simulating streamflow in the Guadalupe River simultaneously with groundwater modeling. The study team considered a repeat of the 1950's drought beginning in 2015. Streamflow simulation results are presented below; the required response of the aquifer using ASR is presented in Section 6. The following paragraphs describe the basis for this analysis and the critical role that ASR could play in satisfying Kerrville's water supply if the drought of record were to occur between 2015 and 2020.

2.3.1 Drought Demands

For the purposes of this study, drought demands for Kerrville were considered to be the same as projected demands for average or normal conditions. During a short-term drought, demands may increase due to greater outdoor use of water, but as a drought becomes longer, conservation and curtailment programs can often reduce demands to below average. No attempt was made to predict the public's response to drought conditions in Kerrville.

The period beginning with year 2015 was selected for evaluation of the drought of record (the 1950's drought). The years 2015 to 2020 were chosen because this is the final 5 years when the sustained capacity of present water sources will just exceed demand, or the most critical period under existing conditions. This is also the year that we recommend reevaluation of supply and demand.

The end of the planning period (2040) was also selected for evaluation, but considered to be less critical than the near-term evaluation.

In order to perform the simulation of drought demands, monthly demand as a percentage of the annual demand was converted to daily demand in millions of gallons per day (mgd). The drought demand levels used in the 2015 simulation are shown by the solid line in Figure 2-8. This line depicts the annual demand cycle shown in Figure 2-6 repeated for four annual cycles, corresponding to the critical period of the drought of record (January 1954 to December 1957).

2.3.2 Drought Supplies

When a drought occurs, surface water supplies are the first to be affected. Streamflow declines and inflow to lakes and diversion reservoirs is reduced. Lakes or reservoirs can usually store enough water to weather a short-term drought, but there are no lakes or reservoirs of appreciable storage capacity on the Guadalupe River near Kerrville. A related concern is the potential decline in water quality as less flow is available to flush suspended sediment and contaminants down the river.

Streamflow Simulation

Drought impacts on surface water supplies are usually evaluated by considering the actual streamflow conditions during the drought of record. For Kerrville this is the drought of the late 1950's. A daily computer model for the UGRA Lake in Kerrville and downstream reaches of the Guadalupe River to Flat Rock Lake was prepared by Espey, Huston & Associates (1988) and employed for this analysis. Daily operation of the UGRA Lake was based on the following water balance components:

- The daily inflows to the reservoir based on actual gaging station records
- The daily evaporation from the reservoir
- Daily diversions to the UGRA Riverside Water Treatment Plant
- The daily instream flow requirement needed to maintain water quality and aquatic ecosystems [a minimum of 15 cfs based on Espey, Huston & Associates' (1990) water quality analysis]

The maximum amount of diversion would be limited both by the reduced streamflow and by an instream flow requirement that would be established to maintain water quality and aquatic ecosystems, and to protect downstream water rights. The minimum instream pass-through requirement considered in this analysis was 15 cfs.

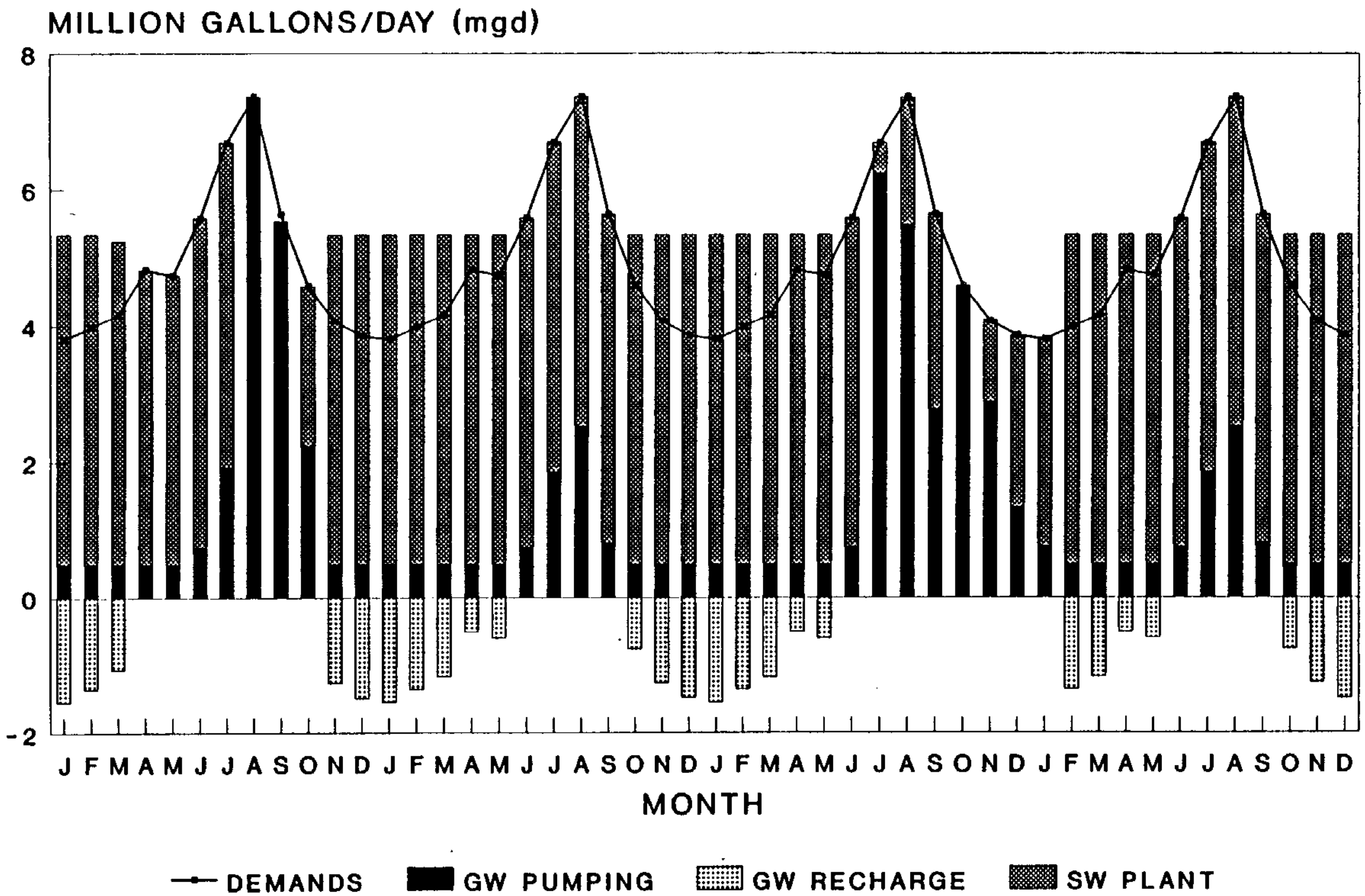


Figure 2-8
2015 Drought
Recharge and Recovery

CH2M HILL

Groundwater Role

Groundwater is usually the most drought-resistant water source. It is not subject to evaporation and existing storage is usually the result of recharge occurring from years earlier. The Lower Trinity is capable of producing 8 to 10 mgd for short periods. This is insufficient to meet total demand when surface water is not available. The duration of these high groundwater production periods can be extended by using ASR to store surplus surface water in advance.

2.3.3 Comparison of Drought Demand and Supply

Comparison of water demand and supply under drought conditions is usually accomplished by simulating the drought impact on streamflow and evaluating the limitations of the supply. The basic question to be answered here is: ***How much water must be stored in the Lower Trinity aquifer in order to meet demand, using surface water when it is available and groundwater at other times, without adversely affecting the aquifer?***

A 40-year simulation was used to evaluate the aquifer storage requirements. The simulation rules were:

- The initial increment of demand (0.5 mgd) was satisfied with groundwater from the City's Lower Trinity wells.
- The UGRA's water treatment plant was assumed to be operated near maximum capacity whenever sufficient surface streamflow was available.
- If surface water production exceeded demand, the surplus water was assumed to be stored (recharged) in the Lower Trinity.
- If surface water production could not satisfy the demand, stored water was assumed to be recovered from the Lower Trinity.
- If no surface water was available, enough stored water was assumed to be recovered from the Lower Trinity to meet the total demand.

A trial and error approach was used, and the simulation was repeated until it was determined that 2,300 ac-ft of storage was required during the critical 4-year drought period beginning in 2015. Using this scenario, the 2015 demands could be satisfied during the critical 4-year drought period without creating a deficit (i.e. without recovering more than was stored or exceeding the long-term safe yield of the aquifer). The results of the simulation are shown in Figure 2-8 and indicate the following:

- Groundwater/stored water was used exclusively to meet demands during some months.

- Even during a sustained drought, the surface water plant could operate at near maximum capacity for many months.

- Some surface water is available in nearly all months.

- Significant quantities of water can be stored during a sustained drought.

The same approach was used to evaluate a repeat of the 1950's drought of record using 2040 demands. We determined that 3,500 ac-ft of storage would be required.

The questions that remains to be answered are:

- How will the Lower Trinity aquifer respond to the storage and recovery rates used in the simulations?

- Can the existing wells handle the recharge and recovery flow rates, including max-month demands?

These issues will be addressed in the groundwater modeling effort presented in Section 6.

2.4 CONCLUSIONS

The key conclusions of this evaluation are:

- 1991 projections from the TWDB predict a significant **decrease** in demand from the 1988 projections used in the Phase I study. A planning horizon of 2015 was adopted for this study to account for uncertainty in future (2040) demand projections.

- The existing water treatment plant and wellfield are capable of treating and/or supplying 5,600 ac-ft/yr which is sufficient to meet the **average** annual demands through 2020, **assuming no drought.**

- The existing water sources **cannot** supply 5,600 ac-ft/yr to meet 2020 demands until:

    - A new surface water diversion permit is obtained
    - ASR or surface storage is implemented

- The need for additional treatment facilities to meet **annual** demands should be reevaluated in 2015, or sooner if demand grows faster than projected or if the existing treatment plant experiences problems. The existing treatment facilities have little excess uncommitted capacity and additional capacity will be required for future demands.

- **Peak month** demands can be satisfied with the existing water treatment plant and wellfield until about 2000 or 2001.

- ASR and the existing wellfield can be utilized to meet **peak month** demands with the existing water treatment plant until about 2020.

- ASR can be used to meet 2015 demands during the **drought of record** if 2,300 ac-ft can be stored and recovered from the Lower Trinity aquifer.

- ASR can be used to meet 2040 demands during the **drought of record** if 3,500 ac-ft can be stored and recovered from the Lower Trinity aquifer.

- Groundwater modeling or a similar analysis must be performed to confirm the aquifer's capacity for storage and recovery will accommodate the needed rates and volumes.

Section 3
ASR DESIGN AND CONSTRUCTION

Section 3
ASR DESIGN AND CONSTRUCTION

Based on the positive results of Phase I and Phase IIA investigations, a prototype, full-scale ASR well system was designed and constructed. The system is located at the UGRA's water treatment plant near Monitoring Well PZ-1 (see Figure 3-1). The system allowed testing and evaluation of the ASR concept and now provides a functional ASR well. All wells including the ASR well and three monitoring wells meet TWC standards for water well construction. The following pages describe the functional and operational criteria of the system and significant construction activities during this phase of work. A brief summary of Well PZ-1 construction is also presented.

3.1 DESIGN AND OPERATIONAL CRITERIA

The prototype ASR system consists of one injection and recovery well (Well R-1) plus three monitoring wells (MWCC--Cow Creek formation monitoring well, MW-GR--Glen Rose formation monitoring well, and PZ-1--production zone or Hosston-Sligo formation monitoring well). All wells are located at the UGRA water treatment plant as shown in Figure 3-1. The system allows for variable recharge rates ranging from 200 to 1,000 gpm and is manually operated. A process schematic of the ASR system is presented in Figure 3-1; design and operational parameters are summarized in Table 3-1.

Potable water flows from the plant clearwell to R-1 for metering, injection, and storage. A submersible multi-stage deep well pump pumps stored water from the aquifer to the clearwell. Flow meters, valves, and sample ports are provided to monitor and control flow to and from the well. Nearby monitoring wells indicate recharge and recovery effects on the storage and overlying formations.

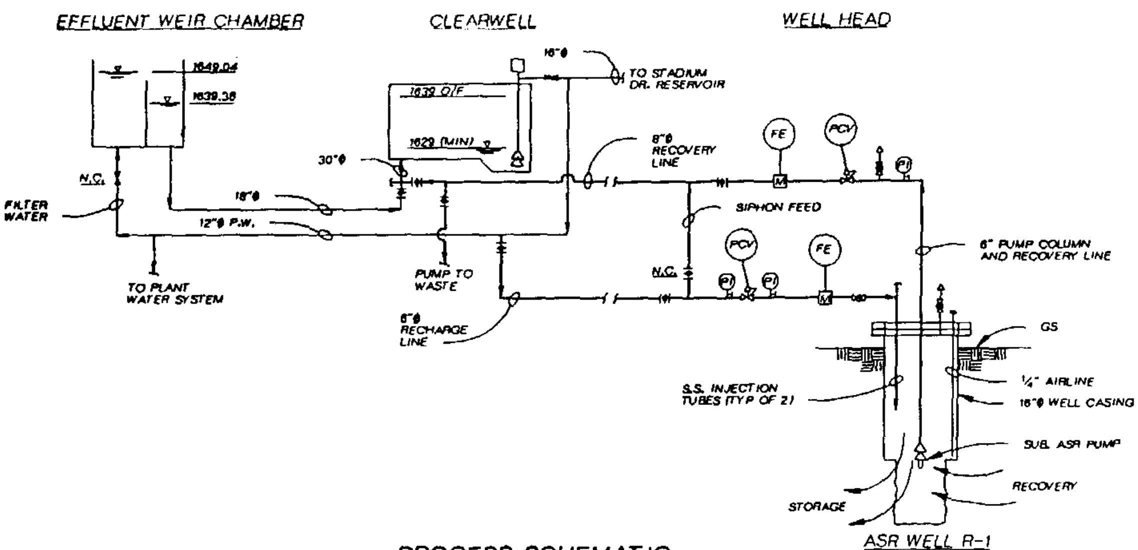
Disinfection of recovered water is accomplished using existing plant chlorination equipment. During the recovery phase, the well water is mixed with the plant water prior to entering the clearwell. The desired chlorine residual in the combined stream is maintained by adjusting the chlorine concentration in the plant water to account for the additional well water flow. The chlorine residual is monitored at the discharge of the distribution pumps.

3.2 R-1 CONSTRUCTION AND TESTING

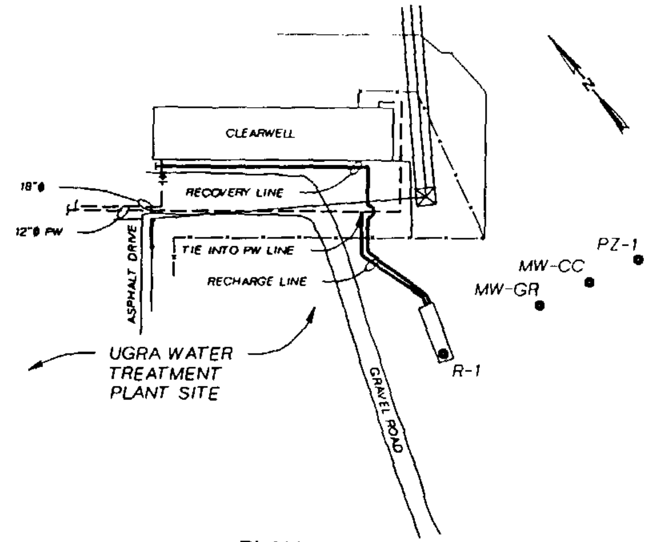
Construction of ASR Well R-1 began on October 3, 1990; well installation and preliminary aquifer testing were completed December 28, 1990. The sequence and

Table 3-1
ASR System Design Criteria

Criteria	Design Condition
Well and Wellhead	**Casing:** Steel cased, epoxy lined. **Wellhead:** Sealed wellhead designed to allow pressure recharge. Maximum operational pressure = 30 psig.
Recharge Capacity	Design 600 gpm; but allow variable feed of 200-1,000 gpm.
Recharge Mechanism	**Primary:** Injection tubes with discharge below minimum groundwater level of 200 feet below ground surface (BGS). **Secondary:** Free fall down annular space between the well and pump.
Recharge Water Source	**Primary:** Pressurized plant water line from clearwell high service pump. **Secondary:** Siphon feed from clearwell.
Flow Monitoring and Control	**Flow Control:** Manually operated control valves. **Pressure:** Automatic pressure control valves to reduce recharge water to 15 psig at wellhead. **Flow Monitoring:** Propeller type with manual readout.
Recovery Capacity	800 gpm.
Recovery Pump	Submersible multi-stage deep well pump, rated at 800 gpm at 500 feet TDH.
Other	See Contract Documents for construction of ASR Phase IIB: Well R-1 and Well No. 7 (CH2M HILL, July 1990).



PROCESS SCHEMATIC
NTS



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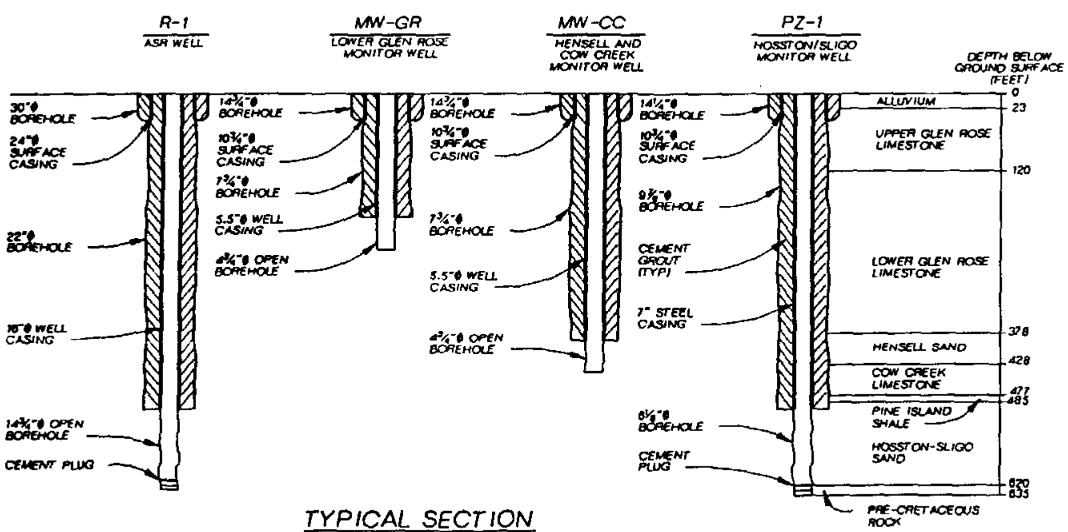

TYPICAL SECTION
NTS

WELL DRILLING AND COMPLETION SCHEDULE				
WELL IDENTIFICATION	R-1	MW-CC	MW-GR	PZ-1
TOTAL DEPTH OF WELL	625'	440'	240'	635'
SURFACE CASING				
BOREHOLE DEPTH	40'	30'	30'	42'
BOREHOLE DIA	30"	14.75"	14.75"	14.25"
SURF CASING LENGTH	40'	30'	30'	40'
SURF CASING DIA	24"	10.75"	10.75"	10.75"
SURF CASING TYPE	STEEL	STEEL	STEEL	STEEL
WELL CASING				
BOREHOLE DEPTH	495'	390'	190'	495'
BOREHOLE DIA	22"	7.75"	7.75"	9.875"
WELL CASING LENGTH	495'	390'	190'	495'
WELL CASING DIA	16"	5.5"	5.5"	7"
WELL CASING TYPE	EPOXY STEEL	STEEL	STEEL	STEEL
OPEN BOREHOLE				
DEPTH INTERVAL	495' TO 613'	390' TO 440'	190' TO 240'	495' TO 620'
BOREHOLE DIA	14.75"	4.75"	4.75"	6.125"

Figure 3-1

ASR PROCESS SCHEMATIC AND WELL COMPLETION SCHEDULE

CH2M HILL

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duration of field activities incorporated in this task are presented in Table 3-2. Activities included air rotary drilling to a depth of 625 feet, installation of 495 feet of well casing, acidification of the well, approximately 100 hours of well development, pumping tests and videotape documentation of the completed well.

Data obtained included deviation survey data, lithologic descriptions of formation samples, water level data from both R-1 and the observation well (PZ-1), post acidification water quality data, and pumping test time-drawdown data.

3.2.1 Borehole Drilling and Well Installation

Borehole drilling and well installation were performed by Driller's Incorporated (D.I.) of Houston, Texas. Initially a 24-inch O.D. surface casing was installed to a depth of 40 feet employing a foundation rig equipped with 36-inch diameter augers. A Speedstar SS-25 II rig was mobilized to the site on November 12, 1990 to complete borehole drilling using air and mud rotary techniques. The study team conducted directional survey tests at 30-foot drilling intervals throughout the length of the borehole and collected and described lithologic samples from 5- to 10-foot intervals during borehole advancement.

The borehole was advanced below the surface casing by air rotary methods to a total depth of 625 feet using a 14 3/4-inch roller bit. The Hosston-Sligo aquifer portion of the borehole (495-625 feet) was then backfilled with clean sand to maintain the integrity of the well during mud rotary drilling and casing installation. After sand placement, the upper portion of the borehole, from a depth of 40 feet to 495 feet, was reamed with a 22-inch roller bit using mud rotary methods. Deviation for the borehole to 495 feet was less than 0.5 degrees. A 16-inch O.D. steel epoxy-lined well casing was installed in the 22-inch hole. The internal epoxy coating (Valspar 78-D7) was applied offsite. Casing joints were threaded and coupled. A concrete floatshoe was coupled to the base of the casing string to facilitate grouting. During makeup of the coupled joints, an epoxy coating (Aquatapoxy) was applied in the field to threads which would be exposed on the internal portion of each coupling.

Pressurized grouting operations were conducted through a 2-3/8-inch O.D. string of pipe seated in the floatshoe valve. The grout was forced out at the base of the floatshoe and up through the annular space between the borehole and casing.

At the completion of grouting operations, no grout had returned to the surface, indicating that at some depth below ground level a permeable zone was allowing the grout to move into the formation. To top off the hole, cement grout was installed through a tremie pipe from the surface.

Upon completion of casing installation, the concrete floatshoe was drilled out and the sand which had been installed to protect the lower aquifer portion of the borehole was removed using airlift eductor methods.

Table 3-2
Sequence of R-1 Construction

Activity	Duration
Drill & Install 24-inch O.D. surface casing (Watson 100 Rig)	10/03/90
Mobilization & setup (Speedstar SS-25 II Rig)	11/12/90-11/15/90
Drilled to 625 feet using 14 3/4-inch roller bit	11/16/90-11/18/90
Backfilled aquifer portion of borehole (495-625 feet) with clean sand	11/19/90
Redrilled (reamed hole) to 495 feet using 22-inch roller bit	11/20/90-11/27/90
Installed 16-inch O.D. casing to 495 feet (grout 139 feet 495 feet)	11/29/90-11/30/90
Install 1 1/4-inch piezometer @ 139 feet in annular space between borehole and casing; grout from surface to 95 feet below ground level	12/04/90
Drilled out concrete floatshoe @ base of casing	12/06/90
Air jet sand out of aquifer	12/08/90
Acidize and air develop well	12/09/90-12/14/90
Set cement plug from 613-625 feet below ground level	12/15/90
Conduct straightness and plumbness test	12/17/90
Pump develop well & conduct pumping tests	12/18/90-12/23/90
Conduct downhole camera survey	12/28/90

3.2.2 Well Acidification

To improve well yield, the well was acidified with fifteen thousand gallons of 15 percent hydrochloric acid through 2-3/8-inch steel tubing set at a depth of 500 feet. A 14-inch diameter neoprene gasket was installed on the steel tubing and secured approximately 3 feet above the base of the casing to prevent acid from moving up into the casing. The acid was pumped at a rate of 1.5 bbls/min (65 gpm).

Following acid placement, 6,000 gallons of fresh water was pumped through the steel tubing in 1,000-gallon increments at half hour intervals to displace the acid further into the formation. The acid was allowed to react with the formation for approximately 36 hours prior to initiating well development. During initial well development, the spent acid solution was brought to the surface, neutralized with caustic soda, and held in a pH stabilization tank prior to discharge to the UGRA sludge lagoon.

3.2.3 Well Development

Airlift methods were initially employed to develop the well until the discharge water was free of visible particulate matter. A 5-inch O.D. eductor pipe was lowered into the well, below the casing, and a 1-inch PVC air line was installed inside the drill string below the water level, to a depth of 200 feet. Airlift development was initiated at a depth of 536 feet and was progressively stepped down to the bottom of the well (625 feet). Water samples were collected periodically throughout well development to monitor chloride concentration and pH. All water produced during airlift development was discharged to the sludge lagoon. Development water in the first two hours of testing contained chloride levels in excess of 9,900 mg/L. It took more than 120 hours of development to bring levels under 100 mg/L.

The well was developed using airlift methods for a total of 73 hours over a 5-day period. After an extended period of development, it became evident that the lowermost portion of the aquifer, below a depth of 615 feet, was not a productive zone. Consistent sloughing of clay and gravel below 615 feet did not allow complete development to the base of the aquifer. Because this lower unit did not appear to contribute to well yield and potentially could be detrimental to overall water quality, a cement plug was installed from a depth of 613 feet to the base of the borehole or approximately 12 feet of plug.

3.2.4 Straightness and Plumbness Test

The straightness and plumbness test was conducted using a 40-foot long rigid dummy. The dummy consisted of two 20-foot links of 4-inch O.D. steel pipe coupled together and three 14-inch diameter neoprene rings. The dummy was suspended from a plumb line on the rig and lowered into the 16-inch casing. As the dummy was lowered into the casing, the deflection of the plumb line from exact center of the casing was measured at each 10-foot interval. Maximum measured deviation off

(MW-GR), began February 6, and was completed February 16, 1991. These wells monitor water levels in the overlying aquifer zones during recharge and recovery operations in the Hosston-Sligo, enabling estimates of the degree of hydraulic connection between the formations and the potential amount of available storage.

The monitor wells are nominal 5.5-inch steel cased with 50-foot open well intervals. MW-CC--located approximately 110 feet east of ASR Well R-1--reaches a depth of 440 feet. The open hole interval straddles the lower Hensell Sand and upper Cow Creek formations. This interval represents the first productive zone overlying the Hosston-Sligo aquifer. MW-GR was completed to a depth of 240 feet in the Lower Glen Rose Formation, approximately 70 feet east of R-1.

The sequence and duration of Phase II-B field activities are presented in Table 3-3. Activities included air rotary drilling, installation of well casing, well development, and collection of water samples.

3.3.1 Monitor Well MW-CC

Page Drilling Company of Kerrville, Texas, performed borehole drilling and well installation. A Gardner-Denver rig was mobilized to the site and set up at location MW-CC on February 6, 1991. After installation of 30 feet of surface casing, an 8 3/4-inch diameter borehole was advanced below the surface casing to a depth of 390 feet for installation of the well casing. Casing joints were butt welded together. The well was capped with a lockable protective cover.

Following casing installation, grout was installed under pressure through a 2 7/8-inch O.D. string of pipe seated at the base of the casing. The grout was forced out at the bottom of the casing and into the annular space between the borehole and casing. The well was grouted to the surface. The grout was allowed to cure for 48 hours prior to additional down hole operations.

Once the grout had cured, the borehole was advanced below the casing to a depth of 440 feet with a 4 3/4-inch bit. The 50-foot open well interval for MW-CC extends from 390 to 440 feet.

The well was developed using airlift methods for approximately 4 hours. During development the well yielded water at an approximate rate of 5 to 10 gpm.

3.3.2 Monitor Well MW-GR

Drilling of the Lower Glen Rose monitor well (MW-GR) began on February 12, 1991. Construction techniques and installation of MW-GR were similar to those used for MW-CC.

The well casing was installed to a depth of 190 feet. The nominal 30-foot casing joints were butt welded and three centralizers were installed at the top, center, and center was approximately 1/2 inch. Movement of the dummy was smooth, no obstructions were noted.

3.2.5 Pumping Tests

Following development by airlift methods, a variable speed submersible pump was installed in the well. An 8-hour step drawdown test was conducted on December 20, 1990. The test consisted of four 2-hour segments operating at 300 gpm, 450 gpm, 600 gpm and 800 gpm. The well was allowed to recover for 40 hours before starting the constant rate pumping test.

A constant-rate pumping test was performed at a flow rate of 805 gpm for a period of 16 hours beginning December 22, 1990 and ending December 23, 1990. Time-drawdown data were recorded for both the pumping well (R-1) and the observation well PZ-1 during the variable and constant rate tests. Recovery data were also obtained for a 4-hour period following the constant rate pump test.

The pump test data was analyzed to determine transmissivity and storativity using Theis drawdown and recovery methods and Jacob Straight Line Approximation methods for drawdown and residual drawdown data. Analysis revealed that after about 90 minutes of pumping there was a marked decrease in transmissivity values. Transmissivity values calculated using early drawdown data range between 10,000 and 13,000 gallons/day/foot (gdf), whereas values calculated using late drawdown data range between 6,600 to 9,000 gdf. The change in transmissivity values indicates the presence of a boundary condition which in this case may represent the boundary between the acidized portion of the formation and the unaltered formation. Similarly, storativity values varied from 1.0×10^{-5} for the early data to 8.5×10^{-5} for the late data.

3.2.6 Video Survey

On December 28, 1990, a video camera survey was conducted to inspect the epoxy-lined casing and well bore after acidification. Both color and black and white video logging runs were recorded on video tape using an onsite video recorder. The video log indicated that the epoxy coating remained intact during acidification and development. Also, the borehole showed no significant caving.

3.3 MONITORING WELL CONSTRUCTION

The ASR system includes three monitoring wells. PZ-1 was designed to monitor activities in the Hosston-Sligo and was constructed as part of the previous Phase IIA activities. MW-CC and MW-GR were constructed in this work phase. Construction of these two monitoring wells, the Cow Creek (MW-CC) and the Glen Rose

3.2.2 *Well Acidification*

To improve well yield, the well was acidified with fifteen thousand gallons of 15 percent hydrochloric acid through 2-3/8-inch steel tubing set at a depth of 500 feet. A 14-inch diameter neoprene gasket was installed on the steel tubing and secured approximately 3 feet above the base of the casing to prevent acid from moving up into the casing. The acid was pumped at a rate of 1.5 bbls/min (65 gpm).

Following acid placement, 6,000 gallons of fresh water was pumped through the steel tubing in 1,000-gallon increments at half hour intervals to displace the acid further into the formation. The acid was allowed to react with the formation for approximately 36 hours prior to initiating well development. During initial well development, the spent acid solution was brought to the surface, neutralized with caustic soda, and held in a pH stabilization tank prior to discharge to the UGRA sludge lagoon.

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Airlift methods were initially employed to develop the well until the discharge water was free of visible particulate matter. A 5-inch O.D. eductor pipe was lowered into the well, below the casing, and a 1-inch PVC air line was installed inside the drill string below the water level, to a depth of 200 feet. Airlift development was initiated at a depth of 536 feet and was progressively stepped down to the bottom of the well (625 feet). Water samples were collected periodically throughout well development to monitor chloride concentration and pH. All water produced during airlift development was discharged to the sludge lagoon. Development water in the first two hours of testing contained chloride levels in excess of 9,900 mg/L. It took more than 120 hours of development to bring levels under 100 mg/L.

The well was developed using airlift methods for a total of 73 hours over a 5-day period. After an extended period of development, it became evident that the lowermost portion of the aquifer, below a depth of 615 feet, was not a productive zone. Consistent sloughing of clay and gravel below 615 feet did not allow complete development to the base of the aquifer. Because this lower unit did not appear to contribute to well yield and potentially could be detrimental to overall water quality, a cement plug was installed from a depth of 613 feet to the base of the borehole or approximately 12 feet of plug.

3.2.4 *Straightness and Plumbness Test*

The straightness and plumbness test was conducted using a 40-foot long rigid dummy. The dummy consisted of two 20-foot links of 4-inch O.D. steel pipe coupled together and three 14-inch diameter neoprene rings. The dummy was suspended from a plumb line on the rig and lowered into the 16-inch casing. As the dummy was lowered into the casing, the deflection of the plumb line from exact center of the casing was measured at each 10-foot interval. Maximum measured deviation off

(MW-GR), began February 6, and was completed February 16, 1991. These wells monitor water levels in the overlying aquifer zones during recharge and recovery operations in the Hosston-Sligo, enabling estimates of the degree of hydraulic connection between the formations and the potential amount of available storage.

The monitor wells are nominal 5.5-inch steel cased with 50-foot open well intervals. MW-CC--located approximately 110 feet east of ASR Well R-1--reaches a depth of 440 feet. The open hole interval straddles the lower Hensell Sand and upper Cow Creek formations. This interval represents the first productive zone overlying the Hosston-Sligo aquifer. MW-GR was completed to a depth of 240 feet in the Lower Glen Rose Formation, approximately 70 feet east of R-1.

The sequence and duration of Phase II-B field activities are presented in Table 3-3. Activities included air rotary drilling, installation of well casing, well development, and collection of water samples.

3.3.1 Monitor Well MW-CC

Page Drilling Company of Kerrville, Texas, performed borehole drilling and well installation. A Gardner-Denver rig was mobilized to the site and set up at location MW-CC on February 6, 1991. After installation of 30 feet of surface casing, an 8 3/4-inch diameter borehole was advanced below the surface casing to a depth of 390 feet for installation of the well casing. Casing joints were butt welded together. The well was capped with a lockable protective cover.

Following casing installation, grout was installed under pressure through a 2 7/8-inch O.D. string of pipe seated at the base of the casing. The grout was forced out at the bottom of the casing and into the annular space between the borehole and casing. The well was grouted to the surface. The grout was allowed to cure for 48 hours prior to additional down hole operations.

Once the grout had cured, the borehole was advanced below the casing to a depth of 440 feet with a 4 3/4-inch bit. The 50-foot open well interval for MW-CC extends from 390 to 440 feet.

The well was developed using airlift methods for approximately 4 hours. During development the well yielded water at an approximate rate of 5 to 10 gpm.

3.3.2 Monitor Well MW-GR

Drilling of the Lower Glen Rose monitor well (MW-GR) began on February 12, 1991. Construction techniques and installation of MW-GR were similar to those used for MW-CC.

The well casing was installed to a depth of 190 feet. The nominal 30-foot casing joints were butt welded and three centralizers were installed at the top, center, and

Table 3-3
Sequence of MW-CC and MW-GR Construction

Activity	Duration
Monitor Well MW-CC:	
Mobilization & setup on MW-CC	02/06/91
Drill and install 10 3/4-inch surface casing to 30 feet	02/07/91
Drill to 390 feet using 8 3/4-inch roller bit	02/07/91-02/08/91
Install and grout 5.5-inch steel casing to 390 feet	02/08/91-02/09/91
Drill 390 - 440 feet using 4 3/4-inch roller bit (open borehole)	02/11/91
Develop well and collect water sample	02/11/91-02/12/91
Monitor Well MW-GR:	
Set up on MW-GR; drill and install 10 3/4-inch surface casing to 30 feet	02/12/91
Drill to 190 feet using 8 3/4-inch roller bit	02/12/91-02/13/91
Install and grout 5.5-inch steel casing to 190 feet	02/13/91
Drill 190-240 feet using 4 3/4-inch roller bit (open borehole)	02/15/91
Develop and collect water sample	02/15/91

base of the casing string. The casing was grouted into place by pressure methods. The grout was allowed to cure for 48 hours prior to advancing the hole with a 4 3/4-inch bit to a total depth of 240 feet. The open borehole interval for MW-GR is from 190 to 240 feet.

The well was developed using airlift methods, as described for MW-CC, for approximately 5 hours. Well yield during development was approximately 1 to 5 gpm.

3.3.3 Monitor Well PZ-1

During the Phase IIA investigation, which occurred from July through September 1989, a 7-inch diameter monitoring well (PZ-1) was installed in the Hosston-Sligo Sand to conduct tests to determine the suitability of the unit to accommodate an ASR facility. In addition, this well will serve as the primary well to monitor the response of the storage formation to recharge and recovery testing. The well was drilled and installed by the Texas Water Development Board and a private contractor. Completion details are summarized in Figure 3-1. Additional details are found in the report entitled *Aquifer Storage Recovery Feasibility Investigation Phase IIA Monitoring Well PZ-1* (CH2M HILL, 1989).

Section 4
ASR TEST CYCLES

Section 4
ASR TEST CYCLES

INTRODUCTION

4.1 OVERVIEW OF TESTING

After construction of the ASR pilot facilities, two recharge and recovery test cycles were conducted at the R-1 site. The goals for these test cycles were:

- To evaluate the effects of injecting treated drinking water into the aquifer and then recovering water from the aquifer
- To test the operational performance of the facility
- To determine hydraulic properties of the aquifer
- To monitor response of the aquifer across the City

Each test cycle consisted of an injection or recharge period, a storage period, and a recovery period. Water samples were collected and water levels were monitored during each period to aid in evaluating the system. Table 4-1 summarizes test cycles.

Table 4-1
SUMMARY OF CYCLE 1 AND 2 TESTS

Cycle	Begin	End	Total Time	Gallons Recharged/ Recovered	Average Recharge/ Recovery Rate
Cycle 1					
Recharge	4/2/91	4/5/91	3 days, 1 hr.	2.93 million	669 gpm
Storage	4/5/91	4/7/91	2 days	---	---
Recovery	4/7/91	4/9/91	2 days, 7 hrs.	2.86 million	867 gpm
Cycle 2					
Recharge	4/15/91	5/14/91	29 days, 3 hrs.	24.9 million	595 gpm
Storage	5/14/91	6/13/91	30 days	---	---
Recovery	6/13/91	7/3/91	20 days	25.0 million	868 gpm

4.1.2 Cycle 1

The first ASR cycle had short recharge, storage, and recovery durations in order to allow for a quick evaluation of the operational portions of the facilities. Recharge of treated water began on April 2, 1991. Water was recharged at an average rate of 675 gallons per minute (gpm) down the injection tubes for most of the test cycle. Some short periods of experimentation occurred during the test, including single tube injection and injection down the annular port.

The majority of the recharge was conducted under pressure from pumps that carry water to the city. However, at one time during the cycle, the system was operated in the siphon mode where water is drawn under vacuum from the clear well to R-1. During siphoning, recharge rates were reduced to about 400 gpm.

Cycle 1 injection ended on April 5, 1991, after a volume of 2.93 million gallons of treated surface water was recharged into R-1.

Water was stored for 2 days. The recovery period began on April 7, 1991, and ended on April 9, 1991. The pumping rate during recovery averaged 890 gpm. One minor pump failure occurred during recovery; however, the downtime was less than one hour. A total of 2.87 million gallons of water was recovered.

At the end of the recovery cycle, 97 percent of the recharged water volume had been recovered. The remaining 60,000 gallons was left in the aquifer as a buffer around the well to prevent the formation water from contacting well materials.

4.1.2 Cycle 2

The second ASR cycle had a longer duration for recharge, storage, and recovery periods. This cycle provided data for evaluating the effects of longer periods on water quality and water level changes. Recharge began at a rate of 600 gpm on April 15, 1991, and ended 29 days later on May 14, 1991. During this cycle all water was recharged through the injection tubes. A total of 24.96 million gallons of treated water recharged into R-1. Water was stored underground for approximately 20 days and then recovered. Recovery of the stored water began on June 13 and ended on July 3, 1991; 100 percent of the recharged water volume was recovered.

4.2 REGULATORY APPROVALS

Operation of the ASR well required approval from two regulatory agencies: The Texas Water Commission (TWC) and the Texas Department of Health (TDH). Injection activities require a Class V injection permit from the TWC. A 1-year testing permit was approved on May 22, 1990. Well construction and operation require approval from the TDH. Well construction approval was received December 14, 1990; the well was certified as a public water supply well on May 20, 1991.

4.3 AQUIFER AND WELL HYDRAULICS

Field data collected during the ASR testing effort was used to develop specific aquifer performance characteristics for use in the groundwater model. These parameters included the following:

- Transmissivity (T)
- Storativity (S)
- Leakance
- Specific capacity

These parameters were developed based on Cycle 1 and Cycle 2 injection and recharge data and water levels measurements at PZ-1, MW-CC, MW-GR, and City wells around the City of Kerrville. A description of the evaluation method and results follows.

4.3.1 Transmissivity and Storativity

Onsite Cycle Testing Results. Aquifer tests at the UGRA water treatment plant included hydraulic analysis during the Cycle 1 and Cycle 2 recharge and recovery periods. During the cycles, recovery and recharge rates were kept constant and recorded manually from the flow meters. Water level in PZ-1, MW-CC, and MW-GR were recorded through a combined use of water level indicators and pressure transducers. Figures 4-1 and 4-2, respectively, show hydrographs of R-1 (as recorded in PZ-1) for Cycles 1 and 2.

Aquifer transmissivity (T) and storativity (S) were determined for each of the tests and compared favorably to the baseline data established during the 50-hour pump test at R-1 prior to Cycle 1. The T and S for the 50-hour pump test was 7,000 and $5x10^{-4}$, respectively. Cycle test T ranged from 6,700 to 7,700 gdf and S ranged from $5x10^{-4}$ to $5x10^{-5}$. A summary of R-1 aquifer properties is presented in Table 4-2.

Table 4-2
AQUIFER PROPERTIES AT R-1

Test	Transmissivity (gdf)	Storativity
16-Hour Pump Test	7200	$3x10^{-4}$
50-Hour Pump Test	7000	$5x10^{-5}$
Cycle 1 Recharge	6700	$6x10^{-5}$
Cycle 1 Recovery	6700	6x10-5
Cycle 2 Recharge	7700	$5x10^{-4}$
Cycle 2 Recovery	7900	$5x10^{-4}$

Figure 4-1

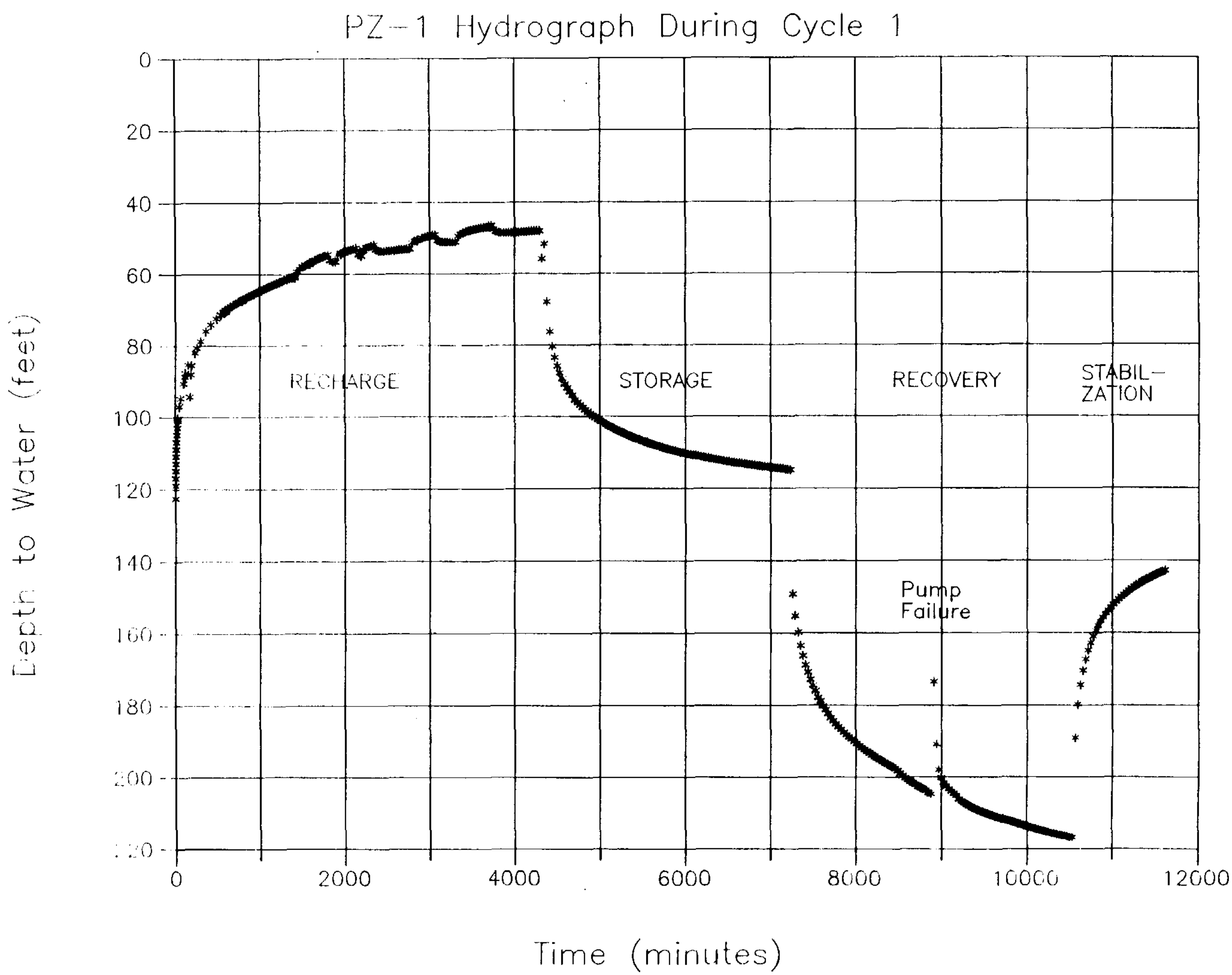
PZ-1 Hydrograph During Cycle 1
Depth to Water (feet)
Time (minutes)
RECHARGE
STORAGE
RECOVERY
STABIL-
ZATION
Pump
Failure
0
20
40
60
80
100
120
140
160
180
200
220
0
2000
4000
6000
8000
10000
12000
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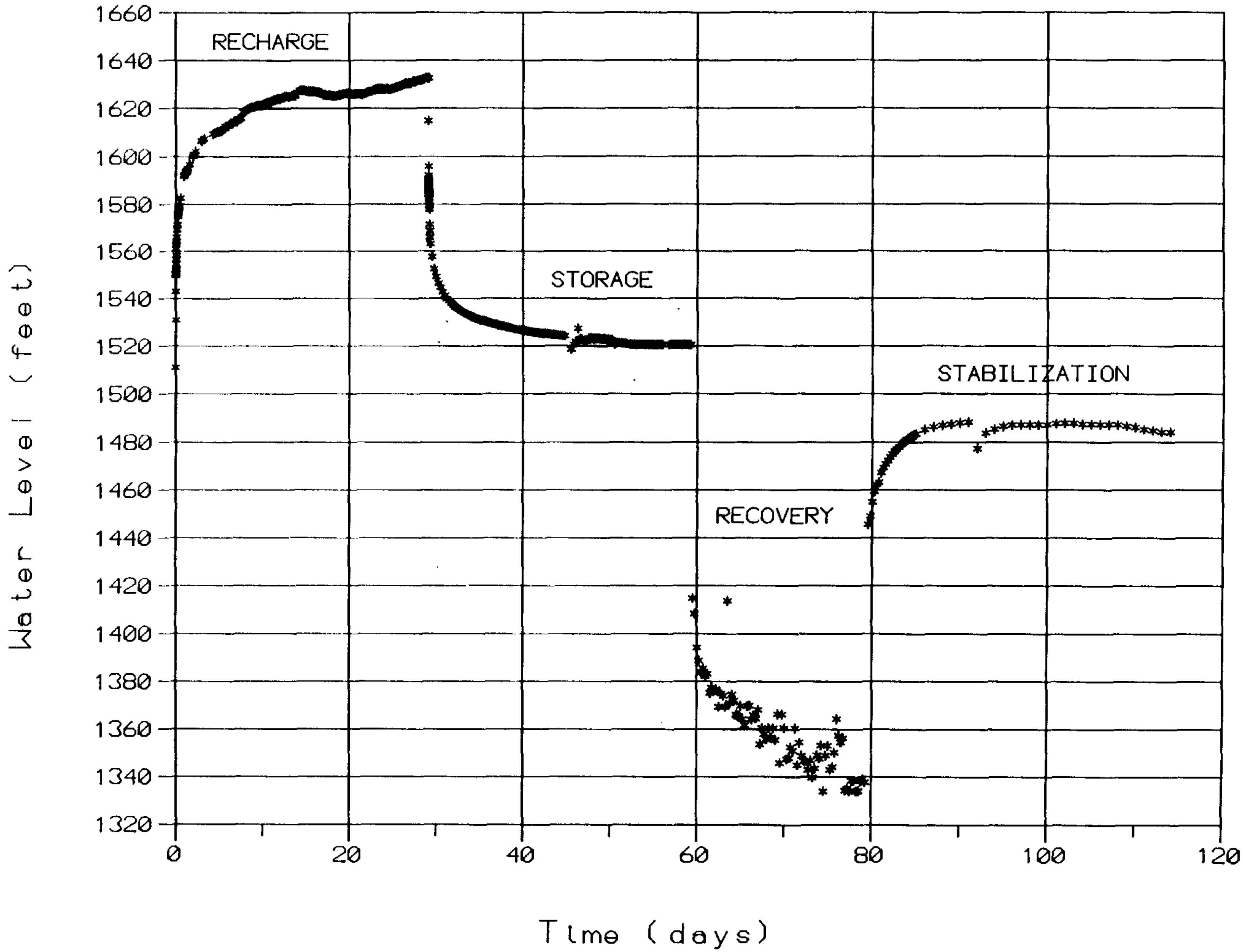
Figure 4-2

R-1 Hydrograph During Cycle 2



Offsite Cycle Testing Results. Offsite measurements and analyses were conducted several times during Cycles 1 and 2 to fill in data gaps for T and S across the city and to compile a record of response of the aquifer during recharge and recovery operations for calibrating the groundwater model. The offsite aquifer tests included monitoring city wells during Cycles 1 and 2.

During the Cycles 1 and 2 tests, water levels were monitored daily by UGRA personnel at the following city wells:

Well No.	Name
4	Plant Well
8	Lewis Street Well
9	H Street Well
10	Lois Street
13	Park Well
14	Travis Street Well
15	Alpine Well

Water levels from these wells were used to assess the effects of recharge and recovery on water levels throughout the city. During the tests, the Lois Street well was the most affected during the recharge recovery periods. This well, which is approximately 5,000 feet from R-1, had water level rises of 36 feet during the Cycle 1 test. During Cycle 2, the water level rose 58 feet at this location.

Figures 4-3 through 4-5 depict the water level changes across the city during Cycle 2 recharge. Figure 4-3 is the water level map for the pre-test conditions. In general, groundwater movement is toward the south. Figure 4-4 portrays water levels after 10 days of recharge or about 9.5 million gallons. This figure displays a well- pronounced cone of impression around the R-1 facility. Towards the Lois Street well (Well No. 10), the cone of impression is higher than towards the other city wells, probably because of the presence of lower transmissivity zones to the west and north of the Lois Street well. Water level rises east and northeast of R-1 were not as great as at Lois Street well because of higher aquifer transmissivity in these directions.

Finally, Figure 4-5 is the water level map at the completion of Cycle 2 recharge. The configuration of the water level surface at this time is similar to Figure 4-4, but more pronounced. These changes may have been greater than shown because of the interference created when several city pumps were turned on.

In summary, the aquifer responded as expected during the Cycle 1 and 2 tests. Water levels rose the most in areas of low transmissivities. Wells in the central

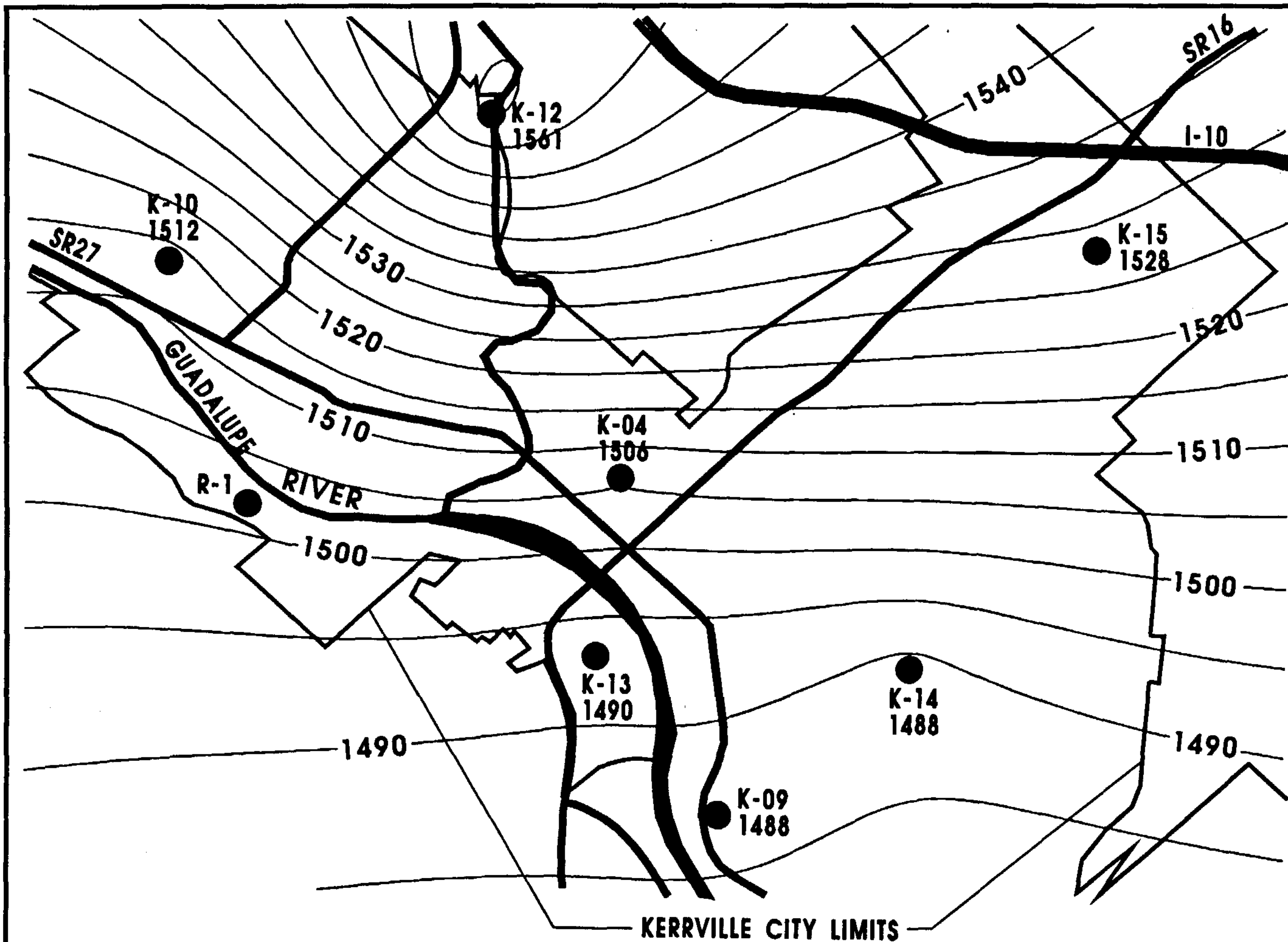


FIGURE 4-3
GROUNDWATER LEVELS PRIOR TO CYCLE 2

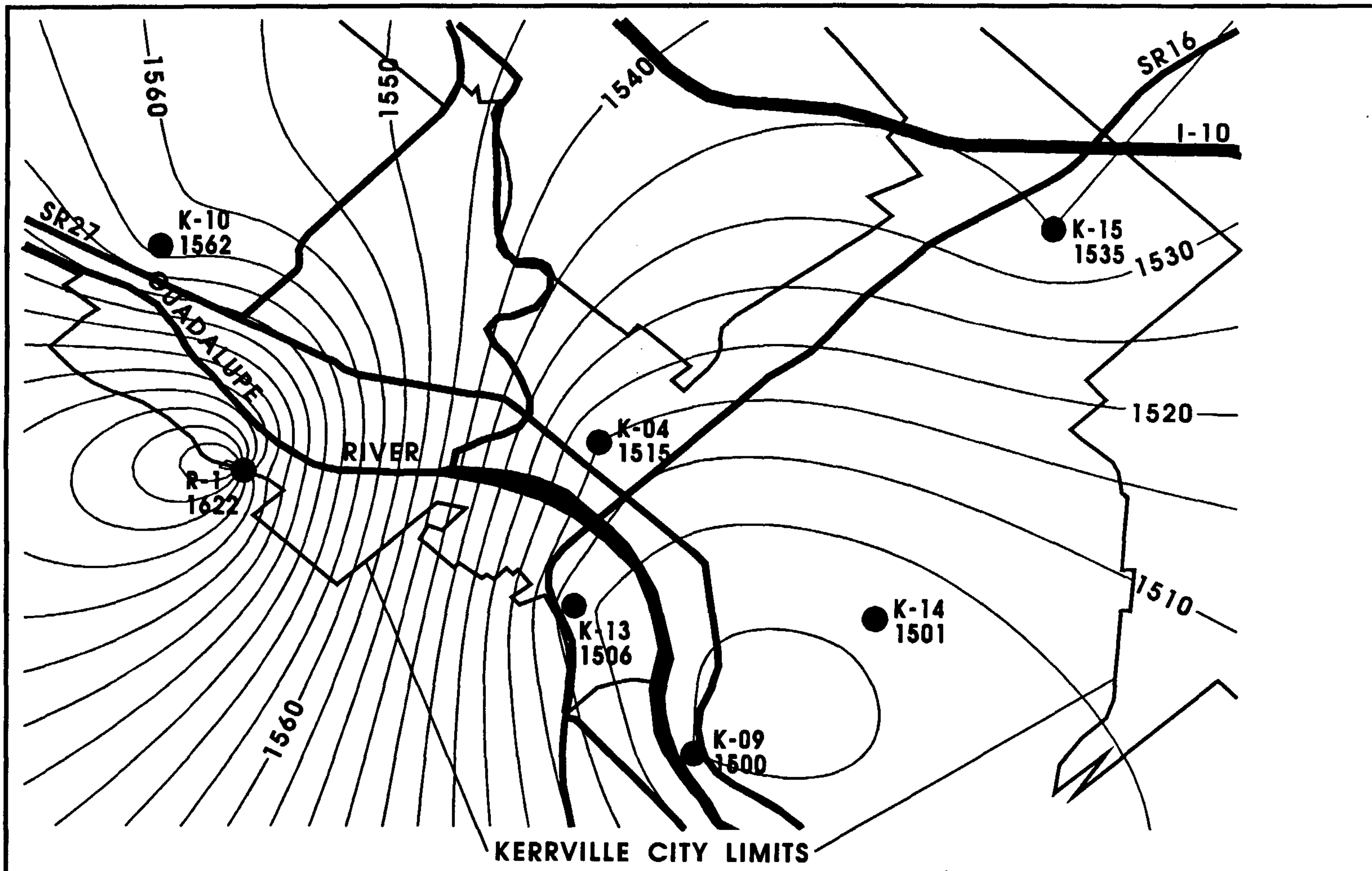


FIGURE 4-4
CYCLE 2 GROUNDWATER LEVELS
9.5 MILLION GALLONS OF RECHARGE

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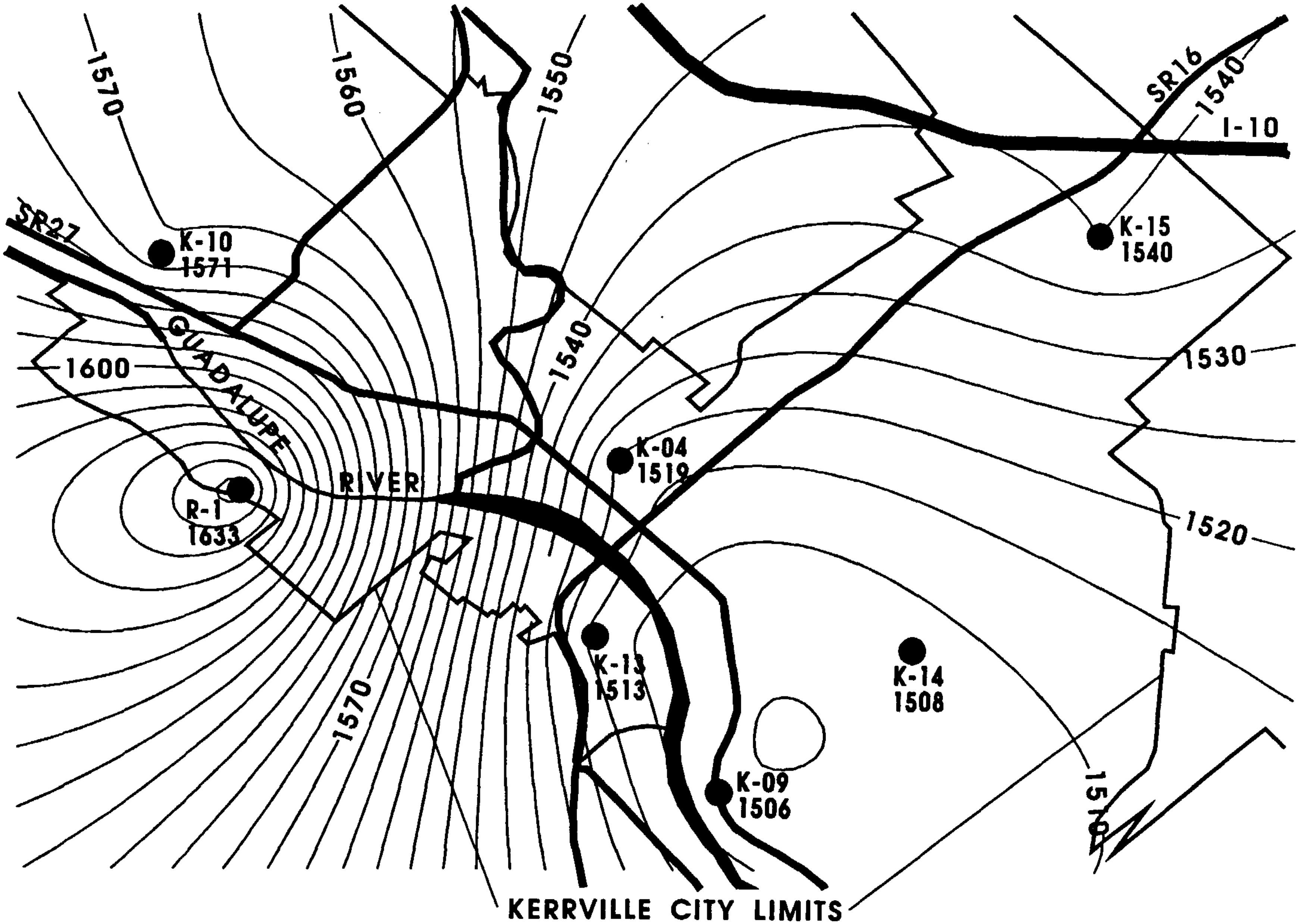

FIGURE 4-5
CYCLE 2 GROUNDWATER LEVELS
25 MILLION GALLONS OF RECHARGE

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portion of the city (the Plant and H Street wells), which exhibit relatively high transmissivities, displayed only minor water level changes. Table 4-3 is a summary tabulation of the water level rises for the monitored wells at various times during Cycle 2 recharge.

Table 4-3
WATER LEVEL RISES DURING CYCLE 2

Well	Rise in Water Level (Feet)		
	1 Day	**10 Days**	**28 Days**
No. 4 Plant	0.4	5.2	12.1
No. 8 Lewis	0.3	2.0	8.0
No. 9 H Street	0.8	-9.6[a]	16.1
No. 10 Lois Street	21.0	47.9	57.7
No. 13 Park	1.8	-1.0	9.8
No. 14 Travis	-3.6[a]	7.5	16.6
No. 15 Alpine	--	7.5	16.3

[a]Trend was affected by pumping of City and/or VA wells.

4.3.2 Leakance

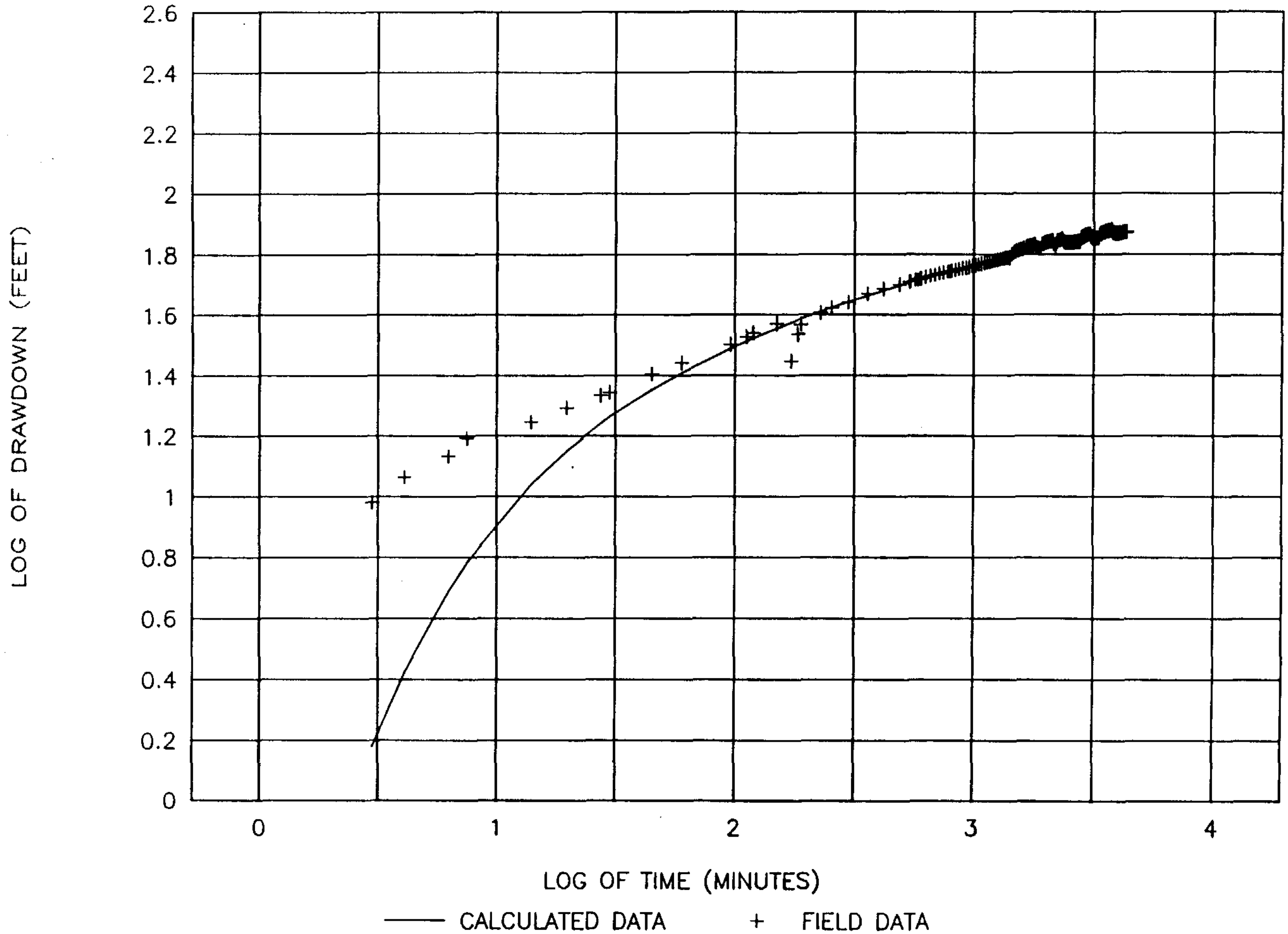
Leakance (water movement into and out of the storage zone) was assessed by two methods: 1) comparisons of the response of observation wells to a standard Theis curve and 2) response of water levels in overlying aquifers. During Cycle 1, review of the Theis curve for recharge and recovery indicated no deviation due to leakance.

Figure 4-6 contains the recharge data for Cycle 1 at PZ-1 and recovery data from the 50-hour pump test at R-1. The theoretical Theis curve data is superimposed over the field data and shows that after approximately 100 minutes of recharge, the field data for recharge and the Theis curve merge to a near match. This close match suggests minimal leakance is occurring. The early data deviation is most likely the result of initial plugging or well effects from the acidization process.

Additionally, water level changes in MW-GR and MW-CC were too small to definitively determine whether leakance had occurred during recharge or recovery during Cycle 1. It was concluded that the test duration was too short to detect leakance.

Figure 4-6

Theis Curve for PZ-1



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Cycle 2 had a much longer duration than Cycle 1, which improves the basis for analysis of leakance. Review of the Cycle 2 recharge data reveals a close fit of the Theis curve and the observed data (Figure 4-6), which suggests minimal leakance of injected water at the vicinity of R-1.

However, review of water levels in MW-CC and MW-GR show an increase in water levels during recharge (Figure 4-7) suggesting some type of leakance is occurring. Water levels rose in MW-CC approximately 7.5 feet during the Cycle 2 recharge period. Similarly, at MW-GR water levels rose about 4 feet during recharge. As shown in Figure 4-7, the water levels in MW-CC quickly stabilized and fell about 3 feet during the storage period. In contrast, MW-GR water levels continued to rise another 2 feet during the storage period before declining at the mid-point of the storage period. From the response of MW-CC during the recharge, storage, and recovery periods, it can be concluded that activities at R-1 have an effect on the Cow Creek aquifer; however, its mechanism is not due to vertical leakance because of the close fit to the Theis curve data.

Review of water levels at the Lewis Street well (Well No. 8) may explain the response at MW-CC. This city well is an open borehole well with an open interval that extends from the Hensell through the Cow Creek and Pine Island into the Hosston-Sligo. This type of completion has resulted in an interconnection of the Hosston-Sligo and overlying aquifers by eliminating the confining zone (the Pine Island Shale). Water is free to exchange between these units. For example, when city wells are pumping and water levels in the Hosston-Sligo are lower than in the Cow Creek-Hensell aquifers, water will flow from the Cow Creek and Hensell down the Lewis well and into the Hosston-Sligo. The reverse is true during recharge at R-1. When water levels are higher than in the Hosston-Sligo because of recharge, flow in the Lewis well will be from the Hosston-Sligo into the Cow Creek and Hensell aquifers.

Figure 4-8 portrays water level responses at the Lewis Street and MW-CC wells. From this figure it can be determined that the two wells exhibited similar responses during the Cycle 2 test.

One important aspect of response is the sequence. For the majority of the cycle the Lewis well responded much more quickly to changes in the Hosston-Sligo aquifer than did MW-CC. This lag time is probably the result of preferential leakance through the Lewis well's open borehole rather than through the intact areas of the Pine Island.

To quantify the leakance through the well bore, a simulated pump test was developed from water level changes in MW-CC. For this simulated pump test, it was assumed that all leakance or injection to the Hensell-Cow Creek aquifer was through the Lewis well. The T and S properties of the aquifer have been previously estimated, approximately 7,000 gdf and 5×10^{-4}, respectively. Pumping rates were then iterated until the T, S, and observed water level changes were approximated. Based on these results, the flow through the Lewis well was estimated at 75 gpm during Cycle 2.

Figure 4-7

MWCC Versus MWGR Water Levels During Cycle 2

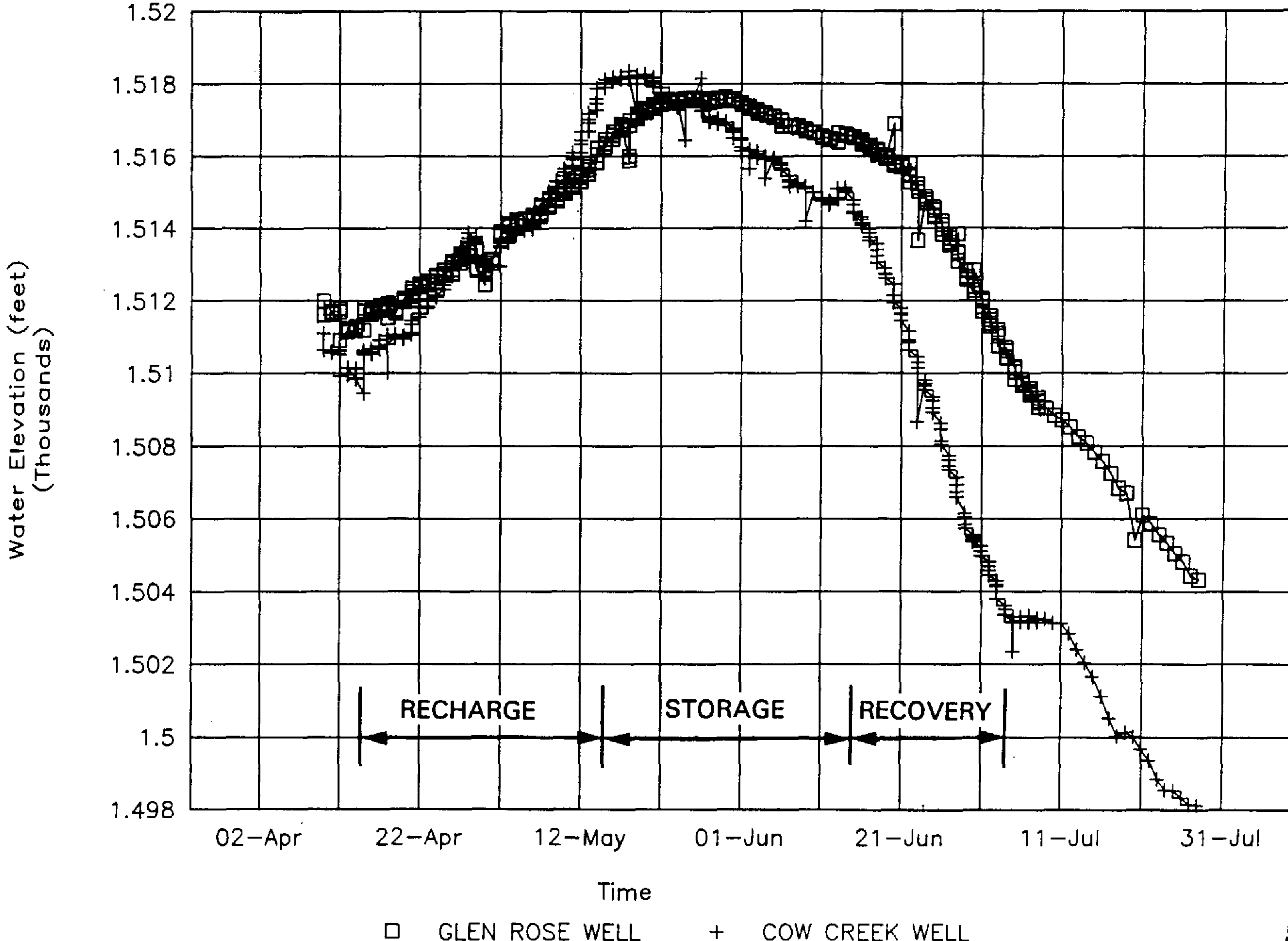

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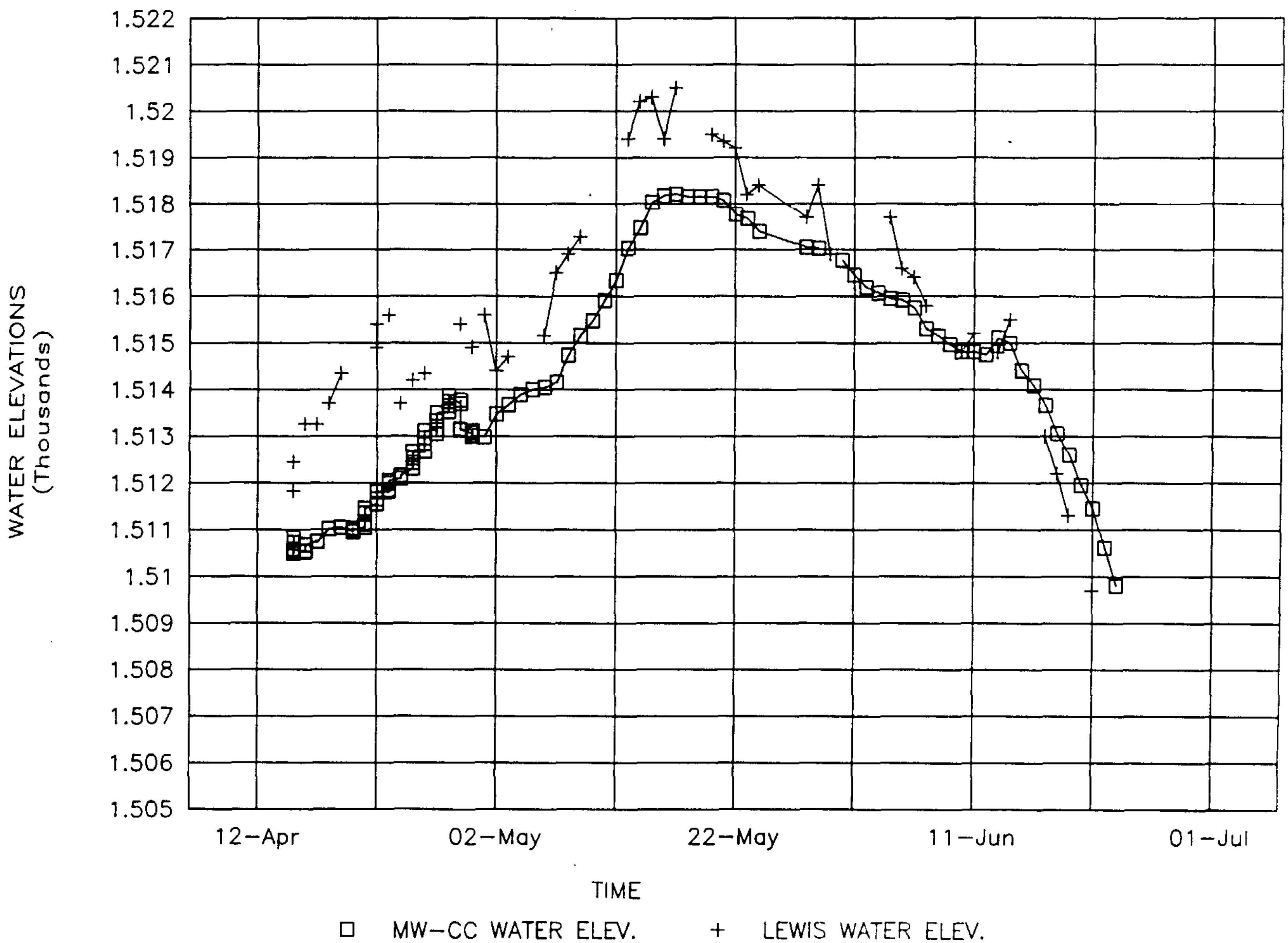
Figure 4-8

MWCC Versus Lewis Well Water Levels During Cycle 2



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Discussion with local well drillers indicates that small industrial and residential wells within the City have also penetrated the upper Hosston-Sligo, which may also contribute to the leakance flow. However, because the Lewis well is so much larger than these industrial and residential wells, the majority of the flow will be predominantly through the Lewis well.

4.3.3 Specific Capacity

The specific capacity of R-1 was determined at various times during the Phases IIA and IIB investigations. The specific capacity ranged from 5.8 to 7.3 gallons/minute/ft (gpm/ft). Table 4-4 summarizes the specific capacity of each test. Each of the specific capacities in the table was determined or adjusted for the first 24 hours of recharge or pumping.

Table 4-4
R-1 SPECIFIC CAPACITY RESULTS

	Specific Capacity (gal/min/ft)	
Test	**Recharge**	**Recovery**
16-Hour Pump Test	--	5.8 at 800 gpm
50-Hour Pump Test	--	6.1 at 890 gpm
Cycle 1	6.8 at 660 gpm	6.8 at 870 gpm
Cycle 2	7.3 at 600 gpm	6.8 at 900 gpm

As shown in Table 4-5, the specific capacity data for recharge in Cycles 1 and 2 gradually decreased with time. In Cycle 2, recharge specific capacity decreased rapidly from 7.2 to 6.0 gpm/ft after two days; then during the remaining 27 days of recharge, specific capacity slowly declined to 4.9 gpm/ft. Cycle 2 recovery specific capacity shows a similar response; it decreased from 6.8 to 6.3 gpm/ft after the first three days of recovery, then slowly decreased over the remaining 15 days from 6.3 to 4.9 gpm/ft.

This type of behavior is normal for recharge wells. Typically, the specific capacity remains fairly constant for the first day or two of recharge, then gradually decreases. This can be the result of either well plugging or mounding around the well.

In this case, mounding is the probable cause. High initial recharge specific capacities occur when area groundwater levels are low; then recharge water gradually begins to mound around the well, increasing local groundwater levels and reducing specific capacities.

Table 4-5
SPECIFIC CAPACITY AT WELL R-1
(gal/min/ft)

	Recharge		Recovery	
Day No.	**Cycle 1**	**Cycle 2**	**Cycle 1**	**Cycle 2**
1	6.8	7.2	7.2	6.8
2	6.2	6.7	6.8	6.6
3	6.2	6.0		6.3
4		6.0		6.0
5		6.0		6.1
6		5.8		5.8
7		5.7		5.7
8		5.5		5.4
9		5.4		5.6
10		5.3		5.8
11		5.3		5.2
12		5.2		5.6
13		5.2		5.2
14		5.1		5.0
15		5.1		5.4
16		5.1		5.1
17		5.2		4.8
18		5.2		4.9
19		5.2		
20		5.2		
21		5.2		
22		5.1		
23		5.1		
24		5.1		
25		5.1		
26		5.0		
27		5.0		
28		4.9		
29		4.9		

4.4 WATER QUALITY

Baseline water quality conditions in the storage zone aquifer were determined during the pre-Cycle 1 pump test. Five samples were collected during this test and averaged to determine baseline conditions. Water samples were also collected during the recharge and recovery periods of Cycles 1 and 2. During recharge, the treated water was sampled on a regular basis, and results were averaged to determine the average water quality of the injected water. Recovered water was also sampled at regular intervals to determine water quality changes that may be caused by reactions with aquifer materials or by mixing.

4.4.1 Cycle 1

Recharge water from Cycle 1 testing was sampled and analyzed by the UGRA laboratory. Seven samples were collected at various times during the recharge test and were analyzed for major cations and anions, trihalomethanes (THMs), coliform, and selected metals. Table 4-6 summarizes the analytical results of the recharged water. The water has low total dissolved solids concentrations (191-268 mg/l) and is mildly alkaline (pH = 7.2-7.6). The major anion is bicarbonate and the major cation is calcium. Residual chlorine in the recharge water averaged about 1 ppm, and THM values averaged 69 ppb, with a range of 33-98 ppb. Chloroform and dibromochloromethane were the major THMs present. Iron was present at concentrations of 0.004-0.24 mg/l. The remaining metals were at or near detection limits. Total suspended solids of the recharge water averaged about 1.6 mg/l, while average turbidity for the samples was 0.26 NTU.

Five Hosston-Sligo water samples were collected during a pre-Cycle 1 pump test to determine background water quality. Results are summarized in Table 4-7. The aquifer water is similar to the recharge water; however, the aquifer water samples exhibited higher TDS concentrations (407-441 mg/l). The groundwater is slightly alkaline (7.2-7.5 pH). Like the recharge water, the major anion is bicarbonate. However, calcium and magnesium, the major cations, are present in nearly equal concentrations. Chloride and sodium, the other major constituents, had average concentrations of 40 and 44 mg/l, respectively. Potassium was present at approximately 6 mg/l in the aquifer samples, whereas it is generally below detection limits in the recharge water. Iron concentrations averaged less than 0.2 mg/l. Remaining minor metals were present at or near detection limits.

The degree of mixing between injected and native groundwater is difficult to estimate because both waters are so close in quality. Typically, the degree of mixing between the injection and formation waters is estimated by evaluating water quality parameters for the recovered water that (1) are assumed to be conservative because they are not involved in geochemical reactions within the aquifer, and (2) tend to show a large difference in concentration between the formation and treated drinking water. This

Table 4-6
Cycle 1 Water Quality Results

Parameter	Units	Cycle 1 Recharge Average	Cycle 1 Recharge Minimum	Cycle 1 Recharge Maximum	Cycle 1 Recovery Average	Cycle 1 Recovery Minimum	Cycle 1 Recovery Maximum
Total Alkalinity	ppm	199	192	208	253	199	312
TOC	ppm	1.8	1.7	2	1.0	0.8	1.1
Chloride	ppm	18.6	17	20	15	8	25
Residual Chloride	ppm	1.03	0.8	1.3	0	0	0
Conductivity	umhos/cm	429	409	477	559	458	643
Flouride	ppm	1.0	0.6	1.4	1.5	1.2	1.7
Total Hardness	ppm	235	220	246	272	242	324
Ammonia	ppm	0.04	0.002	0.08	0.08	0.05	0.09
Nitrate	ppm	1.07	1	1.21	0.6	0.3	0.8
Nitrite	ppm	<0.01	<0.01	<0.01	0.005	<0.01	0.011
Dissolved Oxygen	ppm	8.26	6	9.8	1.2	2	2.5
pH	su	7.42	7.2	7.64	7.7	7.4	7.95
Total Phosphate	ppm	0.014	0.003	0.033	0.008	0.001	0.01
Suspended Solids	ppm	1.6	0.2	5.5	1.1	1.2	1.5
Dissolved Solids	ppm	223	191	268	355	271	430
Sulfate	ppm	13	10	14	20	14	26
Temperature	c	17.5	15	18	23.1	21.5	25
Turbidity	ntu	0.256	0.095	0.8	0.20	0.07	0.39
Coliform	col/100ml	0	0	0	0	0	+
Calcium	ppm	53	48	59	51	50	53
Cadmium	ppm	---	---	---	0.0003	0.0003	0.0003
Chromium	ppm	<0.0003	<0.0003	<0.0003	---	---	---
Iron	ppm	0.055	<0.05	0.24	0.04	<0.05	0.07
Sodium	ppm	8.9	8.2	9.8	22.1	10.4	31.8
Zinc	ppm	<0.02	<0.02	<0.02	---	---	---
Chloroform	ppb	24.9	11	39.1	7.2	1.4	14
Bromodichloromethane	ppb	24.2	11.5	35	8	1	16
Dibromochloromethane	ppb	19.5	<1	38.6	2	<1	12
Bromoform	ppb	<1	<1	<1	<1	<1	<1
Total THM	ppb	68.5	22.5	112.7	16.7	3	38

Table 4–7
Chemical Analyses From The 50 Hour Pump Test

Parameter	Units	R1–1 3/20/91 1036 Hrs	R1–2 3/20/91 1423 Hrs	R1–3 3/21/91 0900 Hrs	R1–4 3/21/91 1400 Hrs	R1–5 3/22/91 0835 Hrs	Average	Maximum	Mimimum
Total Alkalinity	ppm	356	348	348	343	339	347	356	339
TOC	ppm	0.8	0.9	1	0.9	1	0.9	1	0.8
Chloride	ppm	48	44	37	36	35	40	48	35
Residual Chlorine	ppm	0	0	0	0	0	0	0	0
Conductivity	umhos/cm	765	765	739	724	718	742	765	718
Flouride	ppm	1.4	1.5	1.3	1.4	1.5	1.4	1.5	1.3
Total Hardness	ppm	339	339	333	337	325	335	339	325
Nitrate	ppm	0.2	0.3	0.2	0.4	0.2	0.3	0.4	0.2
Nitrite	ppm	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01	<0.01
Dissolved Oxygen	ppm	1.1	6.2	8.1	---	5.7	5.3	8.1	1.1
pH	su	7.25	7.47	7.24	7.3	7.3	7.3	7.47	7.24
Total Phosphate	ppm	0.039	---	---	---	---	0.039	0.039	0.039
Total Solids	ppm	422	422	408	419	442	423	442	408
Volatile Solids	ppm	70	68	132	73	1	69	132	1
Suspended Solids	ppm	3.8	0.8	1	2.4	0.8	1.8	3.8	0.8
Vol Sus Solids	ppm	2	0.2	0.2	0	0.4	0.6	2	0
Dissolved Solids	ppm	418	421	407	417	441	421	441	407
Sulfate	ppm	33	37	37	35	37	36	37	33
Temperature	c	22	---	23	---	18	21	23	18
Turbidity	ntu	0.98	0.98	0.92	0.23	---	0.78	0.98	0.23
Coliform	col/100ml	0	0	---	0	0	0	0	0
Aluminum	ppm	<0.3	<0.3	<0.3	<0.3	<0.3	<0.3	<0.3	<0.3
Arsenic	ppm	0.0005	0.0004	<0.0001	0.002	<0.0001	0.0006	0.002	<0.0001
Barium	ppm	<0.5	0.5	<0.5	<0.5	0.6	0.2	0.6	<0.5
Calcium	ppm	63	60	57	58	56	59	63	56
Cadmium	ppm	<0.005	---	---	---	---	<0.005	<0.005	<0.005
Chromium	ppm	0.0007	---	---	<0.001	---	0.0004	0.0007	<0.001
Copper	ppm	<0.2	---	---	---	---	<0.2	<0.2	<0.2
Iron	ppm	0.24	0.12	0.12	0.17	0.19	0.17	0.24	0.12
Manganese	ppm	<0.02	0.02	<0.02	0.02	<0.02	0.01	0.02	<0.02
Potassium	ppm	5.4	5.7	6	6	6	5.8	6	5.4
Lead	ppm	0.003	0.003	0.008	<0.001	<0.001	0.003	0.008	<0.001
Selenium	ppm	<0.002	<0.002	0.003	0.003	0.004	0.002	0.004	<0.002
Sodium	ppm	40	45	45	45	45	44	45	40
Zinc	ppm	<0.02	0.03	0.02	<0.02	<0.02	0.01	0.03	<0.02
Chloroform	ppb	<1	3.1	1	<1	<1	0.8	3.1	<1
Bromodichloromethan	ppb	<1	<1	<1	<1	<1	<1	<1	<1
Dibromochloromethan	ppb	<1	<1	<1	<1	<1	<1	<1	<1
Bromoform	ppb	<1	<1	<1	<1	<1	<1	<1	<1
Magnesium	ppm	35	36	34	34	34	35	36	34

analysis is difficult to accomplish at Kerrville because water quality of the two waters is very similar. Because the two are both excellent waters, the consequences of mixing one with the other is minimal.

Water recovered during Cycle 1 was sampled at six different times, roughly every 500,000 gallons. This water was analyzed for parameters similar to those of the pump test and recharge samples (Table 4-6). In general, there was a gradual concentration increase in most parameters as recovery progressed. Figure 4-9, is the recovery diagram for sodium, and depicts the trend exhibited by most parameters. A system where no mixing occurs would show recovery samples plotting near the recharge water samples. The Cycle 1 plots indicate that some mixing has occurred, but the amount of mixing is not significant for an initial cycle.

Figure 4-10 is the THM recovery plot. This diagram illustrates that although the storage time was short (two days), significant THM reduction was observed. Initial THM concentrations averaged 69 ppb. The range of THM in recovered water was 3 to 38 ppb. No residual chlorine was observed in the recovered water.

Another significant observation noted in the recovery test was the trend of the turbidity analyses. Eight samples for turbidity analyses were collected in the first hour of recovery. Results of these samples indicated that turbidity levels after the first four minutes of recovery were all below 1 NTU units. These results may be attributed to the well construction techniques (reverse air rotary drilling and epoxy lined casing).

4.4.2 Cycle 2

During Cycle 2 recharge, 22 samples were collected and by the UGRA laboratory. Experience from Cycle 1 permitted a reduced set of analytes. A summary of Cycle 2 recharge water quality is found in Table 4-8.

During the recovery period, 28 R-1 water samples were collected and analyzed by the UGRA laboratories. Analytical data are summarized in Table 4-8.

As was the case in Cycle 1, recovery plots were constructed for selected Cycle 2 analytical parameters, including chloride and THM results. As shown in Figure 4-11, background chloride averages approximately 40 ppm in the Hosston-Sligo at R-1. Cycle 2 recharge water quality averaged 16 mg/l. Using chloride as an indicator, recovered water quality indicated that recharge water exhibited only slight mixing with native water. Maximum chloride concentrations were 24 mg/l, which more closely resembles the recharge than the native groundwater.

The analysis of THM recovery data is more complex as it appears that additional reactions may be occurring in the formation. A plot of Cycle 2 THM data is presented in Figure 4-12. It shows THM levels during initial recovery to be much

Table 4-8
Cycle 2 Water Quality Results

Parameter	Units	Cycle 2 Recharge Average	Cycle 2 Recharge Minimum	Cycle 2 Recharge Maximum	Cycle 2 Recovery Average	Cycle 2 Recovery Minimum	Cycle 2 Recovery Maximum
Total Alkalinity	ppm	191	142	231	247	186	297
TOC	ppm	1.6	1.5	2	1	0	1.4
Chloride	ppm	16	9	20.3	21	18	24
Residual Chlorine	ppm	1.68	0.89	3.54	0	0	1.1
Conductivity	umhos/cm	417	408	440	497	195	602
Flouride	ppm	1.0	0.7	1.1	2	1.1	2.8
Total Hardness	ppm	231	174	301	264	196	407
Ammonia	ppm	0.5	0.08	0.8	0	0	0
Nitrate	ppm	0.82	0.4	1.2	0	0	0.8
Nitrite	ppm	0.01	<0.01	0.02	0	0	0.05
Dissolved Oxygen	ppm	6.8	5.6	8.3	0	0	2.68
pH	su	7.67	7.4	8.1	8	7.21	8
Total Phosphate	ppm	0.0035	0.003	0.004	0	0	0.019
Suspended Solids	ppm	0.39	<0.067	1.4	0	0	1.6
Dissolved Solids	ppm	252	191	340	292	0	381
Sulfate	ppm	14	6.3	18	19	0	27
Temperature	c	22.1	20.5	24	3	0	24
Turbidity	ntu	0.156	0.1	0.232	0	0.1	1.1
Coliform	col/100ml	0	0	0	0	0	0
Calcium	ppm	46	29	56	49	33	67
Iron	ppm	0.02	<0.01	0.1	0	0	0.06
Sodium	ppm	8.0	7.3	10.4	14	0	19
Magnesium	ppm	17	15	17	19	0	23
Chloroform	ppb	15	2.5	80	33	0	87
Bromodichloromethane	ppb	13	4	64	9	0	40
Dibromochloromethane	ppb	7	<1	21	0	0	0
Bromoform	ppb	1	<1	21	0	0	0
Total Thm	ppb	35	6.5	186	42	0	127

Figure 4-9

Cycle 1 Sodium Recovery Plot

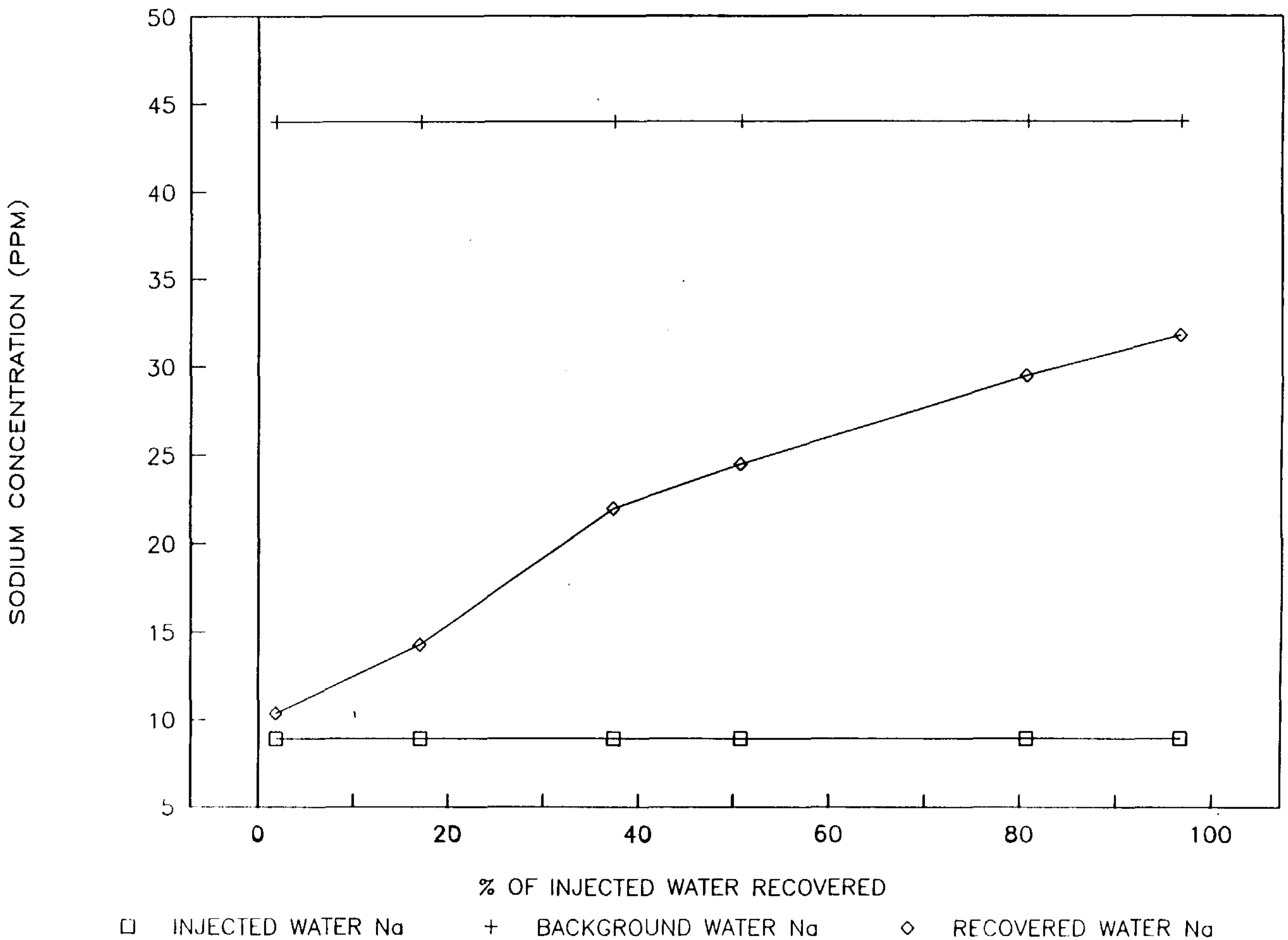

Figure 4-10

Cycle 1 THM Recovery Plot

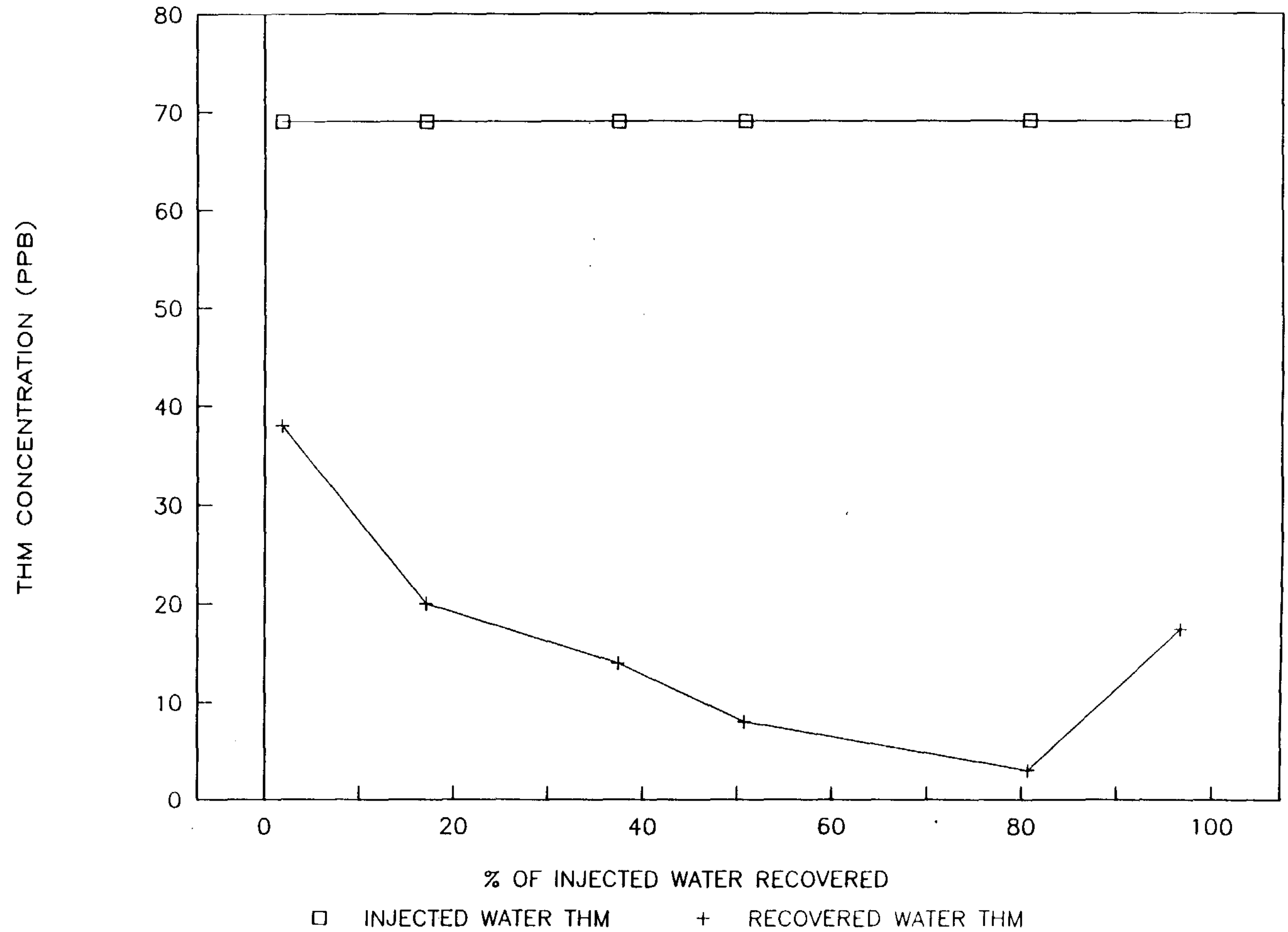

Figure 4-11

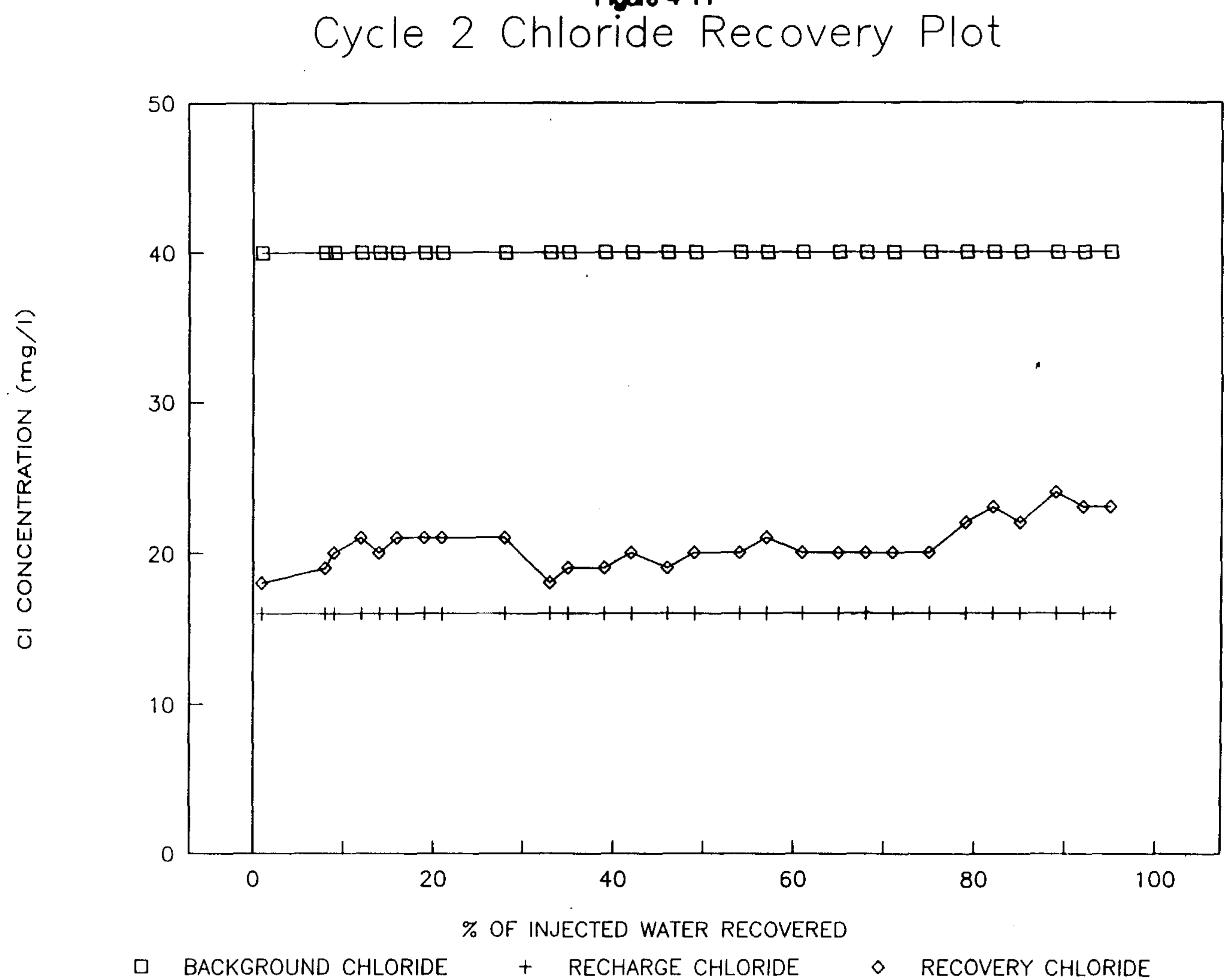

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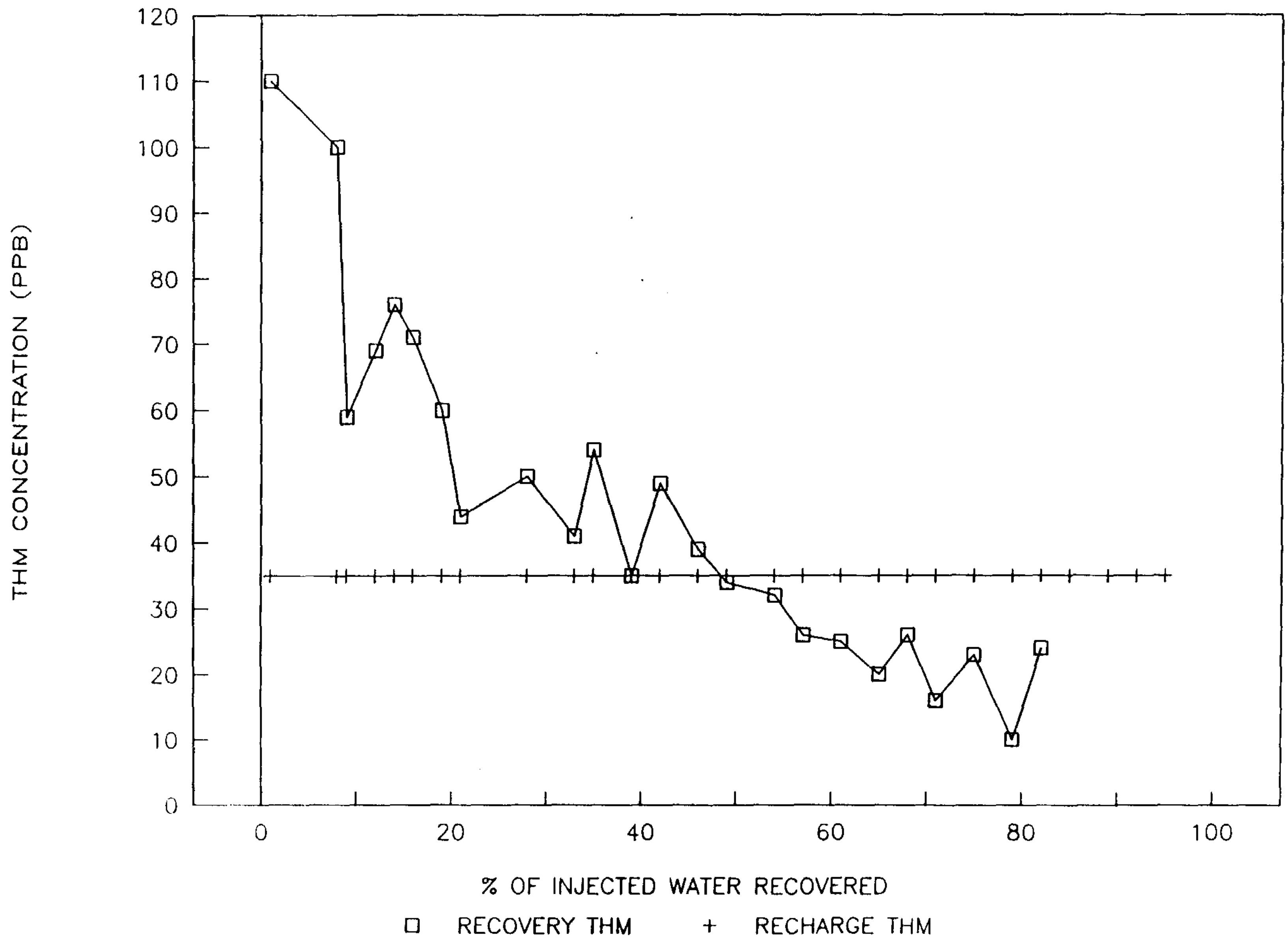
Figure 4-12

Cycle 2 THM Recovery Plot



higher than THM levels during recharge, suggesting THMs are being formed down-hole. THM concentrations in the recovered water did not fall below recharge levels until approximately 50 percent of the recharge volume was recovered. At this point, THM concentrations decrease through the end of the test.

There are several possible explanations for this THM behavior. Initial high THM levels in the recovered water may signify continued THM production within the aquifer following recharge. Recharge water contains free chlorine which is reactive. The subsequent decrease during recovery may indicate diffusion with native water or the loss of THM production caused by the reduction of residual chlorine in the recharge water.

Further review of THM components also reveals interesting trends. Although all four components of THM were found in the recharge water, only chloroform and bromodichloromethane were found in recovered water. UGRA has analyzed water from various locations within the distribution system and has shown that all four THM components can be found throughout the city. This suggests that the fate of THM in the subsurface may not be the same as within the distribution system.

The cycle of THM formation and reduction has been observed at other ASR sites. The mechanism for this phenomenon is an area of much research, but to date, no complete explanation has been developed.

4.5 SUMMARY AND CONCLUSIONS

Cycle testing of R-1 was successful in meeting the goals of the tests. The R-1 prototype well was operated under short and long term conditions and by the end of the test, the facility was functioning properly. The tests have confirmed that aquifer recharge is possible through either the injection tubes or the annular port. Recharge was also possible under vacuum (siphon feed) or pressure conditions.

Field tests indicate that recharge rates are possible over a range from 750 gpm under pressure to 400 gpm by siphon methods. A stable recharge rate of 600 gpm was demonstrated but aquifer plugging may occur over long durations of recharge. Based on experience at other facilities, recharge rates are estimated to drop to 550 gpm during long-term testing. Field results also indicate that short term recovery rates in excess of 1,000 gpm are possible. Typically, the recovery rates exceeded the 800-gpm design rate and averaged more than 860 gpm. Experience at other ASR facilities suggests that periodic recovery or backflushing at high rates for a few hours can reverse any plugging effects during long-term recharge.

Transmissivity and storage coefficients for the Hosston-Sligo are estimated at 7,800 gdf and 5×10^{-4}, respectively.

Leakance across the confining Pine Island Shale is occurring in areas away from the R-1 test site, but it is relatively small and may be the result of leakage through poorly completed wells and/or through zones where the Pine Island shale is absent.

Water levels in wells across the city indicated that pressure responses could be observed as far away as the Alpine and Travis Street wells. Closer to R-1, water level rises of approximately 60 feet were observed at the Lois Street well during the 29 day recharge period.

The degree of mixing between the recharge water and native groundwater is difficult to assess because of the waters having very similar water qualities. Based on sodium concentrations, there appears to be little mixing occurring. Because both sources are of such high quality, the effect of mixing is of little concern at this site.

THM results are inconclusive. Cycle 1 results indicate that THM levels of recovered water were significantly lower than recharge water. However, initial Cycle 2 recovery water was higher than the recharge concentrations and suggests that THM may form in the subsurface. Additional testing of THM response in future ASR cycles is recommended.

Section 5
GROUND WATER MODEL

Section 5
GROUNDWATER MODEL

A groundwater flow model was developed for the Kerrville area to simulate the main supply aquifer's response to predicted pumping demands and ASR. The model enabled us to represent various pumping and injection rates and locations and to observe the corresponding predicted rise and fall in aquifer water levels.

The United States Geological Survey (USGS) three-dimensional finite difference groundwater flow model MODFLOW was used for the aquifer simulation. This model was chosen for its three-dimensional simulation and time-discretization capabilities, and for its variable grid feature.

5.1 MODEL DESCRIPTION

The Kerrville model was developed to represent two of the inter-related aquifer systems beneath the Kerrville area:

- The Hosston-Sligo of the Lower Trinity
- The Middle Trinity (consisting of the Hensell Sand and the Lower Glen Rose)

Since the aquifer of interest was the Hosston-Sligo, a greater amount of detail went into that portion of the model. The Middle Trinity was included because of its inter-connection with the Hosston-Sligo via vertical leakance through the Pine Island confining shale unit.

Figure 5-1 shows the effective area of the model which covers about 156 square miles and is centered on the City of Kerrville. Hydrogeologic characteristics across this area were compiled and reviewed for input to the model where they were available, and estimated based on regional information and hydrogeologic judgement when specific information was not available. Relatively unknown hydrogeologic processes in the Kerrville area, such as river recharge, were not included in the model formulation.

The model utilizes all existing City wells, including the ASR well R-1, for pumping to meet predicted demands, and utilizes R-1 and Well No. 5 for injection during recharge simulations.

5.2 MODEL DEVELOPMENT

The area of the model was first discretized spatially by dividing the area into a grid block of "cells". Aquifer characteristics were assumed to be uniform within each cell, but may vary between cells. Aquifer parameters for each cell was estimated based on recent aquifer tests, published groundwater resource reports, and hydrogeologic

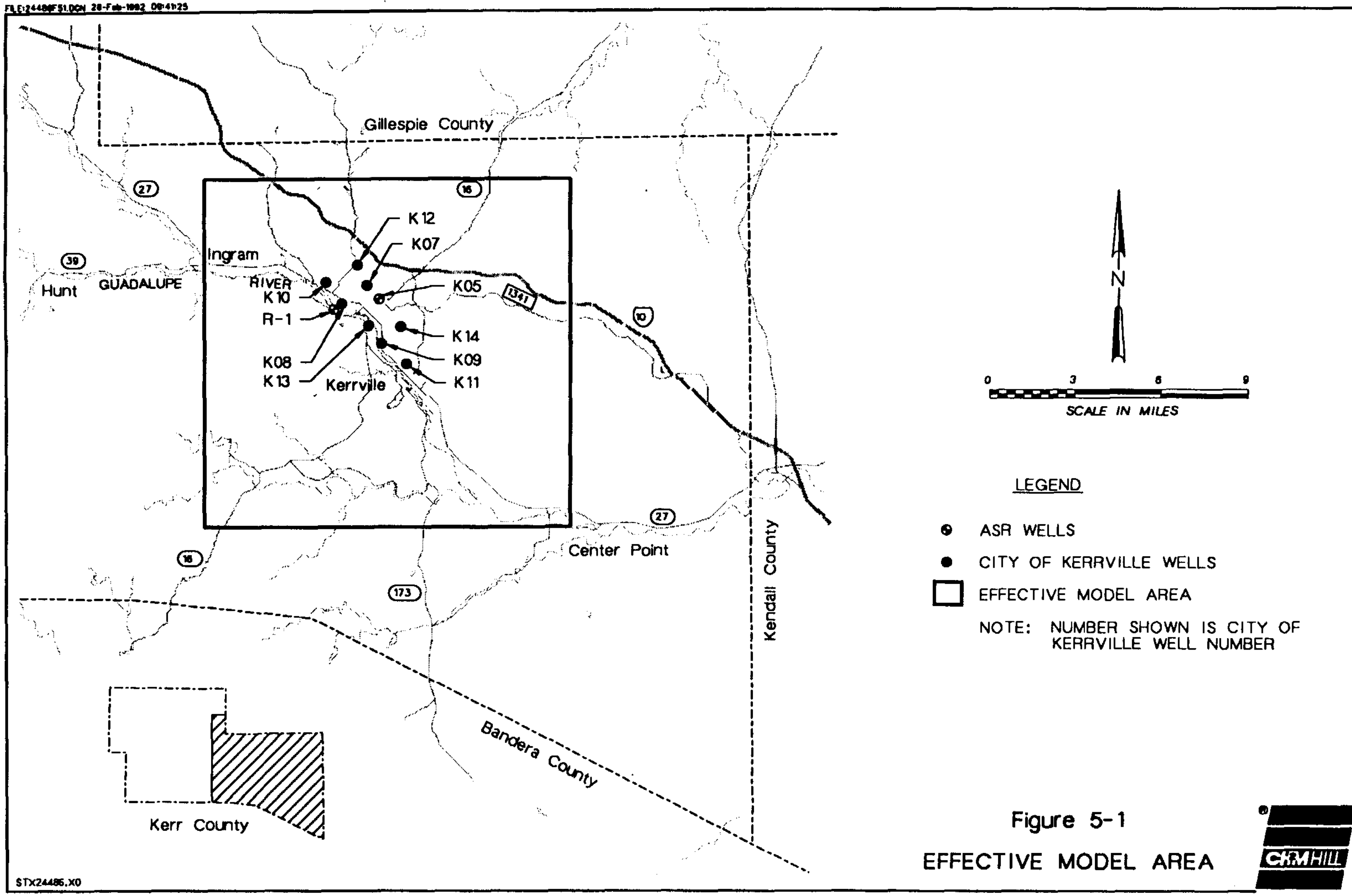


Figure 5-1
EFFECTIVE MODEL AREA

judgement. These parameters were then calibrated within the model using actual water level measurements and aquifer test field data. The determination of aquifer parameters used in the model and the model development process are described in the following paragraphs.

5.2.1 Aquifer Characteristics

Several sources were utilized to provide information regarding aquifer characteristics for initial input to the model and for refinement of the calibration. First, information from published sources such as the TDWR Report 273 (Ashworth, 1983) and TWDB Report 102 (McDonald, 1988) were reviewed for regional information and existing well-specific data for the Hosston-Sligo and the Middle Trinity. Site-specific data for the Hosston-Sligo were obtained from off-site pump tests and from the ASR well cycle tests conducted as part of the study.

5.2.2 Off-Site Pumping Tests

Review of data collected in Phase I investigations indicated that there were several areas in the city where aquifer properties were uncertain or unknown. These included the far western portions of the City, near Kerrville Well No. 10, and the eastern and northern portions, near Well Nos. 9, 13, 14, and 15.

Planned cycle testing would provide data for the western portion of the city; however, a separate pump test was necessary in the eastern section. Well No. 9 (H Street well) was chosen as the point test well because of its central location relative to the wells of concern and its good working condition.

The H Street pump test was conducted during the early portion of Cycle 2 recharge. Water levels during pumping and recovery were measured by UGRA laboratory staff at the Alpine, Travis Street, and Park wells in addition to the pumping well.

Beginning on April 17, 1991, the H Street well was pumped at a constant rate for 48 hours. The pumping rate was approximately 660 gpm. Pump test water, with the exception of the first 5 minutes of flushing of the well, was added to the city water distribution system.

Data from the pump test was used to calculate transmissivity and storage coefficients for the aquifer. At H Street, the transmissivity was calculated to be 14,900 gdf. At the Travis Street location, transmissivity was calculated to be slightly lower (14,000 gdf), with a storage coefficient of 1.9×10^{-4}. Because of interferences by Cycle 2 and pumping effects outside of the City, transmissivity and storage capacity could not be determined at the Park and Alpine well locations during this pump test.

During the H Street pump test, drawdown in the pumping well reached a maximum of 178 feet. During the first 24 hours of the test, drawdown was 173 feet, resulting in a specific capacity of 3.82 gpm/ft.

At the Travis Street well, which is located approximately 4,500 feet from the H Street well, drawdown was 13 and 17 feet after 24 and 48 hours, respectively.

The Lois Street well was tested on April 16, 1991. The well was pumped in order to obtain water quality samples for a TWC underground storage tank study in the area. The pump test was of very short duration (15 minutes) and occurred during the first two days of Cycle 2 recharge at R-1. The pumping rate for the 15 minutes averaged 800 gpm, with a total pumped volume of 12,000 gallons. Water level declines during the test reached a maximum of 39.6 feet at the end of pumping (15 minutes). Recovery was monitored for 17 minutes after the pump was shut off. During this time, water levels recovered to within 10 percent of original levels. Transmissivity at the Lois Street well was calculated to be 21,500 gdf.

The transmissivity value for the Lois Street well should be used with caution because the small volume of water removed from the aquifer resulted in the testing only of the formation very close to the well. Any effects of well casing storage or formation effects caused by acidization may have not been completely overcome during such short duration test. However, when compared with other tests by Guyton (1973) there is some consistency. Guyton has reported transmissivity of 24,400 gdf for the Lois Street well.

5.2.3 R-1 Cycle Tests

Two cycle tests were conducted at R-1 in the spring and summer of 1991 and are described in detail in Section 4. Cycle 1 was a short-duration 3-million-gallon cycle that was used for testing the ASR facility. The longer Cycle 2 test involved recharge and recovery of 25 million gallons of water. The resulting hydrogeologic and chemical information from these tests indicated that the ASR facility was functioning as designed and aquifer response was as expected.

The hydrogeologic data obtained from the cycle test was used to develop the ground-water model. Transmissivity and storage coefficients were used to fill in data gaps on aquifer properties in the western portion of the aquifer. Furthermore, the hydrographs developed for city wells monitored during the test were used in the transient calibration of the model.

5.2.4 *Summary of Aquifer Characteristics*

Aquifer parameters required for confined system model input are transmissivity, storage coefficient, and vertical leakance, if any. These parameters were estimated for both the Hosston-Sligo and the Middle Trinity from available historic and current aquifer tests and reports.

Data gathered from the literature and field tests for the Hosston-Sligo aquifer are presented in Table 5-1. Not unexpectedly, considering the nature of permeability in the Hosston-Sligo, a wide range in transmissivity values is observed. It appears that a higher transmissivity zone extends through the area that parallels the Guadalupe River, with lower transmissivity observed to the north-northeast (Well No. 15). Lower transmissivity is also expected to the south-southwest based on regional information which describes a decreasing permeability of the Hosston-Sligo in that direction.

The Middle Trinity aquifer is described by Ashworth (1983) as demonstrating an average transmissivity value of 1700 gpd/ft for the entire Hill Country area. No current tests of wells screened in the Middle Trinity were available to definitively refine that value for the immediate Kerrville vicinity.

Storage coefficients determined by historic and recent pump tests in the Kerrville area generally fall in the 10^{-5} range. This range is accepted as appropriate for confined aquifer systems.

Vertical leakance between the Middle Trinity and the Hosston-Sligo through the Pine Island formation was determined from laboratory vertical permeability tests and the Pine Island thickness. The vertical permeability tests demonstrated a vertical permeability in the Pine Island of $5x10^{-6}$ feet/day. The thickness of the Pine Island directly beneath the City of Kerrville is generally reported to be an average of ten feet thick but gradually thins out in the northern areas of the county.

5.2.5 *Model Setup*

The model was constructed on a quasi-three dimensional grid, with the confining bed of the Hosston-Sligo (below the Middle Trinity) represented by a vertical leakance factor between the two aquifers. The effective area of the Kerrville model grid, which covered approximately 156 square miles, was subdivided into 30 columns, 34 rows, and two layers (representing the Hosston-Sligo and the Middle Trinity) resulting in a total of 2040 cells. The cell size is smallest within the city limits at 200 feet to a side. Cell widths range up to 6 miles in the outer portions of the grid.

Based on the field and literature data, we developed a transmissivity profile for the Hosston-Sligo in the modeled area to allow input of varying transmissivity to the model on a cell-by-cell basis. This profile was created with the aid of an interpolative contouring program and was modified and refined based on regional information, well

Table 5-1 Summary of Aquifer Characteristics			
Well	**Transmissivity** (gpd/ft)	**Storage Coefficient**	**Source**
ASR Well (R-1)	7000	0.0007	Phase IIB
No. 3 - Plant Well	22000 21500 23500	0.00005 0.0031 0.0006	Phase I Guyton Guyton
No. 4 - Plant Well	24000 23500 24800	0.00005 0.00092 0.000014	Phase I Guyton Guyton
No. 5 - Plant Well	23500 34343	---- ----	Phase I Guyton
No. 7 - Harper St Well	24800 16500 20000	0.00002 0.00019 0.000022	Phase I Guyton Guyton
No. 8 - Lewis St Well	23218 40000 23200	0.00074 ---- ----	Phase I Phase I Guyton
No. 9 - H Street Well	14900 15007 15100	---- 0.00003 0.00003	Phase IIB Guyton Guyton
No. 10 - Lois St Well	21500 24400	---- ----	Phase IIB Guyton
No. 11 - Meadow View Well	22000	----	Guyton
No. 12 - Harper Rd Well	20000	----	Phase I
No. 13 - Park Well	16000	----	Guyton
No. 14 - Travis St Well	14000	0.00019	Phase IIB
No. 15 - Alpine Dr Well	1450	----	Phase I

capacities, and hydrogeologic judgement. The disadvantages to such an approach are that it assumes transmissivity varies interpolatively between given data points, and it does not allow for varying hydrogeologic conditions. The advantages are that it provides a simplified approach to varying transmissivity, and--although it cannot predict unexpected highs and lows of transmissivity--it can represent the average variance in values across the modeled area.

Storage coefficient for each of the two aquifers was assumed to be equal. Based on results of this study, the range of aquifer storage coefficients utilized in the model calibration were varied between 1.4×10^{-4} to 1.4×10^{-5}.

The Middle Trinity aquifer transmissivity was initially input to the model as a uniform value. Transmissivity that varied across the area of the model was also evaluated as a possibility for the Middle Trinity layer based on its known thickness change across the Kerr County area. Both uniform and varying values were developed based on the thickness of the Middle Trinity, field and literature results, model calibration, and hydrogeologic judgement.

Vertical leakance between the Middle Trinity and the Hosston-Sligo was determined by taking the vertical permeability of the confining unit (the Pine Island) and dividing it by the thickness of the confining unit. The leakance was assumed to increase to the north of Kerrville, where the Pine Island is expected to pinch out.

5.3 MODEL CALIBRATION

Through the process of calibration, initial model parameters are refined so that the model can better represent historic conditions. This process ensures that the data used in the model do not represent geographically-isolated test results, but are applicable on a more regional scale.

More confidence in a model's ability to represent future conditions exists if its ability to represent historic conditions has been documented. The Kerrville model was calibrated to actual data collected during the ASR study period for both steady-state conditions (non-pumping) and transient (pumping or recharge) conditions.

5.3.1 Steady-State Calibration

Water levels for the city wells were collected by UGRA personnel during the winter of 1990-1991 so that water levels for a non-pumping period could be used for steady-state calibration. The best city-wide representation of water levels using the most wells was for the end of February 1991. A contour map is shown in Figure 5-2. It shows a 40- to 50-foot elevation difference across the City. Elevation 1500-ft msl approximates a median steady-state groundwater elevation.

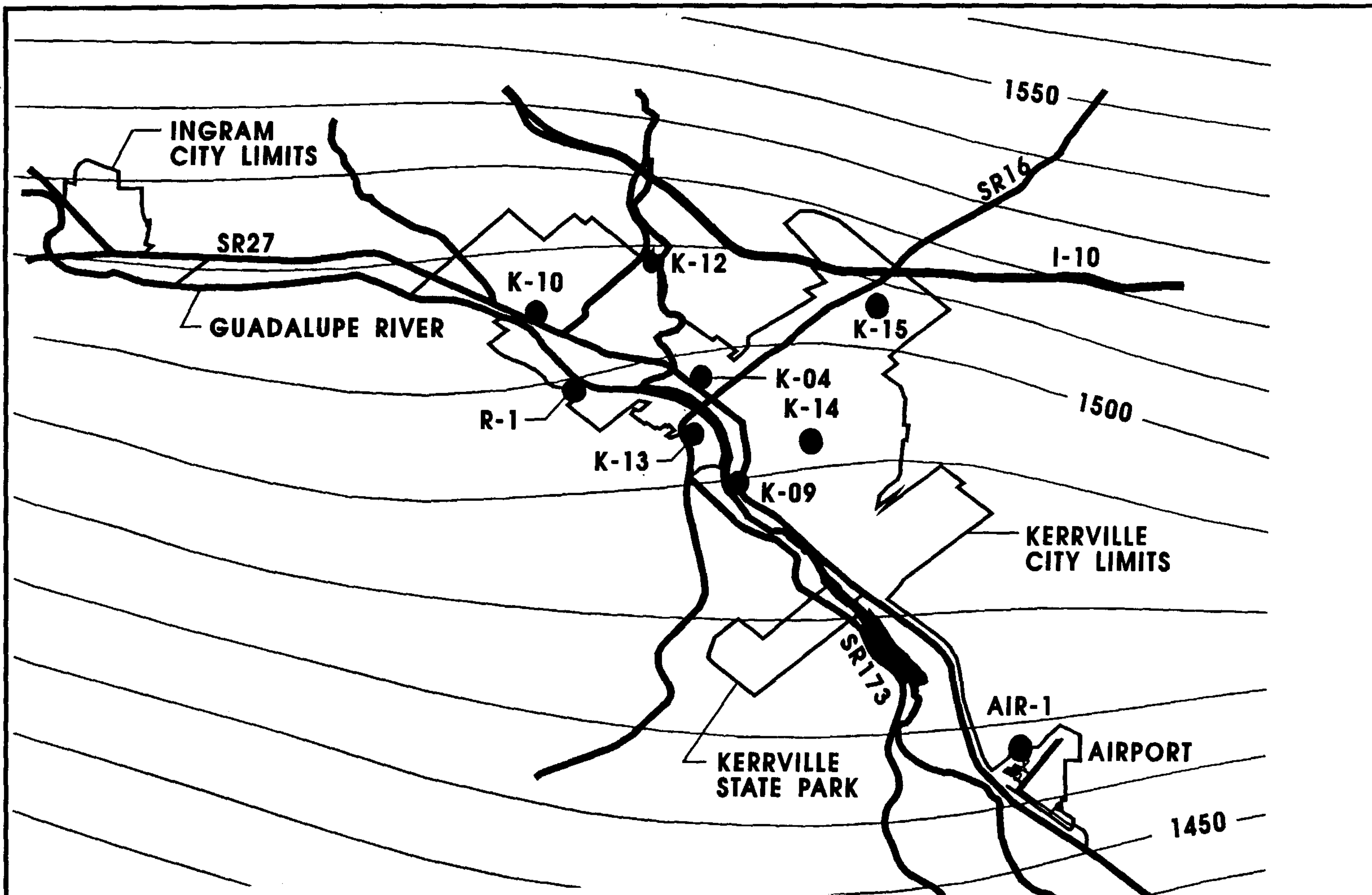


Figure 5-2
STEADY STATE FLOW SIMULATION

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Unfortunately, these water levels may not accurately represent steady-state conditions since other non-city wells may have been pumping during this period, and because the model indicates that the aquifer would likely take longer than just a few months to recover completely to steady state from peak summer pumping. A steady state elevation above 1500-ft msl is possible, but additional long-term water level data will be required to make this determination.

Fifty-two simulation runs were completed during the calibration. In each run, various parameters were adjusted until the best fit between model predicated levels and measured steady-state levels was attained. Parameters adjusted included starting water levels, transmissivity, and vertical leakance in both layers.

The comparison between actual February 1991 and model-calculated water levels in city wells is shown in Table 5-2. The resulting correlation coefficient (R^2) between the field data and the model-calculated data is 0.94, suggesting reasonable correlation between model results and field measurements. Although this may not represent true steady-state conditions, it may be considered a conservative estimate since steady-state water levels are likely to be higher.

Table 5-2
Steady-State Calibration

Well	Measured Water Level[a]	Model-Calculated Water Level	Difference (ft)
ASR Monitoring Well PZ-1	1502	1500	(2)
No. 4 - Plant Well	1506	1499	(7)
No. 9 - H Street Well	1488	1490	2
No. 10 - Lois St Well	1512	1506	(6)
No. 13 - Park Well	1493	1495	2
No. 14 - Travis St Well	1488	1493	5
No. 15 - Alpine Dr Well	1528	1508	(20)
Airport Well	1452	1464	14

[a]February 27, 1991.

5.3.2 *Transient Calibration*

Transient calibration was accomplished by running a series of model simulations with varying storage coefficients. In addition, to verify the steady-state calibration further, two of the best steady-state run setups were compared under the transient conditions.

The transient calibration was confirmed using data from each of the city wells collected during the Cycle 2 test. The model-generated and actual field data hydrographs showed good agreement.

Section 6
ASR EVALUATION

Section 6
ASR EVALUATION

In Section 2, supply and demand comparisons were made to identify deficiencies and needs in the water supply system for the City of Kerrville. This comparison indicated that during periods of normal growth and weather, sufficient surface and groundwater supplies exist to meet anticipated annual and monthly demands through the year 2015 and beyond. The analysis also indicated that during periods of historic drought, significant increases in groundwater supplies will be required. This section presents the results of model simulations used to evaluate the ability of surface water stored using the ASR concept to meet these projected shortfalls of water.

Model simulations were developed based on the assumption that the existing water supply system; i.e. the UGRA Water Treatment Plant (WTP) and the City of Kerrville's wellfield, will operate at its rated, installed, capacity (i.e. design capacity). The analysis indicates that these are reasonable conditions given that the system will require additional water rights for the plant, and given the relatively minor equipment rehabilitation that is likely to be required at the wellfield.

6.1 SIMULATION APPROACH

Simulation scenarios were developed to test the existing system's ability to deliver the necessary water supply during critical periods of drought. These scenarios provide answers to the following key questions:

- If area groundwater levels are maintained at the steady-state elevation of approximately 1,500-ft msl, will there be sufficient underground storage to meet projected needs?
- Can the existing "system" meet the projected 2015 and 2040 demands during the peak summer months and still have sufficient capacity to supply water during a repeat of the 1950's drought?
- Will model-predicted drawdowns extend into the Hosston-Sligo and/or exceed minimum pumping water levels in the existing well field?

Three scenarios were modelled and the predicted groundwater drawdowns were evaluated to determine if projected demand could be met. The simulations tested the following conditions:

- **2015 Drought - Without ASR.** Projected increasing water demands occur over the period January 1992 through December 2014. ASR is **not** used and groundwater levels are allowed to drop. A drought then occurs in the year 2015. What happens to water levels in the aquifer if ASR **is not** used?

- **2015 Drought - With ASR.** Projected increasing water demands occur over the period January 1992 through December 2014. ASR is used to maintain groundwater levels at 1,500-ft msl. A drought then occurs in 2015. What happens to water levels in the aquifer if ASR **is** used?
- **2040 Drought - With ASR.** Projected increasing water demands occur over the period January 1992 through December 2039. ASR is in place and groundwater levels are maintained at 1,500-ft msl, then a drought occurs in the year 2040. What happens to area groundwater levels?

All simulations were based on evaluating the ability of the existing system to meet the projected demands for water. The existing system includes the UGRA's 5-mgd WTP, plus the City of Kerrville's existing well field system operating at its design capacity. In addition, the system includes two ASR wells (Well R-1 and Kerrville Well No. 5), with a combined recharge capacity of 1,100 gpm (1.58 mgd). The combined capacity of the water supply system is estimated to be 14.73 mgd, comprised of 4.5 mgd of firm surface water treatment capacity, plus 9.88 mgd of well field capacity. A breakdown of system capacity is presented in Table 6-1.

Table 6-1
System Capacity Assignments

Supplies	Pumping Capacity, mgd	Recharge Capacity, mgd
Groundwater		
ASR Well (R-1)	1.15	0.79
No. 5 Plant Well	1.15	0.79
No. 7 Harper Street Well	1.22	--
No. 8 Lewis Street Well	1.30	--
No. 9 H Street Well	0.68	--
No. 10 Lois Street Well	1.07	--
No. 11 Meadowview Well	1.22	--
No. 12 Harper Road Well	0.72	--
No. 13 Park Well	0.65	--
No. 14 Travis Street Well	0.72	--
Well Field Subtotal	9.88	--
UGRA Water Treatment Plant	4.85	--
TOTAL	**14.38**	**1.58**

The projected demands varied depending on the conditions simulated. For normal conditions, the demands for the period between January 1992 and December 2014 were broken down into monthly demands using the monthly demand factors presented in Section 2. When surface water was inadequate to meet demand, groundwater was used to make up the deficit. The need for groundwater pumping was then distributed among the City well pumps using the capacities presented in Table 6-1.

The water available for recharge was calculated as the excess flow from the WTP available after system demands were met. Recharge was through Well R-1 and Well No. 5 and ranged from 0 to a maximum flow rate of 1.58 mgd (1,100 gpm).

During a drought, it was assumed that the total annual water demand would remain the same as that projected during normal weather periods. Groundwater pumping and recharge rates were based on the EH&A surface water model simulations for the critical low-river flow years of the 1950's. The EH&A simulations used 2015 and 2040 water demands combined with historic river flow records over the period 1945-1984. Available surface water supplies were based on maintaining a minimum river flow-through of 15 cfs.

To the extent possible, water demands were met using surface water. If insufficient flow was available from the river, groundwater was used to make up the difference. The EH&A model predicted monthly shortfalls that were assumed to be made up by groundwater. These computed shortfalls were adopted and used as monthly groundwater demands in the ASR model. A discussion of this model and the predicted demands is presented in Section 2.

6.2 EVALUATION CRITERIA FOR MODEL SIMULATIONS

The simulations provide an estimate of the water levels within the aquifer for all the areas or cells described in the model. These levels oscillate up and down depending on:

- The demands placed on the formation
- The year being simulated
- Whether drought or non-drought conditions are occurring
- The elevation of the initial steady-state water surface
- The unique physical properties of the aquifer in the region of a specific well

The model calculates these water levels with little regard for the physical world. For example, the model may report water levels being drawn down below the setting of a well pump--clearly a condition that is physically impossible. Therefore, we compared model results to a reference or critical elevation in order to be sure this type of "phantom pumping" was not distorting the outcome.

Table 6-2 presents a summary of the critical elevations used in evaluating model results. Because of the natural variation in the subsurface and hydrogeologic gradient across the City of Kerrville, these critical water-surface and top-of-formation elevations vary widely. For this model, there are two critical check points: top-of-formation and minimum pumping water level. These levels were selected because they will

Table 6-2
Critical Well Field Elevations[a]

Well	Approximate Elevations, ft msl: Surface	Top of Formation	Bottom of Casing	Existing Pump Setting[a]	Minimum Pumping Water Level[b]	Steady State Water Level	Available Drawdown, ft
ASR Well R-1	1638	1053	1143	1158	1183	1500	317
No. 5 - Plant Well	1656	N/A	1186	1206	1231	1505	274
No. 7 - Harper Street	1640	1145	1110	1240	1265	1520	255
No. 8 - Lewis Street	1633	N/A	1193	1283	1308	1512	205
No. 9 - H Street	1609	1135	1109	1179	1204	1488	284
No. 10 - Lois Street	1675	N/A	1162	1175	1200	1510	310
No. 11 - Meadowview	1600	N/A	1072	1100	1125	1480	355
No. 12 - Harper Road	1690	1145	1150	1150	1175	1560	385
No. 13 - Park Well	1621	1135	1089	1121	1146	1490	344
No. 14 - Travis Street	1683	1155	1078	1133	1158	1488	330
Average Value	1645	1135	1130	1170	1200	1505	306
Range							
Minimum	1581	1160	1186	1283	1146	1560	205
Maximum	1701	1053	1072	1100	1308	1480	385

[a]All elevations based on existing well logs except R-1 data and steady state water level which were measured.
[b]PWL (Pumping Water Level) = Pump Setting +25 ft.

indicate when the aquifer is being dewatered or when pumps are being exposed. For example, the table shows a minimum pumping water level in Well R-1 to be 1,183-ft msl; therefore, the model-predicted water surface elevation must be greater than elevation 1,183-ft msl to prevent exposing the pump. Similar comparisons can be made to determine if the top of formation is being exposed. However, because the minimum pumping water level elevations are higher than the top of the aquifer, it became the governing criteria in evaluating model results.

The critical pumping water levels presented in Table 6-2 are estimates based upon pump setting information in well logs; they have not been field verified. It is believed that these estimates are acceptable for this level of study, but confirmation based on actual field measurements on individual wells will be required.

6.3 SIMULATION RESULTS

Table 6-3 presents the model-predicted minimum groundwater levels for the three simulation scenarios. These water levels are compared to the critical levels presented in Table 6-2 to determine if the aquifer can realistically meet these demands during sustained periods of high groundwater pumpage. A discussion of these results are presented below.

Table 6-3
Groundwater Levels During Drought Conditions

City Well Designation	Minimum Pumping Water Level[a], msl	Minimum Water Level Reached, ft msl[b]		
		Under 2015 Demands		Under 2040 Demands with ASR
		Without ASR	With ASR	
ASR Well R-1	1183	1170	1264	1149
No. 5 - Plant Well	1231	1196	1267	1173
No. 7 - Harper Street	1265	1217	1274	1174
No. 8 - Lewis Street	1308	1189	1248	1168
No. 9 - H St Well	1204	1207	1255	1195
No. 10 - Lois St Well	1200	1231	1281	1215
No. 11 - Meadow View	1125	1216	1258	1207
No. 12 - Harper Road	1175	1235	1283	1219
No. 13 - Park Well	1146	1200	1254	1182
No. 14 - Travis Well	1158	1210	1256	1194

[a]Minimum Pumping Water Level. Pump Setting +25-ft (see Table 6-2 for development).
[b]Minimum water level is the model-predicted water level minus 50 feet to account for losses between the formation and the well.

6.3.1 2015 Drought - With and Without ASR

Two drought simulations were run for 2015 demands: one with ASR, the other without. Both meet the projected 2015 demand of 5,550 ac-ft/yr with a combination of surface water and groundwater supplies. River water supplies were assumed to be at their historic lows, similar to those recorded during the period April 1954 through December 1957.

Table 6-3 presents a comparison between the minimum required pumping level at each well (developed in Table 6-2) and the level the model predicts that groundwater will fill to. The results show that **without ASR**, water levels will fall below the minimum acceptable depth in four wells (Wells Nos. R-1, 5, 7, and 8). Thus, these wells cannot meet projected demands. However **with ASR**, all model predicted water levels, with the exception of Well No. 8, are above minimum pumping level requirements, indicating that the wells could pump water to meet projected demands. Well No. 8 is completed across the Hensall sand and Hosston-Sligo, and water levels are difficult to accurately predict. However, a preliminary review of well completion logs suggest this well pump can be lowered to meet the 1,248-ft msl pumping water level requirement.

Although ASR can maintain groundwater levels that are adequate to allow pumping, water levels during a drought will drop dramatically. Model results indicate for a 45-month drought like that of the 1950's, average groundwater levels would drop an average of 236 feet from 1,500-ft msl to an average minimum of 1,264-ft msl (Figure 6-1). Without ASR, average groundwater levels are predicted to drop an average of 293 feet to an average level of 1,207-ft msl.

6.3.2 2040 Drought - With ASR

This simulation was run to test if an ASR system could provide adequate groundwater supplies during a historic drought that occurs in the year 2040. Again, groundwater demands predicted from the EH&A model simulations were used as the basis for estimating periods and rates for groundwater pumping and recharge. Initial groundwater levels were assumed to be the February 1991 steady-state levels.

The 2040 simulation was similar to the 2015-with-ASR run, but the water demands were higher and the duration of the drought longer. In a 2040 drought, the duration is projected to last 54 months, and surface water supplies were assumed to be like those recorded during the period March 1953 through December 1957. Annual water demands for the drought period are projected to be 5,850 ac-ft/yr.

Table 6-3 shows the levels to which groundwater is predicted to fall during a 2040 drought. Half of the wells (Wells Nos. R-1, 5, 7, 8, and 9) are predicted to go dry, indicating that the existing well system cannot reliably provide the 2040 projected water demands during critical drought conditions. The table shows that on average,

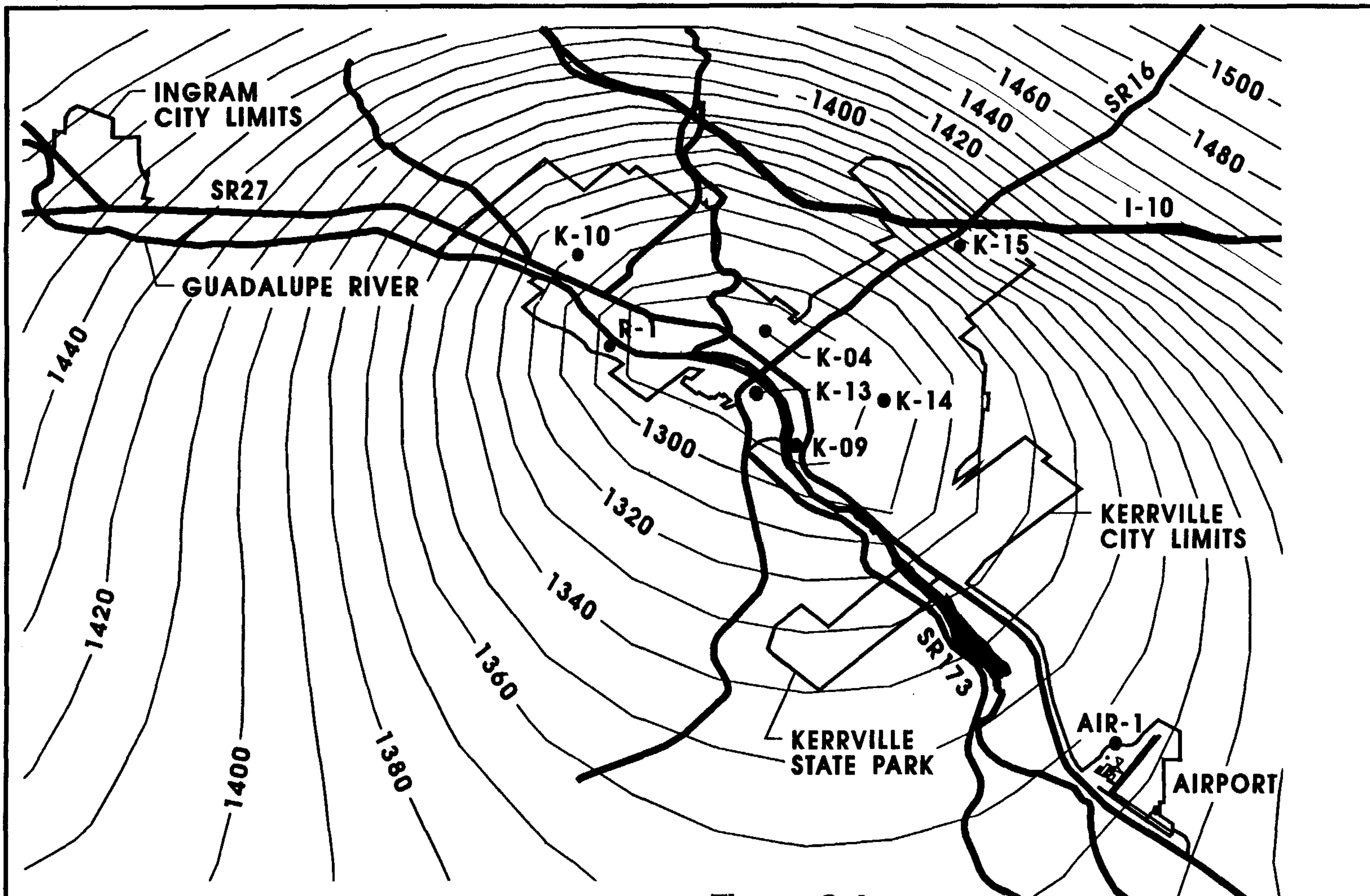

Figure 6-1
WATER LEVELS DURING MAXIMUM DRAWDOWN OF 2015 DROUGHT WITH 5 MGD PLANT

water levels are expected to drop by over 312 feet from the average 1,500-ft msl level to elevation 1,188-ft msl.

6.4 CONCLUSIONS

Based upon these model simulations, the following conclusions have been drawn:

- Under normal or non-drought conditions over the period 1992 through 2014, groundwater levels in the area are predicted to progressively decline. If a drought occurs in 2015, the non-ASR system **will not** be able to meet projected demands.

- The existing 5-mgd surface water treatment plant with two ASR wells and the City's existing well field operating at design capacity **will** be able to meet projected 2015 demands during a repeat of the 1950's drought, if water levels are maintained at 1,500-ft msl and Well No. 8 lowered.

- The existing 5-mgd surface water treatment plant with two ASR wells and the City existing well field **will not** be able to meet projected 2040 demands during a repeat of the 1950's drought if water levels are maintained at 1,500-ft msl.

Section 7
PROPOSED WATER SUPPLY PLAN

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This section presents the recommended action and policies required to implement an ASR-based water supply program to meet the anticipated water demands through the year 2015. Previous sections have demonstrated that these demands can be met with a combination of available surface water and groundwater supplies, but improvements to the existing water supply system will be required. These include acquisition of additional surface water rights, improvements to the well field system, and institutional changes to ensure an adequate and reliable supply. A description of these improvements, as well as an order-of-magnitude cost estimate to implement critical portions of the well field rehabilitation program are presented below.

7.1 GOALS AND POLICIES

The 2015 water plan for the City of Kerrville was developed to achieve three basic goals:

- It must provide adequate water supplies for the City of Kerrville through the year 2015, including during periods of historic drought.
- It must provide a system that is reliable.
- It must contain elements of flexibility to account for changing demographics in the Kerrville area.

Presented below is a summary of the goals and action items required.

Goals	Action
• Provide adequate water supplies for growth and drought protection through 2015.	• Implement an ASR-based Water Management Program to monitor and maintain groundwater levels at approximately elevation 1500-ft msl.
• Improve well system reliability.	• Implement a Well Field Rehabilitation Program over the next 5 years to develop system capacity to reliably meet the 2015 maximum-month demand of 7.58 mgd and a peak-day demand of 10.1 mgd.
• Ensure flexibility in the Plan.	• Periodically verify demand projections by reviewing and revising Plan at census period or every 10 years.

7.2 2015 WATER SUPPLY PLAN

The water supply system used in model simulations is based on the existing City of Kerrville well field and the UGRA's surface water treatment plant operating at design capacity. The model results suggest that no additional wells be drilled or no additional treatment plant capacity constructed, but improvements to the system's reliability and production capacity will be required. In particular, the City's well field must be capable of operating at a maximum month design capacity of approximately 7.58 mgd and a peak-day capacity of 10.1 mgd. The permitting and institutional arrangements required are key elements of this plan.

A primary need is to obtain additional water rights from the TWC so that up to 5,600 ac-ft/yr may be diverted from the Guadalupe River. Additional information will be required on the condition of the City of Kerrville well field to accurately establish rehabilitation needs. As such, an important element of the 2015 Water Supply Plan is to conduct a needs assessment survey of the City of Kerrville well field and water distribution system.

The 2015 Water Supply Plan is comprised of two major programs: The Groundwater Management Program and the Well Field Rehabilitation Program. The Groundwater Management Program will provide the framework for operation of the ASR system and for groundwater monitoring in the Kerrville area. The Well Field Rehabilitation Program will evaluate the existing capacity, identify upgrade needs, and systematically restore well field production to design levels.

7.2.1 Groundwater Management Program

The Groundwater Management Program consists of the following four components:

- **Acquire additional water rights.** The UGRA's existing diversion limit from the Guadalupe River limits the surface water treatment plant production capacity to 3,603 ac-ft/yr (an average of 3.2 mgd). A permitted diversion of 5,600 ac-ft/yr is recommended to allow the plant to operate at its design capacity of 5 mgd.

- **Obtain TWC permits for ASR operation.** Routine operation of Well R-1 and No. 5 as ASR wells will require a Class V permit from the TWC. Well R-1 is currently permitted for testing operations only. An application to operate Well R-1 and Well No. 5 was submitted to the TWC in September 1991. The status of this permit application should be monitored until the permit is issued.

- **Establish Underground Water District to monitor aquifer.** Protection of stored groundwater is critical to ensuring adequate supplies will be available during times of need. The City of Kerrville has enacted ordinances to regulate well drilling within the City limits, but because stored groundwater will extend outside City limits, additional county-wide authority is recommended. On November 5, 1991, voters in Kerr County voted to establish the Headwaters Underground Water District to monitor and protect groundwaters in Kerr County.

- **Implement Aquifer Management Plan to maintain water levels.** Groundwater levels in the Hosston-Sligo must be periodically monitored to ensure they are maintained high enough to provide adequate of storage. It is recommended that groundwater levels in the area be maintained at an approximate elevation of 1500-ft msl. This target elevation should be reevaluated to assess whether a higher target elevation would be beneficial and cost-effective as an alternative to meet drought demands beyond 2015.

  This task could be performed by several agencies--the UGRA, the Headwaters Underground Water District, or the City of Kerrville. The need is to develop a systematic approach to well data collection and a mechanism to implement aquifer recharge. Although data could be collected by all agencies, it is recommended that a single agency be responsible for recharge operations to reduce duplication of effort and minimize operation costs.

  Implementation of the aquifer management plan will require several cycles of operational data to calibrate aquifer response. Typically, aquifer levels are lowest in September and because of natural recharge, will gradually return to steady-state levels during the low-demand months of October through April. How fast the aquifer responds is not precisely known. This study demonstrated that if groundwater levels do not return to steady-state conditions, drought protection may be sacrificed. Therefore, during low-demand periods when excess surface water is available, ASR wells would augment natural recharge such that by the following summer, groundwater levels will again be at their steady-state conditions.

  The amount of ASR water to be injected will vary from year to year, depending on the previous year's groundwater pumping rates. It is recommended that an Aquifer Management Plan be developed to determine the volume, and establish protocols for ASR well operation, monitoring, and periodic review of monitoring data. The ASR groundwater model developed in this study could be modified to assist in developing the ASR operations program.

7.2.2 Well Field Rehabilitation Program

This program is design to systematically restore the existing City of Kerrville well field system to its design condition. It consists of the following three components:

- **Needs Assessment.** Determine and describe the repairs, replacements, etc., required to restore the City of Kerrville's existing system. This assessment includes physical inspection of each well in the system, evaluation of distribution system deficiencies, evaluate the ability and depth pumps can be lowered to, and development of specific repair requirements for each. From this assessment, repair budgets and schedules can be developed. It is recommended that this survey be completed by the end of 1993.

- **Complete Well No. 5 Conversion.** The City of Kerrville is in the process of rehabilitating Well No. 5 to a functional ASR well. The well is a vertical-turbine type and recharge will be through the pump column, a process different from the injection tubes used in ASR Well R-1. Operation, testing, and training will be required to establish recharge and recovery capacities at this well. Well completion is scheduled for 1992.

- **Complete Well Field Rehabilitation.** This step implements the needs assessment outlined above. The goal is to return the existing well field to design condition. It is recommended that this program be implemented as soon as possible. Although the existing well system can meet the needs for the immediate future, postponing implementation of the rehabilitation program increases exposure to the risks of drought or failure of the surface water treatment plant. It is recommended that well field rehabilitation be completed no later than the year 2000.

7.3 IMPLEMENTATION COSTS

The costs for the 2015 Water Plan include annual operating costs to manage groundwater surpluses plus the capital costs required to develop a functioning system. At this level of study, operating costs cannot be accurately defined because of uncertainties as to how much water is to be stored and recovered annually, institutional monitoring costs, and other factors beyond the scope of this study.

Capital costs for the 2015 Water Plan primarily involve rehabilitation of the existing well field system. Assuming a "worst-case" scenario where eight City wells (all except Well R-1 which is new, and Well No. 5 which is assumed to be at design capacity) require complete rehabilitation, including replacement of the well pump, pump column, well head, and associated valves and piping, electrical equipment, and chlorination equipment. An order-of-magnitude cost estimate for this type of program, in 1992 dollars, is approximately $1 million. No estimate has been prepared for improvements to the distribution system.

Section 8
REFERENCES

Section 8
REFERENCES

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