Peter F. Ffolliott

VOLUME 13

HYDROLOGY and WATER RESOURCES in ARIZONA and the SOUTHWEST

PROCEEDINGS OF THE 1983 MEETINGS
OF THE
ARIZONA SECTION —
AMERICAN WATER RESOURCES ASSN.
AND THE
HYDROLOGY SECTION —
ARIZONA - NEVADA ACADEMY OF SCIENCE

APRIL 16, 1983, FLAGSTAFF, ARIZONA

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PREFACE

Every community is forced to confront issues regarding water. The issues change from region to region, yet a simple repetitive note is sounded -- water is important for existence. Whether the issue is too much, not enough, or not enough of the right kind for the right price, water is important. The current articles presented at the 1983 meetings of the AWRA - AAAS in Flagstaff, Arizona present a wide variety of water issues. Important for their ability to use rainfall measurements to predict streamflow characteristics are the articles by Solomon et al. and by Simanton and Osborn. In addition, the articles by Ffolliott and by Baker illustrate the important linkage between land-use utilization and water production.

Water resources planning requires models. If the objective is to optimize wild-land production, the article by Lane and Stone is appropriate. A state-wide approach for the allocation of imported surface water is developed in the article by Buras. And finally, an important determinant of water needs, residential water demand, is presented by Woodard and Rasmussen.

Sophisticated chemical techniques are being used to identify groundwater movement. The use of stable isotopes is an important field of research, allowing investigators the ability to trace regional groundwater movement. The articles by Mohrbacher and by Kaufman et al. present new additions to the family of groundwater tracing techniques. Once a system is defined, evaluation of the data requires models such as the finite element model presented by Contractor. The use of such models extends the understanding by allowing the evaluation of management scenarios which are difficult to evaluate in the field. In addition, the article by Postillion and Esposito indicates the improved management which results when careful analysis of the hydrologic data is performed.

Finally, the articles on the presence of virus in sewage effluent creates doubts about the long-term health effects associated with using the effluent in any area where humans may congregate. In particular, the use of effluent on golf courses and public parks may result in compromising the health of the general population by introducing a vector for disease-causing viruses to infect humans. While pumping drinking water for use on golf courses may appear to be a greater risk to local economic development, the added risk to the regional health is of vital importance and, needless to say, bears further examination.

A new feature appearing in this volume is an index of authors for the thirteen volumes of the proceedings of "Hydrology and Water Resources in Arizona and the Southwest" plus the four AWRA Arizona Section symposiums. The author index was compiled to complement the subject index published as a part of the previous volume in this series. It is hoped that the two indices will provide a useful source of information to students and researchers who wish to access quickly the large amount of information which has been presented under the auspices of the Arizona Section of the AWRA.

Todd C. Rasmussen
July 1983

Instructions to Authors

Beginning with Volume 11, instructions have been modified to accommodate the use of word processors. While the use of a word processor is preferred, a typewriter is acceptable. Each submitted manuscript must be camera-ready. Use papers in Volume 11 or later volumes as typing models.

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Smith, Robert and Arthur Green. 1982. Conflicting theories on the reproduction rate of Felis domesticus during the Great Flood. In: Proceedings of the Society of Arcane Studies. R. Jones (Ed.). San Francisco: Obscure Publications, Inc. 5(14):127-136.

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The deadline for manuscript submission is the final day of the annual meetings of the AWRA. At the meeting, manuscripts may be submitted to any of the AWRA Officers. However, it is prefered that manuscripts be mailed directly to the Editor of the Proceedings at the address given on the Preface page of this volume. In past years, deadline extensions have been liberally granted for a variety of reasons, sometimes delaying publication for several months. Frequently, long delays result from the government agency peer review process and improperly prepared manuscripts. Therefore, the editors request that all authors take special care in following the instructions for manuscript preparation and do everything in their power to meet the meeting date deadline.

IMPORTANCE OF SHORT DURATION RAINFALL INTENSITIES FOR HYDROGRAPH MODELING

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Abstract

Flood flows and water quality in the Southwest are most dramatically influenced by short, intense rainstorms. Runoff from these storms has been modeled with some success. One key element that has often been overlooked, however, is the importance of intra-storm rainfall distribution on runoff response. Actual storms were modeled for small experimental watersheds in the Southwest using different time increments of intra-storm rainfall. Increments of 5 minutes or less proved satisfactory for accurate hydrograph simulation. As increments became longer than 5 minutes, the ability to simulate actual hydrographs became increasingly difficult. Increments of 30 minutes or longer proved unacceptable for most storms. Hydrologic models must be sensitive to short time increments of intra-storm rainfall to accurately predict peak flows in the Southwest. Watershed treatments will be more cost-effective if their design considers intense bursts of intra-storm rainfall in addition to total storm volume.

Introduction

Over the last decade, tremendous strides have been made in computer hardware and software. These advances have made computer systems accessible to natural resource scientists. Many new rainfall-runoff models have been developed, tested and their results published. Many are process-based models that solve complex or tedious differential equations with numerical approximations or other techniques which would require unreasonable amounts of time if done by hand. These process-based models have promoted a clearer understanding of the hydrologic cycle in general, and the factors controlling the rainfall-runoff process in particular.

Extensive work has been done in the Southwest to quantify rainfall-runoff relationships. Much of this research has studied the depth-area and energy-erosion relationships of thunderstorms (Osborn et al., 1980). Investigators have understood and accepted the importance that time plays in the runoff process. Unfortunately, the importance of time in hydrograph synthesis has not been passed on to field engineers and hydrologists that make planning decisions and design flood control improvements. This paper points out the importance of time in hydrograph synthesis, and illustrates the size of error that can be expected by using hourly or even 30-minute rainfall data to calculate peak runoff in areas dominated by intense rainstorms.

The Problem

Engineers and hydrologists are often given the task of predicting flood peaks for small basins. Since local stream records are usually inadequate or nonexistent, runoff models must often be used to make such predictions. With computer technology now available, the Rational, SCS, and other simplistic models are losing favor in preference for more sophisticated process-based models. However, these new models often produce no more accurate results than the traditional methods (Naef, 1981). They require more sophisticated data, much of which are estimated using professional judgment. These estimated data often produce inaccurate results. The models can correctly depict rainfall-runoff processes, but the required data are often insufficient to predict peak flows that agree with observed values.

Almost all rainfall-runoff models require some input of rainfall, the area of the watershed, and some abstraction of infiltration. Even the simple Rational formula contains these factors:

$$q = CIa$$

where q = peak flow

C = runoff coefficient based upon flood-producing characteristics of the basin (abstraction of infiltration and delivery).

I = rainfall intensity, customarily averaged over the time of concentration.

a = area of the basin.

The relationship between I and a is one aspect of accurate hydrograph synthesis, and has received adequate attention (Osborn et al., 1980). The infiltration-rainfall relationship has also been dealt with in part by numerous investigators. Attention has focused primarily on adjustments to the infiltration index, for example, the C factor in the Rational formula or the curve number in the SCS model (Bondelid et al., 1982). The importance of I as a function of time within storms has not been adequately addressed.

An analysis of model performance reveals the importance of intra-storm rainfall distribution. The performance of the Rational formula and the SCS model was tested for over 50 storms (Hiemstra and Reich, 1967). The SCS model substantially underpredicted, while the Rational formula overpredicted peak flows. Hiemstra and Reich (1967) pointed out that the Rational formula can account for rather small time increments of rainfall while the SCS model is limited to longer increments. The original SCS model was not designed to account for intra-storm rainfall distributions and computes infiltration eratically for 5 minute increments (Kumar and Jain, 1982). When Hiemstra and Reich (1967) accounted for intra-storm rainfall distribution, the SCS model performance improved.

Recent studies have recognized the shortcoming of the SCS model and other simple models to accurately predict runoff from short, intense rainstorms (Kumar and Jain, 1982; Aron et al., 1977). Many computer based models can account for short time increments of rainfall, although the importance of corresponding rainfall data is often understated. This problem results in part from data being published by clock hour, with one hour as the shortest time increment (Frederick et al., 1981).

Importance of Short Time Increments

Surface runoff is the result of rainfall excess. Two factors influence excess water, the soil's infiltration capacity and the rate of rainfall. Given a set of initial conditions, the soil infiltration curve can be defined (i.e., the soil will infiltrate water at predictable rates given initial moisture conditions and soil properties). The rainfall rate determines the amount of excess surface water in two principal ways: (1) it can shift the location of the infiltration decay curve because it affects the time to ponding (Smith and Cherry, 1973); and (2) it affects the separation of rainfall excess from infiltrated water (i.e., the higher the rate of rainfall the larger the excess). Rainfall rates averaged over longer time intervals are lower and produce less excess water.

The integrated effect of these two principles is demonstrated in Figure 1. A short, intense (5.0 in/hr) rainfall is shown for a time increment of five minutes. If this same volume were spread across 10 minutes the rainfall rate would be 2.5 in/hr. The excess water produced from these two rates is shown by blocks A and B, respectively. The 10 minute increment underestimates actual excess and resultant runoff volume. This is precisely the problem encountered when using models that have coarse time-increment requirements or use rainfall data published with one-hour or 30-minute increments.

Study Methods

The importance of short time increments of rainfall has been demonstrated conceptually (Engman and Hershfield, 1981). But this concept has not been displayed for actual watershed cases. To better explore the implications of short rainfall time increments, 14 storms on three experimental watersheds were modeled. The three watersheds are located near Albuquerque, New Mexico and range in area from 40 acres to 246 acres (Hickok et al., 1959). The 14 storms ranged in duration from 19 minutes to over three hours. The rainfall data were collected by recording rain gages, and the streamflow data were collected by recording gages on precalibrated triangular weirs. The period of record was 1948 through 1967.

A simple computer model that simulates infiltration and runoff processes was used to show the effects of varying time increment. The model and its performance have been described by Solomon et al., (1982) and Solomon (1983). Regardless of which process-based model is used, the results should be comparable.

Each of the 14 storms was partitioned into one minute time increments and modeled. Adjustments were made to infiltration coefficients until observed hydrographs matched predicted hydrographs. The time increment was then increased from one to two to five minutes, etc. the longest time increment used was one hour.

Of the 14 storms modeled, only two had one-minute partitioning as part of the original reported data. Six storms were reported with two-minute increments. All storms were reported with at least

five-minute increments. The assumption was made that rainfall values in the original data were given for the smallest increment that showed differences in rainfall intensity (i.e., if the rate of rainfall was constant for five minutes, the data were reported for a five-minute increment).

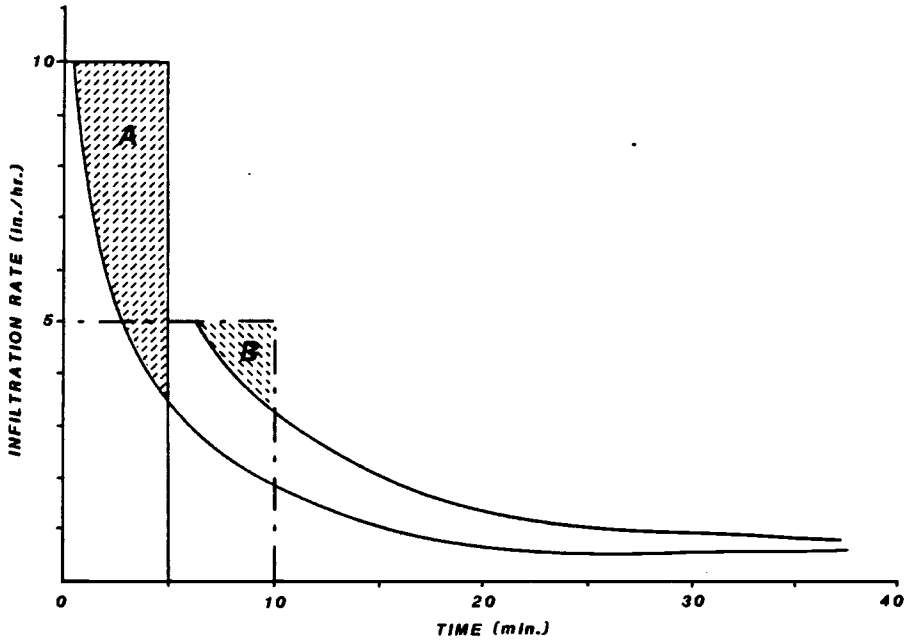

Figure 1. Precipitation Excess for Two Rainfall Intensities and Durations.

Results

These 14 summer storms serve as a base for evaluating the effects of intra-storm rainfall distribution on hydrograph syntnesis. Table 1 shows the effect of increasing the length of intra-storm time increments on predicted peak flow. Peak flows using one-minute increments match observed values. As increments are lengthened, predicted peak flows drop dramatically and are substantially underpredicted.

Figure 2 illustrates these effects graphically for intra-storm time increments of one minute to one hour. The use of time increments up to five minutes produces predicted peak flows that average 86% of the observed peaks. As time increments are expanded to 10 and 15 minutes, ability to model peak flows diminishes dramatically. Beyond 30 minutes, differences in peak flow are not as abrupt but errors are substantial. Note in Figure 2 the range in peak flows for any given intra-storm time increment. This range is due to the extremely variable rainfall distributions for the storms evaluated. Some storms were dominated by one short, intense burst of rainfall less than 5 minutes long while other storms were characterized by a 10 to 15 minute dominant period. No storm had a maximum rainfall intensity longer than 15 minutes. Other hydrograph characteristics, such as time to peak and runoff duration, did not change substantially with changes in length of time increments.

Changes in hydrograph characteristics are shown in Figure 3. The storm of August 12, 1967 was modeled for various intra-storm time increments. The 1 minute, 5 minute and 30 minute intra-storm time increments are shown. For this storm, the 30 minute time increment produced a peak less than one-half the actual peak and a total runoff volume at about 60 percent of the actual runoff volume.

Implications

For rainfall-runoff models to accurately predict peak flows in the Southwest, they must be sensitive to short time increments of rainfall. Engineering designs should evaluate short time increments of intra-storm rainfall and not rely on prediction tools that use one hour or longer time periods of precipitation accounting. Rainfall for intra-storm time increments might be estimated by establishing a standard distribution for the one hour rainstorm (Farmer and Fletcher, 1972; Engman and Hershfield, 1981; Frederick et al., 1981) or by using five minute or 10 minute rainfall probabilities (Kangieser, 1959).

Table 1. Peak Flows From Hydrograph Synthesis Using Varying Intra-Storm Time Increments.

Watershed	Event	Total Rainfall (in)	Highest Intensity (in/hr)	1 min 2/	2 min	3 min	4 min	5 min	10 min	15 min	20 min	30 min	60 min
				Peak Flows (cfs)									
47.001	Sept. 5, 1963	.45	1.9	24.2	-	-	-	22.9	19.7	16.4	-	12.4	2.8
47.001	Aug. 3, 1964	.97	2.2	51.2	50.9	-	-	46.7	43.6	42.3	43.5	32.8	27.5
47.001	July 31, 1965	.65	5.9	125.1	-	-	123.4	112.0	77.2	65.0	-	35.1	12.3
47.001	Sept. 12, 1965	.75	2.0	48.5	-	-	98.5	46.8	36.9	30.7	32.6	24.2	20.3
47.001	Jun. 10, 1966	1.0	10.2	223.0	213.9	-	198.0	170.9	158.3	134.5	-	64.0	38.4
47.001	Aug. 11, 1967	1.3	3.4	151.5	151.3	-	143.0	125.3	117.1	90.8	94.6	71.4	64.4
47.002	Aug. 4, 1948 1/	.49	2.4	26.1	-	24.9	-	22.5	15.1	12.2	-	6.2	0.0
47.002	Aug. 4, 1948 1/	.41	2.8	40.5	40.0	-	35.3	36.1	21.3	21.0	-	8.9	3.2
47.002	Aug. 3, 1964	.90	3.0	42.1	41.7	-	-	39.2	32.2	25.3	27.3	10.3	8.5
47.002	Aug. 14, 1965	.69	3.5	52.1	51.8	-	37.0	43.5	29.4	28.3	-	16.5	6.5
47.002	Sept. 2, 1965	.62	2.2	41.7	-	40.7	-	35.4	27.6	24.2	24.2	17.3	5.1
47.002	Jun. 10, 1966	1.20	12.0	112.0	107.0	-	-	101.4	74.6	79.7	-	44.4	26.1
47.002	Aug. 13, 1967	1.2	5.7	91.2	-	-	85.4	81.3	66.7	63.2	52.1	43.0	28.0
47.003	Aug. 4, 1948	.52	3.3	27.9	27.3	-	-	26.7	20.2	16.9	11.5	3.2	0.0

1/ There were two storms within the same day.

2/ 1-min predicted peaks approximately match observed peaks.

Peak flows depend more on short, intense bursts of intra-storm rainfall than on total storm volume. Small volumes of rain falling over a short enough time period can produce significant runoff. Time is the lever that land managers can use to control surface runoff. Introducing short delays in the runoff process reduces the effective rainfall intensity. In the short term, the manager can increase surface detention storage through structural treatments such as contour furrowing and ripping. Properly designed contour treatments can usually accommodate the small runoff volumes involved. In the long term, the manager can increase ground cover of plants and litter to increase infiltration as well as detention storage. These implications suggest that land management agencies need to measure rainstorms with greater resolution and design standard rainfall distributions that reflect local intra-storm characteristics.

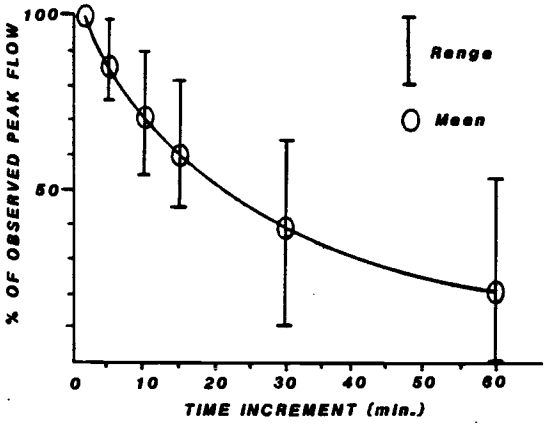

Figure 2. Changes in Predicted Peak Flow Resulting from Changes in Length of Intra-Storm Time Increments.

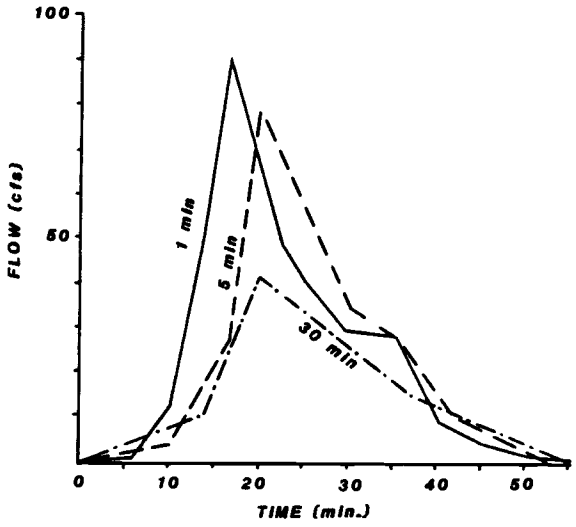

Figure 3. Comparison of Hydrographs Resulting from Changes in Length of Intra-Storm Time Increments.

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Stock Tank Characteristics and Performance in the Beaver Creek Watershed, North-Central Arizona

Julia Hughbanks
Northern Arizona University
Flagstaff, Arizona

ABSTRACT

The Beaver Creek Watershed is located in southern Coconino and southeastern Yavapai Counties, Arizona. This 472 square mile watershed contributes to Beaver Creek which flows into the Verde River. The objectives of this study were: 1) to monitor a network of stock tanks in the watershed on a bi-weekly basis to document seasonal fluctuations in water levels over one year; 2) to investigate the impact of stock tanks on local hydrology by determining the number of times the tanks fill during one year; and 3) to determine a set of dependent variables which would quantify stock tank performance, and a set of independent variables representing characteristics of the tanks and their drainage basins which could affect the performance of the tanks. The relative importance of each of these independent variables in influencing tank performance was then determined through statistical analysis.

RUNOFF ESTIMATES FOR THUNDERSTORM RAINFALL ON SMALL RANGELAND WATERSHEDS

J. R. Simanton and H. B. Osborn

USDA-ARS, Southwest Rangeland Watershed Research Center, Tucson AZ 85705

ABSTRACT

Almost all runoff from small rangeland watersheds in the Southwest is the result of intense thunderstorm rainfall, and the variability of this rainfall is an important runoff-influencing factor in such areas where high intensity rainfall dominates watershed hydrology. Thunderstorm runoff estimates for small rangeland watersheds can be made using a multitude of estimating techniques ranging from simple table and graph procedures to utilizing high-speed computers, and even the most sophisticated models greatly simplify the rainfall input. In this paper, the combined effects of rainfall quantity and intensity, and the rainfall energy factor, EI, in the Universal Soil Loss Equation (USLE), were analyzed, and simple procedures for estimating semiarid rangeland runoff volumes were developed. Equally good correlations with runoff volumes were found for EI, and for total storm rainfall times maximum rainfall intensities for 5, 10, and 30 minutes and the square of the maximum 60-minute rainfall.

INTRODUCTION

Almost all runoff from very small (less than 1 mi^2) rangeland watersheds in the Southwest is the result of intense thunderstorm rainfall. Thunderstorm runoff estimates for small rangeland watersheds can be made using various estimating techniques ranging from simple table and graph procedures to utilizing high-speed computers. Most of these models greatly simplify the actual rainfall input to the system. For example, the curve number method, developed by the Soil Conservation Service (Kent, 1973), incorporates rainfall volume, along with certain watershed characteristics such as soil cover and antecedent moisture conditions, to estimate runoff volumes. The method does not allow for varying rainfall intensity, although the effect of intensity is known to be imporatant (Simanton et al., 1973; Hawkins, 1978). Linear regression is another rather simple procedure for runoff estimation. Schreiber and Kincaid (1967) and Osborn and Lane (1969) applied multiple linear regression analyses to semiarid rangeland watershed hydrologic data and found that 70% of the variance in predicted runoff was attributed to differences in rainfall amounts. They also found that, because of the interrelation between rainfall quantity and intensity, there was no significant improvement in the prediction when both parameters were included in a stepwise linear regression.

In this paper, the combined effects of rainfall amounts and intensities, and their relationships to runoff, are analyzed, and simple procedures for estimating semiarid rangeland runoff volumes are discussed.

Study Area Description

Data used in this study were collected from the Walnut Gulch Experimental Watershed located in southeastern Arizona (Fig. 1). There are 95 recording raingages and numerous instrumented subwatersheds within the 58-mi^2 experimental watershed. The semiarid climate of the area is characterized by a bimodal precipitation pattern, with approximately 70% of the 12-inch average annual rainfall occurring during the summer months of July, August, and September. Almost all watershed runoff occurs during this summer rainy season, and is a result of short-lived, high-intensity airmass thunderstorms. The sub-watersheds studied (63.101, 63.104, 63.105, 63.201, and 63.214), ranging in size from 0.45 acre to 372 acres, have similar physical characteristics, and are typical of the many thousands of acres of semiarid rangeland with mixed grass and brush cover in southeastern Arizona, southwestern New Mexico, and northern Sonora, Mexico. Creosote bush (Larrea divaricata), white-thorn (Acacia constricta), and tarbush (Flourensia cernua) are the principle species making up the 40% canopy cover of the brush-dominated vegetation. The soils are a well-drained, gravelly loam formed in calcareous old alluvium. The soil surface has minimal vegetative basal cover, with up to 60% gravelly erosion pavement. Watershed side slopes range from 3 to 15%, with about a 9% average. Grazing has been eliminated on the smaller watersheds (63.101, 63.104, and 63.105) since 1963, but cattle have been continuously grazing the larger watersheds (63.201 and 63.214) at the rate of about 5 animals per section per year. Rainfall data were tabulated from 24-hour recording raingages located on, or very near (within 1,000 ft of), the studied watersheds. Runoff was measured with small flumes or broad-crested weirs equipped with water-level recorders.

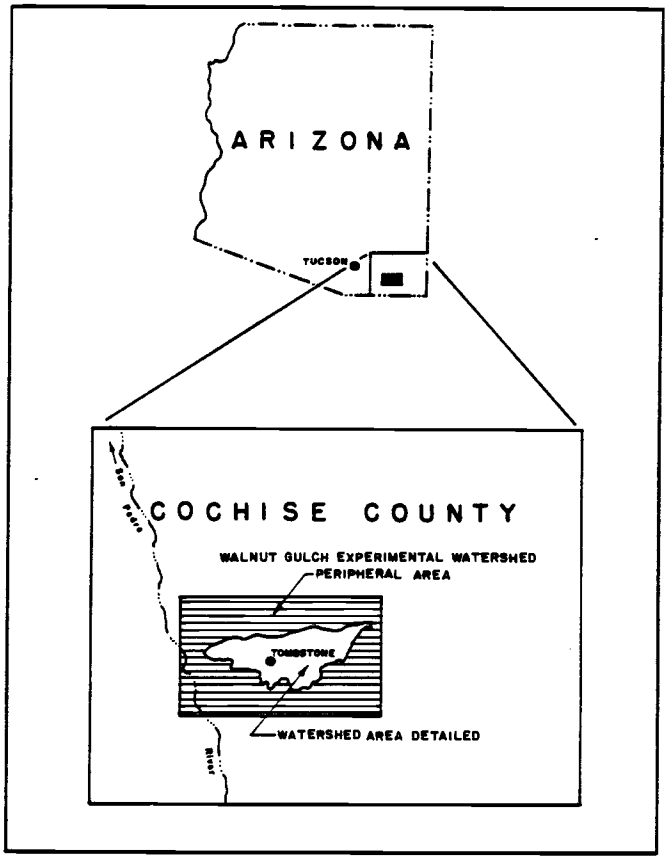

Figure 1. Location of Walnut Gulch Experimental Watershed.

RESULTS

In the initial analysis, a basic rainfall-runoff linear regression relationship

$$y = ax + b, \quad (1)$$

where x = rainfall factor, EI, in the Universal Soil Loss Equation (USLE) (Wischmeier and Smith, 1978), and y = runoff, Q (inches), was developed using runoff events from the smallest watershed (63.105, 0.45 acre). The smallest watershed was used because it best represented a homogeneous plot with no channel abstractions. The EI factor for each runoff-producing storm was computed from recording raingage data and an equation expressing the relationship between energy and rainfall intensity. The EI units are usually expressed in hundreds of ft-ton/ac-in. hr. The EI factor is the product of the total storm energy per acre (E) and the maximum 30-min intensity (I). In some years, over half of the annual EI (or R) has been the result of one storm (Renard et al., 1974). The probability distribution of total summer EI values and summer rainfall amounts for a raingage near 63.105 are shown in figure 2. For the period of study (1965-1975), summer precipitation varied from 5 inches to a little over 10 inches, while EI varied from about 30 to about 200. The range of EI values is a direct measure of the-varying rainfall intensities, and is a much better indicator of potential runoff and erosion than is total summer rainfall. The correlation between EI and runoff, for 63.105 and the rainfall-runoff equations (with and without the record event), are shown in figure 3. The regression equations were:

$$y = 0.018x + 0.015 \quad (2)$$

and

$$y = 0.020x + 0.005 \quad (3)$$

When the regression equations, developed from the data without the very large event, were used to estimate runoff from the large event, there was only an 18% difference between the actual and estimated volume. This suggests that the sample size was adequate. Either equation would be acceptable for prediction, although Eq. (2) is preferred.

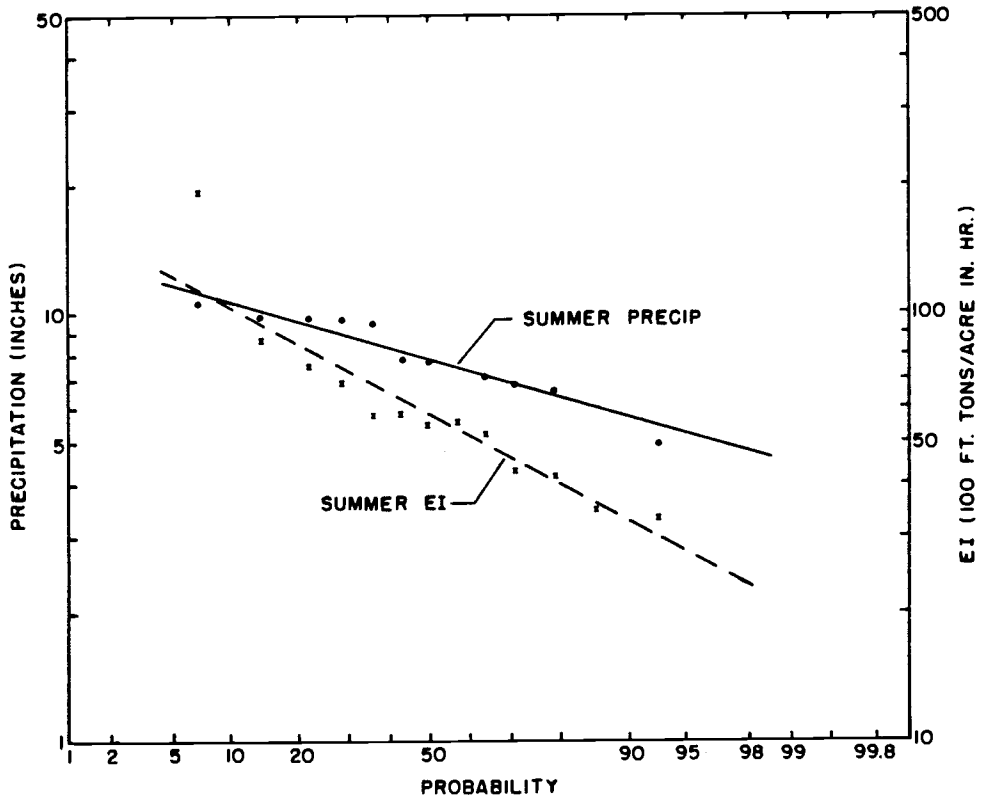

Figure 2. Occurrence probabilities for summer rainfall and summer erosion index (EI) (Raingage 83, 1963-1975).

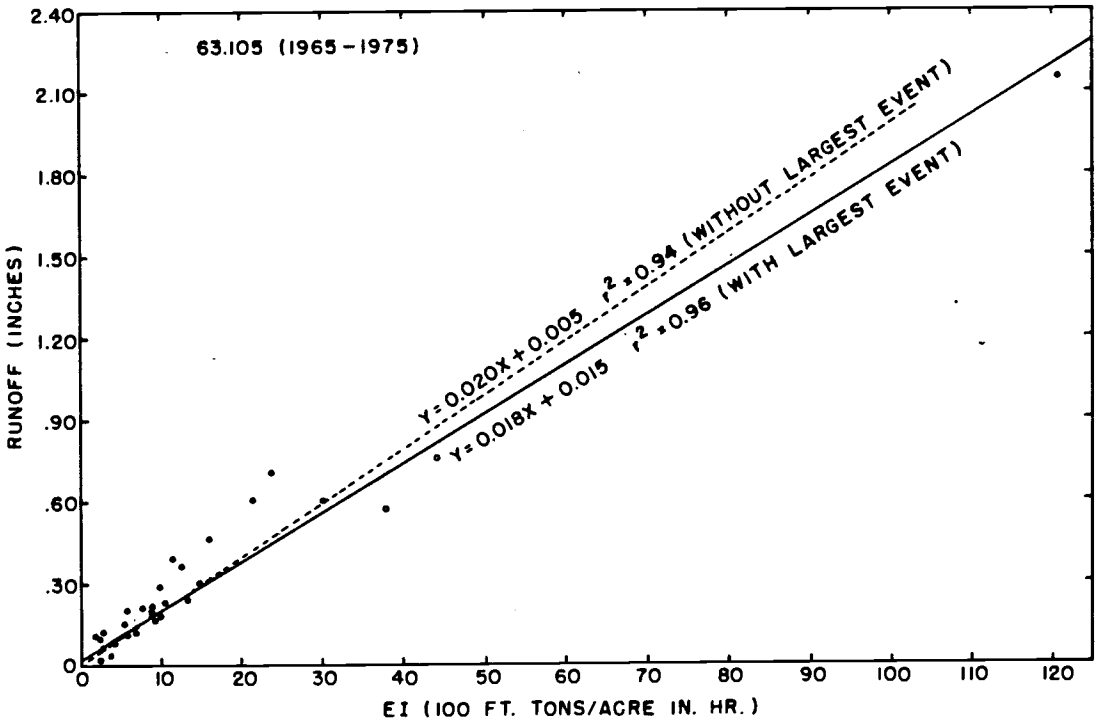

Figure 3. Correlation of EI with runoff for subwatershed 63.105, Walnut Gulch.

The regression equations were developed using EI values from runoff-producing rainfall only. Before the equation could be applied to larger watersheds, it had to be modified to reflect the influence of drainage area. Antecedent storm conditions are usually dry, and the sand-bottomed ephemeral stream channels associated with semiarid rangelands absorb a significant volume of runoff (Keppel and Renard, 1962). Larger watersheds generally have wider channels which abstract an increasingly larger volume of runoff. Furthermore, runoff-producing rainfall is limited in areal extent. Therefore, average annual runoff decreases with increasing watershed size. Based on analyses by Renard (1977) for larger watersheds and data from very small watersheds, the relationship of runoff volume to area was determined as:

$$Q = 1.06\ A^{-.135} \quad (4)$$

where
Q = runoff volume (in.), and
A = area (acres).

Runoff estimates for the larger watersheds, based on Eq. (2), were reduced using Eq. (4) and the ratios of watershed area for the larger watersheds to 63.105.

Watershed Runoff Estimates:

Storm runoff estimates for the four larger watersheds are shown in table 1. Runoff was badly underestimated for the two largest watersheds, which may suggest less channel abstraction than anticipated or an inadequate sample of runoff-producing events, or both. Also, one must remember that Eq. (4) represents the average for Walnut Gulch subwatersheds.

Table 1. Statistics for regression of actual vs estimated storm runoff estimates from area-modified basic equation developed on 63.105.

Watershed	Area (ac)	n	Slope	Intcp	Std error of est	r^2
63.105	.45	--	--	--	--	--
63.101	3.2	54	0.975	0.000	0.038	0.94
63.104	11.2	50	0.972	0.008	0.034	0.93
63.201	110.	32	0.690	0.006	0.021	0.92
63.214	372	25	0.656	0.023	0.014	0.82

Other Precipitation Parameters:

Other precipitation parameters were correlated to the runoff from the smallest watershed to determine if a simpler, more widely-available precipitation parameter (other than EI) could be used in watershed runoff estimation. Thirteen rainfall factors (Table 2) were correlated to each other, to runoff volume, and to peak runoff rate from watershed 63.105. The rainfall factors and their correlation matrix are given in table 2. Of the six parameters that were highly correlated ($r > 0.95$) to runoff, the $(P_{60})^2$ (the square of the maximum 60-min rainfall) parameter appears to be the simplest single rainfall parameter to use in estimating rangeland storm runoff volume. Because most of the runoff-producing rainfall is associated with short duration (1 hr or less) thunderstorm events, and because there is usually only one event a day, the 60-min rainfall can be estimated, with some confidence, from the daily total reported in NOAA monthly climatological reports. In figure 4, actual runoff is compared to estimated runoff with linear regression equations using various precipitation parameters, including (P_{60}^2).

The analysis included only runoff-producing storms, and a single very large event ($EI > 120$) was included in the data. Therefore, a modified linear regression technique (Diskin, 1970) was used to estimate runoff from all summer rainfall ($(P_{60})^2$) and runoff data, and both the Diskin method and the regression/reduction method were made on the data exclusive of the largest event. The analyses indicated no significant difference between using the Diskin method and the basic equation.

Peak Runoff Rate:

Runoff peaks and volumes are highly correlated on small watersheds dominated by thunderstorm rainfall (Table 2; Osborn and Lane, 1969; Renard et al., 1974). Therefore, the correlation coefficients (Table 2) also indicated that the peak run-off rate was correlated to P_{60}, EI, and $(P_{60})^2$, though not as well as was runoff volume. Of the linear regression equations associating P_{60}, EI, and $(P_{60})^2$ with peak runoff, the P_{60} gave the best estimate (fig. 5).

Return Frequency Runoff Estimates:

Widely available precipitation frequency information is found in the NOAA Precipitation Atlas for the western United States (Miller et al., 1973). These 1-hr amounts $(P_{60})^2$ were then used to estimate runoff for various return periods. The ratio of estimated runoff to rainfall were plotted versus return

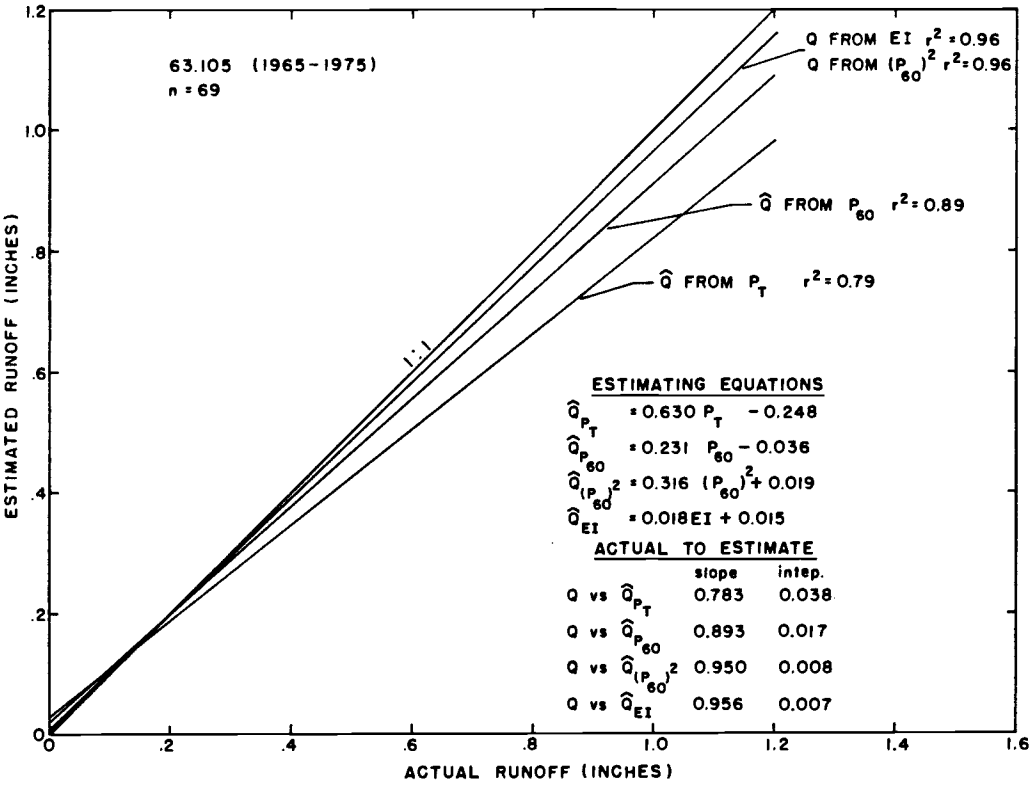

Figure 4. Comparison of actual and estimated runoff with four single-parameter regression equations.

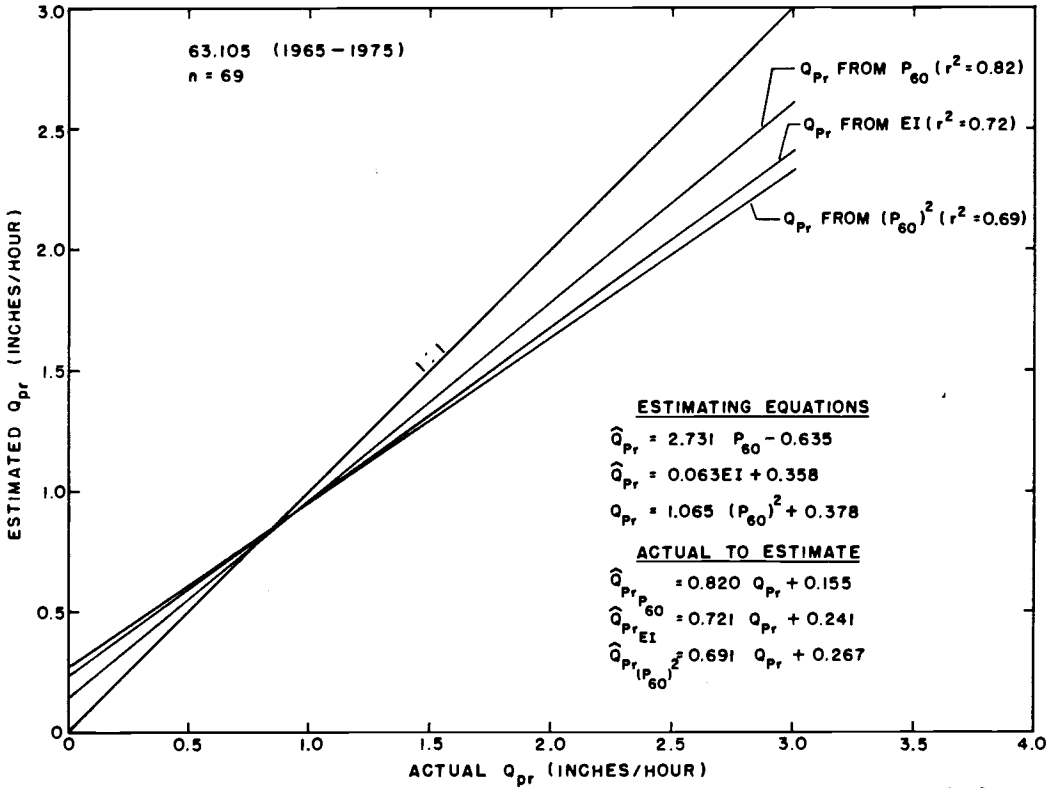

Figure 5. Comparison of actual and predicted peak discharges with three single-parameter regression equations.

period and compared to the plot of the return period versus the ratio of actual runoff to rainfall for 63.105 (fig. 6). The return period of the actual runoff and rainfall was calculated using 22 years of data and the Gumbel method for estimating frequency functions of extreme values (Gumbel, 1958). Estimates based on NOAA Atlas 2 were significantly lower than those based on actual data.

Table 2. Correlation coefficients of 13 precipiation parameters and two runoff parameters (only runoff-producing rainfall used in analysis. N = 69) (All values are significant at the 1% level).

	P_T	P_{60}	P_{30}	P_{15}	P_{10}	P_5	EI	E_{max}	P_{Tx30}	P_{Tx15}	P_{Tx10}	P_{Tx5}	$(P_{60})^2$	Q	Q_{pr}
P_T		.94	.92	.83	.77	.70	.87	.66	.89	.92	.92	.91	.89	.87	.82
P_{60}			.98	.90	.85	.78	.92	.71	.93	.96	.96	.95	.92	.93	.87
P_{30}				.95	.90	.84	.90	.73	.91	.95	.95	.95	.89	.91	.88
P_{15}					.99	.94	.78	.80	.79	.87	.88	.88	.76	.82	.89
P_{10}						.97	.73	.80	.73	.83	.84	.85	.70	.78	.87
P_5							.68	.75	.68	.77	.79	.81	.65	.72	.83
EI								.56	1.00	.98	.98	.97	1.00	.97	.81
E_{max}									.57	.65	.67	.66	.55	.63	.73
P_{Tx30}										.99	.98	.97	1.00	.97	.82
P_{Tx15}											1.00	.99	.98	.98	.88
P_{Tx10}												1.00	.97	.97	.89
P_{Tx5}													.96	.97	.89
$(P_{60})^2$														.97	.80
Q															.91

P_T = Total storm amount (in).
P_{60} = Max 60-min storm depth (in).
P_{30} = Max 30-min intensity (in/hr).
P_{15} = Max 15-min intensity (in/hr).
P_{10} = Max 10-min intensity (in/hr)
P_5 = Max 5-min intensity (in/hr).
EI = USLE rainfall factor (ft tons/ac in hr).
E_{max} = Max energy within storm (ft tons/ac).
P_{Tx30} = Total storm amount x max 30-min intensity (in^2/hr).
P_{Tx15} = Total storm amount x max 15-min intensity (in^2/hr).
P_{Tx10} = Total storm amount x max 10-min intensity (in^2/hr).
P_{Tx5} = Total storm amount x max 5-min intensity (in^2/hr).
$(P_{60})^2$ = Square of max 60-min storm depth (in^2).
Q = Runoff volume (in).
Q_{pr} = Runoff peak rate (in/hr).

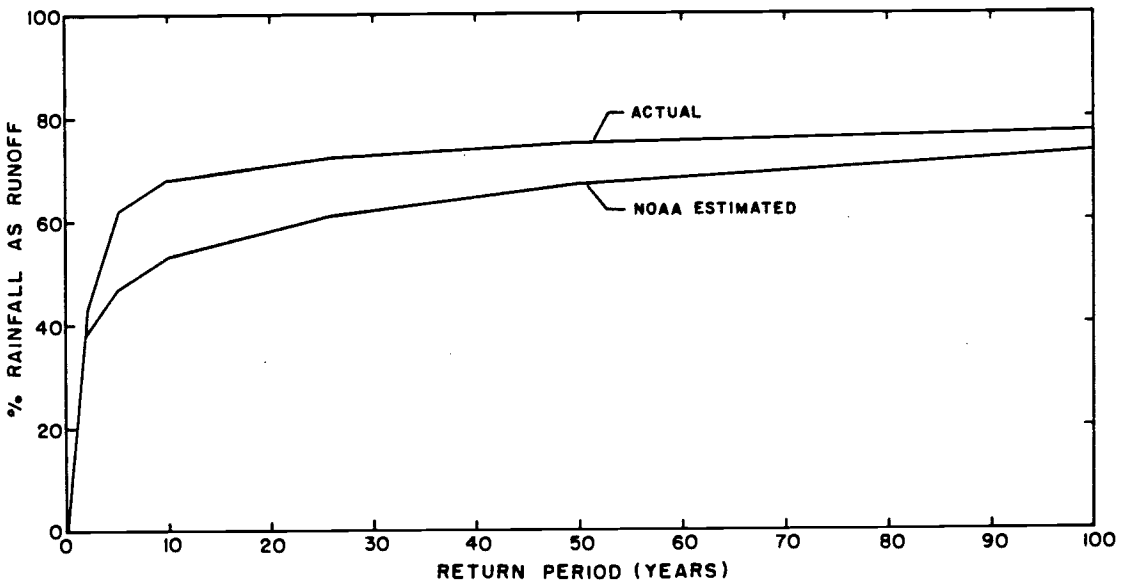

Figure 6. Return periods of ratios of Gumbel fitted precipitation and runoff and of NOAA precipitation and equation estimated runoff for 63.105, Walnut Gulch.

CONCLUSION

Linear regression techniques can be used to estimate runoff volumes from small rangeland watersheds. In the semiarid Southwest, where thunderstorms dominate the runoff-producing rainfall, there are two contributing factors: rainfall quantity and intensity. These factors are reflected in the EI term of the USLE. This term, when regressed with storm runoff volumes from a small rangeland watershed, produced a linear equation that could be used to estimate watershed storm runoff volumes. However, because of the inverse relationship found in semiarid rangelands between watershed drainage area and runoff volumes, an additional step to regression is needed in the method before it can be applied to larger watersheds. Runoff estimates for various-sized watersheds indicated the procedure could be useful if the estimated watersheds were in the same climatic regime and had similar, but not necessarily identical, vegetation cover. These conditions are found in large parts of southeastern Arizona, southern New Mexico, western Texas, and northern Mexico.

In comparing different estimating parameters, we found that $(P_{60})^2$ gave a good estimate of runoff volume, and that P_{60} gave good estimates of runoff peak rate. The P_{60} parameter can be estimated for different return frequencies from the NOAA Precipitation Atlas. Data from this atlas can be used with the runoff-estimating equations to give runoff estimates for various return frequencies. The estimated runoff volumes for the shorter return frequencies were lower than actual volumes. These differences can be attributed to the low NOAA precipitation estimates (Reich and Osborn, 1982).

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TIME-SPACE EFFECTS OF OPENINGS IN ARIZONA FORESTS ON SNOWPACKS

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Introduction

Forest openings affect a snowpack during both accumulation and melt phases. At any point in time, a snowpack is the integrated result of all accumulation, redistribution, and melt processes that have taken place before the time of measurement. Since snowpacks do not always have distinct accumulation and melt phases, it is difficult to determine the effect that an opening will have on a snowpack regime.

This paper describes an analysis of the effects of openings in Arizona ponderosa pine forests on snowpacks in and adjacent to the openings, using readily available input variables.

Methods

Seven study areas were established in ponderosa pine forests to simulate an array of openings (Table 1). Three sites were located 40 miles south of Flagstaff, while the other four were established in the White Mountains. Many of the sites were transmission line rights-of-way. In all cases, the openings were oriented with their long axes up and down the slope.

Two parallel transects, each extending 5H (H = average height of surrounding trees) into the forest on both sides of the openings, were established 50 feet apart on the study areas. All sampling points were located along each transect at an interval of 1/4H.

During a year of record snowfall in Arizona, 1972-73, data were collected on 10 sampling dates approximately two weeks apart. Total snow depth and snowpack water equivalent were measured at each sample point with a Federal snow tube and scale.

To evaluate the effects of an opening on the snowpack for a sampling date, the assumption was made that the influence of the opening is confined somewhere within the boundaries of the measurement transect. This assumption is supported by earlier studies which demonstrated that the effect of an opening is essentially nonexistent by the 2 to 3H distance within the forest (Anderson, 1963; Rothacher, 1965; Hansen and Ffolliott, 1968; and others).

Table 1. Range of Conditions on Study Areas.

Variable	Units	Range of Conditions		
		Minimum	Mean	Maximum
Timber Density	Square feet of basal area per acre	76	103	155
Opening Width[1]	H (average height of timber surrounding opening)	1.0	1.6	2.0
Slope-Aspect	Langleys (average potential input per day between December 1 and May 31)	599	658	709
Elevation	Feet (above sea level)	6600	7450	8100

[1]Dimension perpendicular to forest edge.

It was further assumed that the effect that an opening has on snowpack conditions becomes less as one proceeds away from the opening. It was felt, and subsequently verified, that the zones between 3 and 5H on both sides of an opening would be most representative of an undisturbed forested condition. However, it may also be possible that the entire influence of an opening at some point in time is confined within the opening itself. If this is the case, the average of all sample points within a forest would be most representative of the undisturbed forested condition.

Since a strict delineation of the extent of influence exerted by an opening into a forest at a point in time is difficult, a weighted average of all sample points within the forested condition was selected at the appropriate method to determine the average snowpack water equivalent of the undisturbed forested condition. As the 3 to 5H zone was most representative of an undisturbed forested condition at all points in time, sample points within this zone were given the heaviest weight. Zones closer to the opening were given less weight due to the likelihood that sometimes these zones would be more affected by the opening. Therefore, sample points from 3 to 5H were given a weight of 4; from 2 to 2-3/4H, a weight of 3; from 1 to 1-3/4H, a weight of 2; and from 0 to 3/4H, a weight of 1.

Once the average snowpack water equivalent of an undisturbed forested condition has been determined, the net effect of the opening was obtained by computing the signed deviation between that of the observed water equivalent of every sample point in the snowpack profile and the average snowpack water equivalent of the undisturbed forested condition. The summation of these deviations indicated the net effect of the opening for the point in time being analyzed.

The net effect of the opening calculated at the end of each time period for the different sites over the snow season was then regressed against readily available input variables that change with time and with location.

Variables that varied with time were:

1. Temperature. The total number of degree-days occurring between each time interval was calculated for each site from temperature records.

2. Potential insolation. Daily potential insolation values were obtained from curves developed for each site from December 1 through May 31, using appropriate values (Frank and Lee, 1966). The variable was expressed as the total number of potential langleys occurring between measurement dates at each site.

3. Precipitation. The amount of precipitation, expressed as inches of water equivalent, that fell at each site was recorded for the intervals between measurement dates.

4. Albedo. The daily snowpack albedo was calculated for each site by determining the number of days between precipitation events from weather records, and then changes in albedo with time relations (U.S. Army Corps of Engineers, 1956). The average daily albedo was computed for each time interval at each of the sites.

Variables that changed with location, but were independent of time, included:

1. Opening width. This variable was expressed in terms of the average height (H) of the surrounding forest.

2. Forest density. Average forest density along the measurement transects on either side of the opening, expressed as square feet of basal area per acre, was estimated by point sampling techniques (Avery, 1975).

3. Elevation. Elevation, expressed in feet, was obtained from USDI Geological Survey 7-1/2-minute quadrangle maps.

4. Tree height. Average tree height of the forest surrounding the opening was expressed in feet.

5. Slope-aspect. Using data previously obtained to determine potential insolation, the average number of langleys received from December 1 through May 31 was computed for each site; this is a way of numerically expressing slope-aspect combinations for different sites and, therefore, does not change with time once computed for a given site. (In comparison, the potential insolation variable listed under "time dependent variables" was used to evaluate potential insolation inputs over a given time period at a particular site.)

The dependent variable, the actual value of the net effect of an opening at the end of each time period, was regressed on the nine independent variables, using multiple regression techniques. To empirically evaluate their relative significance, the independent variables were expressed in linear form, squared form, and in cross-products. Data compilations from all study areas over all time periods were included in the analysis.

Results and Discussion

While there were several significant regressions at the five percent level of significance, an equation including precipitation, slope-aspect, and a joint variable of forest density-opening width suggested potential use in practice. This regression, which accounts for nearly 60 percent of the variation in the net effect of an opening, is:

$$Y = 117 + 0.471\ (X_1^2) - 0.119\ (X_2) - 0.226\ (X_3X_4)$$

where Y = the net effect of the opening expressed as inches of snowpack water equivalent

X_1 = the precipitation variable

X_2 = the slope-aspect variable

X_3X_4 = the forest density-opening width joint variable

Information contained in the above equation is two-fold. First, if a watershed manager wanted to maximize the net effect of an opening, he would select a site where X_3X_4 (forest density x opening width) and X_2 (slope-aspect) were low. Second, having selected the site, if the watershed manager wanted to predict the net effect of such an opening at any point in time, he could evaluate the amount of precipitation that had occurred within a specified time period, and then substitute this value for X_1 (precipitation) in the equation, along with the appropriate values for the site variables. For the range of conditions studied, an optimum net effect is obtained when a 1H opening is cut in a forest density of approximately 75 square feet of basal area per acre on a site receiving an average of nearly 600 langleys per day from December 1 through May 31 and "high" amounts of precipitation.

Opening width is of primary importance during snowpack accumulation stages. Narrow opening widths have been shown as early as the 1900's to be most effective as snow traps (Church, 1912). The amount of precipitation received affects the magnitude of this trapping effect; the greater the amount of precipitation, the greater the trapping effect. This general relationship holds so long as snow storage in the opening is less than its capacity.

It is important to realize that in determining the net effect of an opening, one is comparing the average water equivalent of the snowpack lying within the "sphere of influence" with that of the undisturbed forested condition. At a point in time immediately following a heavy precipitation event, this net effect may be relatively large. However, if melt processes are more active in the opening than in the forested condition, the net effect will diminish during the melt phases. Also, the time distribution and sequence of precipitation events is important in determining the magnitude of the net effect of an opening at a given time. (Ten storms of one-inch water equivalent may differ in magnitude from one event yielding 10 inches of water equivalent.)

Forest density also has a large effect on the melt characteristics at a given site. Trees on the site affect several processes simultaneously (Anderson, 1967). The density of the forest determines the amount of short-wave radiation directly impinging on the snowpack. Forest density also determines the amount of short-wave radiation that is absorbed and reradiated as long-wave radiation (Reifsnyder and Lull, 1965). Wind movement through a forest and, consequently, the amount of evaporation is affected by the same trees. If the trees surrounding an opening are dense, the undisturbed forested condition is shaded, resulting in less impingement of direct solar radiation; and snow in openings is subjected to comparatively large amounts of long-wave radiation from the dense forest surrounding the opening, resulting in accelerated melting along the forest edges.

Openings cut in dense forested areas result in comparatively large differential melting between the undisturbed forested condition and the "sphere of influence" of the opening. Therefore, previous gains in the net effect of the opening made during accumulation phases may be rapidly offset during active melting. In contrast, rates of melt in the undisturbed forested condition progressively approach those of the opening as the forest density is reduced because: the undisturbed forest condition becomes increasingly open to both incoming solar radiation and wind movement, resulting in higher rates of ablation; and there are fewer trees around the edge of the opening, resulting in comparatively lower magnitudes of "back radiation."

It is well documented that sites having low potential insolation inputs have less intense melt characteristics than do those with high inputs (Anderson, 1967; Ffolliott and Hansen, 1968; and others). Therefore, the potential insolation variable is important when determining the net effect of an opening. Since melt is not as intense on "cool" sites, an opening is not subjected to the same amount of ablation as openings associated with "warm" sites. Consequently, the differential melting between the opening's "sphere of influence" and that of the undisturbed forested condition is less on "cool" sites, resulting in a maximization of the net effect of the opening.

Conclusions

The analysis of the net effect of forest openings on a snowpack regime may be useful in designing cutting practices to affect snow melt. However, additional data sets representing other sites and additional years are needed to verify the results presented herein. Use of the results of this study in other locations in the Southwest and for other years should be predicated on discerning acceptable values for "confidence limits" to be used for prediction purposes, and on validating the manipulation of site variables in determining locations of proposed forest cuts.

Finally, a question that remains unanswered is what proportion of a watershed should be occupied by openings to insure optimum snow melt increases. In other words, the relative spacing between openings and forested areas required to achieve desired water yield increases derived from snow melt must be determined.

Acknowledgements

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INFLUENCE OF SLASH WINDROWS ON STREAMFLOW

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Abstract

Although the effect cannot be isolated, removal of slash windrows has apparently caused some reduction in annual water yield response from a cleared ponderosa pine watershed. Removal of the slash windrows has eliminated any influence on snow distribution, retention, and melting and has also eliminated any possible influence on losses to soil water recharge and on runoff efficiency of the watershed.

Introduction

Ponderosa pine forests occupy about 1.7 million ha in Arizona, including more than 0.7 million ha or 20% of the Salt-Verde River Basin. These forests are the largest single contributor of water to many of the river basins in the state (Ffolliott and Thorud 1975) and produce nearly one-half of the water yield in the Salt-Verde Basin (Barr, 1956).

Because current water consumption in Arizona includes the mining of an estimated 250-370 thousand ha-m of ground water annually (Ffolliott and Thorud 1975), any effects that ponderosa pine management have upon water yield become very important and significant to land management planners in Arizona.

The Beaver Creek research watersheds in north-central Arizona were established by the USDA Forest Service to quantify treatment effects on water yield from pinyon-juniper woodland and ponderosa pine forests. Two types of vegetation manipulation have been studied in the ponderosa pine forests on Beaver Creek: thinning forest overstory and creation of cleared openings (Brown et al. 1974). This paper documents results of the 4-year period following the removal of slash windrows on a cleared watershed.

Study Area

The paired watersheds used in this study are located 80 km south of Flagstaff, Ariz. at an elevation of about 2,170 m. Physical characteristics of the experimental watersheds are presented in Table 1.

Soil in the study area developed from volcanic basalt parent material is predominately a Brolliar stony clay loam; while the soils derived from volcanic cinder parent material are a Sponseller or a Siesta stony silt loam. These soils are generally less than 1 m deep (Williams and Anderson 1967).

Prior to the application of the clearing and windrowing treatment, the watersheds were stocked with uneven-aged stands of ponderosa pine (Pinus ponderosa, Laws.) and with interspersed Gambel oak (Quercus gambelii, Nutt.) and alligator juniper (Juniperus deppeana, Steud.). Overstory basal area, prior to treatment, averaged about 25 m^2ha^{-1} on both basins.

Average annual precipitation for the period 1960-1981 is 611 mm. Winter precipitation averages 403 mm, or 66% of the annual total. Most of the remaining precipitation falls during July, August, and September. A more detailed description of the hydrologic regime of the study area can be found in Baker (1982) and Campbell and Ryan (1982).

Continuous streamflow records on each watershed were obtained from a precalibrated, concrete, trapezoidal flume (Brown et al. 1974). Mean annual streamflow amounts for the treated and control watersheds are presented in Table 1. Annual streamflow on the treated watershed has varied from 12 to 700 mm.

Most of the annual water yield on the experimental watersheds is produced in the winter. During the period of this study, few years had less than 90% of the annual discharge occurring during the winter season. The soil mantle is recharged during the winter when water from rain or snowmelt

Table 1. Physical Characteristics for Two Experimental Watersheds on Beaver Creek.

Characteristics	100% Cleared	
	Treated	Control
Watershed Number	12	13
Size (ha)	184	369
Aspect	SW	SW
Warm slope (%)	87	74
Cool slope (%)	13	26
Mid-area elevation (m)	2150	2195
Basal area (m^2 ha^{-1})	24	28
Annual precipitation		
Before clearing treatment (mm)	617	609
After clearing treatment (mm)	552	592
Annual streamflow		
Before clearing treatment (mm)	150	93
After clearing treatment (mm)	186	94
Soil water holding capacity		
(weighed mean) (mm)	241	335

is available and evapotranspiration demands are low. Most streamflow appears a snowmelt runoff in March and April, terminating in early May. Most stream channels in these headwater basins become dry before the onset of summer rains.

Slash from the original clearing treatment was windrowed rather than piled because it was felt windrowing would improve water response from the clearing treatment. It was hypothesized that windrows would trap and retain snow, reduce evapotranspiration losses, and increase drainage efficiency of the watershed. It was not intended to separate the effect of overstory removal from that of windrows.

The juniper and oak have been allowed to sprout and grow during the 14-year period following the clearing treatment. The size and number of oak sprouts, in particular, have increased significantly. Periodic measurements of the overstory show that total basal area on the cleared watershed was 2.7 and 3.4 m^2ha^{-1} in 1975 and 1980, respectively. Canopy density of the overstory was 15-20% in 1980.

Covariance analysis of the clearing and windrowing treatment for the period of 1968-1977 indicated a significant increase in annual water yield. However, after water year 1973, the increases in individual annual water yields were reduced or even lost as the size and number of Gambel oak sprouts increased. The ability of the Gambel oak sprouts to utilize the gain in water derived from reduction in evapotranspiration and interception (Tew 1966, 1967, and 1969) was apparently responsible for the diminished annual streamflow response.

Treatment and Analyses

The slash windrows were removed in the fall of 1977 to determine if they had any influence on water yield. The paired watershed method was used to evaluate changes in annual water yield

resulting from removal of the slash windrows. A covariance analysis was used to test significance of windrow removal on annual water yield. An α level of 0.10 was used to determine statistical significance of the covariance analysis. A multiple regression analysis was also used to determine if a time trend for the water yield response could be detected.

Results

Analyses of the water yield response since 1977 have shown significant changes. It is hypothesized that these changes are due to removal of the slash windrows and the continued growth of the Gambel oak. The effect of removing windrows cannot be isolated because of the continued regrowth of Gambel oak. Any difference caused by the removal of windrows appears to be masked by more significant changes caused by the regrowth of oak. Any analysis is also weakened by dry years preceding the removal of windrows and by the comparatively wet years immediately after (figure 1). However, the following analyses display data anomalies which may be at least partially caused by windrow removal.

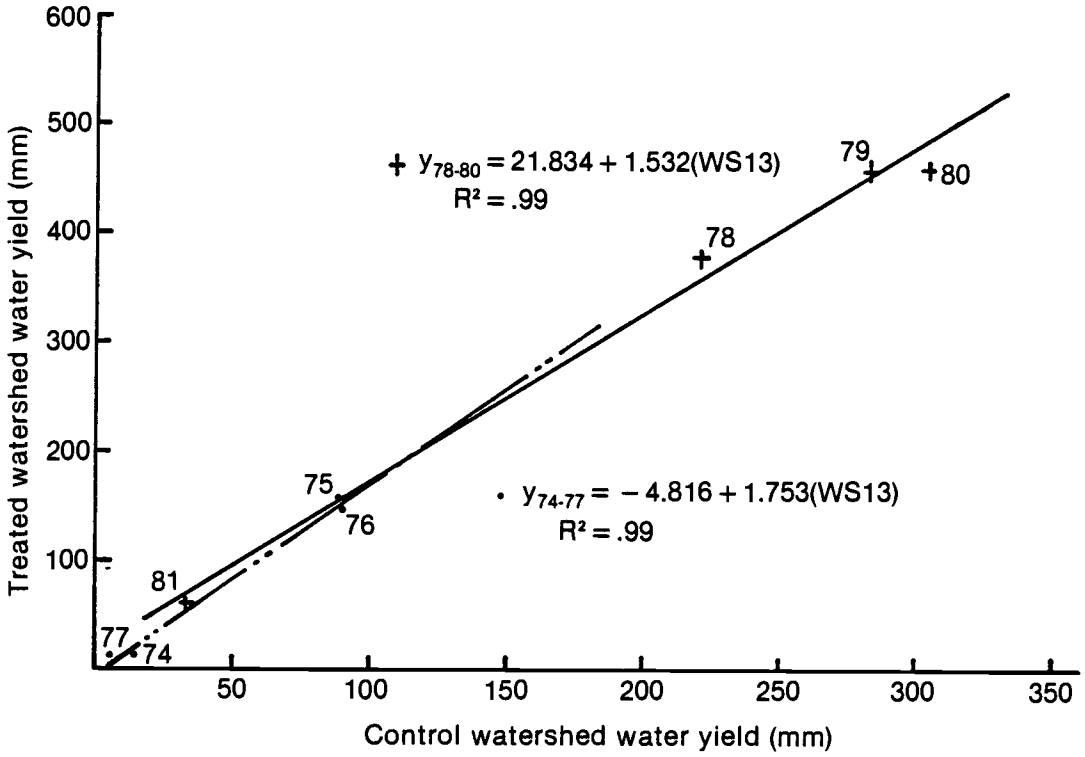

Figure 1.--Pre- and post-treatment regressions for the analyses of the removal of slash windrows.

Analysis of annual water yield for the period 1978-1981 with windrows removed and for the period 1974-1977 with annual water yield response apparently lost, shows there is no significant difference between the two regression relationships (figure 1). Therefore, no effect on water yield can be attributed entirely to the removal of windrowed slash.

However, a regression model is developed using years 1968-1977 with a time factor included to show change in response caused by the clearing and windrowing treatment (figure 2). The model predicted well (r=0.996) until 1977 when the windrows were removed. During the last 4 years, the model overpredicts water yield for all years except 1981, which was relatively dry. This supports the hypothesis that removal of the windrows has caused further reductions in water yield from the watershed. An explanation is that the windrows did affect snow accumulation and melting of snow during the winter and provided for more efficient waterways to the main channel areas.

Discussion

Determination of the effects of windrow removal is confounded by the sprouting and growth of the Gambel oak on the watershed. As shown, the significant increase in annual water yield caused by the original clearing and windrowing treatment was lost 4 years prior to removal of the windrows.

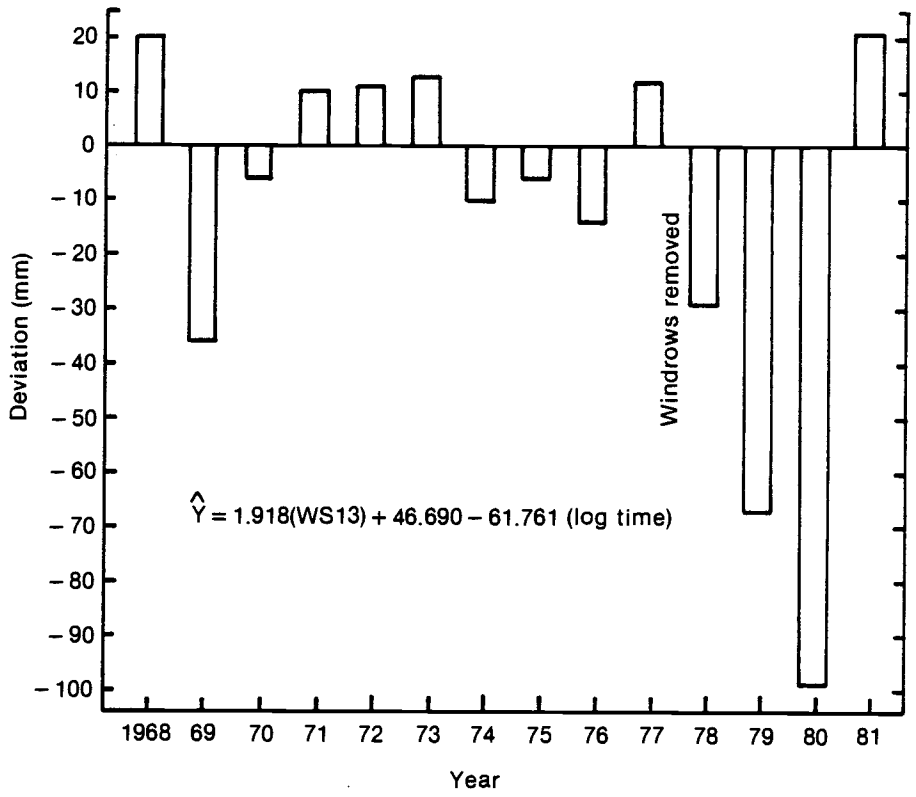

Figure 2.--Deviation of the annual water yield from the regression model.

This leaves the sprouting and growth of the oak as a significant contributor to the loss in the water yield response. Although the canopy density was only 15-20% in 1980, the additional leaf area of the shrubs and herbage plants must have an effect on water yield. The ability of hardwood species to use soil water more efficiently also influenced the amount of water available for runoff.

Removal of the slash windrows apparently has some influence on annual water yields. Although not specifically measured, anomolies in the data suggest that removal of windrows has further reduced water yields by eliminating their influence on snow distribution, retention, and melt and by eliminating any possible influences on losses to soil water recharge and runoff efficiency of the watershed.

The possible influence of slash management on water yield provides land managers the opportunity to complement silvicultural and water yield improvement practices. Managers desiring to maximize streamflow should consider leaving windrowed slash in place, rather than burn it.

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WATER BALANCE CALCULATIONS, WATER USE EFFICIENCY, AND ABOVEGROUND NET PRODUCTION

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Abstract

A discrete form of the water balance equation is used to illustrate the interaction among precipitation, runoff, percolation below the root zone, bare soil evaporation, plant transpiration, and plant available soil moisture. Under rangeland conditions, water availability is often the limiting factor in plant survival and growth. Therefore, the water balance equation is used, together with soils data and water use efficiency factors, to estimate annual aboveground net primary production of perennial vegetation.

INTRODUCTION

Arid and semiarid rangelands are extensive (e.g., Branson et al., 1981), and in these regions, water availability is most often the limiting factor in plant production. Water availability is also the most important environmental factor controlling survival of range plants (e.g., Brown, 1977). Therefore, water balance calculations are essential in soil-water-plant relationships studies. Plant water use, as a component of total evapotranspiration (ET), affects the water balance and soil moisture content, and thus, infiltration and runoff. As will be discussed later, soil moisture often limits the rate of ET so that consideration of feedback is necessary in water balance calculations.

If we assume no net subsurface water movement in the horizontal direction and a limited rooting depth well above the permanent water table, then a discrete form of the water balance equation in terms of unit area is:

$$\frac{\Delta S}{\Delta t} = P - Q - ET - L \tag{1}$$

where:

S = soil water content (mm representing units of volume per unit area),
Δt = time for the calculation period (e.g., hr, day, month, etc.)
P = rainfall depth for the time interval (mm),
Q = net runoff for the area (mm),
ET = combined evaporation and plant transpiration (mm), and
L = seepage or percolation below the root zone (mm).

The rate of ET in Eq. (1) depends upon atmospheric demand (a potential rate), soil texture, and vegetation characteristics (e.g., the leaf area index (LAI)) when soil moisture (SM) is nonlimiting, and upon soil water content, soil texture, atmospheric demand, and vegetation characteristics when water is limiting. Veihmeyer and Hendrickson (1955) suggested that the ratio of actual evapotranspiration (AET) to potential evapotransporation (PET) stayed about constant as the soil moisture decreased from field capacity to near the wilting point. Thornthwaite and Mather (1955) suggested a linear relationship between the ratio AET/PET and available soil water. Other investigators have proposed nonlinear relationships, as summarized by Hanson (1973, Fig. 3, p. 16). This relationship can be expressed as:

$$AET = \begin{cases} PET & SM \geq SM_1 \\ f(SM, PET) & SM_0 < SM < SM_1 \\ 0 & SM \leq SM_0 \end{cases} \tag{2}$$

where SM_1 is a limiting soil water content (between wilting point and field capacity) where soil water content above SM_1 does not influence AET, and SM_0 is the permanent wilting point where plant available soil water is exhausted. The function f controls the ratio of AET to PET when soil moisture is between the limits SM_1 and SM_0. The parameters SM_1 and SM_0 are often expressed in terms of the soil water content at -1/3 and -15 bars, respectively.

Leaf area index (LAI) is the leaf area per unit area of land surface (e.g., m^2/m^2), and is often used in models to relate evapotranspiration and plant growth (e.g., Chang, 1968; Stern, 1965; Penman, 1963; and Brougham, 1956). Hanson (1973, Fig. 2, p. 15) illustrates two relationships between LAI and AET/PET. For LAI = 0 (bare soil), the ratio AET/PET varied from zero to about 0.26, depending upon the

relationships assumed and on the potential rate, PET. For LAI = 1.0, the ratio was about 0.5, and for LAI > 2.5, the ratio approached 1.0. Therefore, LAI can be used to estimate the magnitude of AET and to partition total AET into bare soil evaporation, E, and plant transpiration, T (e.g., Parton et al., 1978; Ritchie, 1972; Ritchie and Burnett, 1971; Ritchie et al., 1976; and Smith and Williams, 1980).

Ritchie (1972) describes the two stages of bare soil evaporation: stage 1 (constant rate limited by supply of energy to the surface) and stage 2 (falling rate stage where water movement to evaporation sites at the surface depends upon soil properties). The volume of evaporation during stage 1, u, is:

$$u = a\,(c - b)^{d} \tag{3}$$

where u is in mm, a, b, and d are fitting coefficients (e.g., a = 9, b = 3, and d = 0.42 fit Ritchie's data), and c is a parameter dependent upon soil texture. Stage 2 evaporation rate is:

$$E_{S2} = 1/2\ ct^{-1/2} \tag{4}$$

where E_{S2} is evaporation in mm/day, t is time in days since initiation of stage 2, and c is the soil texture parameter. The total soil evaporation during a drying period, t, can then be approximated as:

$$E_t = u + \int E_{S2}\,dt \tag{5}$$

which, as a first order approximation for a finite time period, can be represented as:

$$E_t = u + Au = (1 + A)u \tag{6}$$

where A is the ratio of stage 2 to stage 1 evaporation volume.

Now, if this volume is assumed to represent water extracted when soil moisture is between field capacity, FC, and wilting point, WP, then by continuity, we can write:

$$y(FC - WP) = (1 + A)u \tag{7}$$

where y is the effective depth influenced by bare soil evaporation. Values of c, in Eq. (3) and (4), are reported by Ritchie (1972) and by Jackson et al. (1976). The value of E_T, in Eq. (5), was compared with data from Ritchie (1972), Hillel (1976), and Jackson et al. (1976) to estimate the value of A in Eq. (6). Data presented by Jackson et al. (1973, Fig. 8) show the locus of the zero flux surface (vertical direction) in a soil profile (16 to 38 cm with a mean of about 20 cm) which provided experimental data to compare with y values calculated from Eq. (7). Finally, SCS (1982) presents additional data on values of c in Eq. (4) vs. soil texture.

Rawls et al. (1982) analyzed data from over 1300 soils in 32 states to generalize soil water properties as a function of textural class. Interpolated values from the texture triangle (from Rawls et al. (1982) and from SCS (1982)) were combined to produce the gross soil properties summarized in Tables 1 and 2. Table 1 lists 12 texture classes, representative proportions of clay, silt, and sand, mean rates of saturated hydraulic conductivity, and soil evaporation parameters. Notice that the data labeled "avg" represent a central, or representative, value for mean hydraulic conductivity and c in Table 1, and for mean water holding capacities in Table 2. The columns labeled "low" and "high" refer to low and high estimates on the mean, not to the maximum expected range for the given parameter. Moreover, the parameters in Tables 1 and 2 were derived predominately from agricultural soils, and probably do not represent desert soils with high gravel content.

Table 1.--Selected soil properties based on soil textural class.

Soil texture class	Representative composition			Saturated hydraulic conductivity (cm/hr)			Bare soil evaporation parameter c(mm/day 1/2)		
	Clay	Silt	Sand	Avg	Low	High	Avg	Low	High
Sand	3	7	90	23.	11.7	43.2	3.3	3.05	3.32
Loamy sand	5	15	80	6.1	3.6	11.7	3.3	3.05	3.32
Sandy loam	10	20	70	2.2	1.7	3.6	3.5	3.10	4.06
Loam	20	40	40	1.3	.91	1.7	4.5	3.20	4.57
Silt loam	15	65	20	.69	.46	.91	4.5	3.20	4.57
Silt	5	87	8	.51	.30	.61	4.0	3.15	4.40
Sandy clay loam	30	10	60	.30	.25	.46	3.8	3.15	4.32
Clay loam	35	35	30	.20	.19	.25	3.8	3.15	4.32
Silty clay loam	35	55	10	.18	.15	.19	3.8	3.15	4.32
Sandy clay	45	5	50	.13	.11	.15	3.4	3.10	3.56
Silty clay	45	50	5	.10	.09	.11	3.5	3.10	3.81
Clay	65	20	15	.08	.06	.09	3.4	3.10	3.56

Table 2.--Porosity and water holding capacity (water content in % by volume) based on soil textural class.

Soil texture class	Total porosity			-1/3 Bar Water holding capacity			-15 Bar Water holding capacity		
	Avg	Low	High	Avg	Low	High	Avg	Low	High
Sand	41	39	43	9	7	15	3	2	6
Loamy sand	43	39	45	12	10	20	6	4	8
Sandy loam	45	39	52	20	14	29	9	5	12
Loam	47	45	52	26	20	36	12	9	18
Silt loam	50	49	55	31	20	36	13	7	20
Silt	51	49	55	28	26	30	9	6	12
Sandy clay loam	42	38	45	27	17	34	17	11	21
Clay loam	47	40	51	34	29	38	20	16	24
Silty clay loam	47	46	51	36	33	40	21	18	24
Sandy clay	42	40	44	31	27	40	21	18	30
Silty clay	48	46	49	40	35	46	27	23	32
Clay	49	44	52	42	34	49	29	23	38

Smith and Williams (1980) describe the AET calculations used in the CREAMS model (Knisel, 1980). Their basic equations follow Ritchie's model, and were formulated as follows: Potential daily soil evaporation is adjusted for the plant canopy such that

$$E_{so} = E_o e^{-.4 LAI} \tag{8}$$

where E_o is the potential evaporation, and E_{so} is the reduced potential soil evaporation which is then used to compute stage 1 and stage 2 evaporation rate, as described earlier. Plant transpiration is computed as:

$$T_o = \begin{cases} (E_o)(LAI/3) & 0 \leq LAI \leq 3 \\ E_o - E & LAI > 3 \end{cases} \tag{9}$$

where E is the actual soil evaporation. If soil water is limiting, then the reduced transpiration is computed as:

$$T = (T_o)(SM)/(0.25\ FC) \tag{10}$$

where T_o is computed from Eq. (9), SM is soil moisture $\leq$ 0.25 FC, and FC is field capacity of the soil. At each time step, the sum of soil evaporation, E from Eq. (3), (4), and (8), and transpiration, T from Eq. (9) and (10), cannot exceed E_o calculated from temperature and radiation. Therefore, total AET is partitioned into soil evaporation and plant transpiration according to the value of LAI. This value of LAI varies throughout the season to reflect crop growth stage or plant phenological stages.

Therefore, to estimate AET and plant water use using a model such as Ritchie's equations in the CREAMS model, it is necessary to know soil properties (Tables 1 and 2), plant rooting depth, the seasonal LAI, and the other data used to solve the water balance equation. The CREAMS model is typical of many water balance-climate models in that it uses LAI to partition AET into E and T, and AET is limited by available soil moisture, as described by Eq. (2). More complex models (e.g., the ELM model, Innis (1978) and the EPIC model, Williams et al. (1982)) explicitly include plant growth components and direct feedback with the water balance components. In the absence of sufficient information to estimate the parameters of these complex models, or if the objective is to obtain first-order approximations for plant production when water is limiting, then simple water balance-plant production models (e.g., Wight and Hanks (1981) or Lane et al. (1983)) can be used to relate the water balance and plant production.

WATER USE EFFICIENCY

Water use efficiency (WUE) has been defined in many ways, but in each case, the intent was to relate a quantity of plant production to a quantity of water used for that production. Briggs and Shantz (1914), and others (e.g., Pendleton, 1966; Viets, 1966), have used the term water requirement to mean the amount of water required to produce a gram of aboveground dry matter. Others (e.g., Dwyer and DeGarmo, 1970) have used total dry matter production (roots, shoots, and fruits) as a measure of plant production in defining water use efficiency. Still others (e.g., Szarek, 1979; Tadmor et al., 1966) have used annual precipitation as a predictor for annual actual evaportranspiration, which in turn was used as a predictor of actual plant water use. McGinnies and Arnold (1939) illustrated seasonal influences on WUE, and Dwyer and DeGarmo (1970), among others, have illustrated the influence of soil moisture status on WUE. Unless we state otherwise, water use efficiency shall mean the ratio of the amount of aboveground dry matter production (g, g/m^2, kg, kg/ha) to the amount of water used in that production (g, kg, kg/m^2, mm) on an annual basis. The term water use excludes evaporation from bare soil and evaporation of intercepted

water from vegetation surfaces. Thus, production refers to aboveground only, and water use refers to plant water use. Finally, it is not known, explicitly, how WUE factors, measured under controlled conditions, apply to field conditions where water stress, competition, spatial variability in weather and soil characteristics, and related factors might affect water use efficiency.

Selected WUE data, from greenhouse or carefully controlled plot studies, are listed in Table 3. The first column of Table 3 lists the vegetation types, the second column lists WUE in units of g of aboveground dry matter production per kg of water use, which is the same as g d.w./m^2 over mm H_2O/m^2, or grams dry weight per square meter per mm of plant transpiration per square meter. The third column of Table 3 provides comments, and the fourth column lists the appropriate reference. The references were selected to be representative of results obtained from the literature for arid and semiarid rangelands. Data collected by McGinnies and Arnold (1939) were from 76-liter galvanized Witt cans filled with 125 to 145 kg of soil. Because of the rather large-sized soil containers and duration of the experiment (1932-1936), these data are probably among the most reliable of those shown in Table 3. These data represent annual means, with summer WUE being from near to one and a half times as large as the annual values. The mean annual WUE for the six desert grasses was 1.8 g d.w./kg H_2O and ranged from 1.7 to over 1.9. In contrast, the mean annual WUE for the desert shrubs and trees was 0.57 g d.w./kg H_2O and ranged from 0.42 to 1.30. Therefore, these data suggest that, on the average, the selected desert grasses have a WUE some three times higher than the selected trees and shrubs (the range of the ratio of WUE grass to WUE trees and shrubs was 1.3 to 4.6). Dwyer and DeGarmo (1970) used pots (18 cm diameter by 25 cm deep) to study the WUE of four desert grasses and four desert shrubs. Means for the desert grasses ranged from 1.0 to 1.09, with an overall mean of 1.03 g d.w./kg H_2O. The means for Dwyer and DeGarmo's four shrubs ranged from 0.23 to 0.57, with an overall mean of 0.28 g d.w./kg H_2O. These data suggest that, on the average, the desert grasses have a WUE some 3.7 times higher than the selected desert shrubs.

Table 3.--Selected water use efficiency data.

Vegetation	WUE	Comments	References
(Type)	(g d.w/kg H_2O)		
Desert grasslands	1.70 - 1.92	Annual values for 6 species	McGinnies and Arnold (1939)
Plains grasslands	1.42 - 2.09	Annual values for 3 gramma grasses	McGinnies and Arnold (1939)
Southern tall grasss	1.41 - 2.12	Annual values for 3 species	McGinnies and Arnold (1939)
Desert shrubs and trees	0.42 - 1.30	Annual values for 6 species	McGinnies and Arnold (1939)
Desert grasses	1.00 - 1.09	Greenhouse study of 4 species, 120 days	Dwyer and DeGarmo (1970)
Desert shrubs	0.23 - 0.57	Greenhouse study of 4 species, 120 days	Dwyer and DeGarmo (1970)
Native and seeded within the pinyon-juniper type	-	Brief survey of values reported in the literature	Branson et al. (1981) from Gifford (1976)
Selected grasses	0.31 - 2.95	Brief survey of values reported in the literature	Teare (1977)

The ratio of mean WUE for the desert grasses, in McGinnies and Arnold's study to Dwyer and DeGarmo's, was 1.81/1.03 = 1.75, and, for the trees and shrubs, a similar ratio was 0.57/0.28 = 2.04. The reasons for these differences between the two experiments are not clear. One possible reason might be distortions due to root bound plants in the small pots. This situation, if it did in fact occur, would tend to lower the mean water use efficiency. Finally, Branson et al. (1981) and Teare (1977) present brief summaries of water use efficiency data selected from a variety of sources. Data, such as those presented in Table 3, may be used to estimate aboveground production. However, the magnitude of the variability in WUE values, as discussed above, should be kept in mind.

ANNUAL ABOVEGROUND NET PRIMARY PRODUCTION

Annual aboveground net primary production (ANPP) is the total aboveground dry weight (d.w.) biomass produced per unit area in a growing season. Net refers to the gross production minus respiration per unit time of an individual plant, and primary is the sum of the net productions of all individual plants in a unit area (Whittaker and Macks, 1975). Total above and below ground net primary production of general ecosystem types is summarized by Whitaker and Likens (1975). They suggest values for desert and semidesert scrub of 10 - 250 g/m^2/yr, with a mean value of 90. Corresponding estimates for woodland and shrubland range from 250 - 1200, with a mean of 700, while values for temperate grassland vary from 200 - 1500, with a mean of 600. These values are for total annual net primary productivity so that the aboveground values would be significantly less, depending on the root-shoot ratios. Lieth (1975) lists similar estimates for selected climatic regions, and Murphey (1975) lists estimates for tropical terrestrial ecosystems. Webb et al. (1975) present productivity data, including grasslands and desert ecosystems,

and Szarek (1979) summarizes data for four North American deserts.

Generalized relationships relating annual actual evapotranspiration, AET, and annual aboveground net primary production, ANPP, have been developed by several investigators (e.g., Rosenzwieg, 1968 and Webb et al., 1978). Data on ANPP and AET for three North American deserts (Szarek, 1979 and Lane et al. 1983) and for a site in the Tunisian steppes (Floret et al., 1982) are shown in Fig. 1. Notice that, for the data shown, Rosenzweig's equation tends to provide an upper limit on ANPP, and that the Webb et al. equation seems to fit the overall trend, but underestimates for low values of AET < 120 mm.

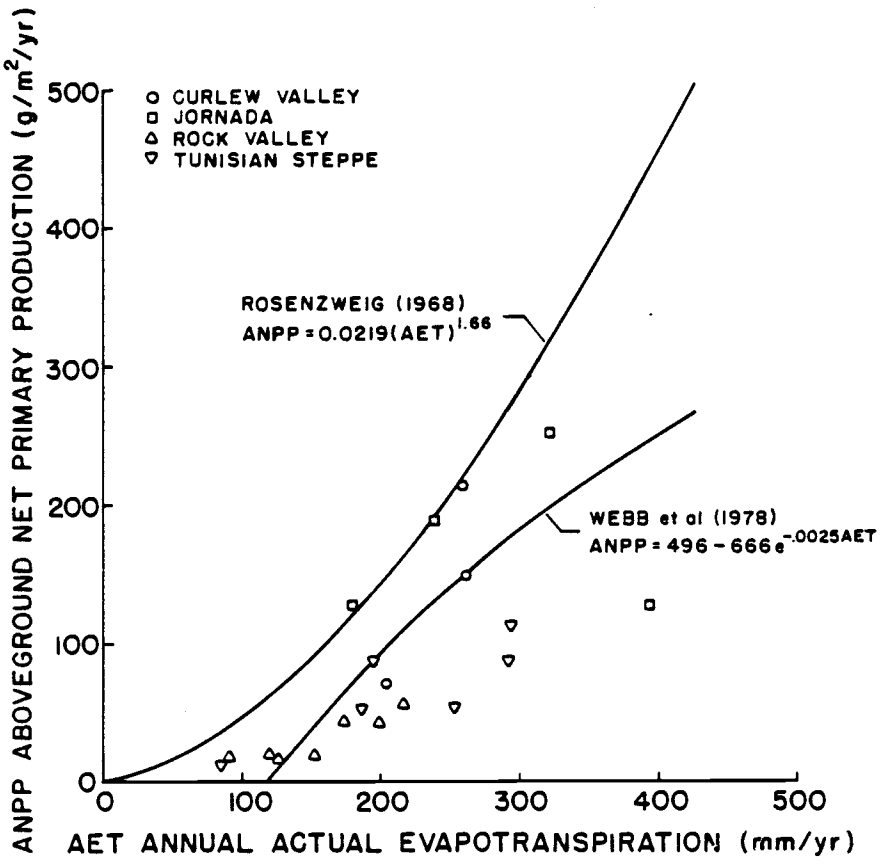

Figure 1.--Relation between AET and ANPP for selected data from desert and semidesert areas and proposed general relationships.

An expanded plot of the Rock Valley, Nevada data are shown in Fig. 2. The curve labeled 1 is the equation from Webb et al. (1978), and the curve labeled 2 is a linear regression between ANPP and AET. The least squares equation derived for this particular data set provides a better fit than the generalized equation which is intended for broader application over wider ranges of AET. For comparison, annual actual transpiration, AT, is plotted against ANPP, as shown in the left portion of Fig. 2, identified as curve 3. If seasonal estimates of transpiration from a continuous simulation model are used, then the unexplained variance in ANPP is further reduced (R^2 = .90), as described by Lane et al. (1983). The slope of the line relating AT and ANPP can be used as an approximate estimate of the water use efficiency, WUE, provided the intercept is near zero. From curve 3, in Fig. 2, the WUE for Rock Valley data is about 0.7, which is well within the range of values suggested in Table 3 (WUE values of 0.42 to 1.30).

If the product of AT and WUE provides a first order approximation of ANPP (curve 3 in Fig. 2), then relationships, such as curves 1 and 2 in Fig. 2 and the curves shown in Fig. 1, provide a somewhat coarser approximation using AET as an approximate predictor for actual plant water use, AT. The relationship between ANPP and AET is inherently weaker than the one between ANPP and AT, because AET includes bare soil evaporation, and is thus an imperfect predictor for AT or plant water use. However, less information is required to estimate AET than is required to estimate AT, so that the choice of approximating relationships depends upon the degree of accuracy required, the amount of information available, and the intended scale of application.

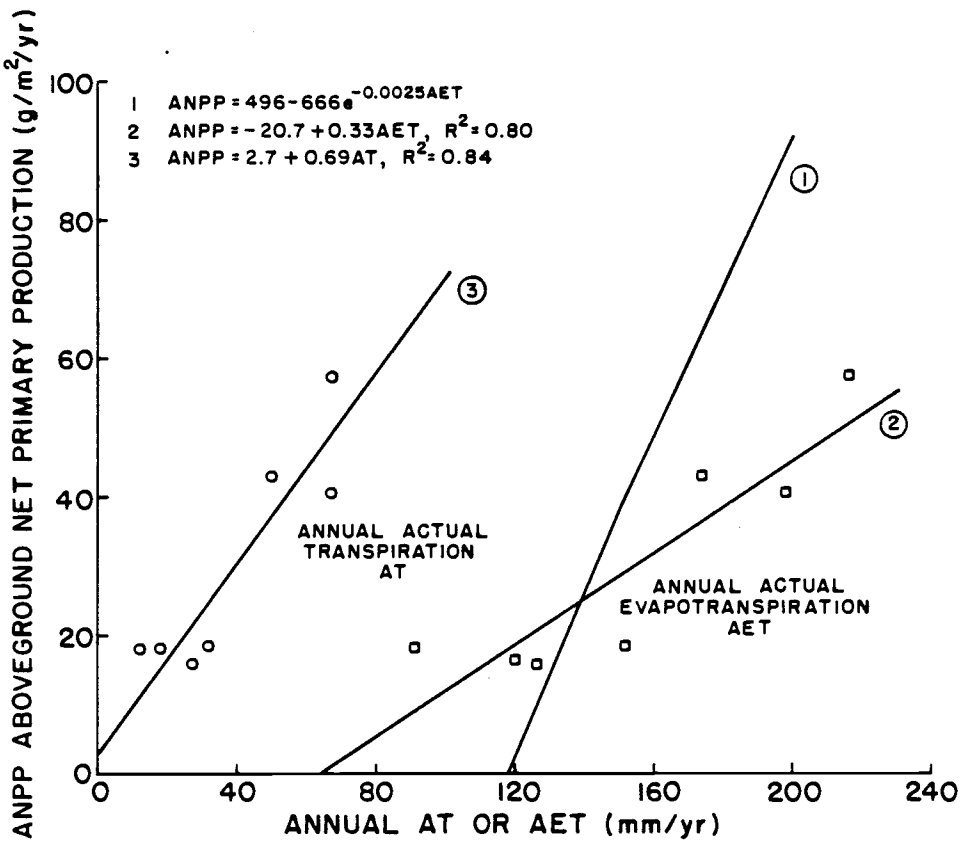

Figure 2.--Relation between annual AT, AET, and aboveground net primary production for Rock Valley, Nevada.

SUMMARY

Water balance calculations are important in soil-water-plant relationships studies and essential to predict net primary production when water availability is the limiting factor. These circumstances are often realized on arid and semiarid rangelands so that the water balance equation can be used together with soils data and water use efficiency factors to estimate net primary production. Equations (1) and (2) describe a simple water balance in a discrete form, selected soil properties are summarized in Tables 1 and 2, and water use efficiency factors are presented in Table 3. Figures 1 and 2 illustrate relationships between actual evapotranspiration, actual transpiration, and annual aboveground net primary production. Depending upon scale of application, available information, and desired accuracy, relationships such as these can be used to estimate annual aboveground net primary production of perennial vegetation.

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WATER HARVESTING: AN ALTERNATIVE USE FOR RETIRED FARMLANDS

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Introduction

Tucson, a rapidly growing city with a current metropolitan population of about 500,000, is located within the Sonoran Desert of the southwestern United States. Beautiful scenery, mild winters and vast open spaces are expected to attract an additional 350,000 people by the year 2000 (The Comprehensive Plan for the City of Tucson, Pima County, The City of South Tucson, and Pima Association of Governments, 1975). Groundwater is the area's primary resource impacted by current demands.

Tucson, in the Upper Santa Cruz Sub-Basin as shown in Figure 1, relies totally upon groundwater for its water supply. The city currently pumps about 90,000 acre-feet per year (AFY) of groundwater from the Santa Cruz Basin. Tucson shares its water supply with agricultural and mining users within the basin. Pumpage has exceeded natural recharge since the 1940s, and as a result water levels have been declining. Of the estimated 395,000 acre-feet withdrawn in 1980, agriculture and mining accounted for 272,000 acre-feet (McNulty, 1982).

The total dependable supply for the Tucson Active Management Area (which includes the Upper Santa Cruz Sub-Basin and Avra Valley) currently does not exceed 160,000 AFY; therefore the difference, 240,000 AFY, must come from storage as aquifer over-draft. Importation of water through the Central Arizona Project and reuse of effluent will reduce, but not eliminate, the need for more groundwater to meet urban needs as the city continues to grow (McNulty, 1982).

Avra Valley, approximately 20 miles west of Tucson, is a separate groundwater sub-basin within the Tucson Active Management Area, and supports a predominantly agricultural economy. Groundwater development there began in the late 1930s and by 1954 supported about 30,000 acres of agriculture.

Since 1975 the City of Tucson has purchased and retired about 12,000 acres of farmland in the Avra Valley to allow transfer of groundwater to the city for municipal use. In fiscal year 1981 Tucson imported about 13,000 acre-feet of water from Avra Valley compared to 9,000 acre-feet in fiscal year 1982 (Bruce Johnson ,Tucson Water, 1982; personal communication).

The Problem

Retired Avra Valley farmlands present a management challenge and an untapped economic potential. Once farming ceases, Russian thistle (Salsola kali) infestation rapidly invades fallow ground. The young weeds emerge in response to spring rains and eventually grow to several feet in diameter. They are easily windblown and can cause great management problems to neighboring farmers (Karpiscak et al, 1981).

Fragile semiarid ecosystems such as those in Avra Valley present rehabilitation challenges once they have been disturbed and abandoned. These lands do not revert to aesthetically pleasing desert growth without considerable passage of time, if at all, and weeds and dust become problems (Figure 2). Range grasses are established with difficulty in relatively low annual rainfall regions (11 inches or less) where evapotranspiration is relatively high. Grasses do not survive prolonged drought without additional water.

Significance of Water Harvesting Agrisystem

In Avra Valley the average annual rainfall does not exceed 10 inches, and therefore agricultural crops cannot be grown there without the benefit of a suplemental water supply. Previously this source of additional water has been groundwater; however, this supply is no longer available for agriculture production on land retired by the City of Tucson. One possible management alternative is to concentrate existing rainfall from a large area (catchment) onto a cropped area and to store the excess water for later use. This practice is called water harvesting.

A five-acre water harvesting agrisystem was developed on retired Avra Valley farmland (Figure 3) by the University of Arizona, College of Agriculture, Department of Soils, Water and Engineering, in cooperation with the

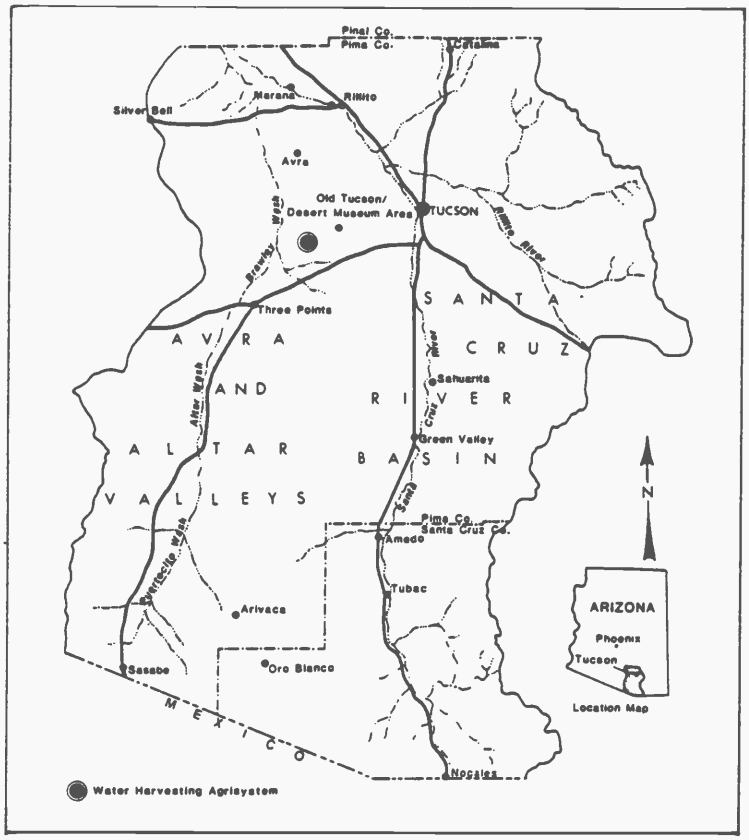


FIGURE 1. The Upper Santa Cruz Sub-Basin and Avra Valley.

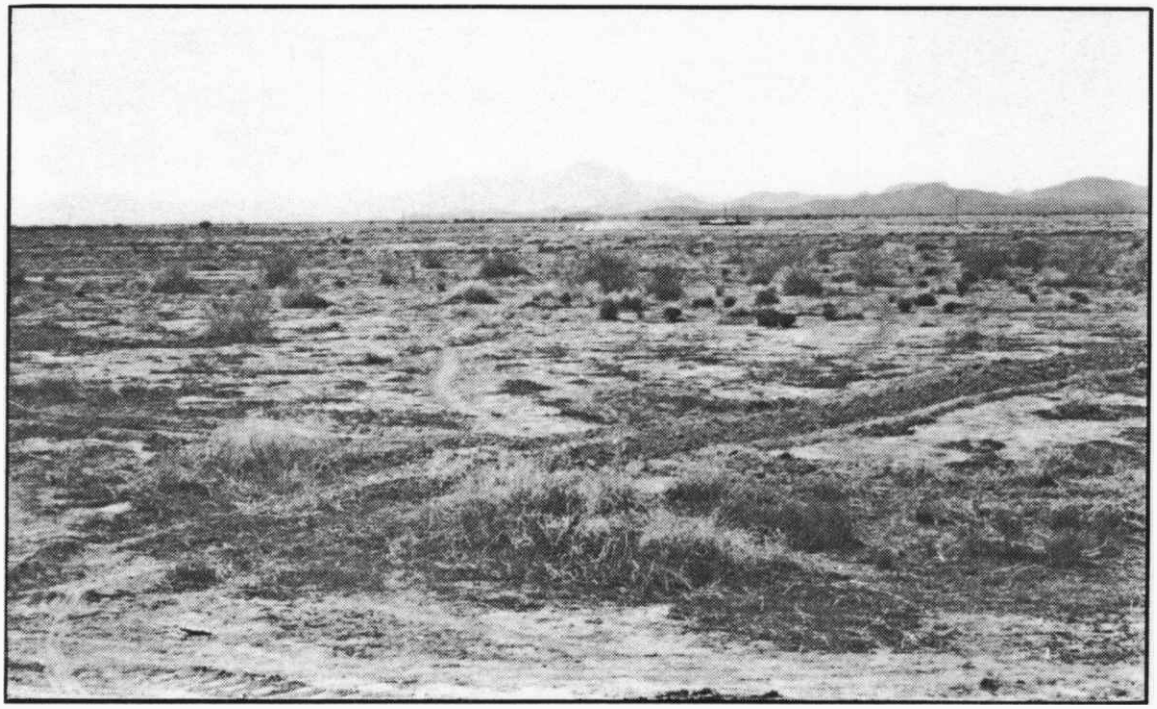


FIGURE 2. Retired farmland before construction of the Water Harvesting Agrisystem. Russian Thistle was the dominant plant with scattered mesquite and burroweed.

Office of Arid Lands Studies and the City of Tucson Real Estate Division. The system was established: (1) to demonstrate a potential and economic use for retired farmland, (2) to indicate the city's commitment to water conservation and land management by determining the feasibility of farming with rainfall as the only source of water for irrigation, and (3) to show the city's concern for all sectors of the water-using community. Crops grown in 1982 included jojoba, eucalyptus, olive, ornamental date palm, mondell and aleppo pine, Arizona cypress, corn, and watermelon.

Initial evaluation of the program indicates that the salt-treated, compacted-earth water harvesting system is a technical success. Data collected from continued monitoring, modifications and expansion of the system will substantiate the economic potential of water harvesting systems, water conservation strategies and plant species for potential economic use of retired agricultural land.

Although this project is primarily directed toward the economic agricultural use of City of Tucson property, the related conservation techniques, if adopted, will affect beneficially all of the area's major water users. Agricultural water users will benefit even if the irrigation, cropping and soil treatment technology used in the demonstration system is only partially practiced. More significantly, the reduced irrigation water need will benefit all. Municipal supply can be augmented with variations of the system. Water harvesting systems can be used directly as a source of potable water or indirectly by aquifer recharge. Mining interests also may benefit by using some of the techniques developed in this program. Future research will explore the potential for water harvesting techniques within the urbanized area of Tucson and for aquifer recharge.

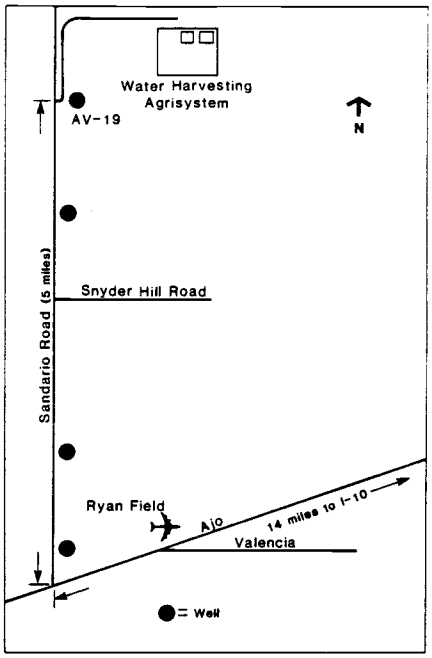


FIGURE 3. Location of the Water Harvesting Agrisystem.

The Agrisystem

The Theory

The quantity of runnoff increases per unit area as the size of watersheds decreases. Applying this principle to catchments, Shanan and Tadmor (1979) found that, under given climatic and soil conditions, runoff per unit area was increased from 10 to 30 times when catchment areas were reduced from 12,500 to 0.05 acres. The principle reason for this increased water yield is the reduction in water loss by infiltration preceding and following peak flow. In general, water yields increase when the time-to-peak-flow is minimized.

Runoff also can be increased by reducing the infiltration rate, which in turn reduces the time it takes for the flow to reach its peak. Both techniques result in an increase in runoff and a proportionate increase in the water-supply rate. Infiltration rates can be reduced from catchments by removing weeds, adding Na^+ and compacting soil (Baver, 1961). Common table salt (NaCl) is usually the least expensive source of Na^+, which increases the conductivity of water, retards weed growth, and decreases the infiltration rate (Dutt and McCreary, 1974).

Site Selection

The success of a water harvesting agrisystem depends primarily on the soils on which it is constructed. In arid and semiarid regions, soils should be deep (usually more than 5 feet) to store water in the crop area. Soils also should be clayey (moderate-to-high clay content, usually greater than 20 percent in the B, or argilic, horizon) to shed runoff on catchment areas. In the catchment area, this clayey, fine-textured soil should be close enough to the surface to be exposed by grading or plowing. In the crop area, it needs to extend deep enough below the surface to provide adequate soil-moisture storage for the crops being grown. The soil should be of mixed clay-mineral content (kaolinite, illite and montmorillonite) or low in montmorillonite to avoid excessive cracking. Soil samples taken at the site confirmed the necessary structure needed for construction (Appendix A).

Topographically, the site should be fairly level and slopes should not exceed 10 percent to minimize erosion.

Catchment Configuration

The land surface was shaped as shown in Figure 4. Catchment widths vary as shown in Table 1. The length of run for each catchment is 300 feet, with a slope of 0.5 percent. Rainwater runs from the ridged portion into adjacent drainages planted with the various crops. Excess runoff is collected in the shallow sump by a cross ditch (collecting channel) located north of all catchments and is pumped into the reservoir for use during prolonged periods of scant precipitation. Figure 5 shows a diagram of the various crops in relation to the sump, reservoir and irrigation system.

TABLE 1

CATCHMENT CONFIGURATION

Catchment	Crop	Length* (ft)	Total Width 2b + c (ft)	Planting Area Width c (ft)	Catchment Slope a/b x 100 (5)
1	Vegetables	300	46	4	2
2	Eucalyptus	300	46	4	2
3	Jojoba	300	46	4	2
4	Jojoba	300	46	4	2
5	Grapes	300	46	4	2
6	Grapes	300	46	4	2
7	Grapes	300	46	4	2
8	Vegetables	300	60	12	4
9	Vegetables	300	60	12	4
10	Vegetables	300	60	12	4
11	Dates/Olives	300	44	12	2
12	Aleppo Pines	300	32	4	3
13	Mondell Pines	300	20	4	10
14	Mondell Pines/ Arizona Cypress	300	44	4	2
15	Mondell Pines/ Arizona Cypress	300	32	4	3
16	Mondell Pines/ Arizona Cypress	300	20	4	10

*Slope = 0.5 percent.

TABLE 2

PARTICLE SIZE ANALYSIS OF SOIL FROM SUMP/RESERVOIR SITE*

	Depth of Sample (ft)	Sand (%)	Silt (%)	Clay (%)	Soil Classification
Sample 1	0-2	24.9	31.3	43.8	clay
	2-4	21.5	32.4	46.1	clay
	4-6	18.6	42.4	39.4	silty clay loam
	6-8	22.3	40.6	37.1	clay loam
Sample 2	0-2	21.5	32.4	46.1	clay
	2-4	15.9	39.2	44.9	clay
	4-6	31.6	26.8	41.6	clay
	6-8	32.2	41.4	26.4	loam

*Soil, Water and Plant Tissue Testing Laboratory, University of Arizona (April 16, 1982).

Construction

A site was selected on the former Growers Finance Farm now owned by the City of Tucson (Sec. 27, T14S, R11E). Soil samples were taken at the proposed location of the catchments, sump and reservoir (Appendix A). Samples taken from the sump/reservoir site and cropping area indicated that the soil ranged from a clay in the upper 2 feet to a loam 6 to 8 feet below the surface (Table 2). The area was first cleared with a front end loader. Then, in order to increase infiltration, the soil was ripped using rippers attached to the rear of the front end loader. Construction water was then applied from a city well (Avra Valley #19) to raise the soil moisture to near field capacity in the upper 18 inches of soil in the catchment and 36 inches of soil in the sump and reservior area. The application of water was necessary because the soil was extremely dry during the time allocated for construction and proper soil moisture was required to make the soil workable with the equipment designated for construction.

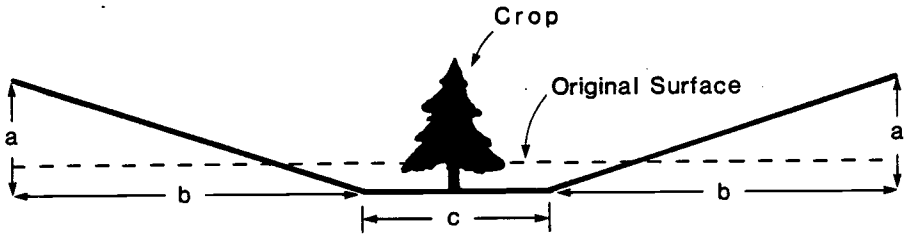


FIGURE 4. Catchment configuration.

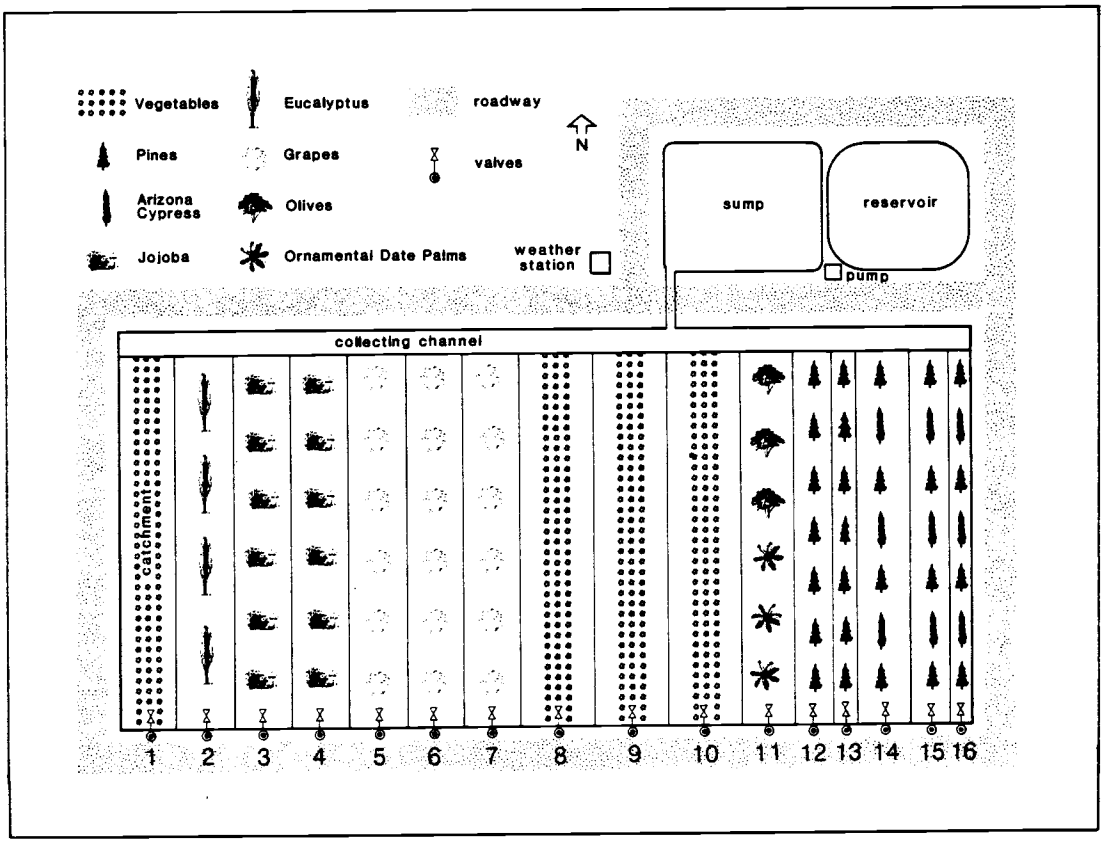


FIGURE 5. City of Tucson Water Harvesting Agrisystem

Sump and Reservoir: The combined design capacity for the sump and reservoir is approximately 2 acre-feet of water. Figure 6 shows the designed side slopes, depth, and bottom dimensions for the sump and reservoir.

To provide a consolidated and uniform soil mixture for the base of the reservoir, soil was first removed from the reservoir site to a depth of about 1 foot below the land surface. A 9 cubic yard earth mover was used. Care was taken to ensure that the depth did not go below 2 feet or penetrate the clay layer, which forms an effective water seal on the bottom of the reservoir. The reservoir walls were then erected using soil excavated during the construction of the sump (Figure 7). A small bulldozer was used to complete the side walls of the sump (Figure 8) and reservoir.

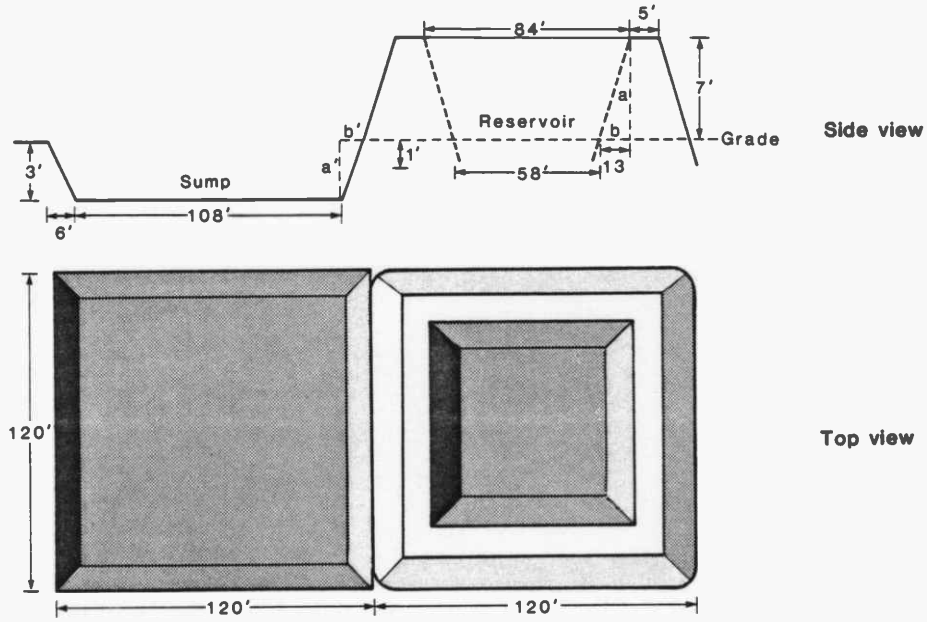


FIGURE 6. Sump and reservoir configuration.

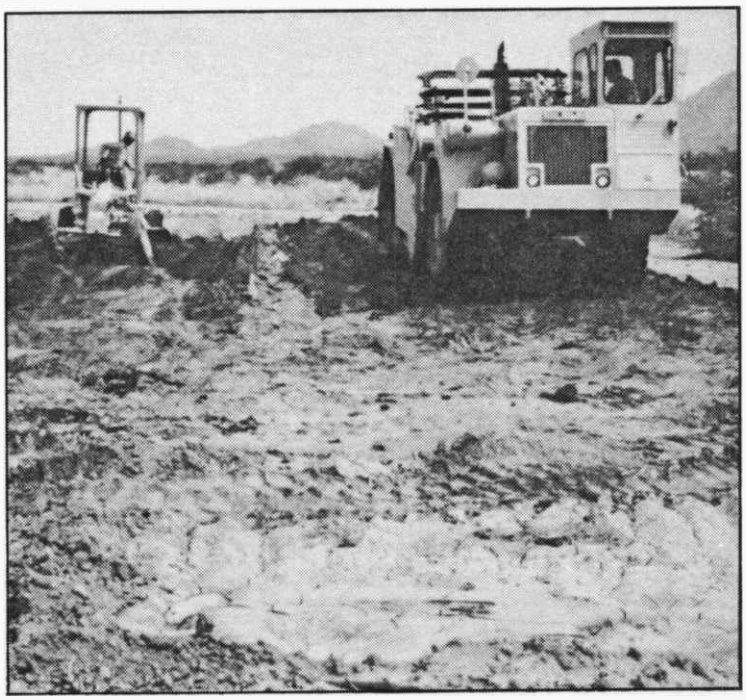


FIGURE 7. Heavy equipment used during reservoir and sump construction included a 9 cubic yard earthmover and a road grader.

Catchments: In order to compare different construction techniques, including time and equipment required for construction, two different approaches were used for catchment construction. Catchment 1 was constructed using a motorized road grader. Shaping began by tilting the grader blade and pushing material out from the cropped area to the ridges. A front end loader was used to remove excess soil from the planting aresa. A rototiller was used to break up the soil in the planting area before planting.

Catchments 2 through 16 were constructed by a multiple-pass operation with conventional farm machinery. The soil first was ripped in both east-west and north-south directions using a 100 horsepower farm tractor. Water was then applied by sprinkler to a depth of approximately 18 inches. Each catchment was plowed three times so that soil was moved from the center of the catchment to the outside, thus creating a slope toward the center planting area (Figure 9). After plowing, the surface was disked using a 72-inch tandem disk pulled by a 30 horsepower tractor. A rock rake was used to smooth the catchment slopes, and in some instances additional smoothing of the catchment slopes with a road grader was required (Figure 10).

Salt (NaCl) was applied at a rate of 5 tons per acre using a fertilizer spreader (Figure 11) and was worked into the sloping surfaces of all 16 catchments with the rock rake (Figure 12). Following a rainstorm, a 10-ton roller/compactor was used to compact the catchment surfaces (Figure 13).

FIGURE 8. The sump being completed with a small bulldozer.

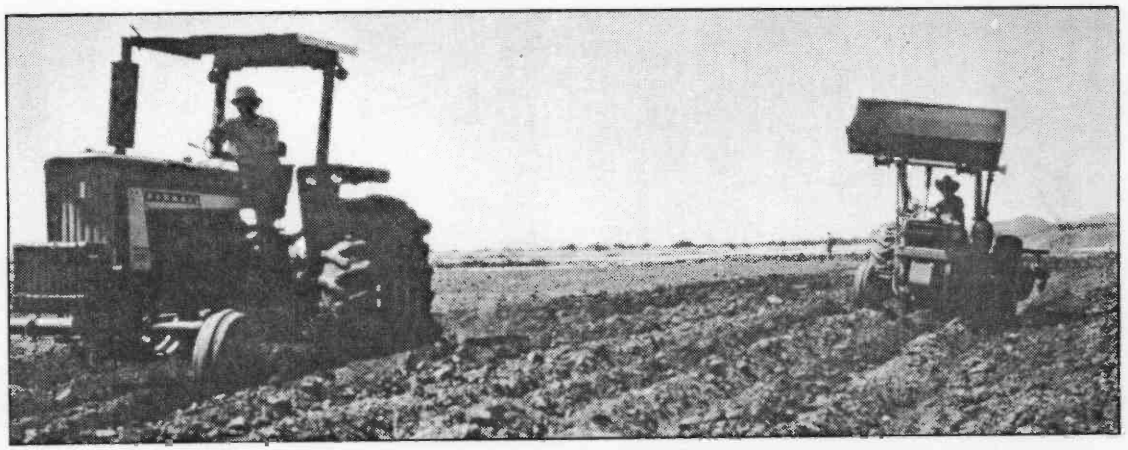


FIGURE 9. Plowing and disking catchments during multiple pass construction operation using conventional farm machinery.

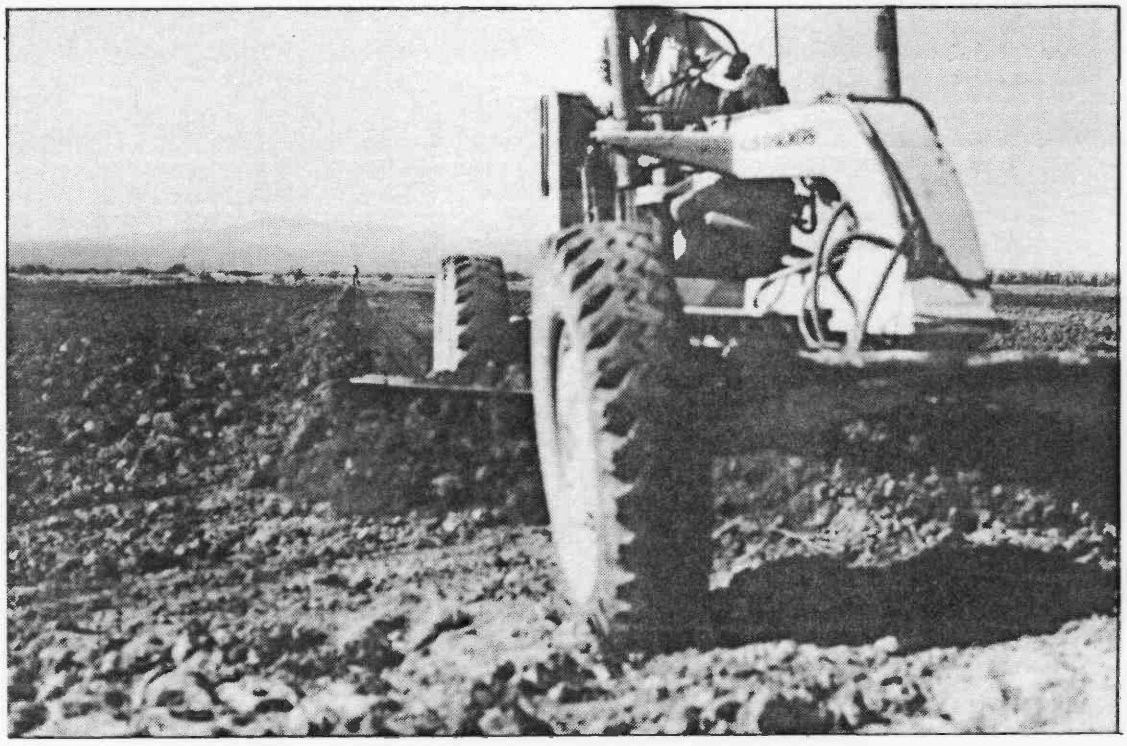


FIGURE 10. Using a road grader to smooth catchment slopes.

FIGURE 11. Applying salt (NaCl) at a rate of 5 tons per acre to catchments.

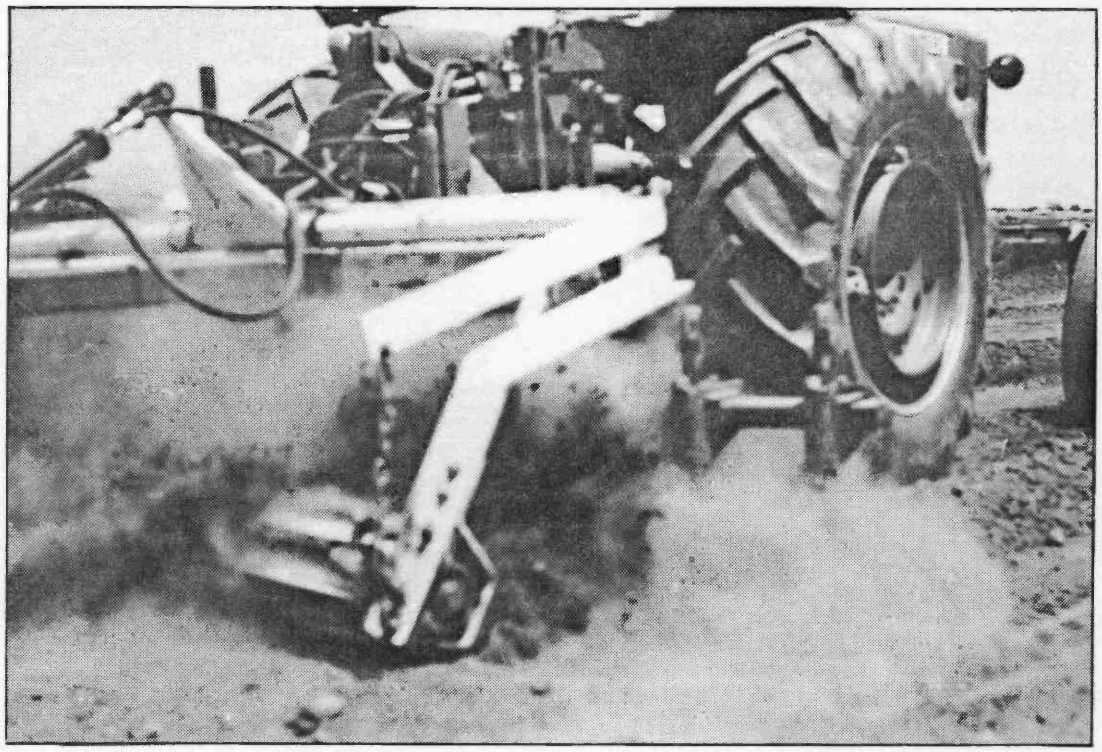


FIGURE 12. Working salt into surface of catchments with a rock rake.

FIGURE 13. Roller/compactor compressing salt treated catchments.

Irrigation System

The irrigation system consists of an 8 horsepower centrifugal pump, a 3-inch PVC pipeline connecting the sump and reservoir to the pump, a 2-inch PVC pipeline connecting the pump to the field plots, and 3/4-inch polyethylene driplines equipped with 2-gallons per hour (Reed Model E-2) drip emitters (Figures 14 and 15). The valving system permits movement of water from sump to reservoir, from reservoir to sump, and from either the sump or reservoir to the field. Screens on the input ends of the 3-inch pipeline help prevent foreign material such as weeds from entering the system, and a fine mesh filter is installed in front of the water meter. The water meter records the total amount of water applied to the plants. Sixteen 1-inch metal risers are capped with 1-inch gate valves at the head of each catchment. Polyethylene driplines are attached to the gate valves and staked to prevent the lines from moving. Removable screw caps are used at the end of each of the driplines and at both ends of the 2-inch PVC line to facilitate cleaning of the system. A breakdown of major irrigation equipment expenditures is presented in Table 3.

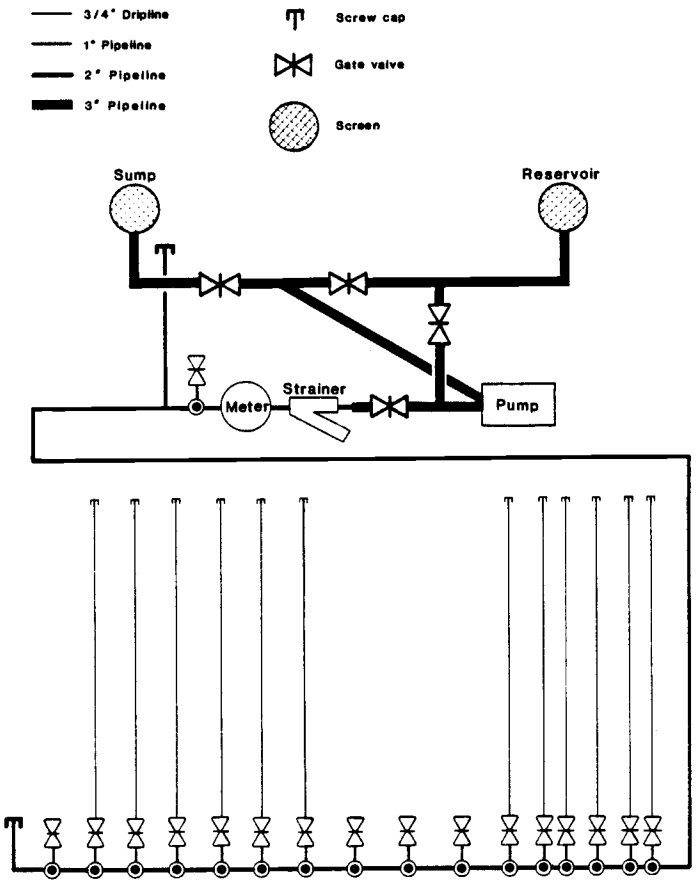


FIGURE 14. Irrigation system at Avra Valley Water Harvesting Agrisystem.

Figure 15. Installing irrigation system at the Avra Valley Water Harvesting Agrisystem.

TABLE 3

COST OF IRRIGATION CONSTRUCTION MATERIALS

ITEM	COST ($)
Pump (8 hp centrifugal)	$ 500
Polyethylene Tubing (3/4"; 3,600 ft)	325
Pipe (2-inch PVC; 1,200 ft)	300
Pipe 3-inch PVC; 100 ft)	60
Salt (25 tons)	950
Valves (3-inch; gate valves)	160
Emitters (2 gal/hr; 600)	100
Assorted pumping supplies	250
Miscellaneous supplies and equipment	600
TOTAL	$3,245

Planting

A 48-inch tandem disk cultivated the soil within the planting areas and destroyed any developing weeds. After cultivation and before transplanting, a 9-inch diameter power auger was used to bore holes. Boreholes were filled with water. Container and tube-grown seedlings (Table 4) were placed in the holes and were backfilled. Perennials were planted during the first ten days of September, 1982. The vegetables planted earlier included watermelon, corn, tepary bean, squash, guar, pumpkins and okra.

TABLE 4

SEEDLING CONTAINER SIZE AND AVERAGE HEIGHT AT TIME OF TRANSPLANTING TO FIELD

Catchment	Species	Plant Size	
		Container Size	Avg. Height (inches)
1	Vegetables	seed	—
2	Eucalyptus	1 gal	23
3	Jojoba	1 gal	17
4	Jojoba	1 gal	19
5	Grapes	1 qt	—
6	Grapes	1 qt	—
7	Grapes	1 qt	—
8	Vegetables	seed	—
9	Vegetables	seed	—
10	Vegetables	seed	—
11	Ornamental Date Palms/Olives	5 gal/2 gal	21/42
12	Aleppo Pines	5 gal	54
13	Mondell Pines	5 gal	16
14	Arizona Cypress/Mondell Pine	tubes*	13/12
15	Arizona Cypress/Mondell Pine	tubes*	15/12
16	Arizona Cypress/Mondell Pine	tubes*	13/11

*Some in one-quart containers.

Monitoring

A program to monitor the water harvesting agrisystem is ongoing. Reservoir water levels, evaporation rates, seepage rates, water quality, cropping area soil moisture, rate of plant growth and percent survival are being monitored.

Rainfall

The dates and amounts of rainfall are shown in Figure 16. Substantial runoff into the sump occurred only twice (September 10 and December 9-10, 1982). Total runoff into the reservoir systems from these two precipitation events was approximately 2 acre-feet. Minor amounts of additional runoff have been generated by other rainstorms.

Figure 17 shows the system in operation during the September 10 rainstorm. Note the concentration of runoff around the plants. Excess runoff filled the sump following the September rainstorm (Figure 18).

Reservoir Water Levels

The need to transplant seedlings during the hot, dry summer of 1982 required that water be available to sustain the plants. Therefore, standing water from a nearby drainage ditch was used initially to partially fill the reservoir. This surface-harvested water was collected from runoff caused by summer storms in adjacent upslope areas of the valley.

Cumulative seepage and evaporational losses following the initial filling of the reservoir are shown in Figure 19 and Table 5. Seepage losses from both the sump and reservoir appear to have stabilized. Total water loss from the reservoir was approximately 37 inches after 90 days; however, the percentage of loss due to seepage has decreased from 70 percent to less than 50 percent of the cumulative loss.

Changes in reservoir water levels are shown in Figure 20. The small aberration in early October and the rapid increase in water levels during December resulted from additional water that was pumped into the reservoir from the sump. A mathematical description of the reservoir water balance is presented in Appendix B.

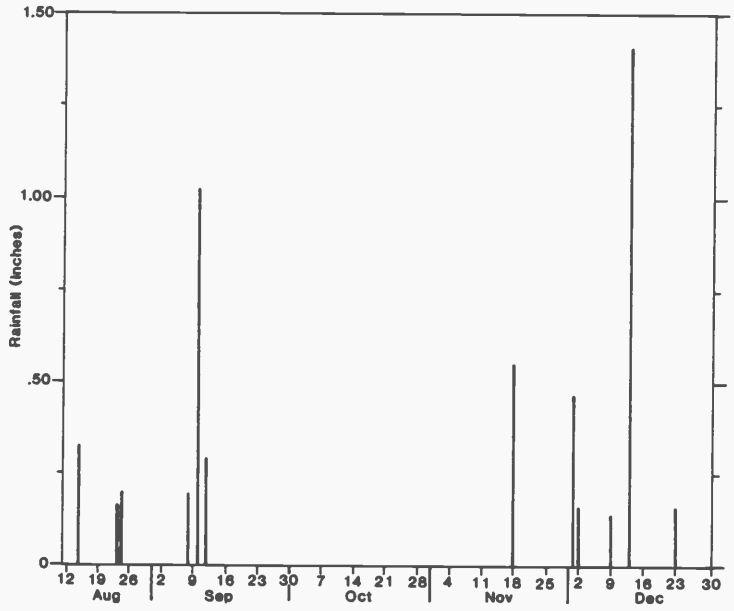


FIGURE 16. Measurable rainfall at Avra Valley Water Harvesting Agrisystem in 1982.

FIGURE 17. Mondell pine catchments filled with runoff from intense summer rainstorm, Sept. 10, 1982.

Total Dissolved Salts

Total dissolved salts (TDS) detected in the runoff from the December storm (225 ppm) were almost half of the amount observed in the water collected from the September storm (403 ppm). The reduction in TDS resulted from the stabilization of the salt-treated catchment by repeated wetting and drying cycles and from the probable decrease in rainfall intensity during the December storm.

Irrigation System

Emitters in the irrigation system occasionally become clogged with suspended sediments and foreign substances such as pipe threading compound. Clogging is virtually eliminated by using an emitter that discharges a stream of water rather than drops. Damage to one emitter was caused by an animal, probably a coyote.

Plant Survival

Plant survival following transplanting to the field was excellent. Many of the aleppo and mondell pines displayed fall growth. Less than 10 percent of the grape seedlings required replacement. One ornamental date palm, which appeared to have died shortly after transplanting, recently has shown signs of recovery.

Growth of corn and melons, especially watermelon, was good when adequate moisture was available. Additional plantings of guar and tepary bean also were good. Other vegetable seed planted failed to germinate or to become established probably because of outdated seed, inadequate moisture and the late date of planting.

APPENDIX A

Particle Analysis of Samples from Soil Horizons A,B, and C.

THE UNIVERSITY OF ARIZONA
College of Agriculture
Department of Soils, Water and Engineering
Soils, Water and Plant Tissue Testing Lab.
Tucson, Arizona 85721

SPECIAL ANALYSIS

COLLECTOR:
Name C. Sands II
Address S.W.E.
Zip
DATE April 7, 1982

BILL TO: (Authority of Dr. W. Gardner)
Name Dr. Brent Cluff
Address Water Resources Research Center
Zip
PO/Fund No.

COMMENTS:
Mechanical Analysis
Moisture Tensions
Please Save Samples.

Sample ID	Substance Sampled	Analysis Requested	Lab. No.	Sand (%)	Silt (%)	Clay (%)						
A1			8213 1563	50.6	19.9	29.5		Sandy	Clay	Loam		
A2			1564	48.2	20.7	31.1		Sandy	Clay	Loam		
B1			1565	49.4	20.5	30.1		Sandy	Clay	Loam		
B2			1566	50.5	20.9	28.6		Sandy	Clay	Loam		
C1			1567	40.3	23.4	36.3			Clay	Loam		
C2			1568	45.3	20.5	34,2		Sandy	Clay	Loam		
C3			1569	26.0	26.9	47.1			Clay			

APPENDIX B

Reservoir Water Balance at Avra Valley Site

In arid areas such as Avra Valley, where surface water must be stored for supplemental irrigation, a system for determining a reservoir water balance and sizing for reservoirs must be employed. Proper sizing of irrigation reservoirs must start by considering all inflows and outflows to and from the reservoir as shown in Figure B-1.

The inflows are precipitation and catchment runoff given by

Precipitation (Qp) + Catchment runoff (Qro)

1. $Qp = PR \times A$ reservoir

   $Qro = RO \times A$ catchment

   PR = precipitation (inches) daily

   RO = runoff (inches)

   A = area in square feet or acres

The outflows are evaporation, seepage, and irrigation demand given by

Evaporation (Q_e) + Seepage ($Qs_1 + Qs_2$) + Irrigation demand (Qi)

2. $Q_e = PET \times A$ reservoir

3. $Qs_1 = 4Kh^2C/9L$

   Qs_1 = seepage discharge through reservoir sides

   K = hydraulic conductivity

   h = reservoir base to present water level

   C = constant

   L = mean length of seepage line (1/3 height)

   PET = potential evaporation

4. Qs_2 = seepage discharge through bottom

5. $Qi = I \times A$ crop/effs

   I = water applied (inches)

   A crop = area of irrigated land (square feet or acres)

   effs = efficiency of the irrigation system

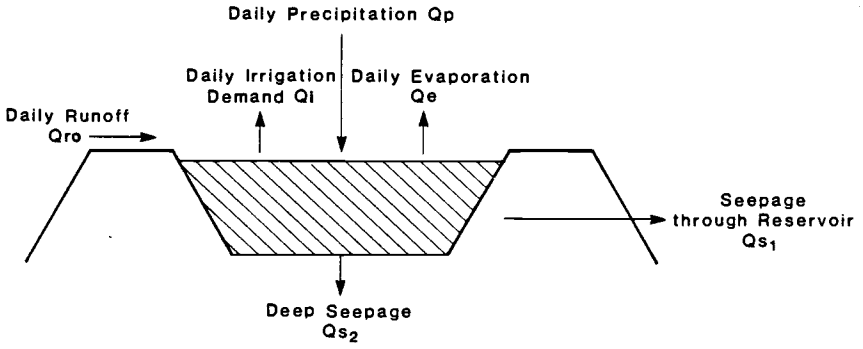


FIGURE B-1. Water balance of an irrigation water supply showing all inflows and outflows.

The daily balance incorporating these inflows and outflows gives

$$\Delta Sj = Qroj + Qpj - Qej - Qs_1j - Qs_2j - Qij$$

The storage at any date is given by

$$S = \sum_{j=1}^{n} \Delta Sj$$

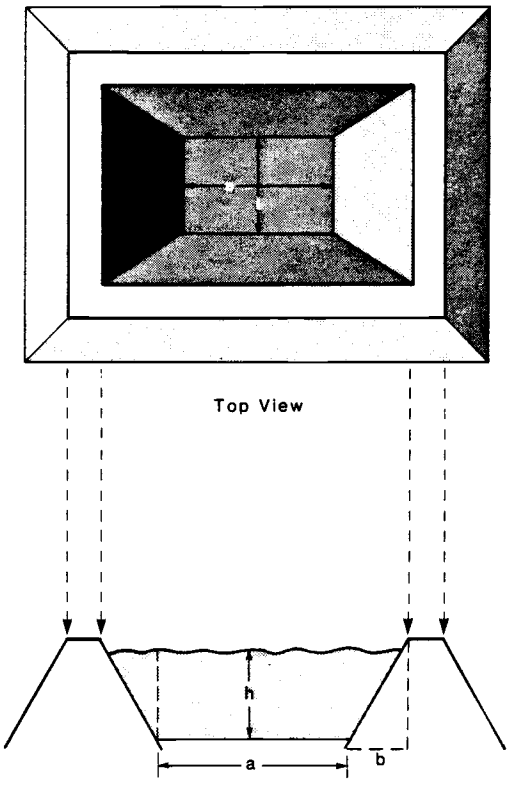
n = number of days since reservoir was last full

S = in cubic feet, acre-feet or gallons

In order to size the reservoir, an initial reservoir height is assumed and daily reservoir balances are calculated starting at the time of the simulated period.

Once the reservoir is sized the model can then be used to determine the number of days water will be available for irrigation. Treatments can also be incorporated to reduce evaporation and for seepage, thus offering a modification to the number of days water will be available for irrigation.

Figure B-2 shows a schematic of reservoir size, side slopes and storage capacity.



S= ahl+ 4(1/2 bh)l
S= hl(a+2b)
a= bottom length (ft.)
l= bottom width (ft.)
b= function of slope

FIGURE B-2. Reservoir schematic.

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A Hierarchical Model for Arizona's Water Resources

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Abstract

Arizona's water resources system consists primarily of four active management areas (Tucson, Phoenix, Pinal and Prescott), the Central Arizona Project, and the Salt River Project. The problem of water allocation among user categories involves pumping from aquifers and diversions of surface flows. In systems less complex than Arizona, allocation policies may appear obvious. In this case, however, a two-level hierarchical management model is proposed to control water allocation to users: the active management areas as a lower echelon, and the Arizona Department of Water Resources at the higher level. A system theoretic approach combined with recent developments in the decentralized control theory are proposed to be included in the model. A significant characteristic of the proposed model is the ability to consider possible interactions among the active management areas as a result of policy decisions at the State level. A dynamic optimization model based on a state space formulation with total energy required as the objective function is solved for each of the subsystems. Detailed information thus generated at the regional level is then appropriately aggregated for statewide decision making. An iterative algorithm is suggested.

Introduction

Management of water resources at the State level is becoming increasingly more complex so that systems theoretical concepts are very useful in analyzing the issues involved (Major and Lenton, 1979). The application of these concepts to water resource systems analysis and planning has been a matter of considerable interest in recent years (Loucks et al., 1981) leading to a variety of mathematical models. In many instances, state-space models (in contrast with decision-space models) seem to be particularly useful because of the possibility of defining clearly the state of the system (e.g., groundwater levels, volume of water stored in a surface reservoir). Thus a number of models exist for groundwater aquifers (Burt, 1976) as well as for surface water resources (Maidment and Chow, 1981). At the same time, input-output models are being developed for water resources management (Hendricks and Dehaan, 1981), as well as models of large-scale systems using decomposition techniques (for example, see Haines, 1977). Lastly, state and dynamic optimization formulations exist for water resources management problems. However, dynamic optimization (Buras, 1972) seems to be more accepted by water management agencies because of the dynamic nature of most variables identified in a large number of water resources systems. Typical optimization criteria for dynamic models are energy requirements, needed storage capacity, water shortages or water spills.

The study of socio-economic frameworks, whether at the national or at state level, often require the construction of complex, large-scale models. However, the use of large-scale models in a planning exercise of whatever scope is still an infrequent event and there are serious reservations regarding their possible use in this context (Dantzig 1981). One such reservation arises from the observation that planning and management models are often designed to achieve a higher rate of performance in resource use, without much consideration of the behavior and reaction of various decision makers at low levels of the hierarchy, who are the actual users of the resource. It appears, therefore, desirable to attempt, at least, a reformulation of large-scale planning models so as to represent regional problems at an adequate level of detail, and linking these sub-systems in an effective manner with the broader decision-making framework.

The Problem

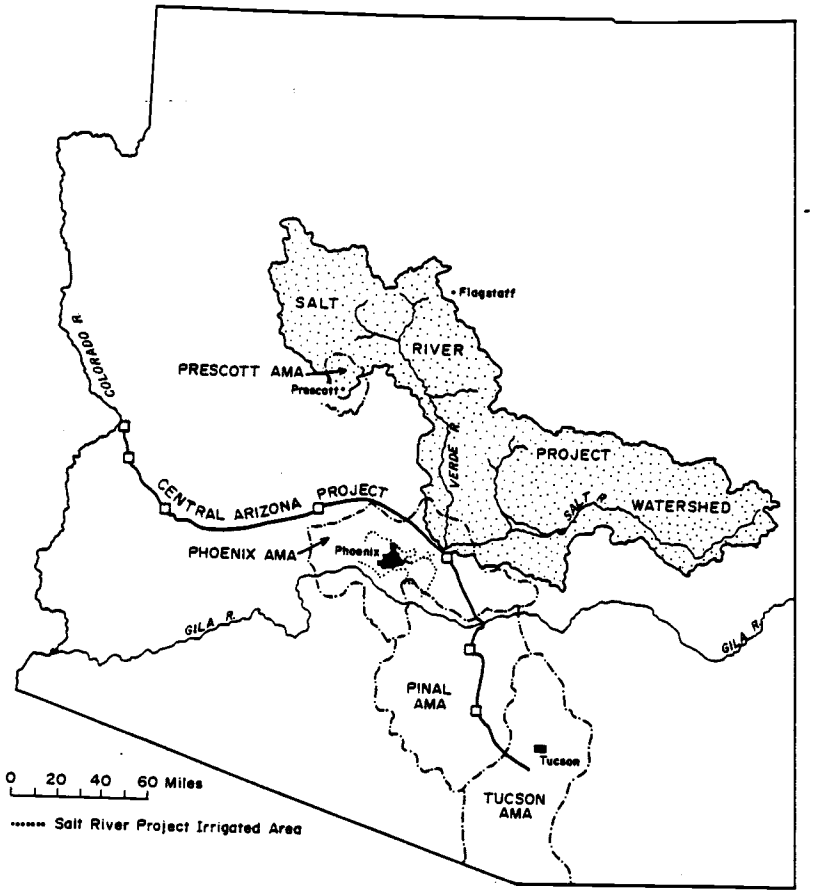
Arizona's water resources present a picture of considerable complexity. The area of the State, about 113,900 square miles (291,600 sq. km), makes it the sixth largest in the U.S. The average precipitation is approximately 13" (330 mm), with about half the State receiving less than 10" per year (250 mm). Of an estimated average of 80 million acre feet (Maf) of precipitation falling over Arizona each year (98.7 billion cubic meters), more than 95 percent is lost to evaporation and plant transpiration (Arizona State Water Plan, 1975). The remaining 4 Maf/yr (4.9 Bcm/yr) represent the yearly runoff, both surface and subsurface. It is estimated that the yearly groundwater recharge amounts to 0.3 Maf (370 million cubic meters) and the dependable surface flows do not exceed 2.5 Maf (3.1 Bcm).

However, the groundwater which accumulated in the Arizona aquifers over a period of several millennia represents a considerable resource. Within the top 700 ft (about 200 m) below land surface, the groundwater is estimated to amount to 794.55 Maf (980 Bcm); the next 500 ft (about 150 m) contains an estimated quantity of 407.25 Maf (502 Bcm). The total of 1.201 Maf (over 1,480 Bcm) represents a substantial water resource which, if managed judiciously, could last several centuries.

Current water utilization in Arizona amounts to about 4.8 Maf/yr. (5.9 Bcm), 89% of which is used in irrigated agriculture, 7% by municipalities and industries, 3% by mines and the energy sector, and about 1% to maintain fish and wildlife. There is a strong inclination by the State water managers to improve the efficiency of water use in all the economic sectors of the State (Arizona State Water Plan, 1977, 1978).

The economic development of Arizona relies heavily on groundwater; yet significant surface water supplies were developed early in this century by the Salt River Project, and another major project--the Central Arizona Project--is currently under construction for the diversion of about 1.2 Maf/yr. (1,480 Bcm/yr) from the Colorado River. Nevertheless, aquifers will remain major suppliers of water and their rational exploitation is an issue of major concern, as evidenced by the promulgation in 1980 of the Arizona Groundwater Management Act. Under this law, the Arizona Department of Water Resources was established, as well as four Active Management Areas: Tucson, Phoenix, Prescott, and Pinal (Arizona Department of Water Resources, 1981).

A map showing the major elements of the Arizona Water Resources systems appears in Figure 1.



ELEMENTS OF THE ARIZONA WATER RESOURCES SYSTEM

Figure 1

The major components of the State water system can be divided into three groups:

- The Salt River Project (SRP);
- The Active Management Areas (AMA's);
- The Central Arizona Project (CAP).

These groups also represent three organizations connected rather loosely: (i) the Salt River Users' Association (established in 1903); (ii) The Central Arizona Water Conservation District which was established to operate the CAP; (iii) Arizona Department of Water Resources (ADWR). Now, the decision-making process related to the operation of AMA's takes place at several levels within the ADWR. For the purpose of this analysis, a two-level hierarchy of decision-making is considered, as shown in Figure 2.

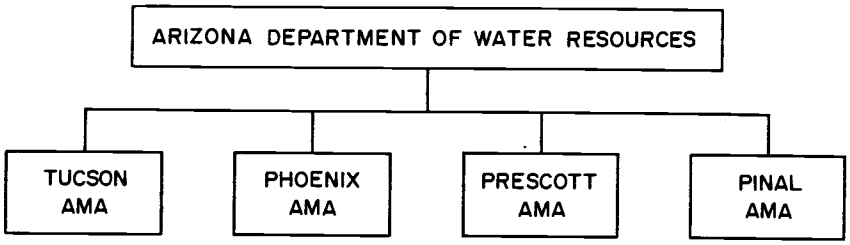


Figure 2

Management of Groundwater Resources in Arizona

This simplified scheme, which reflects the bulk of the water used in Arizona, could be extended to also include the CAP (Yeh et al., 1980) and the SRP. The model proposed in the next section is within the framework of dynamic optimization, to which recent developments in decentralized control theory (Siljak, 1978) could be applied. The problem can now be formulated as follows: What are the alternative policies available to water managers in Arizona, both at the central (State) and at the regional (AMA) levels, given the scarcity of the resource, the possible interactions between components, and the stochastic nature of the demand?

The Model

The formulation of the problem indicates its complexity, yet any operating decision must consider the reliability of system's performance. A basic approach to the reliability problem in water storage systems is mentioned by Sniedovich (1979). A simple concept related to the reliability of dynamic systems states (Siljak, 1978) that

"A dynamic system composed of interconnected subsystems is reliable if all subsystems are self-sufficient and their interdependence is properly limited."

Here "interdependence" is interpreted as interaction between subsystems, such as those between aquifers and surface streams (Illangasekare and Morel-Seytoux, 1982). In the case of the Arizona Water Resources, the interactions between the subsystems are shown in Figure 3. An important detail in this diagram is that aquifers are main sources of water for each AMA. The AMA's are connected with ADWR by a two-way link: the AMA's transmit to ADWR information regarding their respective state (including demand to be satisfied), and ADWR may issue guidelines for groundwater abstraction.

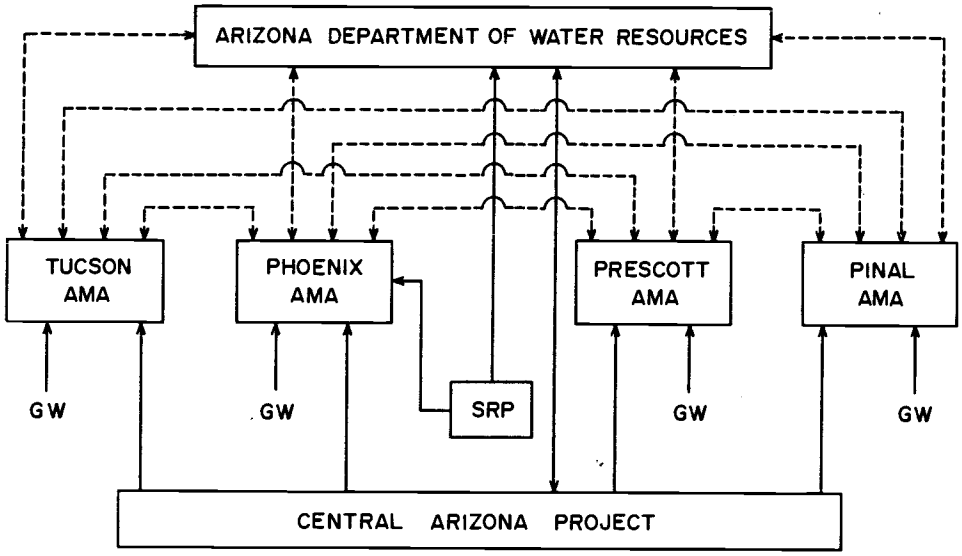


Figure 3

Schematic representation of Arizona Water Resources System

Colorado River water will be supplied to all four AMA's, either directly (Tucson, Phoenix, Pinal) or indirectly by an exchange arrangement via SRP (Prescott). A two-way connection is assumed to exist between CAP and ADWR: information regarding flows diverted from Colorado to CAP is transmitted continuously to ADWR; ADWR could modify CAP allocations to AMA's

The SRP supplies water--both from surface diversions and from aquifers--only to the Phoenix AMA, which overlaps much of the SRP irrigated area. The connection with ADWR is mostly indirectional: water-related information flows from SRA to ADWR.

The scheme shown in Figure 3 is modeled using a deterministic formulation. Let the vector X_i

$$x_i = (x_{i1}, x_{i2}, x_{i3})^T , \; i = 1,2,3,4, \tag{1}$$

represent the state of subsystem i, where i=1 is associated with Tucson AMA, i=2 with Phoenix AMA, i=3 with Prescott AMA, and i=4 with Pinal AMA. Here x_{i1} is the average depth to water table, x_{i2} is the availability of CAP water, and x_{i3} is the availability of SRP water. Clearly

$$\begin{aligned} x_1 &= (x_{11},x_{12}, 0)^T, \\ x_2 &= (x_{21},x_{22},x_{23})^T, \\ x_3 &= (x_{31},x_{32},0)^T, \\ x_4 &= (x_{41},x_{42},0)^T. \end{aligned} \tag{2}$$

Let

$$B_i = (b_{i1}, b_{i2}, b_{i3})^T, \; i = 1,2,3,4 \tag{3}$$

be the net "recharge" of the water supply in subsystem i. Here b_{i1} is groundwater recharge; b_{i2} is the diversion from the Colorado river to CAP theoretically allocated to the i-th AMA; and b_{i3} is the SRP target allocation to the Phoenix AMA (i=2).

Define the control and decision vectors p_i

$$p_i = (p_{i1}, p_{i2}, p_{i3})^T, \; i=1,2,3,4, \tag{4}$$

to be the pumping and delivery rates in subsystem i from groundwater (p_{i1}), from the CAP (p_{i2}) and from SRP (p_{i3}). These control and decision variables are mapped onto the state vectors through function $f_k(\cdot)$, k = 1,2,3.

Given a finite time interval (0-t_f), the basic dynamics of the subsystem AMA i are given by the equations

$$x_{ik}(t+1) = x_{ik}(t) - f_k(p_{ik}) + b_{ik}, \; i=1,2,3,4, \; k = 1,2,3. \tag{5}$$

These equations represent a situation in which the subsystems i are quasi-independent. However, should an AMA require more water from CAP while another AMA may be over-supplied, the ADWR could adjust the control variables p_i so as to maintain the stability of the system. These adjustments generate interactions hij between the subsystems, which have to be taken into account. Thus, the dynamic model of subsystem 1 is

$$x_i(t+1) = x_1(t) - F_1(p_i) + B_1 + h_{12}\cdot x_2(t) + h_{13}x_3(t) + h_{14}\cdot x_4(t). \tag{6}$$

The rate at which the state of subsystem 1 changes is given by

$$\dot{x}_1 = x_1(t+1) - x_1(t) = - F_1(pi) + B_1 + \sum_{j=2}^{4} h_{ij}\cdot x_j(t). \tag{7}$$

In general, the dynamics of subsystem i is expressed by

$$\dot{x}_i = - F(pi) + B_i + \sum_{\substack{j=1 \\ j\neq i}}^{n} h_{ij}\cdot x_j(t). \tag{8}$$

Equation (8) represents the behavior of the subsystems, components of the Arizona water resources system.

The management problem at the statewide decision-making level is formulated in terms of dynamic optimization. A possible optimization criterion is the minimization of an energy cost matrix associated with the control and decision vectors p_i, $E(p_i)$. Thus, the objective function to be optimized is

$$\underset{p_i>0}{\text{Min}}\ J = \int_0^t E(p_i)dt. \tag{9}$$

The minimization problem (9) is subject to the following constraints:

(a) demand, d_i, $\int_0^t p_i dt \geq d_i$ (10)

(b) minimum "storage" level, s_i, $x_{ij} \geq s_{ij}$, (11)

(c) subsystem components capacities c_{ij}, $p_{ij} \leq c_{ij}$ (12)

Thus the two-level hierarchical model for Arizona's water resources consists of the dynamic optimization (9), subject to the constraints (10), (11) and (12), where the dynamics of the component subsystems are given by equations (8).

Future Work

The conceptual model given by equations (8) through (12) needs to be refined on the basis of actual data. A crucial set of variables which have to be numerically evaluated are the interactions h_{ij}. Details of past performance of groundwater basins currently aggregated into the Active Management Areas should yield the necessary insights in these interactions.

The functions F_i, B_i and E should be formulated explicitly: E is probably a nonlinear function of p_i, while F_i and B_i could be represented adequately by linear relationships. Similarly, the right-hand side of the constraints (10), (11) and (12) need to be evaluated numerically for each subsystem (AMA).

Finally, an efficient computational algorithm will have to be developed to solve the two-level hierarchical model (8) through (12), leading eventually to an interactive computer program which could be used by decision-makers at the State level for managing Arizona's water resources.

Summary

An important issue recently identified in operating large-scale systems is the integration of detailed information generated at a lower (subsystem) hierarchical level in an effective manner for decision-making at a higher (e.g., statewide) level (Buras, 1982). The hierarchical model for the management of Arizona's water resources suggested in this paper represents an initial step in the analysis, understanding, and resolution of this issue.

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TUCSON'S NEEDS FOR CENTRAL ARIZONA PROJECT STORAGE

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Abstract

The future acceptance and utilization of Central Arizona Project water by the City of Tucson Water Utility present many complex technical, economic, institutional, and environmental problems. Since Congressional adoption of the Colorado River Basin Project Act in 1968, Tucson Water engineers have supported the concept of a large CAP raw water storage reservoir near Cat Mountain west of the City. The United States Bureau of Reclamation, in its Stage Two planning for Phase B of the Tucson Aqueduct, has identified four potential storage sites, including the Cat Mountain location, for economic and environmental evaluation in conjunction with two basic aqueduct alignments. Engineers of the municipal water utility have utilized available computer tools to develop a preferred CAP delivery location and elevation economically advantageous to water rate payers.

This paper discusses the various factors associated with Tucson's projected need for CAP water storage including reliability, operational flexibility, water quality, shortage, and power management. Each of these factors will affect the degree to which the water utility can successfully assimilate Central Arizona Project water into its groundwater supply system. Although a decision regarding storage location and volume has been postponed for the present, the initial years of CAP usage by the City of Tucson will provide sufficient test to justify the decision for no storage or prove its necessity.

Introduction

Eastern Pima County contains a wide diversity of water-using entities, each solely dependent upon groundwater for its current water needs. Groundwater pumpers include the City of Tucson, four mining companies south of the City, Farmers Investment Company (FICO), Cortaro-Marana Irrigation District, Avra Valley Irrigation District, private water companies, and private well owners. The collective impact of all users pumping groundwater five times in excess of natural recharge in the region is water table decline. Impacts of water table declines include increased energy costs, reduced well capacity, reduced water quality and increased potential for land subsidence. In portions of the Avra Valley and the Upper Santa Cruz basin water levels have dropped more than 120 feet over the last thirty years.

Statewide groundwater problems and the threat by the Federal Government to withhold funds to complete the Central Arizona Project encouraged the Arizona State Legislature to adopt the Groundwater Management Act on June 12, 1980. Primary elements of the Act affecting municipalities in general and the City of Tucson specifically have been discussed in a previous paper by this author (Davis, 1982). Four Active Management Areas (AMA's) were created by the statute including the Prescott, Phoenix, Pinal, and Tucson AMA's. The goal of the Tucson AMA is to attain safe yield (the long-term balance of groundwater withdrawals with natural and incidental recharge) by January 1, 2025. Staff of the Tucson AMA have recently prepared an updated baseline estimate and projection of existing and anticipated water uses and supplies from 1980 through 2025. This data, summarized in Table 1, demonstrates the groundwater mining problem which currently exists within the AMA and which will persist without effective management of all available water resources. Potential water supplies for the Tucson Active Management Area include Central Arizona Project water, reused wastewater, and natural and incidental recharge. In 1980, approximately 426,000 acre-feet of groundwater and wastewater effluent were used in the Tucson AMA. Analysis of Table 1 data indicates that 56.3 percent of the total 1980 water use was agricultural, 22.5 percent was municipal, 14.1 percent was mining, and 7.1 percent was industrial. Under this "no groundwater management scenario" prepared by Tucson AMA staff, overdraft is projected to decline from 261,000 acre-feet in 1980 to 111,000 acre-feet by 2025. Key assumptions include full utilization of wastewater effluent by 1990, gradual conversion of some private agricultural land to urban uses, new irrigation of 5,000 acres of San Xavier and 2,000 acres of Schuk Toak Papago Indian land after CAP delivery to Pima County, and full utilization of CAP water by requesting entities in the amount recommended by DWR and recently approved by Secretary of the Interior, James Watt.

Tucson Water is a municipally-owned and operated water utility providing domestic and industrial water service both inside and outside the corporate limits of Tucson. Water is provided to a service area of over 300 square miles having an elevation differential of over 1,500 feet. Presently, Tucson Water serves over 450,000 people through 131,000 active water services. This represents eighty percent of the Pima County population and over eighty-five percent of the metropolitan Tucson population. In 1981-82, Tucson Water actively pumped 186 wells from four wellfields serving the central metropolitan system and 23 wells serving 13 isolated systems. Sixty-six percent of the 83,000 acre-feet pumped by Tucson Water in 1981-82 was from the central, interior wellfield, and eleven percent was from the Avra

Valley west of Tucson. The remainder was pumped from the Southside and Santa Cruz wellfields near the riverbed south of Tucson.

Tucson Water pumpage statistics for the past ten years are listed in Table 2. The implementation of an increasing block, cost-of-service water rate structure, a winter-summer rate differential, the "Beat the Peak" summer demand management program, and public education through the news media have collectively reduced the Tucson per capita use from 205 gallons per day in 1973-74 to 165 gallons per day in 1981-82. Tucson Water uses population projections officially adopted by the Arizona Department of Economic Security and the Pima Association of Governments for Pima County. The December, 1981 DES figures indicate a County population of 536,000 in 1980 and a 2035 projection of 1,807,000. The corresponding Tucson Water population and water use projections are shown in Table 3. Water use for municipal purposes is projected to more than triple from an average of 68.5 million gallons per day in 1980 to 257.3 million gallons per day in 2035 based upon an assumed average daily per capita use of 160 gallons. To meet projected demands of existing and future customers, Tucson Water will have three primary supplies: groundwater, municipal wastewater effluent, and Central Arizona Project water. The focus of this paper is discussion of the incorporation of Central Arizona Project water into the Tucson Water domestic delivery system and the requirement that raw water storage be ultimately provided within the vicinity of metropolitan Tucson.

The Central Arizona Project

The completion of the Central Arizona Project (CAP) to the Tucson metropolitan area represents the primary ingredient to the formulation of a future water supply plan for the community (McLean and Davis, 1981). Legislation which ultimately resulted in federal approval for the CAP was enacted by Congress in 1921 when the seven Colorado River basin states were authorized to negotiate a compact for the apportionment of Colorado River water. It wasn't until 1944 that Arizona approved the 1923 Colorado River Compact and the 1928 Boulder Canyon Project Act allocating 2.8 million acre-feet of Colorado River's annual flow to Arizona. It wasn't until 1968 when President Lyndon Johnson signed Public Law 90-537, The Colorado River Basin Project Act, that the Central Arizona Project was finally authorized by Congress.

The CAP is a water delivery system which will furnish municipal, industrial, and irrigation water to urban, mining, and agricultural areas within Maricopa, Pinal, and Pima Counties. The CAP is a massive U.S. Bureau of Reclamation water resource development project designed to help solve central Arizona's problem of dwindling groundwater supplies, the overdraft of which exceeds two million acre-feet per year. The importation plan includes construction of a series of pumping plants and aqueducts to lift Colorado River water from Lake Havasu and carry it to the metropolitan areas of Phoenix and Tucson. The project will deliver an average of 1.2 million acre-feet per year via the main aqueduct systems. The Granite Reef Aqueduct will carry water in an open canal from Lake Havasu 190 miles to the Salt River just below the Granite Reef Dam. The concrete-lined canal will continue from the south side of the Salt River as the Salt-Gila Aqueduct for approximately 60 miles through agricultural areas of eastern Maricopa County and central Pinal County and terminate east of the Picacho Reservoir in southern Pinal County.

The Tucson Aqueduct consists of two segments, labelled Phase A and Phase B by the Bureau of Reclamation. Phase A planning is completed, and construction will commence in 1984 to bring the aqueduct southerly to the town of Rillito. Planning for Phase B continues in furtherance of meeting the USBR schedule shown below:

ACTION	DATE
Determine Alignment and Terminus	July, 1983
File Draft Environmental Impact Statement	November, 1984
File Final Environmental Impact Statement	August, 1985
Initiate Construction	Late 1985
Initiate Water Delivery	1991

Tucson Aqueduct-Phase B

Tucson Water has been intimately involved with the planning for the Tucson Aqueduct since 1978 when the Arizona Projects Office assigned a full-time planning engineer to study this major CAP feature. Advance planning for Phase B was initiated in May, 1981 through evaluation of numerous storage and alignment alternatives and preparation of preliminary economic, environmental, and institutional impacts. The number of alternatives were reduced significantly after public meetings in November, 1981. Since that time, the Arizona Projects Office, in conjunction with the major water users in Pima County, has performed further economic and environmental analyses leading to elimination of many Phase B alternatives from further consideration by the Bureau of Reclamation.

In early 1982, the Southern Arizona Water Resources Association (SAWARA) was formed to assist the community in the assurance of a stable, long-term water supply and effective management thereof for the Tucson Active Management Area. A major objective is to support the timely completion of the Central Arizona Project for the benefit of all the people of the State. In June, 1982, SAWARA was approached

by the Bureau to assist in the development of consensus relative to the elements of alignment, terminus and storage associated with Phase B. In response, the Committee on Alignment, Terminus, and Storage (CATS) was formed in August, 1982. Initially, the Bureau presented the CATS with seven plans for its review and consideration as indicated below:

SUMMARY OF USBR PLANS
TUCSON AQUEDUCT-PHASE B

Designation	Alignment	Reservoir	Volume
Plan 1	West of Tucson Mtns.	San Joaquin	10,000 A-Ft
Plan 2	West of Tucson Mtns.	Cat Mountain	10,000
Plan 3	West of Tucson Mtns.	None	0
Plan 4	East of Tucson Mtns.	Marana	10,000
Plan 5	East of Tucson Mtns.	Cat Mountain	35,000
Plan 6	East of Tucson Mtns.	None	0
Plan 7	-No Action Alternative-		

In an attempt to assist the Mayor and Council develop a position with regard to the seven Bureau plans, Tucson Water staff utilized its computerized hydraulic modeling capability to evaluate the annual capital and operating costs associated with delivery of its CAP water to various locations and elevations (Davis, 1980). Assumptions used to generate model results included a fifty-year planning horizon (2035), Arizona Department of Economic Security population projections for Pima County, a per capita daily water use of 160 gallons, peak day demand condition, average day of the peak month supply condition, an annual municipal CAP allocation of 159,000 acre-feet, and a water treatment plant capacity of 190 million gallons per day for single plant alternatives and a hydraulic equivalent for dual plant alternatives. Figure 1 indicates the locations and elevations assumed by staff for Tucson's CAP water treatment plant or plants consistent with the seven plans proposed by the Bureau. Four single plant and two dual plant options were modeled. On the basis of model results, capital and operational cost impacts of various treatment plant alternatives are indicated in Table 4. The least expensive treatment plant location is the Cat Mountain site at an elevation of 2750 feet from both a total capital cost standpoint (in 1982 dollars) and an operational cost standpoint (based on 1982 power rates and 2035 water delivery). This computation, together with storage, blending, and flexibility considerations, has prompted staff to favor the USBR Plan 5 alternative for an east side alignment and a 35,000 acre-foot Cat Mountain Reservoir.

Raw Water Storage Considerations

The Colorado River Basin Project Act of 1968 authorized the construction of the Charleston Dam and Reservoir and the San Pedro Aqueduct partly as a supplemental source of water to serve Tucson. In 1978, Congress was notified by the Commissioner of the USBR that the San Pedro Aqueduct had been eliminated from the Tucson Division of the CAP due to anticipated growth of the Fort Huachuca-Sierra Vista area. At that time it was proposed to extend the Tucson Aqueduct to the south and to add terminal storage to facilitate improved operation of the Tucson Aqueduct and its integration with the Tucson Water system. In view of the aforementioned, Tucson Water staff reviewed the four storage alternatives proposed by the USBR to determine which met Tucson's requirements for reliability, operational flexibility, water quality, and buffer against CAP shortage. Additionally, the future opportunity for power management on the part of the Central Arizona Water Conservation District was evaluated.

As previously discussed, the USBR presented three storage volumes for consideration: 35,000 acre-feet, 10,000 acre-feet, and no surface water storage for which Tucson's water wells would be used to satisfy the criteria heretofore delineated. Under the no surface water storage alternative it was assumed by the USBR that Tucson's wells could be used to provide supply reliability for the period in which CAP delivery is interrupted. The USBR assumed the potential for a two week delivery interruption due to power outage, subsidence cracking affecting the delivery facility, or other maintenance requirements. In 2025, Central Arizona Project water is projected to meet two-thirds of the water demand anticipated for the growing Tucson Water service area. The loss of supply in this amount at the single location of the treatment plant would cause significant pressure problems, potential water outages, and a reduction in fire suppression capability if the CAP outage occurred in the summertime. In addition, new wells drilled in strategic locations would be required at considerable utility cost and constrained by as yet undefined well drilling limitations of the Tucson Active Management Area under the Arizona Groundwater Management Act. For these reasons, storage in some significant quantity must be provided by the USBR, particularly as CAP becomes the primary source of potable supply for the community.

The Bureau of Reclamation has provided two storage volumes for economic and environmental evaluation at three potential sites. The 10,000 acre-foot alternative was computed by the Bureau to be that volume capable of meeting municipal CAP water demands in the Tucson area for a two-week period. This reliability storage would have to remain untouched throughout the year, preserved to meet any CAP delivery interruption. Tucson Water has no operational flexibility with this amount of storage unless allowed by Bureau or Central Arizona Water Conservation District operating policy. Storage volumes larger than 10,000 acre-feet would allow the Utility to take water from storage when customer demands

are greater than CAP aqueduct deliveries and to put water into the reservoir when demands are reduced at night or when it rains, for example. Storage for operational flexibility provides a buffer source to the water treatment plant when projections of CAP water need vary from actual customer requirements. The examination of raw water storage facilities of several similarwater agencies in the southwest suggests provision of two to three months storage for the Tucson area. This results in a recommended volume of 30,000 to 50,000 acre-feet.

Another benefit of storage in excess of that required for emergencies is water quality enhancement. A raw water storage reservoir provides for decreased fluctuation in water treatment plant influent water quality and, therefore, increased operational stability. Reservoirs often reduce turbidity, color and coliform levels. With storage, the potential exists for constructing a direct filtration plant rather than a conventional water treatment plant at a savings of as much as twenty percent in construction costs. This type of treatment process would also yield decreased chemical requirements and sludge production. If blending of groundwater with CAP water is required throughout the year to assure a more uniform water quality as delivered to the customer, then storage is necessary to deposit and withdraw the exact amount of CAP water to effectuate a consistent blend.

A fourth consideration for storage is the potential for CAP water shortages. The Bureau has indicated that shortages in the Colorado River system can be expected after the year 2000. Under the Colorado River Basin Project Act of 1968, Arizona bears the burden of these shortages. If a shortage occurs in 2035, Tucson's allocation of CAP water will be reduced from 150,000 acre-feet per year to approximately 70,000 acre-feet per year under the Bureau's proposed shortage operational mode. With this assumption, and full utilization of locally-developed groundwater sources, an additional 28,000 acre-feet of CAP water drafted from storage would be required for one year. If shortage persists, mandatory conservation is the only alternative. With a large reservoir, Tucson's customers will have the minimal time required to acclimate themselves to a major change in water use habits.

A fifth consideration for storage in the Tucson area is the potential for power management by the Bureau of Reclamation. To pump water from Lake Havasu the Federal Government has purchased 24.3 percent of the Navajo Generating Station in the Four Corners Region. This power has greater value if it can be sold for other uses in the summertime. This is the primary justification for large storage at the proposed New Waddel dam site near Lake Pleasant northwest of Phoenix. The Bureau's large storage option for Phase B would serve a similar purpose near Tucson. The reservoir could be filled in the wintertime and drained in the summertime for use by Tucson. The power savings could help offset the reservoir construction costs.

For all the reasons heretofor presented, on November 8, 1982, the Tucson Mayor and Council unanimously adopted a position in favor of construction of a 35,000 acre-foot reservoir at the Cat Mountain site as an element of Phase B of the Tucson Aqueduct. Additionally, it was recommended that the Bureau deliver water to Tucson at the Cat Mountain site at an elevation of 2750 feet, the most economical location and elevation for Tucson Water customers.

CATS Recommendations

The Committee on Alignment, Terminus, and Storage (CATS), per direction from its parent organization, the Southern Arizona Water Resources Association (SAWARA), adopted a consensus position on the Tucson Aqueduct Phase B on April 15, 1983. These recommendations were forwarded to the Bureau of Reclamation and the Central Arizona Water Conservation District for their mutual consideration. These proposals are included herein for information and future reference.

Alignment Considerations - CATS recommends that Phase B of the Tucson Aqueduct of the CAP follow a route west of the Tucson Mountain Park described by the Bureau as Route 6. It is further recommended that the aqueduct be split into two segments. One segment of the CAP aqueduct at 350 cfs would deliver municipal water to the desired elevation of 2750 feet at the City of Tucson treatment plant. The other segment of the CAP aqueduct would deliver water to the Papagos, mines and other southerly water users at the south boundary of the San Xavier Indian Reservation.

Terminus Considerations - As a consequence of the alignment recommendations, CATS recommends that two termini be designated - one at the south boundary of the San Xavier Indian Reservation and another at the City of Tucson water treatment plant site.

Storage Considerations - CATS recommends that no further considerations by the Bureau be given to storage in the Cat Mountain area. We make this recommendation with the recognition that as the community becomes increasingly dependent upon CAP water to meet its domestic needs, there will be a need for surface storage of not less than 10,000 acre-feet in the future. This need will occur sometime after the year 2000 when it is projected that there will no longer be sufficient well capacity to meet the municipal needs should CAP fail to deliver water to the Tucson area. Prior to the occurrence of this situation, a cost/benefit analysis for different storage volumes and locations should be conducted.

It is apparent that the above recommendations will be adopted and implemented by both the U.S. Bureau of Reclamation and the Central Arizona Water Conservation District. Tucson's long-held position favoring large storage in the Cat Mountain area has been abandoned as a compromise to achieve desireable delivery location and elevation at federal cost. Only time will test the decision to forego storage initially. Ultimately, CAP storage in the Tucson area will prove to be both necessary and cost-effective.

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TABLE 1

TUCSON ACTIVE MANAGEMENT AREA
WATER USES AND DEPENDABLE SUPPLIES
(APRIL, 1983)

	1980		2000		2025	
POPULATION	536,400		936,400		1,587,900	
IRRIGATED ACREAGE	48,000		43,000		31,500	
TOTAL WATER USE (A-FT)		%		%		%
MUNICIPAL	96,000	22.5	168,000	33.3	285,000	47.4
INDUSTRIAL	30,000	7.1	52,000	10.3	89,000	14.8
MINING	60,000	14.1	69,000	13.7	69,000	11.5
AGRICULTURAL	240,000	56.3	215,000	42.7	158,000	26.3
TOTAL	426,000	100.0	504,000	100.0	601,000	100.0
CONSUMPTIVE USE (A-FT)	336,000		373,000		425,000	
DEPENDABLE SUPPLIES (A-FT)		%		%		%
CENTRAL ARIZONA PROJECT						
MUNICIPAL/INDUSTRIAL	--		92,000	23.5	173,000	35.3
AGRICULTURAL	--		55,000	14.1	28,000	5.7
INDIAN	--		38,000	9.7	38,000	7.8
REUSED WASTEWATER	9,000	5.4	80,000	20.5	136,000	27.8
NATURAL RECHARGE	75,000	45.5	75,000	19.2	75,000	15.3
INCIDENTAL RECHARGE	81,000	49.1	51,000	13.0	40,000	8.1
TOTAL	165,000	100.0	391,000	100.0	490,000	100.0
OVERDRAFT (A-FT)	261,000		113,000		111,000	

TABLE 2

TUCSON WATER PUMPAGE STATISTICS

Fiscal Year	Peak Day Pumpage (MG)	Average Day Pumpage (MG)	Active Services	Service Population	Pumpage Per Capita Per Day
72-73	118.8	60.1	95,105	351,889	170.8
73-74	130.4	75.4	99,604	368,614	204.6
74-75	115.2	67.6	102,813	380,408	177.7
75-76	117.6	70.0	105,595	369,583	189.4
76-77	123.7	60.5	109,480	383,180	157.9
77-78	112.1	59.4	113,105	395,868	150.1
78-79	113.2	60.8	117,777	412,220	147.5
79-80	110.0	66.2	122,514	428,799	154.4
80-81	120.6	69.7	125,367	438,785	158.8
81-82	119.1	74.0	127,650	446,775	165.5

TABLE 3

TUCSON WATER SERVICE AREA PROJECTIONS

(1980-2035)

YEAR	PIMA CO. POPULATION	TUCSON WATER SERVICE POPULATION	AVERAGE DAILY WATER DEMAND
1980	536,100	428,344	68.54 MGD
1985	620,000	493,852	79.02
1990	710,100	599,275	95.88
1995	810,700	684,612	109.54
2000	921,900	822,293	131.57
2005	1,048,400	935,754	149.72
2010	1,174,900	1,047,803	167.65
2015	1,301,400	1,159,852	185.58
2020	1,427,900	1,271,903	203.50
2025	1,554,400	1,383,952	221.43
2030	1,680,900	1,496,001	239.36
2035	1,807,400	1,608,051	257.29

TABLE 4

ECONOMIC EVALUATION OF TUCSON DELIVERY ALTERNATIVES

ALTERNATIVE	CAT MTN. I	CAT MTN. /TANGR.	SAN JOAQUIN	MARANA	CAT MTN. /MARANA	CAT MTN. II
DEL. ELEVATION	2617	2617	2400	2060	2617/2060	2750
WTP. CAPACITY	190 mgd	166/77 mgd	190 mgd	190 mgd	166/77 mgd	190 mgd
CAPITAL COSTS						
WTR. SYSTEM	$183.3m	$187.9m	$204.8m	$247.9m	$196.6m	$173.3m
WTP	41.2	55.8	41.2	41.2	55.8	41.2
TOTAL	$224.5m	$243.7m	$246.0m	$289.1m	$252.4m	$214.5m
OPERATIONAL COSTS						
WTR. TREATMENT	$ 2.43m	$ 5.46m	$ 2.43m	$ 2.43m	$ 5.46m	$ 2.43m
BOOSTER POWER	4.11	4.24	7.85	12.18	6.12	2.99
TOTAL	$ 6.54m	$ 9.70m	$ 10.28m	$ 14.61m	$ 11.58m	$ 5.42m

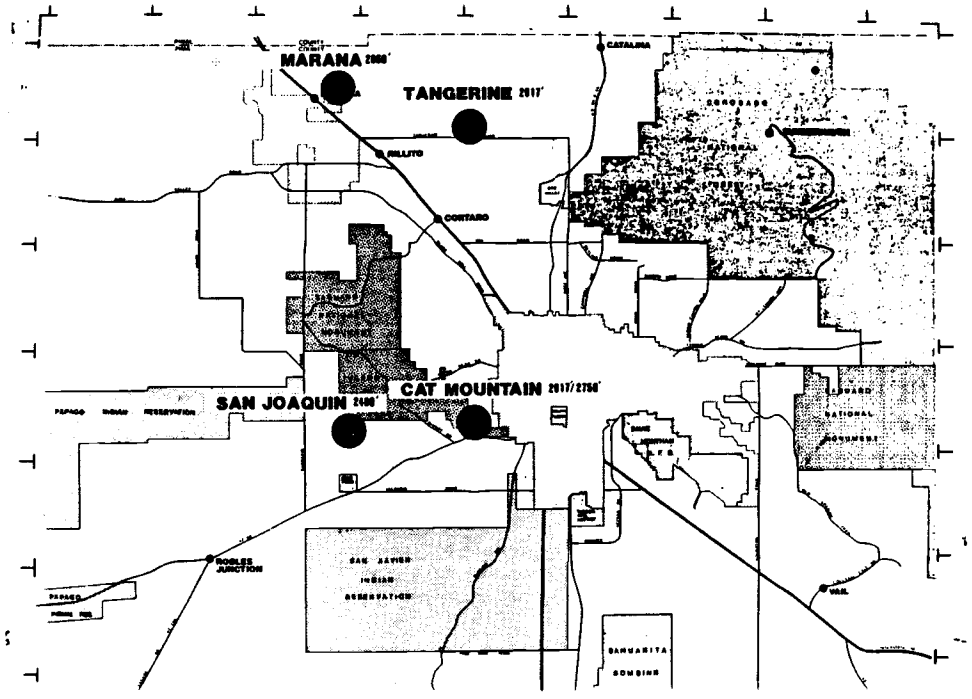

FIGURE 1. MAP OF MODELED CAP TREATMENT PLANT LOCATIONS

Models of Indoor and Outdoor Water Demand for Single Family Residences in Tucson, Arizona

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ABSTRACT

Structural models for single family residential water demand in Tucson, Arizona are identified using cross-sectional data from the 1980 census. Indoor and outdoor water usage are modeled separately. Indoor water demand appears to be a simple function of household size and fraction of the population that are children. Outdoor water demand is a function of several variables, including length of residence in the area and lot size. The indoor and outdoor models are shown to be significantly different. The impact of the results on forecasting water demand is discussed.

INTRODUCTION

The Tucson Active Management Area (TAMA) was created by the State of Arizona to manage the water resources of the Upper Santa Cruz and Avra-Altar valleys of southeastern Arizona. Currently, the major water users within the TAMA depend almost entirely on groundwater to meet their water demands and are pumping at about three times the natural recharge rate. Even after effluent is fully utilized as a source of water and the Central Arizona Project (CAP) reaches Tucson, groundwater pumping will still exceed recharge by a large margin.

The TAMA's long-range goal is to halt the decline of the water table by bringing the pumping rate in line with the recharge rate. A wide variety of policy options, including mandating the use of low-flow plumbing fixtures, promoting desert landscaping, prohibiting wasteful uses of water, and buying and retiring farmland, are being considered or implemented. In order to help make water planners aware of all the indirect effects of various water management decisions, and to help predict the long-term impacts of various demographic and economic trends on patterns of water use, the Division of Economic and Business Research (DEBR) at the University of Arizona is developing a computerized water budget model. This model allows various hydrologic and econometric relationships between supply and demand sectors to be described quantitatively. The authors do not intend to model all the relationships -- that would be an undertaking beyond DEBR's resources. Instead, the model is intended to be a framework for incorporating the data gathering efforts and analyses of others into a useful management tool.

One function of this framework is pointing out areas where data are lacking or further analysis is needed. When encountering such an area, the authors have gathered and analyzed data. For example, climatic factors such as temperature, precipitation, humidity, and evaporation have been expressed as functions of elevation in the TAMA so that better estimates of sources of recharge and net evaporative losses from open bodies of water can be obtained.

Another area of original analysis, which is the subject of this talk, is modeling residential water demand. This component of municipal demand accounts for about three-quarters of the water supplied by the Tucson Water Department (see Figure 1). Therefore, accurate projections of the residential component of municipal water demand are essential for effective water management. However, many residential water use projections are made by assuming that demand will increase proportionately with population. In other words, a current per capita water use figure is multiplied by a future population forecast to obtain a future water demand estimate. But water consumers are changing in many ways -- the population is getting older, household size is shrinking, incomes are rising, lots are getting smaller, and the percentage of children in the population is falling, to name but a few trends. To the

Figure 1

1980 Water Usage by Sector

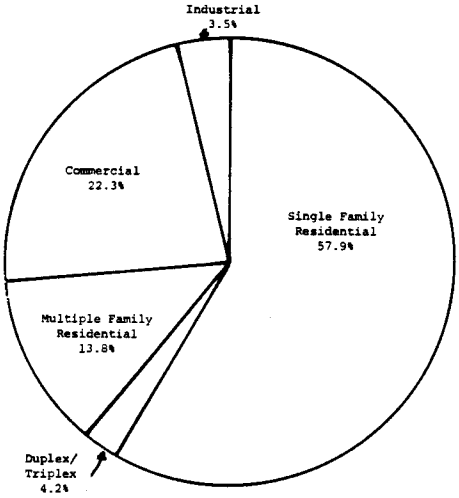

Figure 2

1980 Single Family Residential Daily Water Use by Month (Gal./Connection/Day)

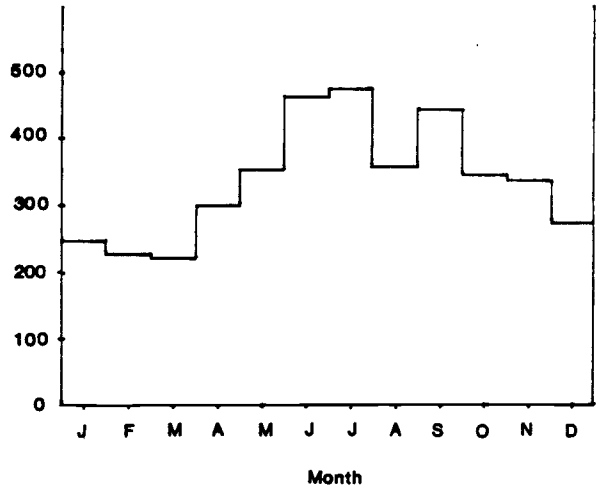

Figure 3

Total Water Use = Indoor Water Use + Outdoor Water Use

Indoor Uses	Outdoor Uses
shower/tub	turf irrigation
sinks	landscape irrigation
toilets	pools/jacuzzis
dishwashers	evaporative coolers
clothes washers	car washing
garbage disposals (most flows into sewage system)	pavement cleaning (most evaporates)

Dependent Variables Modeled

March water use ≈ Indoor water use

July water use = Indoor + Maximum outdoor water use

July - March water use ≈ Maximum outdoor water use

extent that these factors affect water consumption, such simplistic projections of water demand may be seriously in error. The models presented in this paper describe residential water demand in terms of some of these changing factors.

METHODOLOGY

In particular, Single Family Residential (SFR) water demand is analyzed, which accounts for some 57% of the total water demand in the Tucson Water service area. This is a cross-sectional study, with the basic unit of data being the census tract. Water consumption data are from Tucson Water; other data are from various 1980 U.S. Census reports and the 1980 Tucson Trends survey.

Certain census tracts have been excluded from our study, for one of three reasons:

First, some tracts have many apartment and duplex dwellers without individual meters, so that census data may or may not accurately describe SFR occupants.

Second, some census tracts are served entirely, or to a large extent, by other water companies, so that water consumption data are not available.

Third, a small number of census tracts are excluded because of the large number of 'snowbirds' (winter visitors) who spend time here but are not year-long residents and are excluded from the census.

This leaves about 45% of the tracts in Tucson, which contain approximately 60% of the city's single family residences.

Residential water demand is characterized by yearly cycles. Figure 2 presents average daily water use by month for single family residences served by Tucson Water in 1980. Water use is typically at a minimum during March. It peaks in June or July, depending on the onset of the summer 'monsoon' season. In 1980, it peaked in July. Residential water uses can be classified as either indoor or outdoor uses (see Figure 3). The classification is based on where the water winds up after it has been used. If most of it winds up in the sewage system, it is classified as an indoor use. If most of the water evaporates, it is an outdoor use. That is why evaporative cooling is considered to be an outdoor use.

Unlike previous researchers, the authors have modeled indoor and outdoor water demand separately. This should be valuable for several reasons: Modeling indoor water use would aid in designing sewage systems of appropriate capacity. An undersized sewage pipe has obvious costs; an over-sized sewage pipe is a waste of money and may not be self-scouring, leading to possible blockages. Also, the ability to forecast sewage flows is necessary before long-term contracts for delivery or treatment of effluent can be written.

Modeling outdoor water use also can be valuable. While indoor use is relatively constant, outdoor use is concentrated by month and time of day. The amount of outdoor use determines necessary pump capacities, lift capacities, and size of pipes. To a large extent, it determines the capital costs of the water delivery system and is therefore the most expensive water to deliver. In addition, outdoor uses result in most or all of the water evaporating and leaving the TAMA. Indoor uses, by contrast, return most of the water to the sewage system for reuse or recharge. Thus, a model of outdoor water use could be used to determine effective management options for limiting the cost of the water distribution system and conserving the area's water resources.

Two assumptions are made in order to model indoor and outdoor water demand. First, it is assumed that March water use is essentially all indoor use, or that outdoor use is roughly zero in March. This seems to be a fairly reasonable assumption, since Bermuda grass and many other plant species are dormant at that time, and since evaporation minus rainfall is at most a fraction of an inch. Also, no water is used for evaporative cooling in March. Indeed, it appears that all outdoor water uses are either zero or minimal during March.

Table 1

Explanatory Variables Tried

Economic Variables

- Median home value
- Median monthly rent
- Median household income
- Fraction of homes self-owned

Age of Population Variables

- Fraction of population under 5
- Fraction of population 5 to 9
- Fraction of population 10 to 17
- Fraction of population 18 to 24
- Fraction of population 25 to 64
- Fraction of population over 64
- Median household age

Age of Home Variables

- Fraction of homes less than 5
- Fraction of homes less than 10
- Fraction of homes less than 15

Other Variables

- Persons per household
- Fraction of households living in Pima County longer than 5 years
- Fraction of population of Hispanic origin
- Fraction of households of Hispanic origin
- Median lot size

Table 2

Variables Occuring in Water Demand Models, Census Tract Data, Tucson S.M.A., 1980[1]

Name	Description	Mean	Standard Deviation	Range
(Dependent Variables)				
SFR M	Mean water consumption, single family residences, March, 1980 (CCF/Month)	9.3729	1.1322	6.133 - 11.448
SFR J	Mean water consumption, single family residences, July, 1980 (CCF/Month)	20.0694	3.0408	12.143 - 26.464
SFR O	SFR J - SFR M; Estimated mean outdoor water consumption, July, 1980 (CCF/Month)	10.6964	2.8020	5.839 - 17.009
(Independent Variables)				
PPH	Mean persons per household	3.0116	0.501	2.06 - 3.89
UNDER5	Fraction of population less than 5 years old	0.0756	0.0262	0.037 - 0.132
B5AND17	Fraction of population between 5 and 17 years old	0.2258	0.0597	0.095 - 0.325
B20AND24	Fraction of population between 20 and 24 years old	0.0919	0.0320	0.042 - 0.184
OVER64	Fraction of population older than 64 years old	0.0929	0.0573	0.026 - 0.259
TGT5	Fraction of households living in Pima County more than 5 years	0.7256	0.1671	0.289 - 1.00
LOTSIZE	Mean lot size of single family residences (acres)	0.3385	0.2657	0.169 - 1.293

[1] Dependent variable data from Tucson Water Department; independent variable data from U.S. Census reports and Tucson Trends survey.

The second assumption is that the difference between water use in any other month and March water use is essentially equal to the outdoor use for that month. The underlying assumption here is that indoor use per household is fairly constant throughout the year. There are many arguments that can be made as to whether or not indoor water use remains constant over the year. People take more showers in the summer and clothes may become soiled quicker. On the other hand, fewer bulky winter outfits fill a washer, and the price of water is lower in the winter. There is no reason to expect water used for cooking and dish washing to fluctuate seasonally.

A more conclusive argument may be made with Pima County Wastewater Management Department data, which show an essentially steady flow of effluent, with a slight drop in the summer corresponding to the University recess, and a slight increase in winter months, explained by the arrival of snowbirds. Thus, constant indoor water use per household over time also appears to be a reasonable assumption. To minimize the effect of any fluctuations in indoor use on the estimated outdoor use, the authors modeled the month of maximum estimated outdoor use during the census year of 1980, which was July.

Thus, there are three dependent variables to model: March water use, which approximates indoor use; July use, which is the sum of indoor and maximum outdoor uses; and July minus March, which approximates maximum outdoor use.

FINDINGS

The first potential explanatory variable investigated was geographic location. The question posed was whether water use patterns are regional phenomena. A map showing indoor water use by census tract shows no apparent spatial pattern. Both high and low water use tracts appear in all parts of the metropolitan area, and many neighborhoods contain high, medium and low water use tracts. There is a similar lack of spatial pattern for outdoor water use.

Not only is there no apparent geographic basis for either indoor or outdoor demand, there is also very little correlation between indoor and outdoor use within census tracts, with some tracts having high levels of indoor use and low levels of outdoor use, and vice versa. The simple correlation coefficient of 0.02 between indoor and outdoor water use, which is not significantly different from zero, confirms that knowing either the indoor or outdoor water use in a tract will not help to predict the other. It also means that the models for indoor and outdoor water demand are probably substantially different.

There are a great many other variables which might help explain differences in water demand. Those variables for which data were available and which were fitted to residential water demand are listed in Table 1. Originally, the data were fitted with three different model forms: linear, log-linear, and semi-log. These are the model forms commonly used in the water demand literature. The simple linear form consistently fit the data as well as, or better than, the semi-log and log-linear forms.

Tables 2 through 4 summarize the models determined for indoor, outdoor, and total water demand. Table 2 lists the dependent variables and those independent variables which are significant in at least one demand model and gives summary statistics. Note that in July outdoor water use accounts for about 53% of total water use. Also, note that the standard deviation of outdoor water use is much greater than that of indoor use.

Most of the significant independent variables are demographic. TGT5 stands for the fraction of the population in the census tract that lived in the area prior to 1975, which was when water first became a topic of immediate concern in the community.

Table 3 contains models of indoor, outdoor and total water demand. Of all the independent variables considered, only the number of persons per household (PPH) and the fraction of the population under five years old (UNDER5) are significant predictors of indoor water demand. PPH has the expected sign, while UNDER5 is shown to have a negative impact on indoor water use. Income, the age of the home, and home value are insignificant. The adjusted R^2 is quite low, and an analysis of the residuals indicates that the model is underspecified, i.e., one or more important explanatory variable has not yet been identified and included.

Table 3. Models of Residential Water Demand

Model of Indoor (March) Water Demand

$$SFR\ M = 4.46 + 2.28(PPH) - 25.94\ (UNDER5)$$

$$R^2 = 0.533; \quad Adj.\ R^2 = 0.507$$

Significant Variables	Coefficient	Standard Error	T Statistic	Confidence Level
Constant	4.46	0.83	5.36	99.9
PPH	2.28	0.36	6.26	99.9
UNDER5	-25.84	6.98	-3.71	99.9

Model of Indoor + Maximum Outdoor (July) Water Demand

$$SFR\ J = 15.72 + 3.35(PPH) - 124.55\ (UNDER5) + 5.06\ (TGT5)$$

$$R^2 = 0.634; \quad Adj.\ R^2 = 0.602$$

Significant Variables	Coefficient	Standard Error	T Statistic	Confidence Level
Constant	15.72	2.27	6.92	99.9
PPH	3.35	0.88	3.79	99.9
UNDER5	-124.55	16.97	-7.34	99.9
TGT5	5.06	1.94	2.60	99

Model of Maximum Outdoor (July-March) Water Demand

$$SFR\ O = -5.39 - 126.39\ (UNDER5) + 57.90\ (B5AND17) + 56.50\ (B20AND24) + 30.55\ (OVER64) + 4.87\ (TGT5) + 2.96\ (LOTSIZE)$$

$$R^2 = 0.787; \quad Adj.\ R^2 = 0.746$$

Significant Variables	Coefficient	Standard Error	T Statistic	Confidence Level
Constant	-5.39	4.03	-1.34	90
UNDER5	-126.39	16.00	-7.90	99.9
B5AND17	57.90	13.07	4.43	99.9
B20AND24	56.50	12.97	4.36	99.9
OVER64	30.55	10.39	2.94	99.5
TGT5	4.87	1.60	3.05	99.5
LOTSIZE	2.96	1.11	2.67	99.5

Table 4

Tests for Homogeneity of Coefficients

The three models are regressed on the same set of variables with: none of the coefficients constrained to be the same; each coefficient separately constrained; and all coefficients constrained. F tests are then run to determine whether the indoor and outdoor water demand models are significantly different, and whether individual coefficients are significantly different.

Unconstrained Models		SSE
Indoor	SFR M = 4.44 + 2.28(PPH) - 26.0(UNDER5) + 0.03(TGT5)	22.13
Outdoor	SFR O = 11.28 + 1.07(PPH) - 98.6(UNDER5) + 5.03(TGT5)	108.29
Indoor + Outdoor	SFR J = 15.72 + 3.35(PPH) - 124.6(UNDER5) + 5.06(TGT5)	125.26

Constrained Models: Indoor $\stackrel{?}{=}$ Outdoor

Constrained Variable(s)	SSE	F-stat.	Significance Level
All	262.79	17.25[1]	1%
Constant	147.14	8.72[2]	1%
PPH	133.90	1.81[2]	-
UNDER5	164.17	17.60[2]	1%
TGT5	142.58	6.34[2]	5%

The indoor and outdoor models are different at a 1% level of significance. The effect of the constant and UNDER5 also differ at a 1% level. TGT5 differs at a 5% level, while PPH cannot be shown to differ.

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[1] F-stat. calculated by $\frac{(SSE_c - SSE_{uc})/4}{SSE_{uc}/68}$; significance levels: 1%(3.65); 5%(2.53)

[2] F-stat. calculated by $\frac{(SSE_c - SSE_{uc})/1}{SSE_{uc}/68}$; significance levels: 1%(7.08); 5%(4.00)

The model of outdoor water demand has many more variables that are significant, including several additional age groups. Lot size is significant and of the expected sign. In earlier regressions, income and house value were significant variables; however when lot size is added, they become insignificant. There is a fairly high correlation between lot size, home value and income.

The most interesting variable is TGT5. Its coefficient seems to indicate that newcomers to Tucson use less water outdoors, which runs counter to the stereotypical perception of Midwesterners moving to the Southwest and bringing their water-thirsty landscaping habits with them. It may suggest that water conservation preachments are more effective for influencing newcomers than long-term residents.

Another interesting conclusion that can be drawn is that indoor and outdoor water demand do have very different determinants. Indoor demand is mainly dependent on the number of persons in the household and, the authors suspect, how much of the time they are at home. By contrast, outdoor demand is characterized by a more complex model. Table 4 shows the results of F-tests which demonstrate that the demands are determined by significantly different models.

Figure 4 summarizes the effects on indoor and outdoor consumption of all age groups, including those not statistically significant. The effects have been corrected for the relative percentages of the population in each age group. Small children have the largest (and negative) effect on both indoor and outdoor water consumption. The proportion of the population over 64 years of age (OVER64), a surrogate variable for retirees, shows increased outdoor usage, but constant indoor usage.

CONCLUSIONS

Indoor and outdoor water demand are shown to be characterized by significantly different models. Both demand models contain significant explanatory variables other than PPH, indicating that total residential water demand is not simply proportional to population. While the population of the country as a whole is getting older and family sizes are shrinking, Tucson's high rate of in-migration and its large Hispanic population make local demographic trends more difficult to determine. Additional work needs to be done to estimate these trends, as well as determining the trend in lot sizes and investigating the effects of other variables including the price of water before any serious attempt can be made to forecast residential water demand.

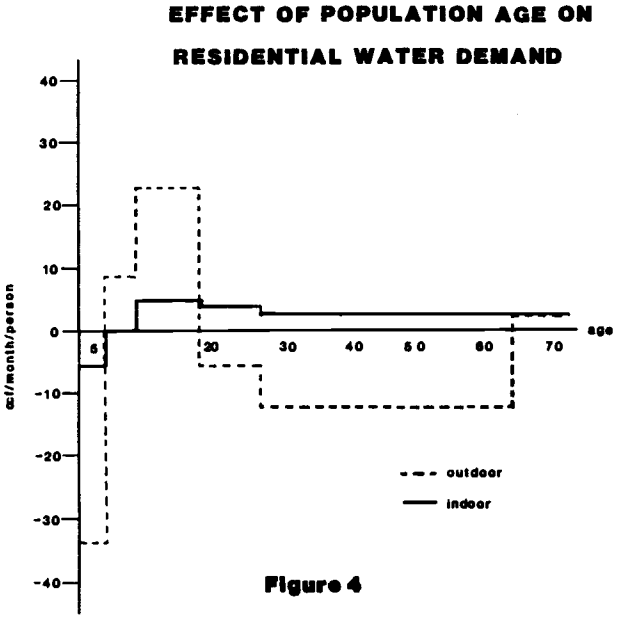

Figure 4

Subsurface Production of Chlorine-36 and Its Impact on Ground Water Dating

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ABSTRACT

Chlorine-36 is an important radioisotope with which to date old ground water. The initial chlorine-36 in ground water originates in the atmosphere by cosmic ray spallation of argon-40. Following precipitation and infiltration processes, the natural decay of this radioisotope is then used to date ground water. One must consider, however, the production of chlorine-36 in the subsurface. The production reaction of most interest is ^{35}Cl + neutron → ^{36}Cl + gamma. Buildup of chlorine-36 in the subsurface can result from cosmic ray secondary neutrons near the surface and natural radioactivity produced neutrons below the surface. These production mechanisms, if not taken into consideration, will contribute to the error in chlorine-36 age determinations. To predict subsurface production rates, field measurements were made of thermal neutron fluxes for various geologic materials and depths below the surface. Thermal neutron fluxes were found to vary by more than three orders of magnitude. Theoretical calculations of neutron flux were compared to filed measurements. Estimates of chlorine-36 production rates were then calculated and compared to measured values of chlorine-36 in very old ground water, where decay rates have been hypothesized to be equal to production rates.

STABLE ISOTOPES AND GROUND-WATER CHEMISTRY AS INDICATORS OF MOUNTAIN FRONT RECHARGE, TUCSON BASIN, PIMA COUNTY, ARIZONA

Carl Mohrbacher (University of Arizona, Tucson, Arizona)

Abstract

The relative importance of mountain front recharge as compared to total recharge was determined for a portion of the Tucson basin aquifer margin by interpretation of chemical and isotopic data. Concentrations of $^{18}O/^{16}O$ lower than δ-10.7 $^{o}/_{oo}$ as compared with a background of about δ-9.3 $^{o}/_{oo}$ in ground water from the base of the mountains in the gneissic rock suggest the presence of recharge from significantly higher elevations. The trilinear diagram of major ions dissolved in ground water from 123 wells in the Santa Catalina foothills indicates three water types. Water from wells in gneissic rock is high in sodium and potassium content and low in calcium and magnesium. Wells in the gypsiferous Pantano Formation yield water high in sulfates. The majority of wells in the study area, which are along major streams and in the regional aquifer, have calcium carbonate type water. Their chemistry indicates only minor contributions from the gneissic mountain block and the underlying Pantano Formation. Funding for this project came from the Spanish Project Register T377017.

Introduction

The city of Tucson, Arizona, and its surrounding area totally depends on ground water for its water supply. Economic and population growth has resulted in overdraft of the regional aquifer. To calculate safe yield, it is necessary to determine and quantify sources of recharge. Mountain-front recharge occurs along a boundary of the regional aquifer system that parallels a mountainous area (Wilson and others, 1980). This study discusses mountain-front recharge along the eastern third of the Santa Catalina Mountains (Figure 1). Tucson is in an alluvium-filled basin which is bounded by mountains on the northeast (Santa Catalina Mountains), east, and west.

Geology

The Santa Catalina and Tanque Verde mountains are composed of gneiss. Average composition of the gneiss is quartz (42%), oligoclase (An. 20-25%)(32%), orthoclase (13%), muscovite (6%), biotite (4%), and microcline (3%) (Medhi, 1964). The crystalline mountain block is bounded on the basin side by a low-angle fault.

On the hanging wall of the low-angle fault is a tertiary alluvium formation called the Rillito Formation (Pashley, 1966). Pashley divided the Rillito Formation into three units. The lowest unit, Rillito I, is interbedded conglomerate, sandstone, and mudstone, and is firmly cemented. Pashley observed massive gypsum beds in the Rillito I unit in an outcrop just east of the present study area. Rillito II lacks the mudstone and is moderately to firmly cemented. Rillito III has more sand and less cementation. The Rillito Formation is deformed by normal faults, some of which may join the main low-angle fault at depth (Figure 2). Overlying the Rillito Formation are Basin-Fill sediments which form the regional aquifer.

In the area studied, the alluvium consists of low- and high-permeability aquifers. The low-permeability aquifer is Rillito I and II, which crops out in a 2-mile-wide band in front of the mountain base. The high-permeability aquifer is Rillito III and Basin-Fill sediments. Three types of wells are present in the study area (Figure 2): wells in gneiss (water samples #1-22), wells exclusively in the lower-permeability sediments (water samples #23-33, 39, 40, 43, and 51-53), and wells drawing mainly from the Basin-Fill and Rillito III.

Data

The author used University of Arizona equipment to analyze samples from two streams, four springs, and 72 wells (Table 1). The cations were analyzed on an atomic absorption spectrophotometer. Fluoride was analyzed with a specific ion electrode, and the rest of the anions were analyzed with a Hach Kit using standard methods. Silica was analyzed with both the atomic absorption spectrophotometer and the Hach Kit; then, the results were averaged. pH was measured in the field and bicarbonate was measured

in the laboratory within a few days of sampling. Data from 48 analyses are included from previous studies. All analyses were checked for accuracy by dividing the difference of cation and anion milliequivalents by the total milliequivalents. Any analyses with more than a 10% difference were rejected or resampled. Wells with several analyses covering a period of years or months were averaged. These wells did not show long-term trends in water chemistry; the data were too sparse to evaluate monthly variations.

Results

A trilinear plot (Figure 3) of all analyses shows three types of water chemistry. The large majority of analyses plot as Ca^{++}-Na^{+}-HCO_3^- water. A second group of analyses has high sulfate water, except for #50 and #53, which have high chloride instead of high sulfate. These analyses are of wells in the less-permeable Rillito units. The third group of analyses has low Ca^{++}, Mg^{++}, high Na^{+}, K^{+} and variable $SO_4^{=}$. These wells are in the gneiss at the mountain base, except for #32A and #32B, which are in the less-permeable Rillito units. Analysis #32B has high chloride in place of sulfate. These three types of water chemistry will be referred to as group I, II, or III, respectively.

Mixing appears to take place between group I and the low sulfate end of group III. Except for #28, all of these wells with mixed water chemistry are more than 2½ miles from the mountain base. Only analysis #30 shows any evidence of mixing between groups I and II. In the cation triangle, most of the analyses of group I below ten percent Mg are from wells south of Rillito and Tanque Verde Creek.

Scattergrams of bicarbonate (HCO_3^-), sulfate ($SO_4^{=}$), and sodium and potassium (Na^{+} and K^{+}) versus total dissolved ions (TDI) (Figures 4a, b, and c) suggest the processes which cause the different types of water chemistry. Figure 4a shows that a very strong positive correlation exists between HCO_3^- and TDI for wells in the Basin-Fill, but no correlation for wells in Rillito I and Rillito II. Sulfate is present in only minor amounts in Basin-Fill wells (Figure 4b) and in moderate amounts in the gneissic hard rock wells. Sulfate increases almost linearly with TDI for wells in Rillito I and Rillito II, except for those wells which have a high chloride concentration. Figure 4c shows that Na^{+} and K^{+} are positively correlated for each type of aquifer. However, the gneissic hard rock wells are separate and follow their own trend line which includes alluvium wells #32A, #32B, and #43.

The plot of δ deuterium versus δ oxygen-18 (Figure 5) shows that most analyses fall along the local winter meteoric line (Turner, 1983). Five samples showed the effect of evaporation (1, 2, 6, 8, 32B), and three fell above the winter line (18, 36, 32A). Table II lists all available isotope data with the type of water chemistry for each well. The five wells with the lowest oxygen-18 values are hard rock wells which have a type III water chemistry.

Discussion

Exactly half of the analyses from wells or springs in the gneiss have group III water chemistry. They also have the five lowest $\delta^{18}O$ contents of all samples (-11.8 to -10.7). The lower $\delta^{18}O$ values suggest a higher point of recharge for these wells as compared to other wells at the mountain base (-7.8 to -9.3). The difference in isotope values cannot be explained by seasonal effects alone, because all of the samples fall close to the local winter meteoric line. Smalley (1982) determined an oxygen-18, altitude gradient of $-0.32\delta^{18}O^{0}/oo/305m$ for the Graham Mountains, 100 km northeast of Tucson. If this gradient is applied to the difference in oxygen-18 values of the most negative group I and group III wells at the mountain base, a recharge altitude of 10,600 feet is obtained for the group III well. This altitude is greater than the highest peaks in the Santa Catalina Mountains. The discrepancy may be explained by seasonal effects including the greater contribution of snow melt recharge at high altitudes.

A high altitude source of recharge for water depleted in oxygen-18 is also suggested by the water chemistry. The ground water with high Na^{+} content (group III) has probably been in contact with the albite-rich gneiss for a greater length of time than water with average values of Na^{+} (group I). The longer flow path from high elevations affords more time for the water to equilibrate with the gneiss than a short flow path from lower elevations.

Other analyses in group III or mixtures of group I and III waters may indicate water flowing up from faults in the gneissic basement rock. For example, wells #32A and #32B are near a major fault in the Rillito Formation, which may continue into the gneissic basement rock. Analysis #32B is of water from a well with twice the depth of #32A, and it has several more feet of head. Sample #32B occupies an extreme point in group III (Figure 3), which suggests that most of its water comes from the gneiss.

The positive correlation of $SO_4^{=}$ to TDI for most wells in Rillito I and Rillito II units (Figure 3, 4b) indicates that dissolution of $CaSO_4$ is the main cause of mineralization of ground water in this area. The lack of correlation of Ca^{++} to TDI is probably due to ion exchange in the clays.

The low sulfate concentration in Basin-Fill wells indicates that the Rillito I and Rillito II units do not contribute significant amounts of water to the Basin-Fill. Assuming that all of the Cl^{-} and $SO_4^{=}$ found in Basin-Fill ground water comes from Rillito I and Rillito II units, the percent

contribution ground water from the latter source can be calculated: average Cl^- and $SO_4^=$ in group II is 19.5 meq/l and 0.8 meq/l for group I. Therefore, the required mixing ratio is 1:23.

The bicarbonate ion is positively correlated with TDI for Basin-Fill ground water. Water percolating through the soil dissolves large amounts of CO_2, creating bicarbonate and relatively low pH's. The more acidic water reacts readily with aquifer materials, principally calcite. Eventually, the water may become supersaturated with calcite, causing its precipitation. Magnesium does not precipitate out with the Ca^{++}. Dissolution and precipitation vary with space and time in the aquifer.

As mentioned earlier, Ca^{++}/Mg^{++} ratios are distinctly lower for wells south of Rillito and Tanque Verde Creek. The slightly acidic water recharged along these creeks is dissolving pure calcite which was deposited earlier under different conditions.

Conclusions

Recharge to the gneissic aquifer occurs at various altitudes, and the system is not well mixed. Faults in the gneissic basement rock are contributing some water to the alluvium. On the basis of water chemistry, it appears that the amount of water flowing from the low-permeability Rillito I and Rillito II units is negligible. Calcium/magnesium ratios indicate that the Rillito and Tanque Verde Creek are recharging the regional aquifers.

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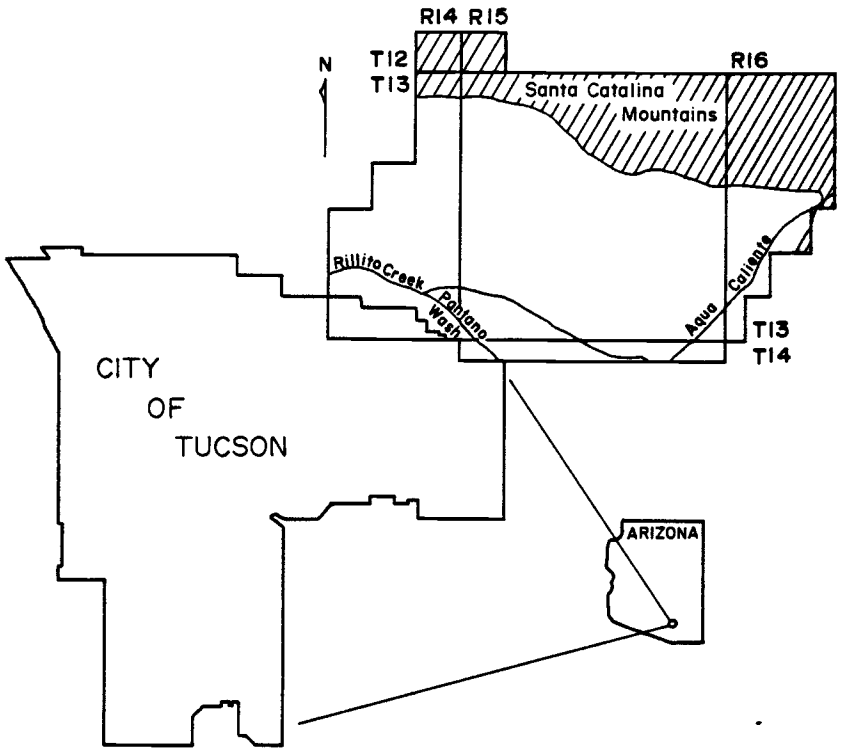

Figure 1. Location of study area

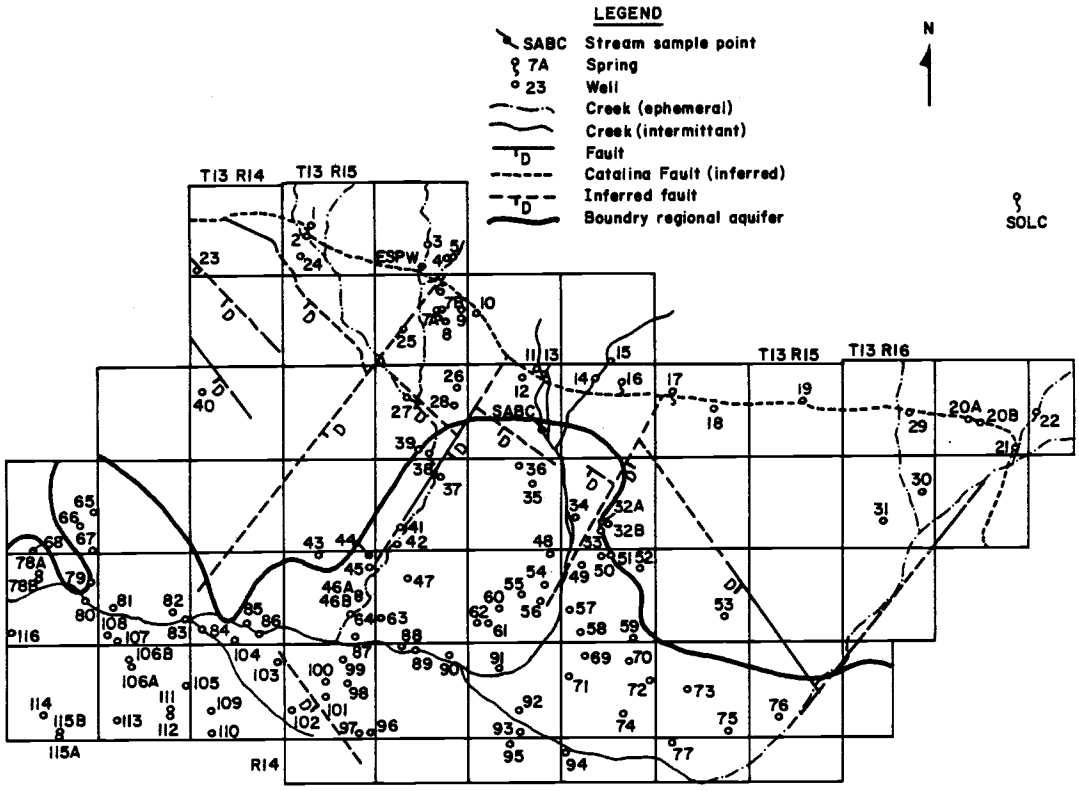

Figure 2. Map of study area showing location of wells, springs, and stream sampling points. Also shown are fault traces, streams, and boundary of the regional aquifer. Fault traces are taken from Pashley (1966).

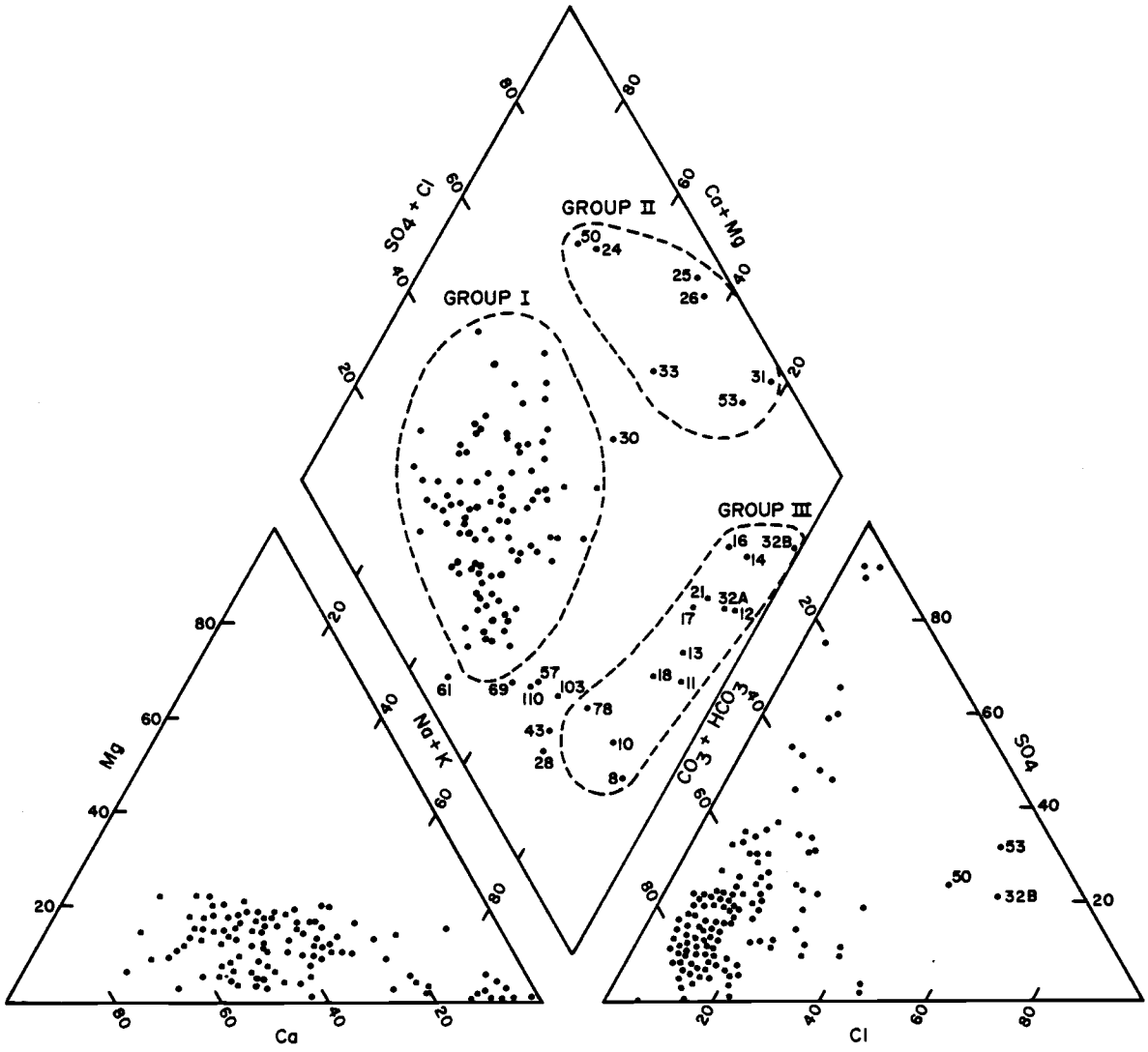

Figure 3. Trilinear diagram of relative amounts of major ions dissolved in ground water. Group I is Ca-Na-HCO_3 type water chemistry; Group II is high sulfate; Group III is high Na and K. Most analyses below 10% Mg and above 25% Ca in the cation triangle are for wells located south of Rillito Creek.

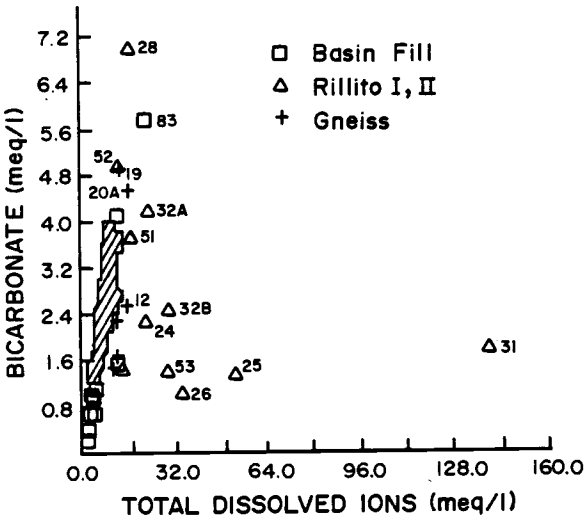

Figure 4a. Scattergram of bicarbonate vs. total dissolved solids in milliequivalents per liter. Hatched area represents a dense cloud of points from all three types of wells.

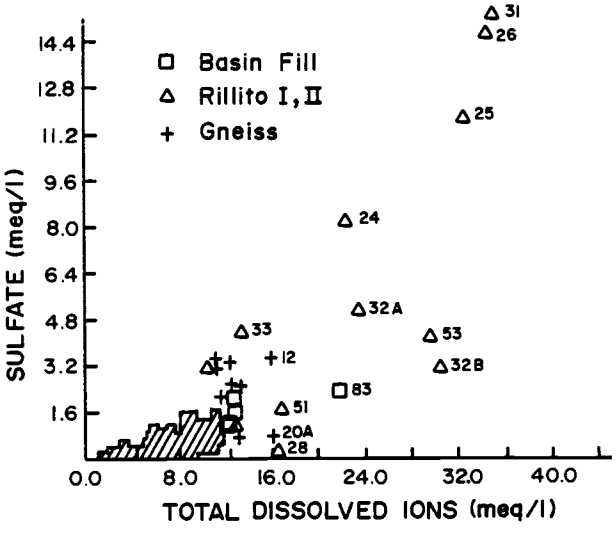

Figure 4b. Scattergram of sulfate vs. total dissolved solids in milliequivalents per liter. Hatched area represents a dense cloud of points from all three types of wells.

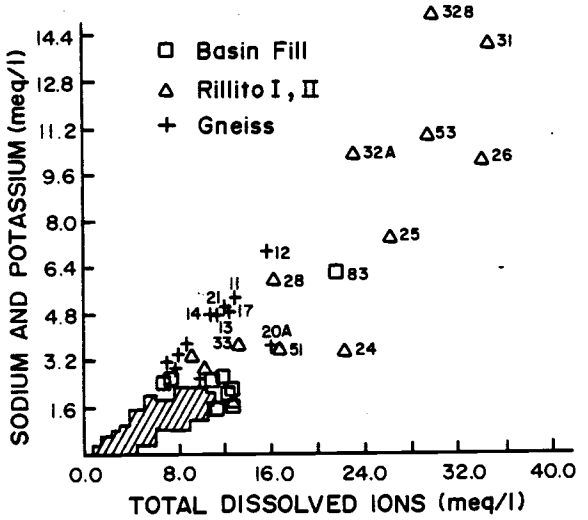

Figure 4c. Scattergram of sodium and potassium vs. total dissolved solids in milliequivalents per liter. Hatched area represents a dense cloud of points from all three types of wells.

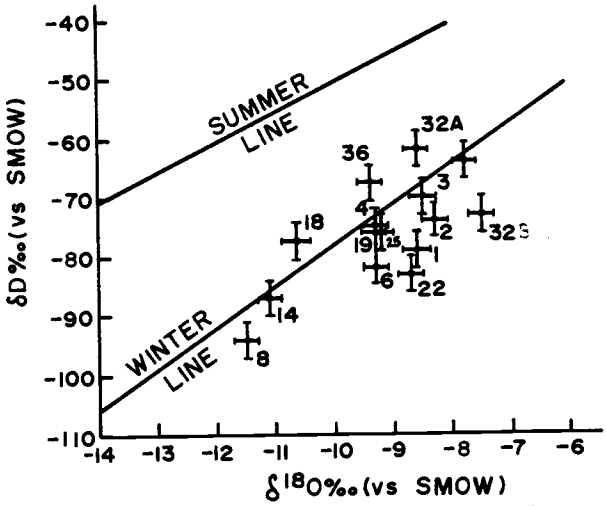

Figure 5. δ deuterium ‰ vs. δ oxygen-18 ‰ for ground water samples. The standard is Standard Mean Ocean Water.

Table 1. Chemical Analyses in Milliequivalents per Liter

Sample No.	Ca^{++}	Mg^{++}	Na^{+} & K^{+}	HCO_3^-	$SO_4^=$	Cl^-	NO_3^-	F^-	TDI	SiO_2	Depth
1	1.22	.56	1.66	2.66	.04	.37	.10	.03	6.60	.85	
2	1.45	.59	1.48	2.38	.48	.01	.03	.03	6.76	.73	190
3	.93	.57	1.38	2.26	.19	.46	.06	.02	5.87	.71	324
4	.45	.64	1.47	1.72	.91	.42	.09	.02	6.20	.76	300
5	.79	.27	1.51	1.56	.85	.42	.04		5.37	.73	
6	2.15	1.02	2.14	3.40	.85	.76	.29	.06	10.70	.78	213
7A	2.40	.16	1.68	2.42	.77	.21	.11	.03	8.12	.70	0
7B	.80	.06	2.96	2.82	.81	.23		.27	7.63	.55	402
8	.30	.02	3.21	2.29	.72	.24	.04	.14	7.26	.53	194
9	1.84	.74	2.34	3.83	.37	.02	.05	.04	9.79	.09	100
10	.55	.05	3.44	2.75	.83	.31		.13	8.06		284
11	.45	.03	5.42	3.56	2.50	.85	.03	.19	13.01	.74	144
12	.50	.03	7.04	2.56	3.45	1.50	.04	.31	15.82	.57	200

Table 1 Continued . . .

Sample No.	Ca^{++}	Mg^{++}	Na^{+} & K^{+}	HCO_3^-	$SO_4^=$	Cl^-	NO_3^-	F^-	TDI	SiO_2	Depth
13	.34	.27	4.91	2.80	2.19	.81	.05	.24	11.01	.05	55
14	.48	.02	4.85	1.50	3.44	.79	.03	.03	11.13	.53	200
15	1.50	.44	1.35	1.50	.69	.73	.13	.06	0.40	.43	33
16	.64	.14	5.06	1.58	3.33	.72	.13		12.47	.73	0
17	.75	.08	5.10	2.44	2.80	.82	.06		12.40	.53	0
18	.34	.26	3.65	2.51	1.56	.41	.04	.31	9.06	.74	
19	3.19	.99	2.18	4.94	.73	.99	.06	.09	13.17	.63	110
20A	3.09	1.40	3.74	4.55	.83	2.48		.05	16.12		40
20B	1.59	.34	2.70	4.00	.59	.47	.05	.04	9.98	.57	
21	.55	.16	5.17	2.34	3.12	.76	.05	.04	12.19	1.04	80
22	1.85	.67	1.65	3.47	.35	.48	.12	.03	8.02	.89	400
23	2.87	.80	1.89	3.85	.33	1.19	.02	.02	10.96	.53	
24	7.53	.35	3.53	2.20	8.12	.42	.01	.16	22.38	.02	200
25	11.46	1.14	14.03	1.34	23.30	1.07	.01	.06	52.01	.40	500
26	7.64	.10	10.00	1.00	14.60	0.85		.13		.27	200
27	1.35	.58	.61	1.28	0.00	1.16			4.98		112
28	.80	1.32	6.00	7.00	.21	1.13			16.46		
29	2.55	1.14	1.92	3.86	1.25	.04	.11	.04	10.53	.03	250
30	1.98	.54	2.94	2.19	3.10	.21	.16	.03	10.45	.24	237
31	14.60	1.83	55.41	1.78	81.22	4.85	.11	.04	159.25	.17	110
32A	.63	.31	10.30	4.19	5.63	1.95	.12	.16	23.20	.93	103
32B	.14	.08	15.12	2.43	3.12	9.36	.04	.17	30.37	.06	300
33	2.15	.91	3.72	1.48	4.33	.74	.03	.09	13.45		
34	.74	.32	.04	.68	.50	.29	.02	.02	3.21	.20	18
35	1.50	.74	2.00	3.11	.62	.50	.05	.03	8.61		60
36	1.90	.85	1.67	3.13	.88	.70	.10	.04	9.35		350
37	1.72	.59	1.30	2.40	.14	.09	.05	.06	6.00	.04	170
38	2.54	.80	1.43	2.64	.00	2.00	.15	.04	10.20	.87	150
39	1.91	.58	1.33	2.94	.27	.79	.00	.02	7.80	.73	
40	2.00	.82	1.09	3.88	.21	.56	.03	.01	8.60		335
41	1.80	.61	1.12	2.79	.19	.02	.05	.02	7.20	.02	190
42	2.50	.61	1.13	2.40	1.05	.48	.04	.01	7.22	.43	
43	1.04	.26	3.39	3.76	.41	.46	.06	.03	9.36	.51	470
44	1.68	.67	1.39	3.25	.18	.50	.00	.03	7.82	.66	280
45	1.77	.65	1.44	3.25	.26	.50	.10	.05	8.00	.54	260
46A	1.81	.04	1.18	2.88	.56	.21	.16	.01	7.75	.70	143
46B	2.21	.65	1.90	3.44	.34	1.02	.16	.03	9.75	.00	190
47	1.31	.48	1.01	2.26	.10	.05	.05	.02	5.86	.67	280
48	.85	.58	1.41	2.06	.42	.73		.03	6.10		
49	2.99	1.11	2.27	2.72	2.06	1.36	.22	.03	12.78	.75	142
50	3.19	1.31	1.66	1.56	1.58	3.41	.13	.03	12.87	.07	120
51	3.69	1.06	3.60	3.72	1.67	2.20	.26	.10	16.90	.64	105
52	3.34	.75	1.74	4.94	1.09	.93	.14		12.93	.43	205
53	3.21	.66	10.90	1.39	4.17	7.07	1.53	.03	29.05	.35	
54	2.60	.80	1.80	2.78	.69	1.41	.09	.04	10.21	.82	125
55	.88	.35	.79	1.50	.16	.37	.12	.01	4.18	.05	100
56	1.90	.99	2.22	2.69	.98	1.41			10.19		125
57	.94	.35	2.49	2.46	.33	.36	.06	.07	7.10	.38	15.
58	1.21	.49	1.78	2.35	.31	.31	.10	.03	6.58	.52	122
59	1.10	.49	1.57	2.56	.34	.23	.02	.03	6.34		
60	.65	.20	.79	1.54	.02	.25	.04	.05	3.55	.60	160
61	.51	.20	.71	1.40	0.00	.10	.04	.01	3.03	.70	105
62	.85	.32	.81	1.60	0.00	.48	.04	.01	4.11	.74	105
63	1.39	.44	2.23	2.51	.50	1.02	.07	.09	8.25	.52	302
64	1.83	.50	1.24	2.10	.58	.65	.16	.02	7.08		263
65	2.59	.74	1.44	3.28	.35	.85	.34	.03	9.02		271
66	2.20	.91	1.35	3.33	.44	.68	.19	.02	9.12		
67	2.20	.91	1.30	3.20	.29	.60	.18	.03	8.79		250
68	1.65	.61	.48	1.87	.54	.17	.09	.03	5.46		60
69	.54	.30	1.30	1.72	.16	.21	.13	.02	4.36	.92	
70	1.08	.49	1.17	1.88	.33	.40	.09	.03	5.69		
71	1.43	.55	1.09	2.18	.48	.44	.01	.02	6.20		
72	1.12	.33	1.08	1.76	.61	.38	.06	.02	5.36		
73	1.15	.30	1.46	2.45	.38	.26	.07	.03	6.10		300
74	2.29	.64	2.52	3.14	1.45	.04	.20	.03	11.11		300
75	1.34	.45	1.48	1.92	.63	.30	.11	.03	6.00		265
76	1.07	.84	1.98	3.56	.29	.48	.03	.02	8.27	.10	100
77	1.25	.41	1.74	2.10	.94	.26			6.76		145
78A	1.12	.21	1.58	2.45	.27	.49	.08	.02	6.22		200
78B	2.42	.45	.98	3.25	.49	.36	.22	.02	8.21		224

Table 1 Continued . . .

Sample No.	Ca^{++}	Mg^{++}	Na^{+} & K^{+}	HCO_3^-	$SO_4^=$	Cl^-	NO_3^-	F^-	TDI	SIO_2	Depth
79	3.74	.33	2.09	4.11	1.25	.73		.01	12.25		76
80	2.99	.33	2.70	3.07	1.15	1.18		.03	12.05		91
81	2.25	.38	1.17	3.44	.38	.56	.07	.02	8.79		235
82	3.30	.91	1.59	3.60	.67	1.14	.45	.02	11.44	.04	303
83	3.99	1.32	6.26	5.75	2.29	1.69	.41	.04	21.75		100
84	1.36	.24	.60	1.56	.29	.16	.03	.01	4.22	.24	140
85	2.40	.38	1.52	2.47	.19	2.20	0.00	.01	9.37		
86	.55	.15	.39	.08	.20	.08	.03	.01	2.09	.38	125
87	2.20	.58	1.09	2.56	.69	.53	.32	.02	7.79		215
88	.87	.21	1.18	1.71	.29	.23	.06	.02	4.57	.21	100
89	.58	.18	.88	1.33	.20	.12	.04	.02	3.41	.22	60
90	.79	.21	1.11	1.71	.17	.21	.05	.04	4.29	.57	135
91	.80	.20	.51	.86	.37	.28	.05	.02	3.09	.32	100
92	1.55	.49	.78	1.52	.83	.23	.12	.02	5.54		100
93	.88	.25	.44	1.01	.23	.41	.01	.01	3.24		
94	1.10	.33	1.70	1.87	.31	1.02	.06		6.39		250
95	1.19	.19	1.12	1.79	.30	.30	.03	.02	5.20		402
96	2.26	.23	1.57	2.65	.96	.55	.07	.02	8.33		247
97	3.08	.40	1.99	3.01	1.49	.89	.20	.01	11.07		245
98	1.47	.28	1.09	1.56	1.01	.27	.05	.02	5.85		150
99	1.08	.12	1.06	1.07	.19	.22	.05	.01	4.40	.46	600
100	2.54	.33	1.78	2.15	1.39	.42		.02	8.61		195
101	2.02	.17	2.15	2.29	1.06	1.06	.10	.01	8.85	.46	609
102	1.90	.33	1.05	1.84	1.04	.39		.03	7.18		190
103	.94	.13	2.52	2.89	.40	.52	.07	.05	7.52	.47	505
104	1.50	.33	.39	1.11	.21	.85		.05	4.42		
105	1.39	.16	1.16	2.01	.33	.27	.07	.01	5.40		376
106A	2.04	.27	.66	2.10	.45	.16	.06	.01	5.74	.39	220
106B	1.72	.26	.67	1.90	.48	.17	.06	.01	5.27	.37	700
107	1.70	.15	1.09	2.44	.50	.34	.11	.01	6.35		
108	1.23	.12	1.04	1.68	.29	.14	.09	.01	4.59	.41	390
109	1.85	.15	1.71	2.37	.75	.35	.12	.02	7.32		300
110	.60	.41	1.87	2.36	.17	.45	.05	.01	5.92		302
111	2.45	.16	1.83	2.79	.92	.42		.15	8.72		208
112	1.30	.19	1.70	2.41	.52	.34		.03	7.21		
113	1.79	.43	1.61	2.88	.49	.39	.08	.01	7.68		432
114	2.20	.65	1.01	3.00	.96	.56	.15	0.00	8.54		500
115A	2.00	.16	1.70	2.84	.54	.26	.11	.01	7.64		500
115B	2.03	.28	1.86	3.06	.52	.50	.13	.01	8.39		505
116	2.00	.16	.52	1.97	.40	.17		.02	5.24		
SABC	.36	.11	.22	.24	.11	.23	.02	0.00	1.29	.19	0
ESPW	.26	.18	.59	.40	.26	.20	.08	0.00	1.99	.33	0
SDLC	.60	.21	.52	.68	.23	.22	.05	0.00	2.50	.35	0

Table 2. Stable Isotope Data

Sample Number	δ oxygen-18	δ deuterium	Aquifer	Water Chemistry
1	-8.6	-7.9 •	gneiss	group I
2	-8.3	-74	gneiss	group I
3	-8.5	-70	gneiss	group I
4	-9.3	-75	gneiss	group I
6	-9.3	-82	gneiss	group I
8	-11.5	-94	gneiss	group III
13	-11.8	--	gneiss	group III
14	-11.1	-87	gneiss	group III
15	-7.8	-64	gneiss	group I
17	-11.1	--	gneiss	group III
18	-10.7	78	gneiss	group III
Gibbon Spring	-10.2	--	gneiss	--
19	-9.3	-76	gneiss	group I
22	-8.7	-8.3	gneiss	group I
25	-9.2	-7.6	Rillito I, II	group II
32A	-8.6	-62	Rillito I, II	group III
32B	-7.5	-73	Rillito I, II	group III
36	-9.4/-9.9	-67.8/	Basin-Fill	group I
37	-9.8	--	Basin-Fill	group I
Soldier Camp	-8.8	-74	gneiss	group I

APPLICATION OF CHLORIDE STABLE ISOTOPE ANALYSIS TO HYDROGEOLOGY

Ron Kaufmann, Harold Bentley, Stanley Davis, and Austin Long (University of Arizona, Tucson)

Introduction

Anomalous concentrations of chloride in groundwater have been found in many geologic settings. In deep aquifer systems, chloride concentrations ten times sea water have been reported. In shallow systems, chloride concentrations are generally observed to increase with depth (White, 1965). Chloride in groundwater is believed to come mostly from six sources: chlorides introduced directly from volcanic emanations, chlorides leached from small amounts of brine present in micropores in igneous rocks, chlorides released from minerals by weathering or diagenetic processes, direct dissolution of halite, sea water entrapped in sediments, and chlorides from the surface transported downward by meteoric water. Occurrence of chloride in unusual concentrations in aquifers may indicate nearby igneous activity, dissolution of aquifer material, presence of trapped sea water, cross formational solute movement, or concentration within the aquifer by ion filtration. Of these five processes, three are rarely suggested to explain chloride occurrence within a specific aquifer. Igneous activity is not a likely chloride source unless there is direct evidence of such. In old aquifers, most soluble components are thought to have already been removed so dissolution of halite is not usually considered as a process that contributes chloride to groundwater. Sea water is only considered a direct contributor of chloride if the overall chemistry of the groundwater appears to have evolved from marine water. As a result, occurrence of unusual chloride concentrations in a specific aquifer are most often attributed to either cross formational solute movement (both movement of water with solutes or movement of solutes alone) or ion filtration. These two processes are difficult to distinguish because the chloride itself has not been thought to provide direct evidence of its origin. This paper proposes that the distribution of stable chloride isotopes in groundwater may be applied to understanding some groundwater systems, and in particular in helping to distinguish the effects of ion filtration and cross formational solute movement.

Theory of Stable Chloride Isotope Fractionation

Previous research indicated that chloride isotope distribution does not vary significantly in natural systems. Hoering and Parker (1960) studied stable chloride (masses 35 and 37) in samples from more than 30 natural materials including halite, sea water, oil field brines, meteorites, and various rock types. Of all the terrestrial samples they analyzed, only two samples, well field brines from the Muddy J formation of Oklahoma, were "significantly" different from the laboratory standard. The Muddy J samples were isotopically lighter by less than 1.0 per mil. The apparent lack of chloride isotope fractionation is not surprising. J. Hoefs, in his book Stable Isotope Geochemistry (1980), says that fractionation of isotopes of light elements is likely to occur as a result of equilibrium reactions in which the elements are in different phases or oxidation states, or during bacterially catalyzed reactions. Chloride would not be expected to fractionate because chloride is rarely in equilibrium between phases, is mostly in one oxidation state, and is not selectively used during bacterial reactions. Still there is the mystery of why the Muddy J samples contained fractionated chloride.

Chloride isotope fractionation does occur when chloride diffuses, but the effect is apparently quite small. Kendall (1928) tried to separate chloride mass 35 from 37 by having chloride diffuse down a tube under an electrostatic potential. He found no fractionation of chloride isotopes. Madorsky and Straus (1947) succeeded in separating chloride isotopes by using counter current migration. They also had a NaCl electrolyte solution migrate along a sand-packed tube under an electrical potential. However, in their experiment, the electrolyte solution flowed in a direction opposite to that of the diffusing chloride ions. The rate of flow was sufficient to reduce the net transport of chloride to zero. During the experiment, more mobile chloride 35 migrated towards the cathode while chloride 37 was carried towards the anode by the flow of the solution. Madorsky and Straus reduced the chloride 37 to 35 ratio at the cathode from an initial value of 0.3205 at the beginning of the experiment to 0.2392 at the end.

Two geologic processes analogous to counter current migration are ion filtration of groundwater and diffusion of solutes between aquifers due to concentration gradients. In an aquifer which is confined by compacted clay or shale under a large hydraulic head gradient, water may be forced out of the aquifer through the confining layers. If the boundary of the confining layer is electrostatically charged by clay minerals, only uncharged water molecules will leave the aquifer unhampered. Charged particles will not pass into the confining layer as readily and are concentrated in the aquifer. This process, referred to as salt filtration or ion filtration, may have produced some of the saturated brines in deep aquifers and other anomalies at shallow depths. Clays in the confining layer may play a similar retarding role when solutes diffuse out of an aquifer due to concentration gradients. Diffusion of solutes from one aquifer to another may also have produced some anomalies.

Phillips (1982) suggested an equation to describe the influence that ion filtration and cross formation solute movement may have on the distribution of chloride isotopes in groundwater. The counter current model is applied to the boundary between the aquifer and confining layer. The negative charge of the clays at the boundary is a barrier to chloride anion diffusion out of the aquifer. Impetus to cross the boundary is provided either by a hydraulic or concentration gradient. When these two opposing forces are in equilibrium, only a portion of the chloride crosses the boundary. Because of its larger ionic mobility, chloride 35 is more efficiently repelled by the boundary charge so the chloride crossing the boundary is isotopically heavier. The resultant distribution of chloride isotopes in the aquifer can be described by an equation in the form of the Rayliegh distillation equation.

$$\frac{R_{ri}}{R_o} = (f_i^{a-1}) \times 10^3$$

R_{ri} indicates ratio (Cl-37/Cl-35) of residual chloride left in the aquifer at time i.

R_o indicates the initial isotopic ratio of the aquifer (time = 0).

f indicates the fraction of chloride remaining in the aquifer at time i.

a indicates a fractionation factor defined as follows in terms of membrane filtering efficiency (E) and ionic mobility ($U_{total}, U_{35}, U_{37}$).

$$a = \frac{(U_{35}-U_{37})(1-E)}{U_{total}-U_{35}}$$

The equations above are independent of whether ion filtration or cross formational solute movement is occurring. The equations predict that where diffusion of chloride is occurring out of an aquifer, the chloride remaining behind in the aquifer will become isotopically lighter. Conversely, chloride diffusing out of one aquifer into another will make the chloride of the receiving aquifer isotopically heavier. Knowledge of potentiometric head and concentration gradients linked with chloride isotope information can indicate if the diffusion of chloride is caused by ion filtration or cross formational solute movement. In light of Phillips' theory, the chloride isotope ratios in the Muddy J samples would suggest that chloride ions are diffusing out of the Muddy J, possibly during the formation of the brine.

Applications to Hydrologic Studies

For our study, we wanted to know if the counter current theory could be applied to understanding real groundwater systems. The first step was to develop an analytic method for chloride isotope analysis, in this case we chose mass spectrometry of methyl chloride gas. The laboratory method is described in detail by Taylor and Grimsrud (1969), and is summarized as follows:

1. Chloride is quantitatively precipitated as AgCl from a water sample.
2. The AgCl precipitated from the groundwater sample is reacted with excess methyl iodide in an evacuated tube for two (2) days at 70°C.
3. The resultant methyl chloride is separated from the unreacted methyl iodide gas by gas chromatography.
4. Methyl chloride is analyzed by positive ion mass spectrometry.

Commercially produced methyl chloride gas was used as a reference gas during spectrometric measurements. A value of the stable chloride ratio was determined for sea water from three Pacific ocean samples. Chloride ratio results of our study are reported with respect to this value of sea water.

The groundwater samples used in our study came from a Louisiana salt dome area and the Milk River aquifer in Alberta, Canada. These two areas represent two different problems in the geochemical interpretation of chloride occurrence in groundwater; chloride origin and chloride concentration mechnism in aquifers.

Chloride origin

In the vicinity of Louisiana salt domes, samples of groundwater have high chloride concentrations. The question is whether the chloride comes from dissolution of salt domes or from some other source. Samples of salt and fluid from the Weeks Island and Avery Island salt domes were collected by J. Fabryka-Martin of the University of Arizona along with well field brines near the domes.

Six well field samples were taken from Tertiary sandstone beds at depths between 3,000 and 12,000 feet. These sandstones are slightly tilted by regional structural activity. The salt domes are intruded into the tertiary rock layers. Chloride concentrations of the well field samples range from 70,000 mg/l to 90,000 mg/l. Sampled wells were adjacent to or above the salt domes.

Four samples of salt solution from within the domes (two from each of the domes) were obtained along with one salt sample from the Weeks Island dome. Chloride concentration in the dome solutions varied among samples from about 185,000 mg/l to 230,000 mg/l. Results of the stable chloride analysis for both well field and dome samples are shown in Table 1.

Table 1. Results of Stable Chloride Analyses of the Louisiana Samples, Including Chloride Ratio Results, Number of Analyses per Sample, and Chloride Concentrations

Sample	Cl Concentration** (mg/l)	Cl Isotope Ratio* (per mil)	Number of Analyses
Avery Island Dome Brines			
AV-2A	189,744	+0.1	1
AV-7706/18	228,330	+0.3	1
Weeks Island Dome Brines			
WI-IGE	201,426	+0.1	1
WIMP	194,700	+0.3	1
Weeks Island Salt		(+0.12)-(+0.2)	4
Well Field Brines			
Gulf PC-3	89,562	-0.2	
WC-W122	82,836	(0.0)-(-.05)	2
WC-W141	70,800	(0.0)-(-0.26)	3
WC-W155	78,588	(-0.13)-(-0.58)	3
WC-W245	81,420	-0.14	1

* All measurements are with respect to sea water. Ranges indicate where multiple analyses were performed.
** Analyses of chloride concentration provided by J. Martin, University of Arizona.

Table 1 indicates that samples from within the dome are .1 to .3 per mil heavier than sea water, while those outside the dome are as much as .58 per mil lighter. Though the difference between the two sample groups is not large (reaching a maximum of 0.88 per mil), the consistency of the results suggests that the difference is significant.

That the chloride ratio value of sea water seems to divide dome chloride from well field chloride may not be accidental. Chloride from both the dome and well field probably came from sea water originally. The difference between the two groups may be that the well field water has been concentrated

by ion filtration, while dome solutions have not. Removal of a fraction of the well field chloride by an ion filtration process would be expected to produce isotopically lighter chloride within the aquifer.

A couple of additional observations may be made about the Louisiana area chloride isotope results. First, chloride of the well field brines probably did not come from the domes. This observation is consistent with a recent study concerning strontium in the Louisiana well fields and salt domes. Pushkar (1983) looked at strontium 87/86 ratio as we looked at the chloride isotope ratio and concluded that strontium in the well field did not originate from the salt domes. Second, while ion filtration may explain fractionation of well field chloride, apparently some other mechanism has caused the dome chloride to be isotopically heavier than sea water. This other mechanism, though unknown, may be associated with equilibrium processes during salt deposition, or time variations in the isotopic ratio of the marine chloride.

Concentration Mechanisms in Aquifers

The second area for which chloride ratio analysis was performed presents a more complex problem than the Louisiana study area. The Milk River aquifer of Alberta, Canada is a sandstone aquifer that outcrops near the U.S.-Canadian border. The aquifer dips to a depth of 1000 feet about 60 miles into Canada. A thick layer of glacial sediments overlies the Milk River. Approximately 1350 feet of shale separate the Milk River from an underlying aquifer, the Bow Island sandstone. Chloride concentration in the Milk River aquifer varies from near zero at the outcrop to 1295 mg/l or more in the deeper portions of the aquifer 60 miles north of the recharge zone. Similar increases are noted for other major ions (Swanick, 1982). Chloride concentration in the Bow Island is as high as 14,000 mg/l (Schwartz et al., 1981). Potentiometric maps of the Milk River and Bow Island indicate that the head gradient is toward the Bow Island (Swanick, 1982).

The unresolved question associated with the Milk River aquifer is whether increased concentration of dissolved constituents in groundwater is due to ion filtration or cross formational solute movement. Hitchon, et al., 1971, examined the ion chemistry of the Milk River aquifer and several other west Canada aquifers and concluded that ion filtration controlled groundwater chemistry in the area. Schwartz and Muelenbachs (1979) reviewed ion chemistry and hydrogen and oxygen isotope distribution in the Milk River, and concluded that mixing of incoming and connate water from adjacent shale beds was responsible for the aquifer chemistry. Phillips (1980) reviewed the same isotope data from the Milk River and suggested that ion filtration could have caused the observed isotope distribution. Bentley, et.al. (1983) concluded from radioactive Cl-36 data that ion filtration is the dominant mechanism for chloride concentration.

To help resolve the question of concentration mechanisms in the Milk River aquifer, we measured chloride stable isotope ratios of six samples from the aquifer. Table 2 contains the results of those measurements.

Table 2. Results of Stable Chloride Analyses of the Milk River, Including Chloride Ratio Results, Number of Analyses per Sample, and Chloride Concentrations

Sample	Cl Concentration** (mg/l)	Cl Isotope Ratio* (per mil)	Number of Analyses
1	627	(-0.31)-(-0.38)	2
17	1093	+0.8	1
43	420	+0.6	1
11	356	+0.7	1

* All measurements are with respect to sea water. Ranges indicate where multiple analyses were performed.

** Chloride concentrations are from Swanick, 1982.

Sample 1 is from the northwestern corner of the study area. Sample 17 is located about 50 miles directly east of sample 1. Sample 11 is midway between samples 1 and 17. These three samples are 60

miles north of the intake area for the aquifer. Sample 43 is located about 20 miles south of sample 11.

Table 2 indicates that all the samples except sample 1 have a chloride isotope ratio of about +0.7 per mil. Sample 1 has a value that is about 1.0 per mil lighter (-0.38 per mil).

A relevant interpretation of the stable chloride ratio results can be made by assuming the chloride at the point of recharge is the same as sea water (0.0 per mil). Using the counter current fractionation model, sample 1 would indicate fractionation during outflow of chloride from the aquifer. The other samples indicate inflow of chloride, possibly from confining layers, perhaps ultimately from the Bow Island. It is not difficult to envision chloride coming from the Bow Island since the concentration in the Bow Island is much greater than in the Milk River. Although the present potentiometric head is higher in the Milk River than the Bow Island, it is possible for chloride to diffuse upward against the hydraulic head. Approximations of these two processes using the darcy velocity equation and Fick's First Law with suitable variables for the Milk River situation indicate that the velocity of the water movement vertically is on the same order of magnitude as the rate of diffusion (about 10^{-11} m/s) upward from the Bow Island. Then the only question is why would the west side of the study area have chloride that was isotopically lighter than the east side. This situation might occur if the west side possessed a potentiometric head which was larger than the east side and sufficient to stop diffusion of chloride into the Milk River. Indeed the west side of the area has a potentiometric head gradient between the two aquifers that is twice the head gradient in the east. In other words, it is possible that water is leaving the Milk River by ion filtration throughout the area but on the east side, flow is sufficiently slow that chloride can diffuse into the Milk River from the Bow Island. Interestingly, this interpretation reconciles Phillips' and Bentley's conclusions with Schwartz' and Muelenbachs' interpretation, in that both ion filtration and cross formational solute movement is suggested to be occurring. Before the chloride isotope interpretation can be accepted for the Milk River aquifer, more work must be done to clearly establish a link between laboratory experiments and actual field situations.

Summary and Final Discussion

Anomalous occurrences of the chloride in groundwater are often attributed to ion filtration or cross formation solute movement. Evaluation of isotopic and ion chemistry has not always been able to distinguish the influence of these two mechanisms. Analysis of stable isotopes of chloride has the potential to differentiate the influence of the two mechanisms. Counter current theory, based on laboratory experiments, suggests that groundwater chloride will be fractionated during diffusion through clay or shale confining layers. The direction of movement (either into or out of an aquifer) of chloride will be indicated by the chloride isotopes ratios.

Utility of this counter current fractionation theory was tested by determining the isotopic distribution of chloride in aquifers in Louisiana and Alberta, Canada. In Louisiana, the chloride isotope results were used to determine whether chloride in brines from wells was contributed by dissolution of nearby salt domes. The results suggest that the chloride in brine from wells had been made lighter by ion filtration than the salt dome chloride, and probably did not originate from the dome. In Alberta, Canada, chloride analysis was used to determine whether ion filtration or cross formational solute movement influenced chloride concentration in the Milk River aquifer. Analysis results suggested that both processes may be involved in the concentration of chloride in the aquifer, though the processes were not operating uniformly throughout the aquifer.

The results of this study indicate that chloride isotopes are fractionated in natural systems and that the fractionation may aid in understanding the origin of chloride and diffusion processes in groundwater. More specifically, chloride isotopes, in conjunction with other information, may allow researchers to distinguish the influences of ion filtration from those of cross formational solute movement. In addition, several general comments should be noted concerning the use of stable chloride isotope analysis for interpreting the groundwater systems. Some of the advantages of chloride isotope analysis are as follows:

1. Accurate isotope measurements are not difficult to obtain.
2. Chloride isotope distribution is probably not influenced by mechanisms other than counter current fractionation.
3. Other information used when interpreting chloride isotope analysis results, such as potential head and concentration gradients, is not difficult to obtain.

Though chloride isotope analysis can become a powerful tool for hydrogeologic research, considerable work needs to be done before there is complete confidence in interpreting the results.

Laboratory experiments which more closely describe diffusion processes in natural systems should be done and chloride ratios evaluated from such experiments. This would help close the gap between the original counter current experiments and natural systems. Also, other chlorine fractionation mechanisms such as those associated with hydrothermal and precipitation processes should be investigated to provide a better understanding of natural processes which could affect isotopic ratios of chlorine.

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Volatile Organic Ground Water Contamination in the Tucson Airport Super Fund Area, Tucson, Arizona

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ABSTRACT

The Tucson Airport Super Fund Area is currently being investigated by the Environmental Protection Agency, the Arizona Dept. of Health Services, the City of Tucson and the Arizona Dept. of Water Resources for volatile organics and heavy metal ground water contamination. The volatile organics include trichloroethylene (TCE), trichloroethane (TCA), dichloroethylene (DCE) and heavy metals, primarily chromium. The area is defined as north of Los Reales Rd. to distinguish this contamination from the US Air Force Plant No. 44 contamination south of Los Reales Rd. The investigation includes defining the hydrogeology, the extent of ground water contamination and potential contamination sources. The aquifer being contaminated is located in the Upper Santa Cruz Basin and is the principal source of domestic water for the City of Tucson. The area of concern currently contains 177 water wells; 24 of these wells are contaminated with TCE concentrations ranging from 5 ug/l to greater than 400 ug/l. This includes 6 City of Tucson public supply wells. There are currently 6 potential contamination sources being investigated. The first phase of Super Fund will enable the State and City governments to collect and analyze data which will be used for remedial action.

COMPUTER SIMULATION OF SALTWATER INTRUSION

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INTRODUCTION

Careful management and planning of groundwater aquifers is essential to assure users of an adequate and sustained yield of water. This process helps to avoid the "mining" of water and excessive drawdown of the water table and the dangers inherent therein. When the aquifer has a coastal boundary, an additional danger is the intrusion of the seawater into the aquifer. Excessive pumping could cause an increase in the salinity concentration of water, rendering it unfit for drinking and other purposes. Management of the aquifer includes knowing the location of the salt water interface under present and future pumping conditions. A numerical model is one method by which the location of the salt water interface can be estimated for a given set of hydrologic conditions.

Two types of models can be used to study saltwater intrusion into aquifers. The first type of model is one in which a sharp interface separates the fresh water and salt water (Bear and Dagan, 1964; Fetter, 1972; Glover, 1959; Kashef, 1975; Pinder and Page, 1976; Rumer and Harleman, 1963; Shamir and Dagan, 1971; Van der Veer, 1976). The second type assumes that the salt water has gradually mixed (diffused) with the fresh water resulting in salt concentration from zero to the concentration that exists in the sea (Bear and Dagan, 1963; Desai and Contractor, 1977; Gelhar et al., 1972; Henry, 1964; Segol et al., 1975). In general, there is always some degree of salt water diffusion in coastal aquifers. However, when the diffusion is confined to a narrow band in the aquifer, it may still be appropriate to use a sharp interface model. This report describes the development and use of a sharp interface model.

Several numerical methods have been used in sharp interface models, including the method of characteristics, finite difference and finite element methods. The concepts and techniques described by Sa da Costa and Wilson (1979) have some unique features and are used extensively in this study (Contractor, 1981a). The model utilizes linear triangular elements and the Galerkin weighted residual method for deriving the element equations. The model has been applied to a variety of flow situations for which analytic solutions are available and the numerical accuracy of the model confirmed. The model was also used to study the behavior of the aquifer in Guam; an aquifer which had a complex boundary geometry, basement topography and boundary conditions (Contractor et al., 1982). The model also has the capability of studying aquifers that are partially confined and partially unconfined. Even though aquifers of this type are rare, they are known to exist in Hawaii. An application of this kind is presented in this paper.

CAPABILITIES OF COMPUTER PROGRAM

The computer program can handle steady or unsteady flows, and can analyze both confined and unconfined aquifers. If the aquifer is confined, the program can consider leaky and nonleaky conditions. Additionally, any number of pumps can be accommodated at the nodes of the network. Recharge is constant in an element but can be varied from element to element. Specified head or flow conditions can be applied at the boundaries. At a coastal boundary, a mixed or third-type boundary condition can be specified. For steady flow conditions, the saltwater head can be specified to be zero along the boundary or at every node in the network. This procedure assures that the Ghyben-Herzberg condition is satisfied. The program can also be run in the unsteady mode with the Ghyben-Herzberg condition. If the saltwater head at every node in the network is specified to be much less than the anticipated freshwater head, the results of the computer program will show that the thickness of the saltwater layer is equal to an arbitrarily small value (BTOE). Under these conditions, the program can simulate flow in a freshwater aquifer. Since the heads are assumed to vary linearly across the triangular element, the velocity in each element will be constant and the program can print out the components in the x and y directions in each element in both the fresh water and salt water layers. These velocities can then be used to calculate the flow rates across any line or boundary. The program can also determine where in the network a freshwater or saltwater toe occurs. A saltwater toe occurs where the interface intersects the lower impervious boundary. The output provides the element number, the node numbers, and the fractional distance between the two nodes where the saltwater toe occurs. The same kind of information is also provided about the freshwater toe. A freshwater toe occurs where the phreatic surface intersects the lower impervious boundary or where the interface intersects the upper impervious boundary in confined aquifers.

VERIFICATION WITH ANALYTIC SOLUTIONS

To promote confidence in the program, it was advisable to check the output of the program against results that are analytically known. The program was checked out against three different groundwater problems. The first problem dealt with the steady state, one-dimensional saltwater intrusion into confined and unconfined aquifers. The second case was the unsteady drawdown of head due to pumping of

a well (Theis solution) in a confined aquifer. Finally, the program was checked against the analytic solution of the one-dimensional gravitational segregation problem (Gelhar et al., 1972). The results of all three simulations (Contractor, 1981(a)) indicated good agreement with the analytic solutions. Many more analytic solutions are available in the literature to check the numerical output of the program. However, the satisfactory agreement obtained in these three flow cases indicated that the basic numerical technique and procedures are sound.

APPLICATION OF MODEL TO THE NORTHERN GUAM AQUIFER

Guam is located within Micronesia at $13^{o}28'$ North latitude and $144^{o}45'$ East longitude. It is the largest and southernmost of the Mariana Islands. It is approximately 30 miles long, ranges from 4 to 11-1/2 miles wide and has a land area of 212 square miles excluding reefs. The island is divided into two nearly equal areas of different geology (WERI Staff, 1980). The northern half is a broad limestone plateau bounded by cliffs and the southern half is a dissected volcanic upland fringed with limestone along the east coast. The limestone plateau of Northern Guam slopes gently to the southwest from an altitude of about 600 feet in the north to less than 100 feet at the narrow midsection of the island. The various limestone units rest on a basement complex of older volcanic rocks. Because of their low permeability, the volcanic rocks act generally as a barrier to groundwater movement and in Southern Guam do not readily yield water to wells. The Mariana and the older Barrigada limestone constitute the main aquifer of Northern Guam. Taken together, the two limestone formations may be described as a permeable, massive, clean limestone that is relatively free of clay and volcanic detritus. The most important characteristics of the limestones in relation to groundwater movement are their high hydraulic conductivity and porosity.

Almost all residences on Guam are served by the public water supply system. The approximate amount of water produced from all sources is 28.5 million gallons per day (mgd). Most of this water (56%) is withdrawn from the aquifer underlying the northern half of the island by means of approximately 75 wells. The remainder of the water is obtained from surface water sources in Southern Guam (28%) and springs (16%). The economic growth and development of Guam is inextricably intertwined with the water resources available on the island. Increased quantity of water made available to the public is essential to future growth in population, industry and agriculture (Marsh and Winter, 1975; PUAG, 1971). Pumping water from the Northern Guam lens is a very cost-effective method of increasing the water supply. Increased pumping, however, should guarantee that the quality of the water is not compromised at the higher level of pumping. Sea water intrusion depends on the recharge to the aquifer and the distribution and rate of pumping from the aquifer. If the rate of pumping exceeds the natural rate, then mining of water from the aquifer will take place, and it will be only a matter of time before the lens is depleted and the aquifer is contaminated with salt water. Recovery of the aquifer from such a condition would be a very slow process.

The aquifer in Northern Guam is bounded by the sea coast on the east, north and west and by a volcanic outcrop on the south. The aquifer also has a complex basement of impervious volcanic rock. The elevations of the basement were the subject of a separate study (Biehler and Walen, 1981). Fifty-six reversed seismic refraction profiles were established throughout Northern Guam in order to define the basement.
Seismic refraction was successful because the volcanic rocks have a consistently higher seismic velocity (7,000-9,000 ft/sec) than the limestones (2,500-3,000 ft/sec) throughout most of the aquifer. On the basis of these data, a contour map of the basement surface was assembled. The basement elevations at the nodes of the element network were interpolated from this contour map.

There are about a dozen rain gages scattered over the island. The rainfall in Northern Guam undergoes a loss due to evapotranspiration and the rest of the rainfall recharges the aquifer. There is no surface runoff (streamflow) in Northern Guam. On the other hand, the rainfall in Southern Guam undergoes a loss due to evapotranspiration and the rest of the rainfall becomes direct runoff and streamflow. Very little infiltration occurs because of the impermeable volcanic rocks that outcrop in the south. This contrasting hydrologic feature is made use of by Mink (1976) to calculate the recharge to the aquifer in the north. The same technique is used in this paper. The USGS publishes streamflow records for many streams in the south of Guam (U.S. Geological Survey, 1978). The annual flow in these streams can be converted to inches of runoff. The evapotranspiration in the south is calculated as the difference between the average rainfall in the south and this runoff. The evapotranspiration in the north can be taken to be equal to that in the south. The recharge to the aquifer is then equal to the average rainfall in the north minus the evapotranspiration.

The Northern Guam aquifer was discretized into 222 triangular elements and 149 nodes (fig. 1). Pumps are located at 70 of these nodes. Thirty-three elements have one side along the coastline and 35 nodes on the coastline have saltwater heads specified as a function of time. Region I of the aquifer consists of the Mariana limestone and Region II the Barrigada limestone. The porosity of both regions was estimated to be 0.25. However, the conductivity of each region had to be determined by calibration of the model. This was done by using the historical hydrologic data of 1978, 79 and 80. The input data consisted of the recharge, pumping rates and elevations of the sea on a monthly basis. The output of the model was compared with the measured elevations of the water in six observation wells. The permeability in each region was varied till satisfactory agreement was obtained between the computed and and measured elevations. The result of the calibration run at one of the observation wells is presented in fig. 2. It was determined that the best estimate of conductivity for region I was 1000 ft per

day (fpd) and for region II was 5000 fpd.

MANAGEMENT RUNS

The model was used to determine the effect of increasing the current pumping rate of 18.5 million gallons per day (mgd) to 60 mgd. An analysis of the monthly rainfall and evapotranspiration over the aquifer showed that substantial recharge occurs only during the months of July to November, the remaining months of the year having zero recharge. This management scenario was modeled on the computer. The initial conditions were taken to be the same as those used for the calibration runs. The simulation was run for a year and the results at the end of the year were used as the initial conditions for simulation of a second year of the hydrologic conditions. This process was repeated for five years, at which time it was felt that steady cyclic conditions would prevail. Fig. 3 shows the water surface elevation at one of the nodes for the five year simulation. It can be seen that steady cyclic conditions were not achieved and that the simulation would need to be continued for several more years. Fig. 4 shows the variation of the saltwater head at the same node. It can be seen that the saltwater has to encroach into the aquifer much more before steady cyclic conditions prevail in the aquifer. Fig. 4 also shows that the interface has risen 35 feet in five years and would rise even more for steady cyclic conditions. The interface moves slowly to its new steady cyclic position because it has to force a large amount of seawater through the porous media.

APPLICATION TO AN AQUIFER THAT IS PARTLY CONFINED AND PARTLY UNCONFINED

Some aquifers have the unusual characteristic that a portion of it is confined and the rest of it unconfined. Aquifers of this kind are known to occur in Hawaii and their analysis has been difficult. The aquifer in Southern Oahu is composed of basalt. However, the basalt has a cap of limestone of lower conductivity than the basalt near the coastline. Fig. 5 shows the details of such an aquifer. The limestone acts as a leaky aquitard above the basalt aquifer. The location of the saltwater interface depends on the flow of fresh water and the hydraulic conductivity of the aquitard. A finite element grid was used to simulate the aquifer in fig. 5 using parameter values close to those of Southern Oahu. Figure 5 shows the location of the interface for different values of the flow rate and the hydraulic conductivity of the limestone cap (aquitard). It can be seen that the interface moves out toward the ocean for increasing values of flow rate and for decreasing values of the aquitard conductivity. The program can handle additional complexities such as pumps, local non-homogeneities, recharge, etc. Thus, this program is very well suited to manage aquifers of this type.

CONCLUSIONS

1. A two-dimensional finite element model has been developed to solve problems of saltwater intrusion into aquifers.

2. The finite element program has been checked for accuracy against analytic solutions of three different groundwater flow cases.

3. The model has been applied to the Northern Guam aquifer. The conductivity of the aquifer was calibrated using hydrologic data of 1978-80.

4. A Management run was made to determine the response of the aquifer to increased pumping. Simulation for a five-year period did not result in steady cyclic conditions. The reason for this appears to be the sluggish movement of the interface.

5. Some aquifers in Southern Oahu, Hawaii have the characteristic of being partly confined and partly unconfined. Such aquifers have been difficult to analyze. This program, however, can be used to locate the interface. An application of the program to such an aquifer is provided.

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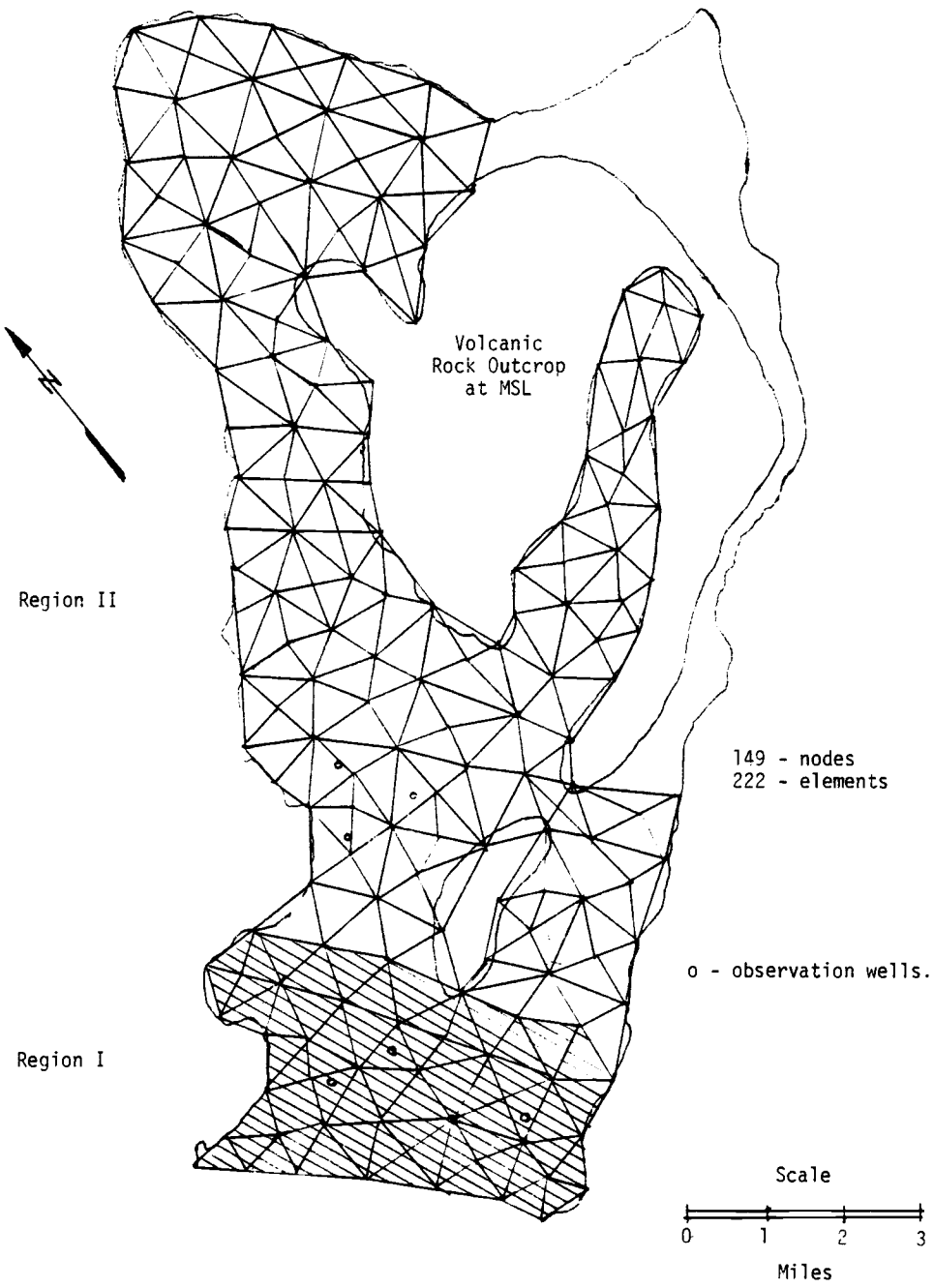

FIG. 1. DISCRETIZATION OF GUAM AQUIFER.

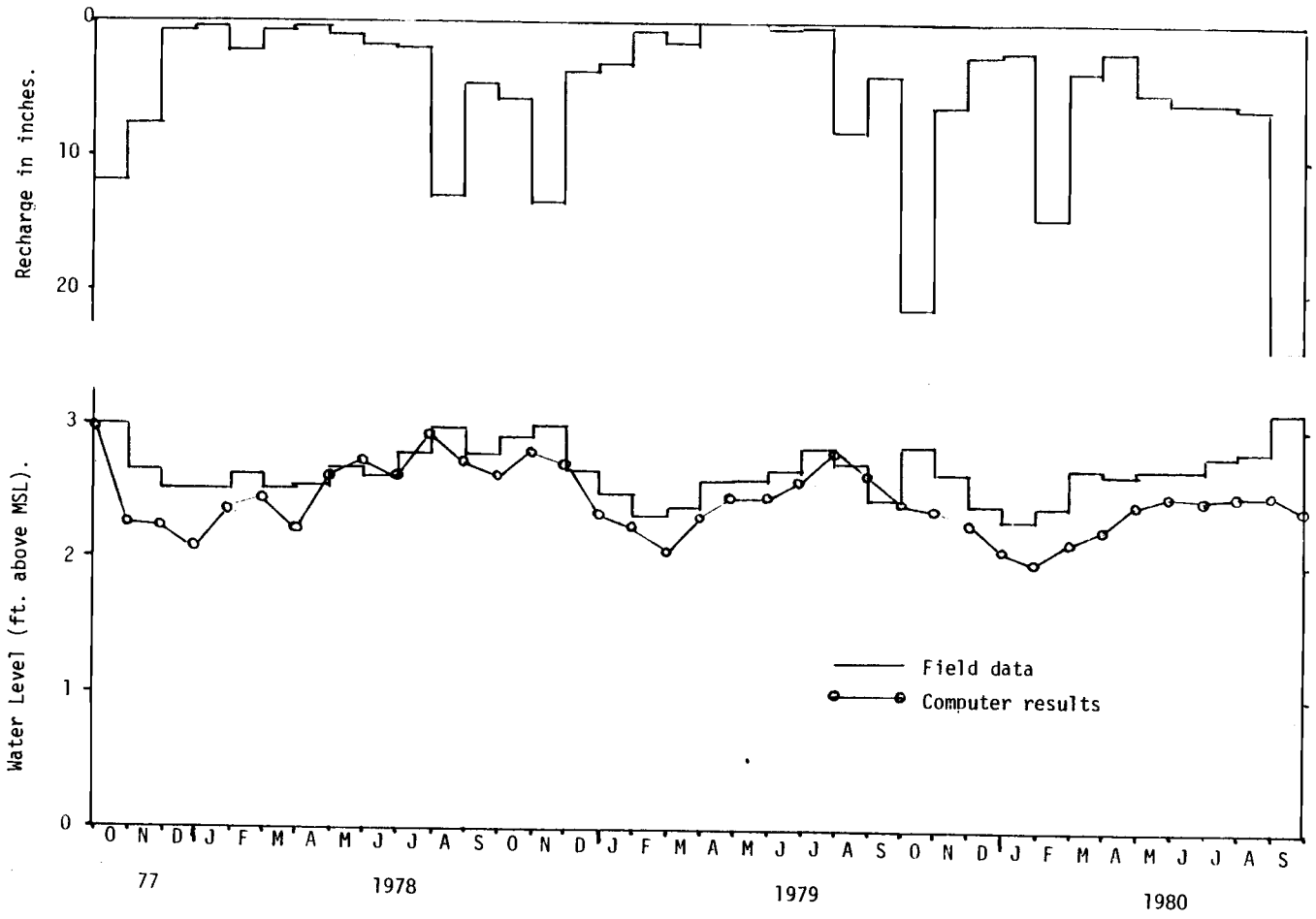

FIG. 2. CALIBRATION RESULTS AT AN OBSERVATION WELL.

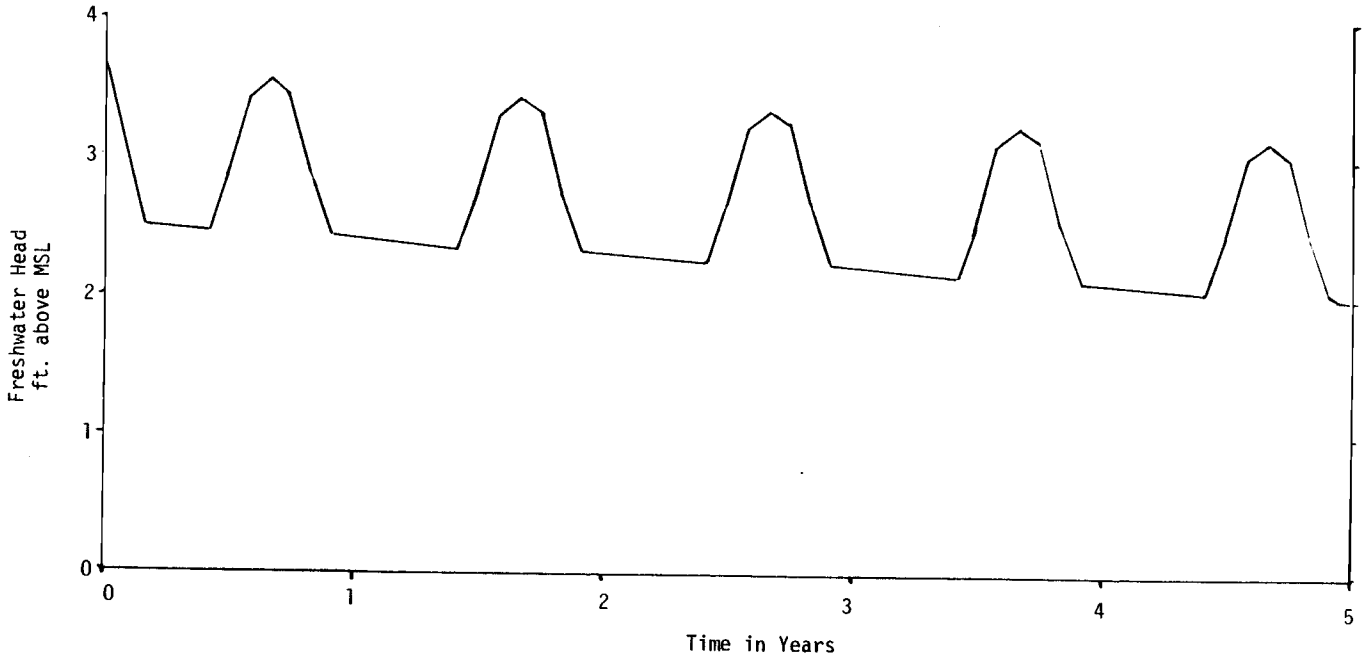

FIG. 3. VARIATION OF FRESHWATER HEAD AT A NODE FOR MANAGEMENT RUN.

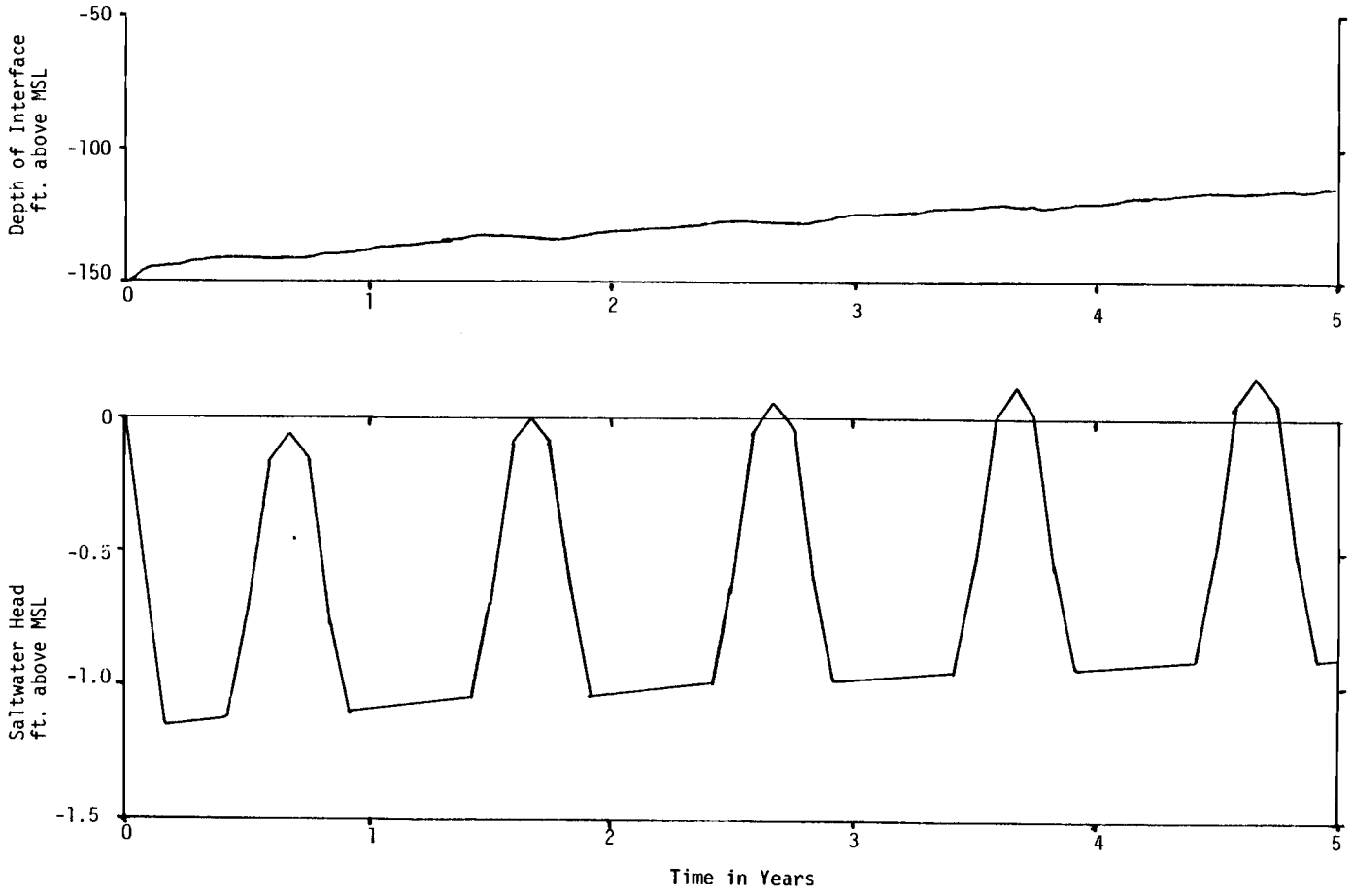

FIG. 4. VARIATION OF SALTWATER HEAD AND DEPTH OF INTERFACE AT A NODE FOR MANAGEMENT RUN.

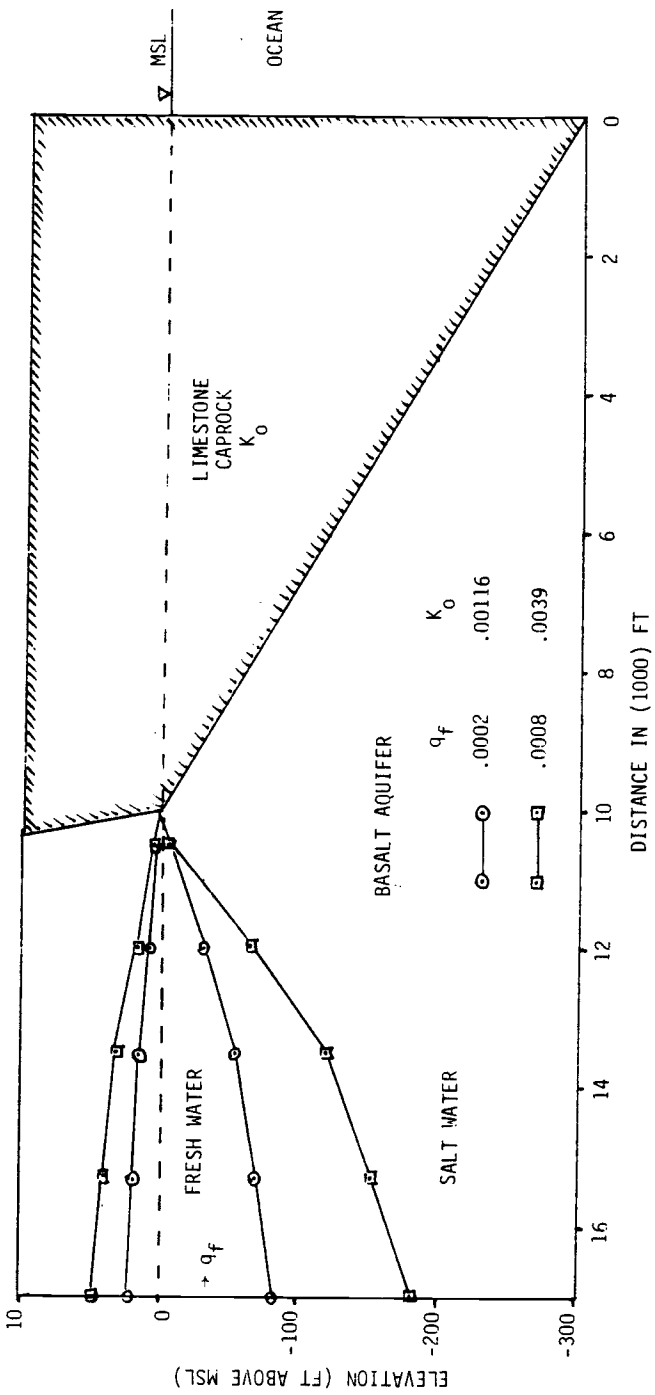

FIG. 5. FLOW INTO AN AQUIFER THAT IS PARTLY CONFINED AND PARTLY UNCONFINED.

EVALUATION OF GROUNDWATER MONITORING METHODOLOGIES IN THE TUCSON COPPER MINING DISTRICT

by

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ABSTRACT

The Upper Santa Cruz Basin Mines Task Force implemented a two year groundwater monitoring program in response to recommendations of an earlier investigation. The work program included monitoring several copper mines' tailing ponds and wells 15-20 miles south of Tucson. ASARCO mine was monitored to determine the source of high sulfates and TDS in the groundwater in the vicinity of the ASARCO ponds. A network of twelve sampling sites was sampled quarterly to look at water quality trends over time. One additional monitor well was drilled at the base of ASARCO's newest pond. The Anamax groundwater monitoring program consisted of investigating changes in water levels and water quality in the vicinity of its two tailing ponds to determine the hydrologic impacts of the ponds. The Duval program consisted of a network of thirteen monitor and seven interceptor wells. It was designed to determine the effectiveness of the interceptor wells as a management practice for preventing migration of mineralized tailing pond seepage to downgradient areas. The programs are compared in relation to their relative merits and their effectiveness in determining the groundwater quality impacts of the tailings ponds.

INTRODUCTION

Studies concerning tailing pond seepage and recharge of tailing pond water south of Tucson began over twelve years ago. Data collected in the early 1970's by the United States Geological Survey indicated recharge of tailing pond water at one mine. Monitoring of groundwater showed increased concentrations of sulfate and total dissolved solids in downgradient wells monitored over a ten year period. Information from a water rights case suggested significant groundwater recharge was occuring through tailing ponds south of Tucson (Thuss, 1978).

Earlier soils and water quality studies of the tailing ponds reached various conclusions. A materials investigation of the tailing ponds south of Tucson characterized the water quality, soil density, particle sizes and permeabilities of the tailings and tailing pond water (Engineers Testing Laboratory, Inc., 1973). The rather high sulfate and total dissolved solids concentrations in the tailing pond water and downgradient wells suggested the ponds were a possible source of degraded water in the wells. Two additional studies (W.A. Wahler and Associates, 1973)(Hail, 1974) completed a drilling exploration program at the mines' tailings ponds. The program included soil sampling and laboratory analyses of the soils. Additionally, these studies included drilling four monitor wells at the base of four different tailings ponds. Water with high total dissolved solids and sulfates, exceeding recommended Arizona State limits, was found at three of the four sites.

The Upper Santa Cruz Basin Mines Task Force completed its first study in 1979. Investigation of water levels and groundwater quality in the Sahuarita-Continental-Green Valley area was generally inconclusive regarding the impact of tailings ponds on groundwater since there were few monitor wells to sample near the tailings ponds and only one round of samples was obtained. The Mines Task Force recommended further studies in the tailing ponds areas to determine the effects of tailings ponds on the local groundwater. The three groundwater monitoring programs to be discussed in this paper were conducted as a result of the Task Force's 1979 recommendations (Upper Santa Cruz Basin Mines Task Force, 1979).

An independent study of the ASARCO tailings ponds effects on groundwater recharge and contamination was conducted during the same period as the Mines Task Force study of the ASARCO ponds (Thurnblad, 1982). The independent study evaluated the effectiveness of stable isotopes of hydrogen and oxygen as tracers of tailings pond recharge and subsurface movement. Isotope information coupled with

sulfate, calcium and total dissolved solids data suggested localized tailings pond recharge as the source of degraded downgradient monitor and industrial well water.

PEDCo Environmental, Inc. has just finished a groundwater and source monitoring study of the Cyprus-Pima tailing ponds area south of Tucson. The study followed the Resource Conservation and Recovery Act guidelines. Five wells were constructed for the program: three wells immediately downgradient, one well upgradient, and one well further downgradient of the suspected source. Results from this study are not yet available, but they will be used for recommendations to the Environmental Protection Agency as to what kinds of regulations are necessary for tailings pond disposal (PEDCo, 1983).

An evaluation of the monitoring programs performed as part of the Upper Santa Cruz Basin Mines Task Force Program is presented in this paper. These programs include the ASARCO, Anamax, and Duval study areas. The Task Force consists of various public entities and includes, or included, the four mining companies operating south of Tucson. Recommendations will be formulated by the Task Force for future activities in each site-specific area.

MONITORING PROGRAMS

ASARCO Program

The purpose of the ASARCO program was to determine the source of increased sulfates and total dissolved solids in the groundwater in the vicinity of the ASARCO ponds, and to develop, as necessary, recommendations for the future (Upper Santa Cruz Basin Mines Task Force, 1980).

The program included wells in Figure 1. All of these wells were measured for water levels in 1981 and 1982 and the underlined wells were sampled for water quality quarterly over a two year period. Two monitor wells at the base of ASARCO Tailings Ponds #1 and #2 were sampled. The City of Tucson monitor well was redeveloped after two years of non-use (since the 1979 Baseline Report). The Bureau of Indian Affairs' well was drilled during the monitoring program and was sampled during two subsequent sampling runs. M-6, M-7, M-8, M-10, M-11, SX-1, SX-2, and SX-3 are ASARCO industrial wells which are downgradient of the ASARCO tailings ponds. Beginning with M-6, the wells range from about one to two miles downgradient of the ponds. Two other water quality sampling sites were established at the ASARCO pits about one to two miles upgradient of the ponds.

The quarterly sampling was sufficient to obtain temporal trends for any plume movement into the wells. From both a monetary and a data interpretive viewpoint, the frequency was more than sufficient.

Constituents that were analyzed included all the major cations and anions, copper, molybdenum, total dissolved solids, electrical conductance (lab and field), pH (lab and field), and field temperature. Analyses were done by BC Laboratories of Bakersfield, California, an Arizona State Certified lab. Ten percent of the samples were split between BC and Arizona Department of Health Services Labs for verification.

Deficiencies of the monitoring program included: the lack of monitoring wells to the east and northeast of the tailings ponds, the need for sampling the tailings ponds, and the need for sampling additional constituents used in the copper milling process. It is difficult to describe the flow regime and water quality to the northeast of ASARCO Pond #1 since there are no monitor wells in that area. There is a known cone of depression described in earlier studies (Upper Santa Cruz Basin Mines Task Force, 1979) which is centered about the ASARCO and Cyprus-Pima well fields and defining the northern influence of the cone is difficult. Table 1 presents pond water quality data for ASARCO. Pond water quality is significantly different temporally and points to the necessity for more frequent pond sampling to obtain better understanding of variations in water quality. Finally, because of the addition of flotation agents, and the type of ore being processed, there is a need to analyze the ponds and wells for selenium, cyanide, and chemicals used in the copper and molybdenum flotation processes.

Anamax Program

The purpose of the Anamax tailings pond monitoring program was to determine the hydrologic impacts of the ponds, and to develop recommendations, as necessary, for future activities (Upper Santa Cruz Basin Mines Task Force, 1980).

The program included wells in Figure 2. Underlined are the wells sampled for water quality while the others were checked only for water levels.

Water quality samples were taken on a quarterly basis. This frequency was again sufficient for observation of seasonal trends and plume movement. Three wells sampled for water quality parameters were at the base of tailings ponds: P-1225, P-1758, and P-1759. The other two wells sampled were public supply wells ST-5, one-half mile east of Tailings Pond #3; and ST-6, about one mile east of the

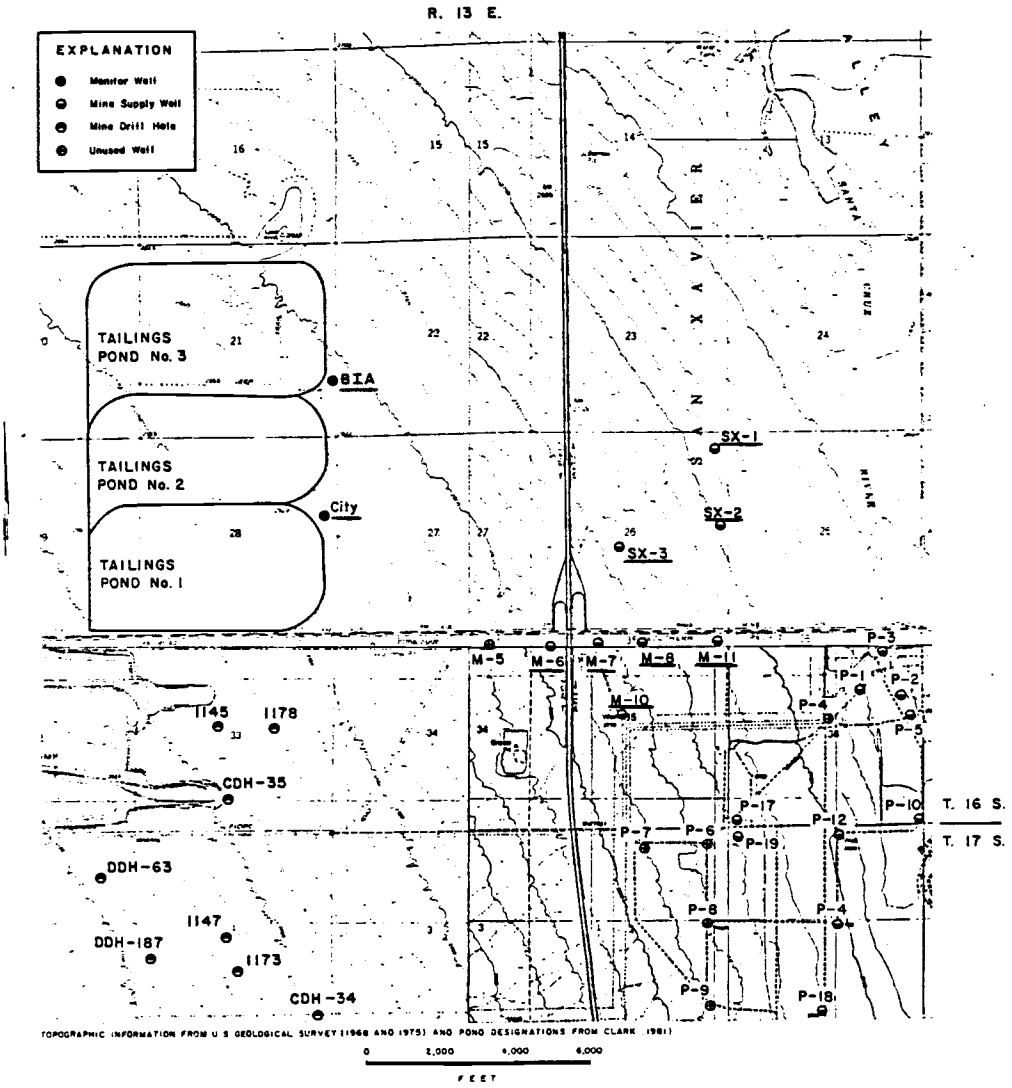

FIGURE 1 - LOCATION OF WELLS AND DRILL HOLES IN THE VICINITY OF THE ASARCO PONDS

TABLE 1 - CHEMICAL COMPOSITION OF WATER FROM ASARCO TAILINGS PONDS

Constituent (mg/l)	Pond No. 1	Pond No. 2		
Calcium	530	552	615	760
Magnesium	<1	5	2	1
Sodium	190	163	166	240
Potassium	49	70	-	72
Carbonate	35	0	0	0
Bicarbonate	0	39	41	18
Chloride	43	26	38	32
Sulfate	1,600	1,558	1,745	2,400
Nitrate	2	2	<1	2
Flouride	2.0	3.7	0.6	2.3
Iron	<0.08			0.05
Manganese	<0.04	-	-	0.08
Arsenic	<0.005	-	-	<0.01
Chromium	<0.04	-	-	<0.01
Selenium	0.013	-		<0.01
Molybdenum	1.9	5.5	16	2.0
Lead	-	-	-	<0.01
Copper	-	-	-	0.01
Zinc	-	-	-	0.14
Cadmium	-	-	-	<0.01
Boron	-	0.4	-	0.13
Vanadium	-	-	-	<0.1
Electrical Conductivity (micromhos at 25°C)	2,993	-	-	3,351
Total Dissolved Solids	3,500	-	2,629	3,333
Total Organic Carbon	7.8	-	-	6.7
Field pH	-	-	-	5.5
Lab pH	10.1	6.7	5.0	6.5
Date	5/4/81	2/17/71	10/18/71	8/20/81
Lab	U of A	ETL	ETL	BC Labs

1971 analyses from Engineers Testing Laboratory, Inc. (1973) and 1981 analyses from Thurnblad (1982).

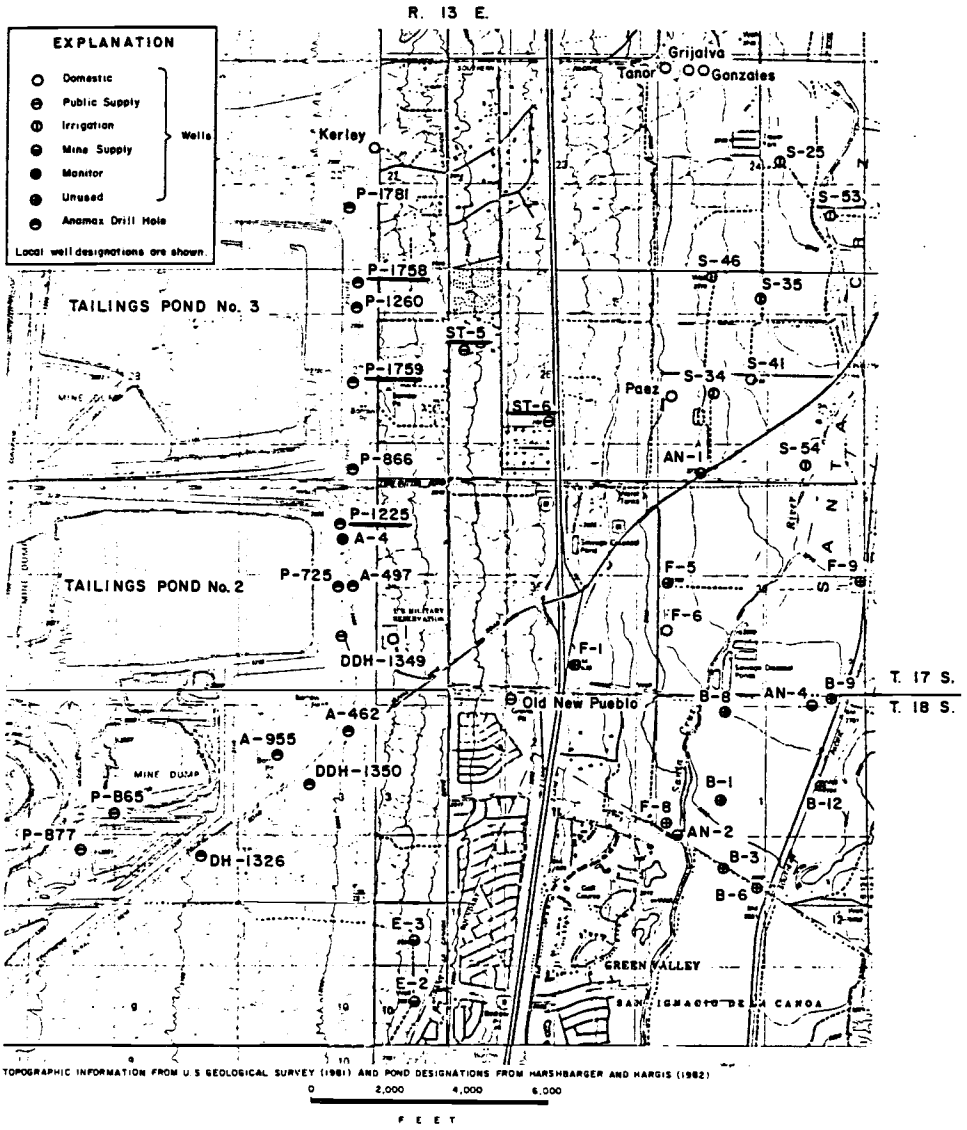

FIGURE 2 - LOCATION OF WELLS AND DRILL HOLES IN THE VICINITY OF THE ANAMAX PONDS

southeast toe of Tailings Pond #3. Constituents analyzed included all the ones sampled in the ASARCO program and also trace constituents such as selenium, arsenic, silver, mercury, zinc and lead. Well P-1225 showed a marked increase in sulfate and total dissolved solids over time indicating tailings pond recharge.

The program's additional needs, which will be recommended for future investigation, include: tailings pond water quality sampling, including products used in the flotation process and radiologic analyses, and placement of additional monitor wells between the wells next to the ponds and the public supply wells. The latest available data for pond analyses is from the early 1970's (Engineers Testing Lab, Inc., 1973) and if trends follow the ASARCO pond water quality trends, Anamax pond water quality could be substantially different. The number of wells in the Anamax program is not sufficient to define the areal extent of subsurface sulfate movement. In comparison to the ASARCO or Duval programs, the areal coverage of the Anamax area is minimal and could benefit by expansion. Water level data suggest an area of depressed groundwater elevation due to pit dewatering (Upper Santa Cruz Basin Mines Task Force, 1979). The necessity of wide areal coverage for water quality monitoring under altered flow directions is evident.

Duval Program

The purpose of the Duval monitoring program was to determine the effectiveness of the Duval interceptor well system as a management practice for mitigating downgradient groundwater degradation, and to develop recommendations concerning the Duval interceptor and water quality monitoring systems.

Figure 3 delineates the Duval monitoring system. Underlined wells were sampled for water quality parameters. Ten three-inch diameter monitor wells ring the Duval tailings ponds. Additionally, three three-inch diameter monitor wells are in a line from north to south about one mile east of the northeast part of the tailings pond. These wells were bailed with a mechanical stainless steel bailer. Seven interceptor or barrier wells also ring the pond. Interceptor wells were constructed to alter the direction of groundwater movement to mitigate potential water quality degradation of downgradient public supply wells. Two Green Valley supply wells were sampled for water quality parameters, CW-3 and GV-1. These wells are about one and one-half miles east of the ponds. Additionally, two Duval industrial wells, E-1 and E-4, were sampled for water quality. These wells are about one and one-half miles northeast of the pond. Finally, tailings pond water samples were also obtained and analyzed.

Sampling frequency varied from once per week for the interceptor wells, to once per month for the monitor wells and CW-3, to once per year for the other wells. Constituents analyzed included all the ones mentioned in the ASARCO program. The laboratory analyzing the water samples was the Duval lab. Ten percent of the samples were split between the Duval and Arizona State Health Labs for verification.

The Duval program was probably the most comprehensive of the three programs described. Over twenty-five wells were sampled for water quality, covering twelve square miles. Sampling frequencies were more than adequate and close monitoring enabled correlations between well water quality and interceptor well pumpage to be described. Monthly water level measurements in the monitor wells allowed examination of the effects of interceptor well pumpage on the direction of groundwater movement. Suggestions for improvements to the monitoring program include: addition of monitor wells between existing monitor wells and the public supply wells to better define the sulfate plume; additional analysis of constituents in the pond water which are used in the flotation processes, including cyanide, selenium, molybdenum, and selected organics; and installation of a pumping system for the monitor wells in order to flush adequate amounts of well water prior to sampling. Finally, it appears that the interceptor well pumping regime could be improved by additional pumping capacity northeast of the ponds.

CONCLUSIONS

1. Evidence from previous investigations suggest tailings pond recharge as the source of degraded water in wells downgradient of the ASARCO and Duval ponds. Evidence from this current study corroborates this observation and also suggests a similar situation for Anamax tailings pond #2. There is no reason to believe the situation at Cyprus-Pima is any different.

2. Future monitoring programs at all study areas should emphasize placement of monitor wells upgradient of public supply wells in order to detect problems and prevent supply well degradation.

3. Monitoring of tailings ponds on an annual to semi-annual basis is necessary to determine constituents for analysis in downgradient wells.

4. Sampling frequency for wells and ponds could be reduced to an annual basis for the complete suite of constituents. However, indicator constituents such as sulfate and total dissolved solids should be sampled quarterly. Annual monitoring should include constituents added in the milling process for flotation purposes.

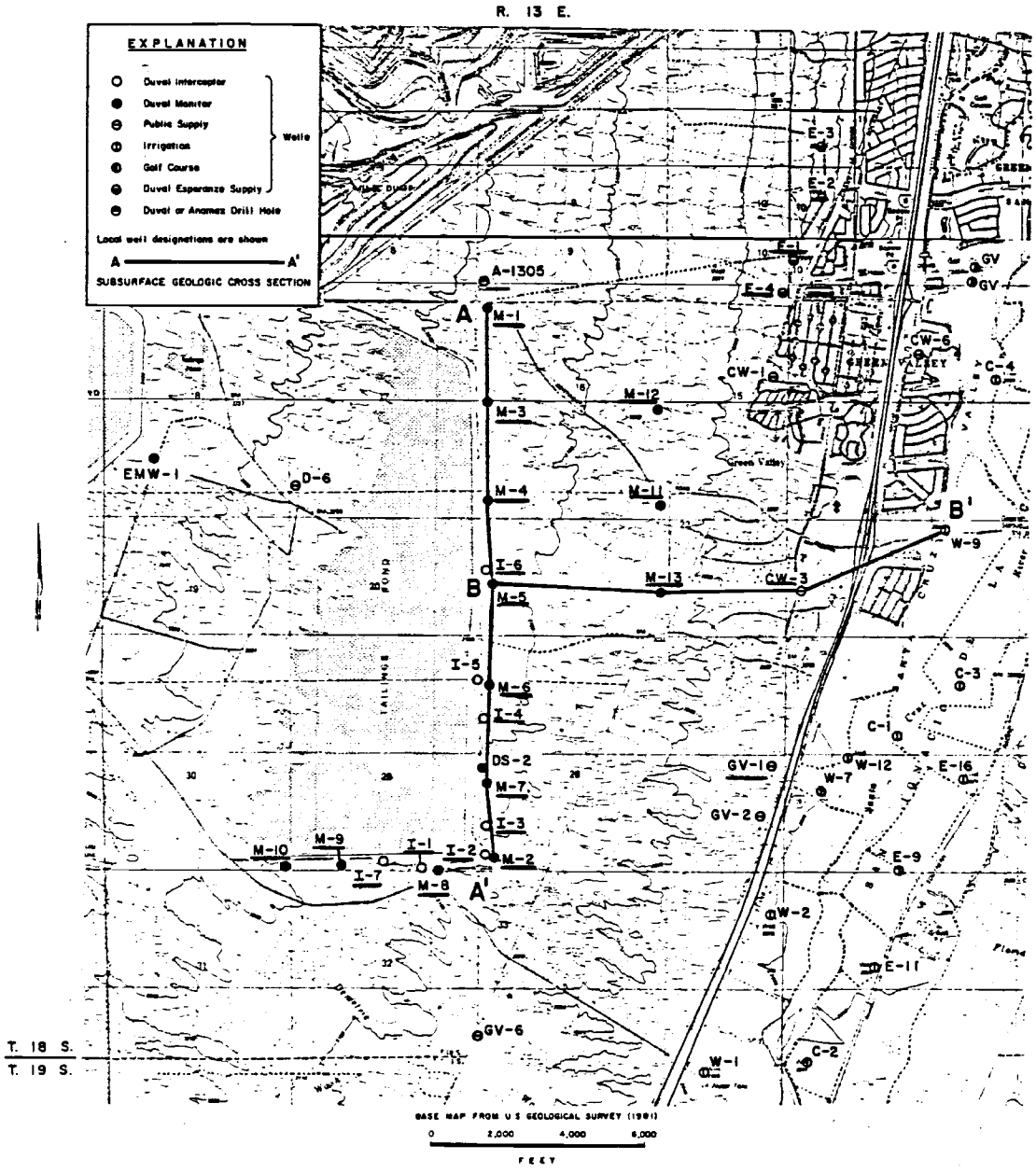

FIGURE 3 - LOCATION OF WELLS, DRILL HOLES AND SUBSURFACE GEOLOGIC CROSS SECTIONS IN THE VICINITY OF THE DUVAL SIERRITA POND

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VIRUS FATE IN GROUNDWATER

by

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INTRODUCTION

Groundwater traditionally has been considered to be safe for consumption without treatment. Currently, however, groundwater accounts for about 50% of the documented waterborne illness in the United States (Craun, 1979). Of these, over 65% have a probable viral etiology. A total of 50 waterborne outbreaks occurred during 1980, increasing the annual average of outbreaks to 39 for the five-year period from 1976-1980. This represents more than 50% increase over the 1971-1975 average of 24. The five-year averages have steadily increased from an annual average of ten during 1951-1955. Before that period, the trend was declining (Lippy, 1981).

It should be emphasized that reporting of waterborne disease outbreaks, particularly in individual systems, is notoriously poor. Given the difficulty of performing epidemiologic studies and inadequacy of waterborne disease reporting, evidence for potential health problems related with land application of wastewater will often rest on our ability to assess the fate of human pathogens in the environment.

More than 110 different virus types may be present in raw sewage including polioviruses, echoviruses, coxsackieviruses, rotaviruses, hepatitis virus, adenoviruses, calciviruses, etc. The list of pathogenic human enteric viruses has continued to grow during the last decade. Rotaviruses are now recognized as a major cause of childhood gastroenteritis, sometimes resulting in dehydration and death in infants. Rotaviruses also cause diarrhea in adults and several waterborne outbreaks have been documented recently (Sutmoller et al., 1982). The Norwalk viral agent has been demonstrated to be the cause of numerous water and foodborne outbreaks of epidemic viral gastroenteritis (Kaplan et al., 1982; Gunn et al., 1982).

In most of the groundwater waterborne disease outbreaks, underground migration of septic tank effluent was believed to be the probable cause. But, viruses may also find their way into groundwater from the intentional land application of domestic sewage for purposes of crop irrigation or groundwater recharge. Viruses also occur in solid household domestic wastes because of the widespread use of disposable diapers and, thus, find their way into landfills. Viruses have been detected in groundwater beneath land application sites where sewage was being applied and near domestic landfill sites (keswick and Gerba, 1980). Viruses were detected at depths as great as 46m and at distances as great as 900m at some sites. An understanding of factors that control virus migration through the subsurface is necessary for the management of septic tanks, landfills and wastewater land treatment systems.

SURVIVAL OF VIRUSES IN THE SUBSURFACE

The fate of pathogenic bacteria and viruses in the subsurface will be determined by their survival and their retention by soil particles. Both survival and retention are now believed to be largely determined by climate, nature of the soil and nature of the microorganism. Climate will control two important factors in determining viral and bacterial survival: temperature and rainfall. The survival of microorganisms is greatly prolonged at low temperature; below 4°C they can survive for months or even years (Gerba et al., 1975). At higher temperatures, inactivation or die-off is fairly rapid. In the case of bacteria, and probably viruses, the die-off rate is approximately doubled with each 10°C rise in temperature between 5°C and 30°C (Reddy et al., 1981). Above 30°C, temperature is probably the dominant factor determining virus survival time. While the survival of human pathogenic enteric viruses in soil has previously been studied, almost no information on the survival of viruses in groundwater exists. Hurst et al. (1980) concluded that temperature and adsorption appear to be the most important factors affecting virus survival in soils. Factors controling virus survival in groundwater have yet to be determined.

MIGRATION OF VIRUSES IN THE SUBSURFACE

Rainfall mobilizes previously retained bacteria and viruses and greatly promotes their transport in groundwater. Several studies have shown that the greatest degree of drinking water well contamination occurs after periods of heavy rainfall (Dewalle et al., 1980; Lewis et al., 1980).

The nature of the soil will also play a major role in determining survival and retention. Soil properties influence moisture-holding capacity, pH and organic matter - all of which will control the survival of bacteria and virus in the soil. Other soil properties such as particle size, cation exchange capacity and clay content will influence retention. Factors that may influence virus movement through the subsurface are shown in Table 1.

TABLE 1. Factors that may influence virus movement through the subsurface

Factor	Comments
Soil Type	Fine-textured soils retain viruses more effectively than light-textured soils.
pH	Generally, adsorption increases when pH decreases.
Cations	Adsorption increases in the presence of cations. Rainwater may desorb viruses from soil due to its low conductivity.
Soluble Organics	Can compete with viruses for adsorption sites. No significant competition at concentrations found in wastewater effluents. Humic and fulvic acid reduce virus adsorption to soils.
Virus Type	Adsorption to soils varies with virus type and strain.
Flow Rate	The higher the flow rate, the lower adsorption of virus to soils.
Saturated vs. unsaturated flow	Virus movement is less under unsaturated flow conditions.

Retention of viruses by soil is, of course, a paramount consideration in protecting groundwater from contamination. Filtration plays a major role in bacterial removal, although adsorption is also involved. Virus removal is believed to be almost totally dependent on adsorption (Keswick and Gerba, 1980). The nature of the soil probably plays a major role in determining the degree of virus adsorption. Generally, soils containing clays are more effective in retaining viruses. Understanding the physical-chemical properties of the soil as related to virus adsorption has been seen as an aid in understanding the potential for groundwater contamination. Virus adsorption to soils is believed to be largely governed by electrostatic double-layer interactions and Van der Waal's forces (Murray and Parks, 1980). More recent work indicates that hydrophobic interactions may also play a role (Farrah et al., 1981).

A number of studies evaluating virus adsorption to soils using batch reactors have been conducted (Goyal and Gerba, 1979). In these studies, a given amount of soil was mixed with virus suspended in a solution and adsorption determined after a given period of time. The results of such studies indicate that virus adsorption is related to cation exchange capacity, pH, surface area and organic matter, but firm predictive correlations between virus adsorption and these factors have not been well established.

Establishment of predictive relationships between soil factors and virus adsorption is further complicated by genetic variability among different types and strains of viruses. The isoelectric point of viruses varies among strains and types of viruses, and this has been shown to effect virus adsorption to the surfaces of charge modified glass beads (Zerda et al., 1981). Recently, we have been conducting studies on the removal of different viruses by soil columns. These studies indicate that the retention of viruses by a given soil can vary markedly as shown in Table 2.

Table 2. Removal of Poliovirus type 1 and coliphage f2 by Pomello fine sand*

Column Depth (cm)	Percentage of Virus Remaining	
	Poliovirus 1	f2
0	100	100
2	41	109
27	13	102
67	2	98
87	1	101

*Suspended in secondarily treated sewage and passed through a 2 cm diameter column at a flow rate of 120-194 cm/day. Initial virus concentration was approximately 10^5 plaque forming units/ml.

The results of these column studies indicate that some viruses will have a greater potential for contamination than others. Differences in adsorption among viruses may be due to variations in configuration of proteins in the outer capsid of the virus, which affects the net charge on the virus. This affects the electrostatic potential between virus and soil, which, in turn, affects the degree of interaction between the two particles. Thus, it appears that no one virus can be used as the sole model for determining the adsorptive behavior of viruses to soils and that no single virus can be used as the model for determining viral adsorptive capacity of all soil types.

From the foregoing discussion, it is apparent that many factors control the removal and persistence of pathogenic enteric viruses during the movement of wastewater through the soil. Additional research in this area is needed to further define processes involved in the survival and transport of viruses in the subsurface.

ACKNOWLEDGEMENTS

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VIRUS SURVIVAL IN GROUNDWATER

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INTRODUCTION

Virus survival in the subterranean environment is believed to be influenced by three interacting factors: the nature of the virus, the nature of the soil and the climate. Different types and strains of virus exhibit varying abilities to survive, based on differences in their genetic makeup. The effect of soil type on virus survival has been extensively studied using various soils (Hurst et al, 1980). The results of that study are presented in Table 1.

TABLE 1. Factors which affect virus survival in soil

1.	As viral adsorption to soil increases, virus survival is prolonged.
2.	Virus survival increases with increasing levels of exchangeable aluminum.
3.	Virus survival decreases with increasing pH and resin extractable phosphorus.
4.	As temperature increases, virus survival decreases.
5.	Aerobic soil microorganisms adversely affect virus survival, while anaerobic microorganisms have no effect.

Climatic conditions also play a role in virus survival. Rainfall is important in that it can cause desorption of virus from soil particles, thus allowing migration to the groundwater (Gerba and Lance, 1978; Wellings et al, 1975). Temperature has also been shown to influence virus survival in the marine environment. In surface waters, sunlight is thought to be an important factor in virus inactivation.

Virus survival in marine waters has been extensively studied, due to the large number of outbreaks of viral illness associated with consumption of contaminated shellfish. Little information is available concerning the persistence of viruses in groundwater. Keswick et al (1982) found that human enteric viruses are capable of surviving longer than 24 days in groundwater. Their experiments, however, were carried out in McFeter's survival chambers, which may have excluded contact with components of the groundwater that could influence virus survival, such as antagonistic bacteria.

The purpose of this investigation is to study the survival of viruses in groundwater, with the ultimate goal of developing a model to predict safe distances between drinking water wells and septic tanks.

MATERIALS AND METHODS

Groundwater samples were collected from seven different sites (Table 2). The samples were collected aseptically in sterile polypropylene containers, and packed on ice for shipment to the laboratory.

TABLE 2. Sample collection sites

Site Description	Groundwater Temperature	Depth
New York 1 - monitoring well northeast of landfill	12°C	< 100 ft
New York 2 - monitoring well north of landfill	12°C	< 100 ft
North Carolina 1 - well near septic system	14°C	deep water table
North Carolina 2 - well near septic system	14°C	shallow water table
Tucson - drinking water well	23°C	> 200 ft
Arizona - monitoring well	23°C	> 200 ft
Wisconsin - drinking water well	12°C	50 ft

Fifty-ml aliquots of water were placed in sterile, polypropylene tubes and seeded with one of the following viruses: MS-2 (a bacteriophage) and one of two animal viruses, Poliovirus-1 or Echovirus-1. To determine the influence of microorganisms on virus survival, duplicate tubes containing water passed through 0.45 μm and 0.2 μm filters were used.

The tubes were incubated at 4°C, 12°C or 23°C. One-ml aliquots were withdrawn on days 1, 2, 3, 5, 7, 10, 15, 20, 25, 30, 40, 50 and 60. Samples containing animal viruses were frozen for future assay.

Determination of the number of MS-2 phage in samples was performed using the agar overlay plaque technique (Adams, 1959). The bottom layer consisted of trypticase soy agar. The overlay contained trypticase soy broth and 1% agar. One ml E. coli ATCC 15597 and a 1-ml sample were added to the overlay which was poured onto the bottom layer. Plaques were counted after 24 h of incubation at 37°C.

RESULTS AND DISCUSSION

Initial experiments were performed to determine the reproducibility of results. MS-2 phage were seeded into 9 tubes containing 50 ml of groundwater and 3 tubes were incubated at each temperature. As no major differences among the replications were noted (Table 3), subsequent experiments were done using only one tube per set of experimental conditions.

TABLE 3. Reproducibility of virus survival determination

Incubation Temperature	Replication	Survival -Log N/N_o* After 10 Days
4°C	a	0.43
	b	0.31
	c	0.37
12°C	a	1.1
	b	1.15
	c	0.95
23°C	a	1.85
	b	1.84
	c	1.89

*N=number of plaque forming units (pfu) per ml after specified length of time; N_o=initial number of plaque forming units per ml.

To determine whether the age of the water sample had any effect on results, water was inoculated with MS-2 one day after collection and after it had been stored for 30 days at 4°C. No large differences in survival of MS-2 were seen (Table 4).

TABLE 4. Effect of age of water on survival of MS-2

	Survival - Log N/N_0	
Day	1 Day Old	30 Days Old
0		
1	0.041	0.079
2	0.56	0.47
3	0.74	0.69
5	1.6	1.2
7	2.2	2.2
10	2.2	2.5
15	2.6	2.7
20	3.2	2.9
25	4.0	4.1

The influence of microorganisms on virus survival is shown in Table 5. The results varied greatly from sample to sample. More investigation needs to be done to more clearly define the role of microorganisms in virus survival.

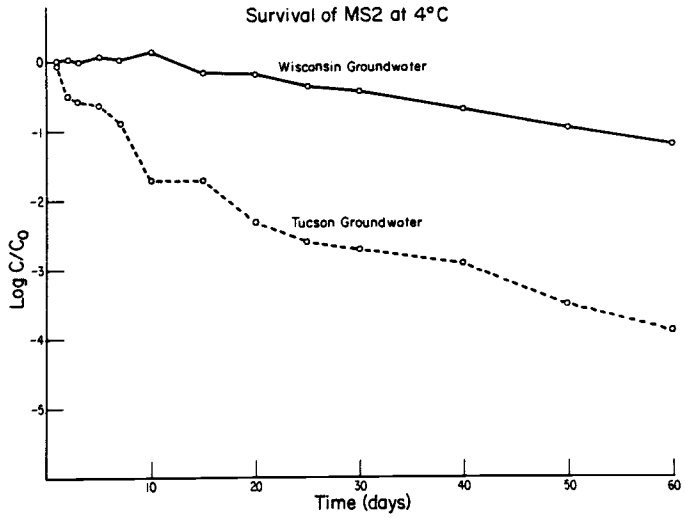
TABLE 5. Effect of microorganisms on the survival of MS-2

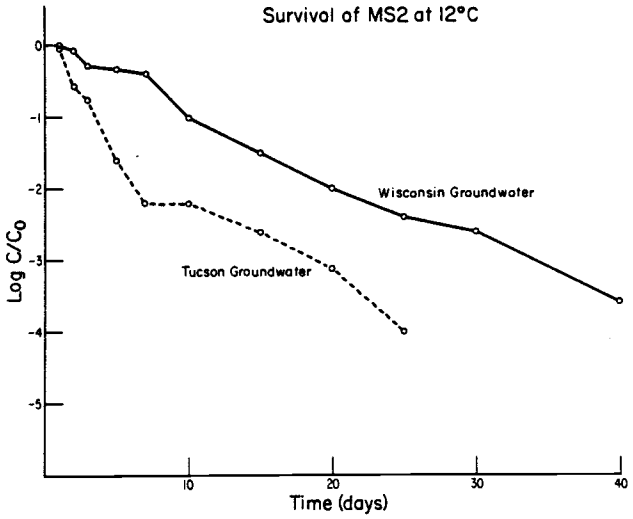
	Survival - Log N/N_0*	
Sample	Filtered	Non-filtered
North Carolina 1	1.8	0.84
North Carolina 2	0.86	3.5
Arizona	3.2	1.6
New York 1	0.95	1.0
New York 2	1.5	0.97

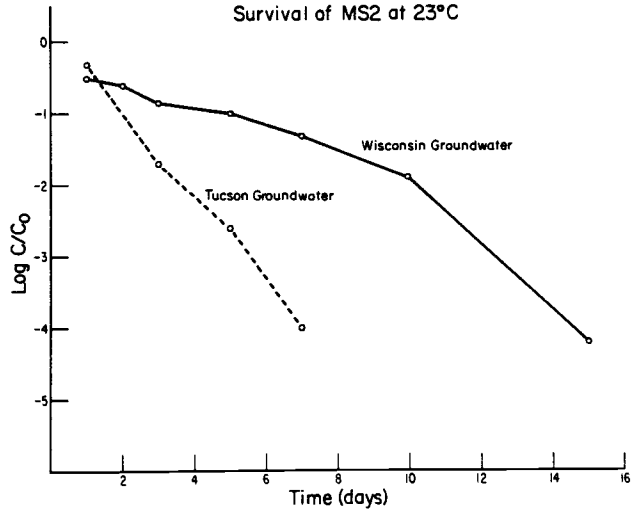
*After 30 days, incubation temperature 12°C.

The temperature of incubation played a major role in the survival of the virus. Figures 1, 2 and 3 show the inactivation rate of MS-2 in the Tucson and Wisconsin groundwater samples. It was found that as incubation temperature increased, the survival rate decreased. However, the rate of the decrease was strikingly different for the two groundwater samples at all three temperatures.

FIGURES 1, 2 and 3. Survival of MS-2 in Tucson and Wisconsin groundwaters at 4°C, 12°C and 23°C

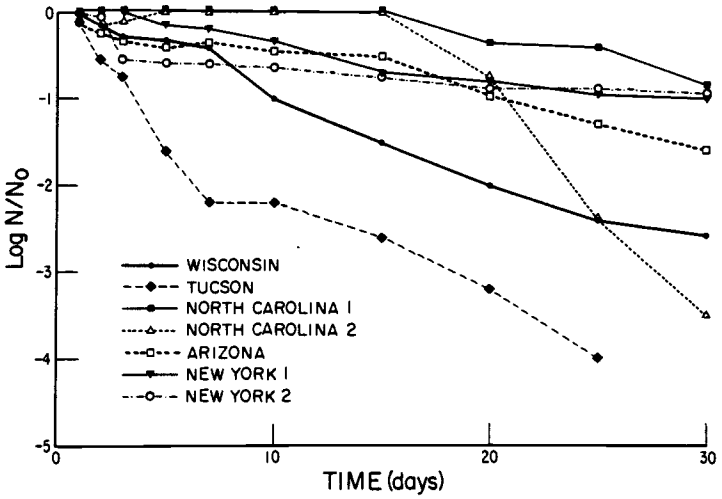






The survival rate of MS-2 at 12°C in all seven groundwater samples is shown in Figure 4. A wide variation in survival rates occurred among the different waters.

FIGURE 4. Survival of MS-2 at 12°C



CONCLUSIONS

From these results it was concluded that: (1) persistence of viruses in groundwater varies with location of sample collection, (2) viruses survive longer in groundwater samples incubated at lower temperatures [4°C] than at 12°C or 23°C, (3) the influence of microorganisms on virus survival varies in different groundwater samples, and (4) virus survival in groundwater is longer than in surface water.

However, because of the wide variation in survival rates from site to site under the same experimental conditions (temperature and the presence or absence of microorganisms), other factors which influence survival of viruses must be present in the water. These factors need to be studied to explain this variation.

ACKNOWLEDGMENTS

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Ground Water Contamination of the Estes Landfill, Phoenix, Arizona

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ABSTRACT

The ground water contamination potential of the Estes Landfill, Phoenix, Arizona has been evaluated by the City of Phoenix and the Arizona Dept. of Health Services. The landfill is located in a recharge zone of the Salt River Valley aquifer. The aquifer is under water table conditions. The depth to ground water ranges from 80 feet to 15 feet. Ground water monitoring wells were installed up-gradient and down-gradient from the landfill. Ground water samples collected from the wells during flow events of the Salt River indicated leachate production from the landfill; a mound of ground water develops and intrudes the solid waste. The leachate characteristics include volatile organics and heavy metals: vinyl chloride, trichloroethylene and barium. Analysis of solid waste borings indicated only small quantities of organics and heavy metals. Currently the ground water is used for industrial and agricultural purposes. However, the ground water could be used as a domestic water supply because it has an acceptable ambient water quality. Ground water monitoring is continuing with the intent of using the data to design a leachate migration control system for the landfill and to distinguish contaminants from an adjacent landfill.

Author Index of Arizona Section (AWRA) Proceedings

The following index is a compilation of all articles published through the auspices of the Arizona Section of the American Water Resources Association (**AWRA**). The index lists the authors who contributed articles for the proceedings of meetings held jointly with the Hydrology Section of the Arizona Academy of Science. In addition, proceedings of four symposia sponsored by the Arizona Section (**AWRA**) are also included.

The listing is arranged in alphabetical order with each author who, either singly or jointly, contributed one or more manuscripts appearing before the title of the works.

Following the title, a location is indicated which identifies the where the work may be found. A volume number (identified by the label V#) refers to publications of the series Hydrology and Water Resources in Arizona and the Southwest (Volumes 1; 1971 through Volume 13; 1983). Symposia (identified by the label P#) can be identified using the following list:

P1 - Symposium on Water Conservation Alternatives
(April 12, 1979; Phoenix, Arizona)

P2 - Symposium on Flood Monitoring and Management
(October 26, 1979; Tucson, Arizona)

P3 - Symposium on Water Quality Monitoring and Management
(October 24, 1980; Tucson, Arizona)

P4 - Symposium on Ground Water Quality Management
(October 29, 1982; Tucson, Arizona)

EXAMPLE:

Agthe Donald E.
An Application of the Almon Polynomial Lag to Residential Water Price Analysis - V12
The Impact of Socioeconomic Status on Residential Water Use: A Cross-Section Time-Series Analysis of Tucson, Arizona - V9

The author, Donald E. Agthe, has written two articles for the AWRA proceedings. The first article, "An Application of the Almon Polynomial Lag to Residential Water Price Analysis", appears in volume 12 (1982). The second article, "The Impact of Socioeconomic Status on Residential Water Use: A Cross-Section Time-Series Analysis of Tucson, Arizona", appears in volume 9 (1979).

Author Index of Arizona Section (AWRA) Proceedings