Peter F. Ffolliott

VOLUME 3

HYDROLOGY and WATER RESOURCES in ARIZONA and the SOUTHWEST

PROCEEDINGS OF THE 1973 MEETINGS
OF THE
ARIZONA SECTION—
AMERICAN WATER RESOURCES ASSN.
AND THE
HYDROLOGY SECTION—
ARIZONA ACADEMY OF SCIENCE

MAY 4-5, 1973, TUCSON, ARIZONA

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PREFACE

Each Spring during the annual meeting of the Arizona Academy of Science, the Arizona Section of the American Water Resources Association joins with the Hydrology Section of the Academy to co-sponsor a program of mutual interest -- the Hydrology and Water Resources of Arizona and the Southwest. This forum traditionally provides a unique opportunity for those engaged in Arizona and Southwest water-related research to present their recently completed or ongoing research. The program also provides excellent opportunities for graduate students to present their research findings in an atmosphere less formal than most professional meetings.

This volume is a compilation of the papers read and discussed at the third joint meeting held at the University of Arizona in May, 1973. Economics has dictated a more compact form from previous editions.

Although this volume is the product of the Arizona Section, AWRA, its fruition would not have been possible without the generous assistance of the Water Resources Research Center, University of Arizona, and its Director, Professor Sol Resnick. Finally, it is both a pleasure and a relief to state that any errors are the responsibilities of the authors.

RON S. BOSTER
Tucson

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ABSTRACT

IMPACT ON THE ENVIRONMENT BY WATER RESOURCES DEVELOPMENT

by
Sol Resnick
Director, Water Resources Research Center
University of Arizona

Are engineers looking at the impact on the environment due to their water resources developments? What is needed is an interdisciplinary approach, imagination, and real concern with regard to our environment. Examples of this type of thinking and research are as follows: Importing water by towing iceberg trains from Antarctica; evaporation suppression using floating coupled styrofoam rafts; water harvesting from graveled-plastic covered desert areas; reuse of treated industrial and sewage effluents; artificial groundwater recharge by means of pits or wells; controlling urban storm runoff by diverting the water onto greenbelts paralleling the washes and using the water for irrigating the greenbelts and parks along the washes, and using lake storage in the parks for taking the peak off the floods and as a facility for water-related recreation; and increasing irrigation efficiencies through newly developed trickle irrigation methods.

USE OF STOCK PONDS FOR HYDROLOGIC RESEARCH

ON SOUTHWEST RANGELANDS [1]

J. R. Simanton and H. B. Osborn [2]

INTRODUCTION

The Walnut Gulch Experimental Watershed in southeastern Arizona is operated by the USDA, ARS Southwest Rangeland Watershed Research Center in Tucson. Hydrologic research on the 58-square-mile rangeland watershed includes the following: estimates of rainfall amounts, intensities, and variability based on records from 95 recording rain gages; estimates of runoff based on continuously recording water level recorders at 11 concrete flume-weir measuring structures, 12 livestock watering ponds, 6 V-notch weirs, 2 Venturi-type flumes, and 2 H-flumes; and, fluvial sediment samples with cableway, wading, and pumping samplers. Sediment deposition is estimated from pond surveys.

In this paper, the use of livestock watering ponds as a relatively inexpensive method of comparing rainfall amounts with runoff sediment volumes is discussed.

CLIMATE

The climate of Walnut Gulch is semiarid, with the rainfall biseasonal and monsoon in type. The annual rainfall distribution is characterized by a strong summer maximum and a weaker winter maximum. The summer rains are usually short-lived, high-intensity, air mass thunderstorms occurring in late afternoons and evenings from June through September. The winter rains are usually prolonged, low-intensity frontal storms occurring from December through February. The average annual precipitation is about 13 inches, of which 70% falls during the summer. Summer thunderstorms produce nearly all of the annual watershed runoff.

WATERSHED DESCRIPTION

There are 19 livestock watering ponds on Walnut Gulch. Semiarid rangeland wildlife also use the ponds as a source of water. Without disturbing either livestock or wildlife, Southwest Watershed Research

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1. Contribution of USDA, ARS Soil, Water, and Air Sciences.

2. Research hydrologic technician and research hydraulic engineer, Southwest Rangeland Watershed Research Center, Tucson, Arizona.

Center hydrologists use the ponds as an inexpensive source of rangeland hydrologic records. Twelve of the ponds have been instrumented with continuously recording water level recorders. Materials and labor for instrumenting a typical stock pond cost about $410. These ponds drain watersheds with widely different soils and covers and represent a wide variety of rangeland combinations. Also, the pond drainage areas are grazed at different intensities. Several of the watersheds are heavily grazed; others receive little or no pressure from livestock (all pond drainage areas have been heavily grazed at one time or another).

In this study, records from 5 of the 12 stock ponds were analyzed. Four of the 5 have the longest continuous records (over 10 years), and the fifth was of interest because its watershed was root-plowed and reseeded.

In addition to the installation of water level recorders, topographic surveys are made of each pond to determine its storage capacity. These surveys provide a sediment accumulation record for each pond, as well as a means to quantify the rainfall-runoff relationship for each watershed.

The vegetation on Walnut Gulch consists mainly of grass and brush. Brush dominates on the lower 2/3 of the watershed, and grass dominates on the upper 1/3. Predominant brush species are spreading creosotebush (Larrea divaricata), whitethorn (Acacia constricta), and american tarbush (Flourensia cernua). Grass species include black grama (Bouteloua eriopoda), blue grama (Bouteloua gracilis), sideoats grama (Bouteloua curtipendula), and tobosa (Hilaria mutica). Vegetative covers on the different pond drainages are grouped as brush, grass, or brush-grass, based on visual observations. Soils are generally deep, well drained, poorly developed, medium to moderately coarse textured gravelly loams.

The five ponds selected for analysis are referred to hereafter as Pond No. 1, 7, 14, 20, and 23 (Figure 1). Pond drainage area, vegetative cover, principal soil type, percent slope, and years of record for each pond are listed in Table 1. The two listings for pond 1 represent the watershed condition before and after root plowing in the spring of 1971. At that time, the pond drainage was reseeded to sideoats grama and blue grama, but these grasses had not become completely established after 2 seasons.

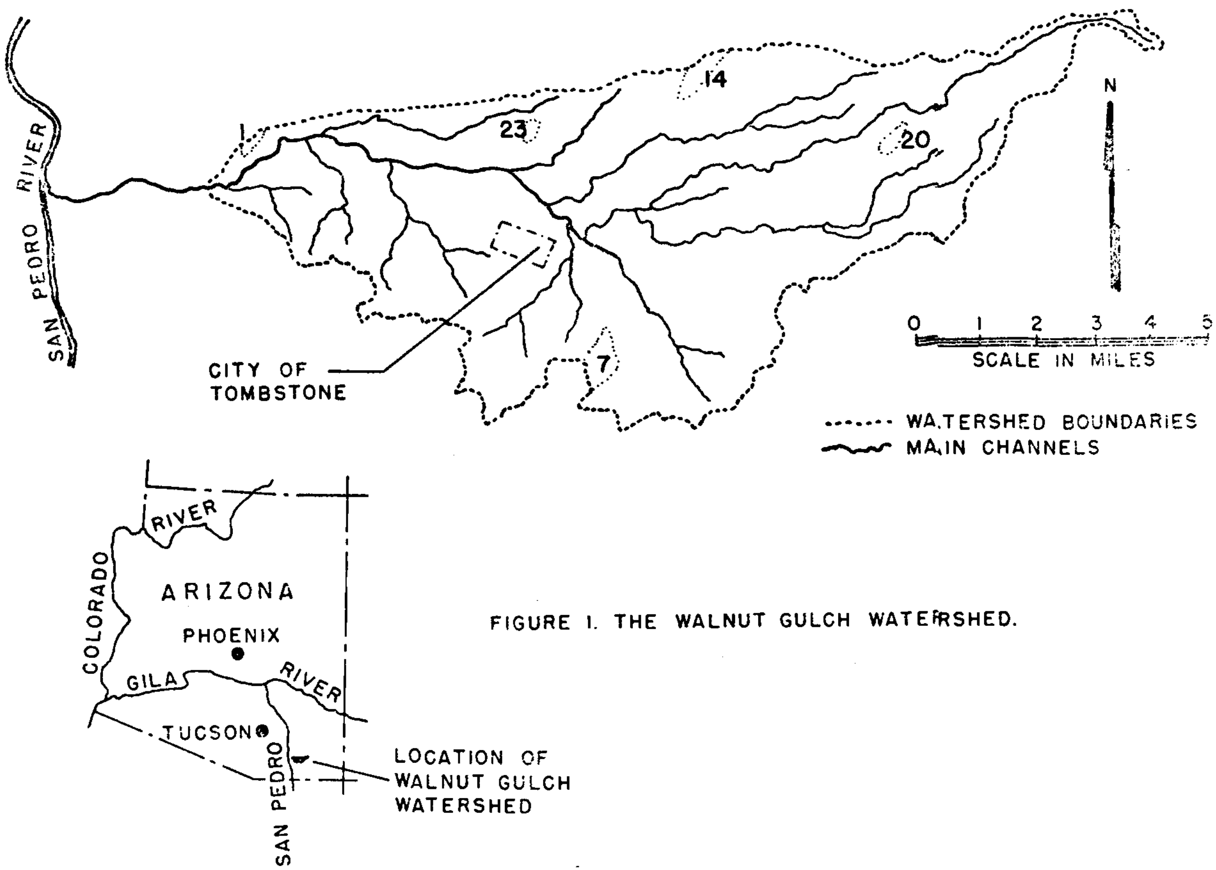

FIGURE 1. THE WALNUT GULCH WATERSHED.

TABLE 1. Description of 5 selected Walnut Gulch Stock pond watersheds.

Pond	Drainage Area(Acres)	Vegetation	Soil*	Slope(%)	Years of Runoff Record
1	109	Brush	Rillito-Karro	3-15	5
1	109	Ripped, reseeded to grass	do	3-15	2
7	253	Brush-grass	Rillito-Cave	3-30	12
14	372	Grass	Hathaway-Bernadino	8-15	10
20	142	Grass	do	8-15	14
23	108	Brush	Rillito-Laveen	3-15	13

*All soils listed are gravelly loams.

RAINFALL-RUNOFF RELATIONSHIPS

Runoff from various sized semiarid watersheds has been related to many different variables. Kincaid and Williams (1966) reported that runoff volume decreased significantly as crown cover increased on 6-x12-foot plots. Osborn and Lane (1969) found that total precipitation was the primary variable for determining runoff volume from very small semiarid rangeland watersheds (0.6 to 11 acres). Reich and Hiemstra (1965) reported that for watersheds larger than one square mile, runoff peaks were best correlated with maximum 30-min. rainfall depth.

Runoff volumes for the 5 selected stock ponds were related to total storm amounts and maximum 15-minute depths, assuming a linear relationship (Figs. 2 and 3). From the figures and the correlation coefficients, there is a suggestion that the runoffs from the watersheds of about 100 acres and less are more highly correlated to total storm rainfall than to maximum 15-min. depth; and, that the runoffs from the watersheds of about 250 to 350 acres are more highly correlated to maximum 15-min. depth. If true, the most likely explanation is that

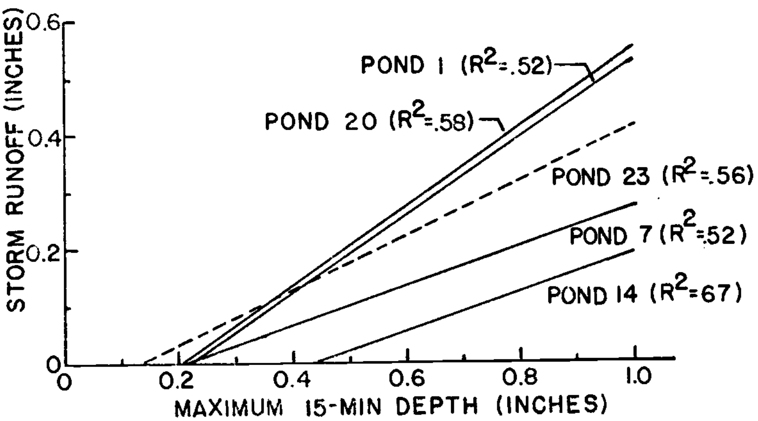

FIGURE 3. Storm runoff versus maximum 15-min. rainfall depth for 5 selected Walnut Gulch stock pond drainages.

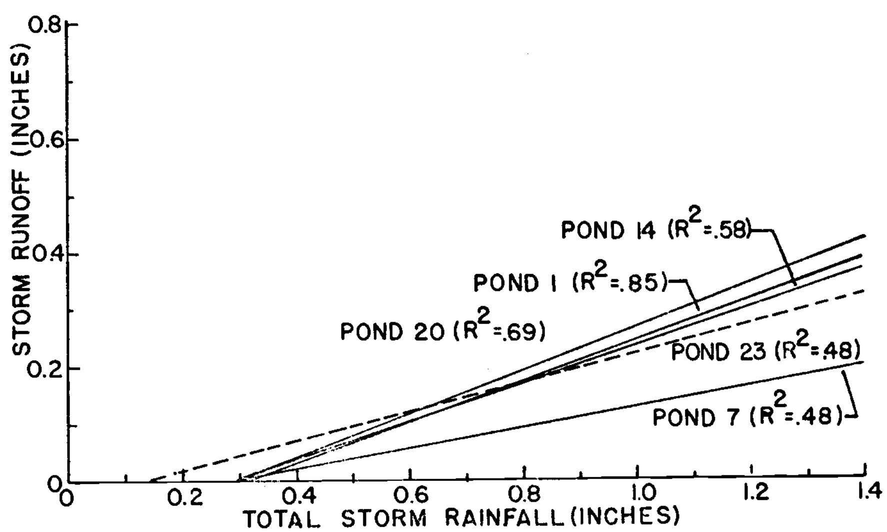

FIGURE 2. Storm rainfall--runoff relationship for 5 selected Walnut Gulch stock pond drainages.

storm rainfall is more variable over the larger watersheds, and therefore more poorly correlated to runoff than for the very small watersheds. However, a more detailed analysis when more data are available from all 12 instrumented stock ponds will be necessary to verify this suggested relationship.

Mass curves of accumulated summer rainfall and runoff indicate some differences in rainfall-runoff relationships between the 5 stock pond drainages (Fig. 4 and Table 2).

TABLE 2. Relationships between rainfall and runoff for 5 selected Walnut Gulch stock pond drainages.

Pond	Drainage Area (acres)	Years of Record	Ave.Summer Rainfall (Inches)	Ave.Summer Runoff (Inches)	Runoff/ Rainfall
1	109	1966-70	9.42	0.90	.10
1	109	1966-72	9.50	1.11	.12
7	253	1962-71	8.83	.33	.04
14	372	1960-68	7.71	.78	.10
20	142	1962-71	9.07	.83	.09
23	108	1961-72	7.54	.70	.09

Except for pond 7, about 10% of the summer rainfall, on the average is measured as runoff into stock ponds. Runoff from the drainage above pond 7 is about 1/2 that of the other 4 watersheds. This indicates that, although the intense highly variable thunderstorm rainfall generally tends to mask differences in rainfall-runoff relationships caused by differing watershed characteristics, in some cases, these differences make meaningful differences in the rainfall-runoff relationship. Possible explanations for the lower runoff volume from the drainage above pond 7 are: This watershed has the densest brush-grass cover of any of the 5 ponds and has recovered the most from earlier over-grazing; the relatively complex soil and rock surface may be more porous than that of other pond drainages; and, the watershed is grazed more lightly than the other watersheds.

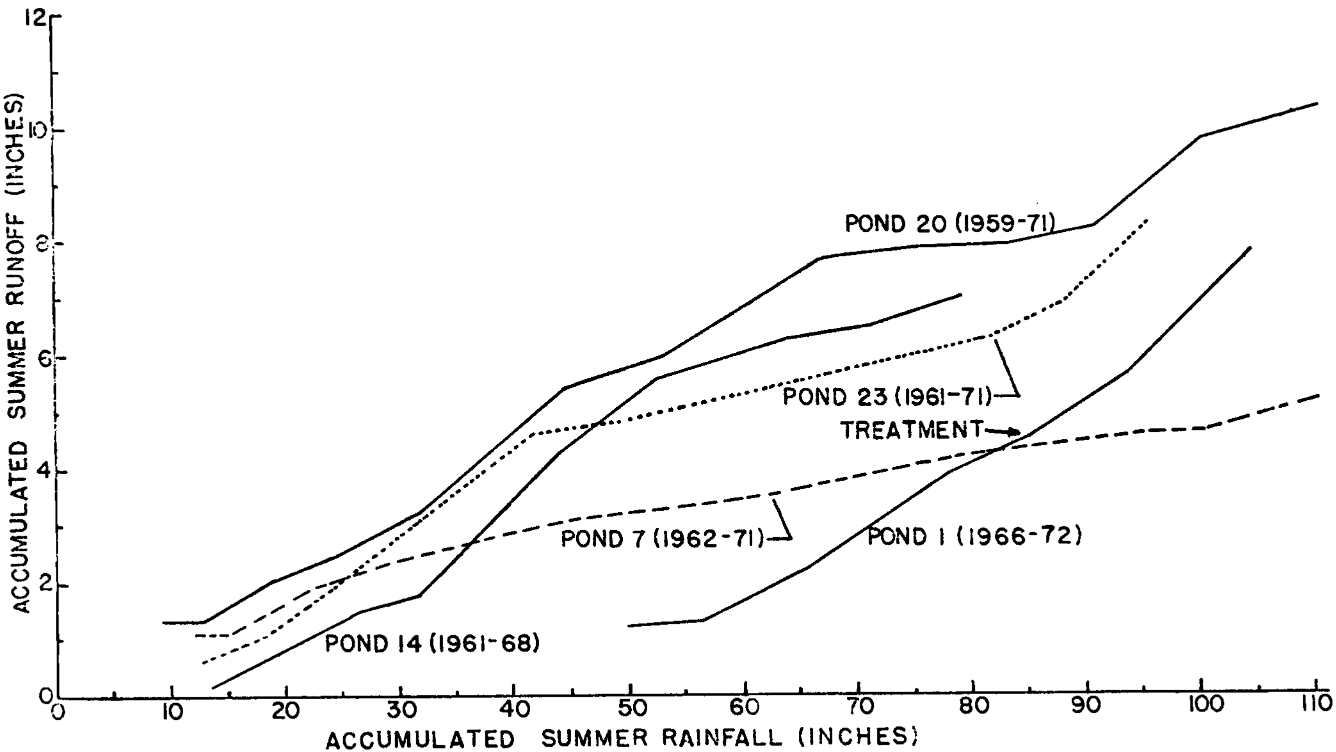

FIGURE 4. Accumulated summer rainfall versus accumulated summer runoff for 5 selected Walnut Gulch stock pond drainages.

Several investigators, including Renard (1970), have reported decreasing water yield per unit area with increasing watershed size for the ephemeral streams of the Southwest. Primarily because of the record from pond 7, but also because of natural rainfall variability and the short records, this effect is not apparent in the analysis (Fig. 5 and 6). Obviously, more quantitative descriptions and longer records are needed to derive rainfall-runoff relationships for the stock pond drainages on Walnut Gulch.

Runoff from the pond 1 watershed increased after the 1971 root plowing and reseeding (Fig. 7). However, the most intense storm rainfall (2.5 inches in 30 minutes) for the period of record occurred in 1972, so the indicated increase in runoff rate per unit area indicated in Figure 7 is misleading. However, the record does not suggest that ripping decreased runoff as might have been expected.

SEDIMENTATION

The principal purpose for instrumenting the stock ponds on Walnut Gulch was to estimate sediment production from very small (less than one-square-mile) rangeland watersheds. The results to date have not been very satisfactory because of a variety of instrumentation and surveying problems and because of the natural variability of thunderstorm rainfall. For example, on several occasions sediment has been removed from ponds before a survey could be made, thus losing one or more years of record. However, most of the problems that have plagued early evaluations have been corrected, and records from all 12 ponds should yield better results in the future.

Sediment yields were estimated from the records available and from surveys at ponds 1, 7, 14, and 23 for various periods (Table 3).

TABLE 3. Sediment yields for 4 selected Walnut Gulch Stock Pond Drainages.

Pond	Years	Ave. Summer Runoff (Inches)	Sed.Yield (ac-ft/mi^2/yr.)	Sed. Yield per 1" of Runoff (ac-ft/mi^2/yr.)	Cover
1	1966-70	0.90	0.33	0.37	brush
1	1971-72	1.64	.54	.33	seeded to grass
7	1962-72	.33	.14	.42	brush grass
14	1960-68	.78	.31	.40	grass
23	1961-71	.70	.41	.59	brush

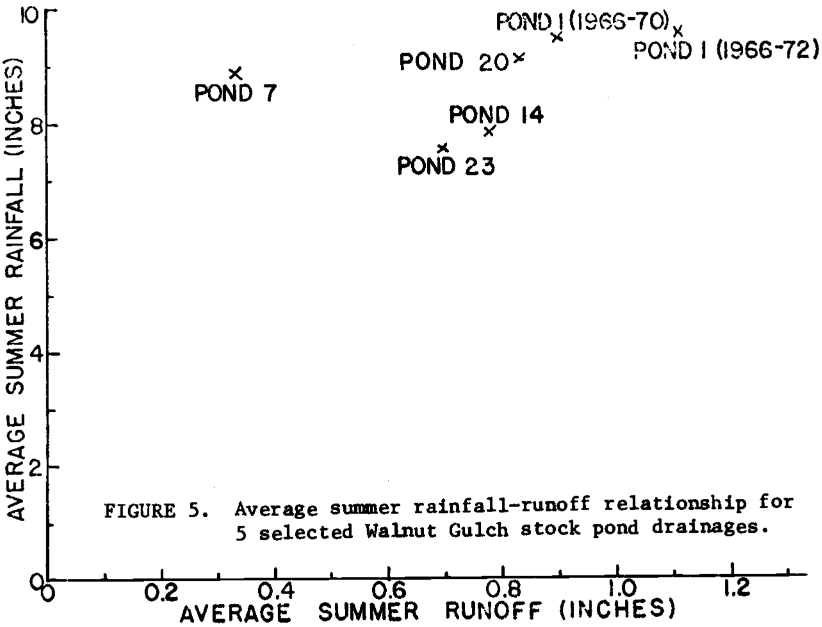

FIGURE 5. Average summer rainfall-runoff relationship for 5 selected Walnut Gulch stock pond drainages.

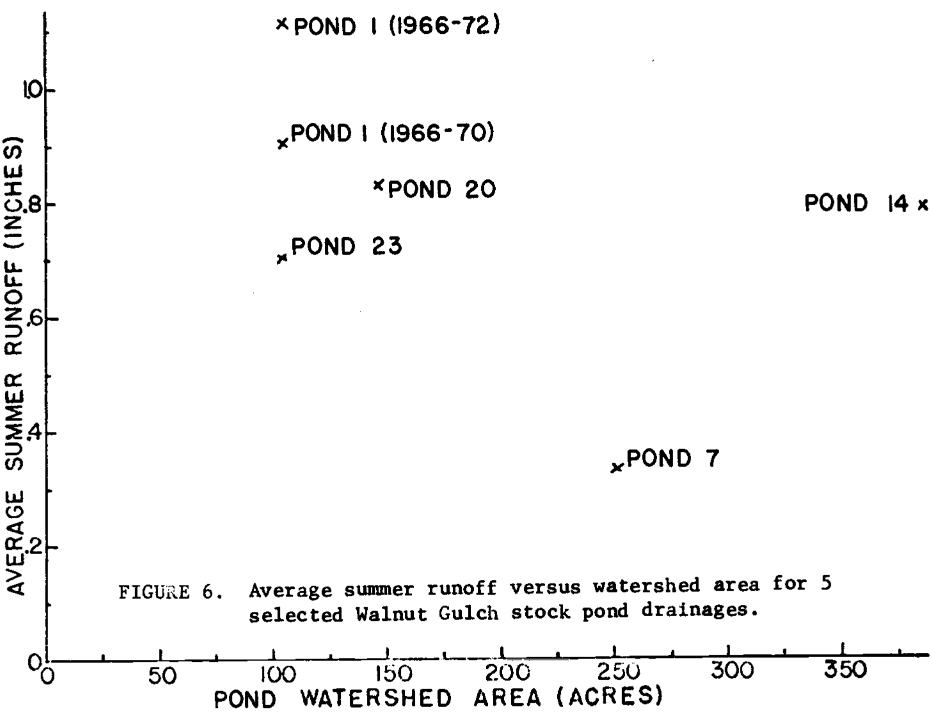

FIGURE 6. Average summer runoff versus watershed area for 5 selected Walnut Gulch stock pond drainages.

Sediment yields from watersheds of ponds 1 (before treatment) 7, and 14 all averaged about 0.4 ac-ft/mi^2/yr/inch of runoff. Sediment yield from pond watershed 23 was about 50% greater than that from the other 3 pond watersheds. The watershed above pond 23, which has been heavily grazed in the past, is highly eroded and almost devoid of grass. Again, however, the available data are insufficient to reach definite conclusions.

There was no significant change in sediment yield from the pond 1 watershed after the 1971 treatment, although higher sediment yield rates generally are expected for exceptional events such as the storm in 1972. The watershed was root plowed along the contour and left heavily furrowed and pitted, which may explain why the sediment yield from the watershed was less than expected. This low yield may be only temporary, or if the grass becomes well established and grazing is deferred, the sediment yield may remain low. Again, analyses of current and future data collected from all 12 stock pond drainages should answer this question.

SUMMARY

Stock watering ponds in the Southwest can be instrumented to provide valuable hydrologic information, particularly for rainfall-runoff relationships and sediment yields. On the Walnut Gulch Experimental Watershed in southeastern Arizona, 12 such stock ponds have been instrumented. Because of a variety of problems, analyses based on early records are inconclusive, but future records should provide valuable information in hydrologic research on the semiarid rangelands of the Southwest.

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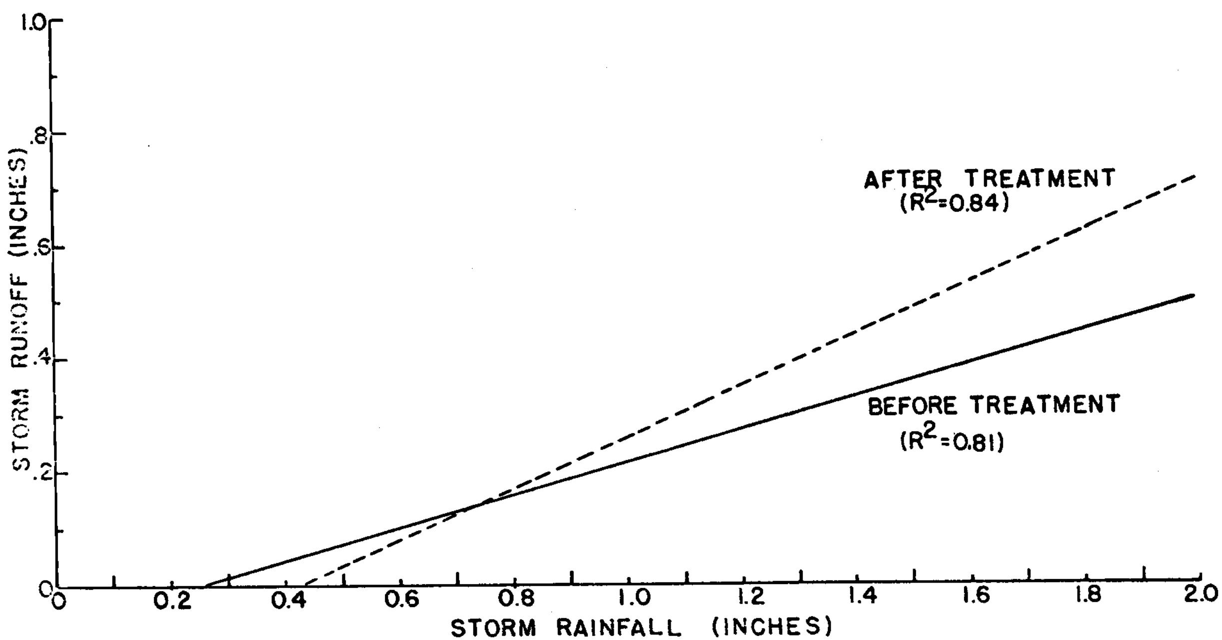

FIGURE 7. Rainfall-runoff relationships before and after treatment for Walnut Gulch stock pond drainage 1.

PROBABILITY DISTRIBUTIONS OF SNOW COURSE DATA FOR CENTRAL ARIZONA

Lawrence E. Cary and Robert L. Beschta

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INTRODUCTION

Arizona represents an area of snowfall scarcity, with most of the state receiving little or no snowfall. Appreciable snowfall does occur, however, in the more mountainous portions of the state where orographic processes have a pronounced effect. Even so, much of this snow does not remain on the ground during the winter, but often "disappears" between succeeding storms due to snowmelt or evaporation. Only at the higher elevations is it common for snowpacks to build up and remain during the winter months. Though these areas are of limited areal extent, they provide a major contribution to Arizona's surface water supply.

The central highlands of Arizona are of particular importance from the standpoint of snowpack accumulation. Encompassing the San Francisco Mountains, Mogollon Rim, and White Mountains, the central highlands extend in a broad arc from the north-central to the east-central portion of the state. Annual temperatures for this region average less than 50°F with winter temperatures often remaining below 32°F, especially at the higher elevations. At lower elevations, daily temperatures commonly exceed 32°F, and contribute to the ephemeral nature of low elevation snowpacks. Average annual precipitation varies from 15- to 35-inches with approximately one-half occurring during the winter. The average annual snowfall is probably in excess of 50 inches (Beschta and Thorud, 1973). Most of this area is characterized by ponderosa pine (*Pinus ponderosa* Laws.) and mixed conifer vegetation types.

The central highlands occupies the headwaters of many of Arizona's major river systems, including the Salt, the Verde, the Little Colorado, and the San Francisco Rivers. Well over 80 percent of the water yield from these snow covered watersheds occurs during the winter and spring periods (Cooperrider and Sykes, 1938; Hansen and Ffolliott, 1968). Because of the significance of this geographical area in the water supply of the state and the relative abundance of snow course data, it was chosen for detailed analysis (Figure 1).

SNOW COURSE DESCRIPTIONS AND DATA

The total length of snow course record available for this study was 33 years (from 1940 to 1972). However, only a limited number of snow courses actually had 33 years of record, and thus it was decided to include

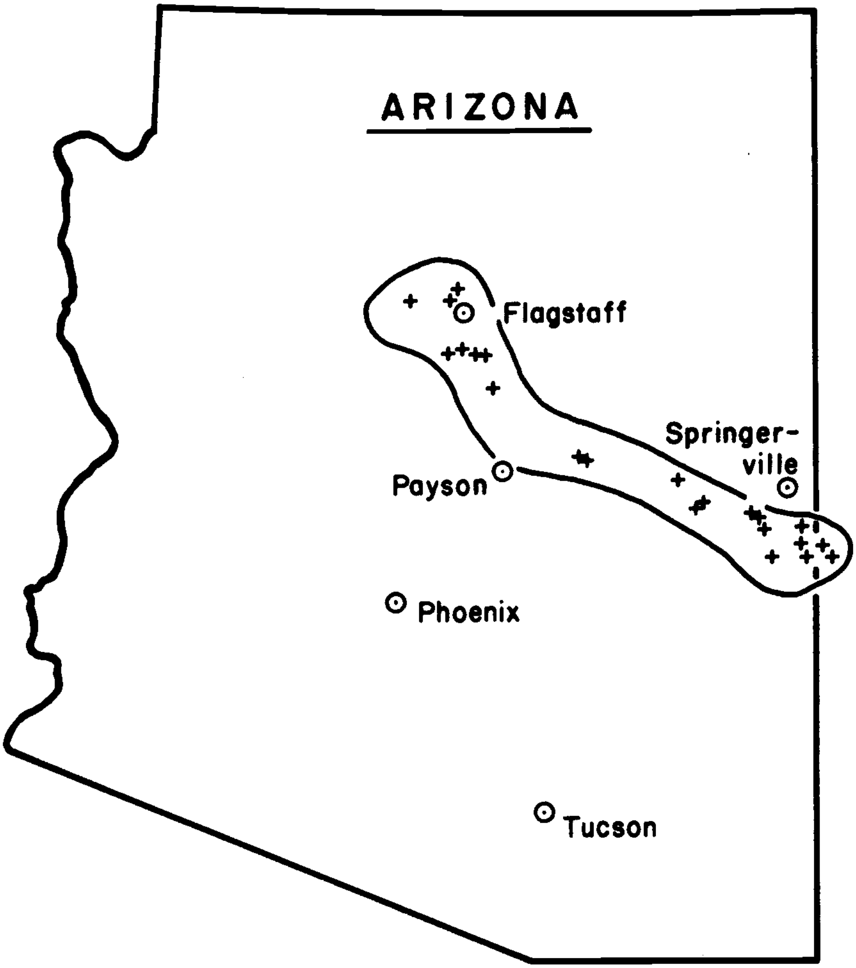

Figure 1. Location of 22 snow courses (+) with ten or more years of record in the central highlands of Arizona.

any courses with ten or more years of record (Figure 2). The choice of ten years represents a compromise. To have included courses with nine, eight or even five years of record would have substantially increased the number of courses available for analysis, but it was felt that such courses had inadequate periods of record for evaluating probability distributions. On the other hand, requiring a longer period of record would have eliminated snow courses from analysis. A total of 22 courses were used in this study.

Snow course elevations varied from 6,430 to 10,260 feet, with a large proportion of the snow courses found between 7,000 and 8,000 feet (Figure 3). The effect of site characteristics (e.g. slope, aspect, and forest density) was not evaluated as snow courses are subjectively located on sites where snow persists into the melt season. Most of the snow courses used in this study were associated with forest densities of less than 90 square feet of basal area per acre and with slopes of less than five percent (Ffolliott, Thorud, and Enz, 1972).

Normally, snowpack depth and water equivalent (i.e., the depth of liquid water obtained from melting the snowpack) are measured on each snow course at two-week intervals, commencing on January 15 and continuing through April 1 of each year. In effect, water equivalent (W) indicates the amount of liquid water stored on a watershed in the form of snow; thus measurements of water equivalent are often used to predict water yields from areas of snowpack accumulation, and it is the basic variable evaluated in this study. The actual water equivalent measured on a particular date represents the integration of both snowfall accumulation and snowpack ablation up to that point in time.

OBJECTIVE

The objective of this study was to determine empirical probability distributions of snow water equivalent at the various snow courses for each of the six measurement dates. It was felt that additional information and insight might be obtained from historical snow course data if the probability distributions of water equivalent were known. Based upon the distributions obtained from this study, limitations of snow course data would be identified and the practical implications of the analysis discussed.

ANALYSIS

The selection of a probability distribution as a possible model of snow course water equivalent involves a number of considerations. It was desirable to have single family of distributions to describe water equivalent at all courses and for all measurement dates. Relationships between the distribution parameters and location, and between the distribution parameters and time, may have a physical basis. If the parameters can be identified as functions of geographic location or elevation, for example, distributions can perhaps be obtained for other locations or elevations.

Statistics and relative frequency histograms were determined for each of the 22 snow courses on each of the six snow measurement dates. Inspection of the statistics revealed that most of the distributions possessed positive skew coefficients. In addition, snow water equivalent is restricted to

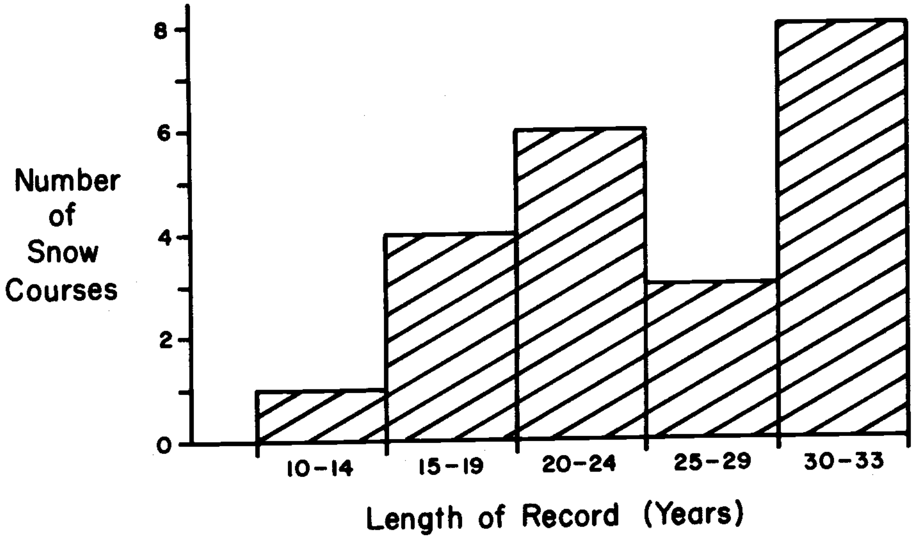

Figure 2. Histogram of the number of snow courses in relation to length of record.

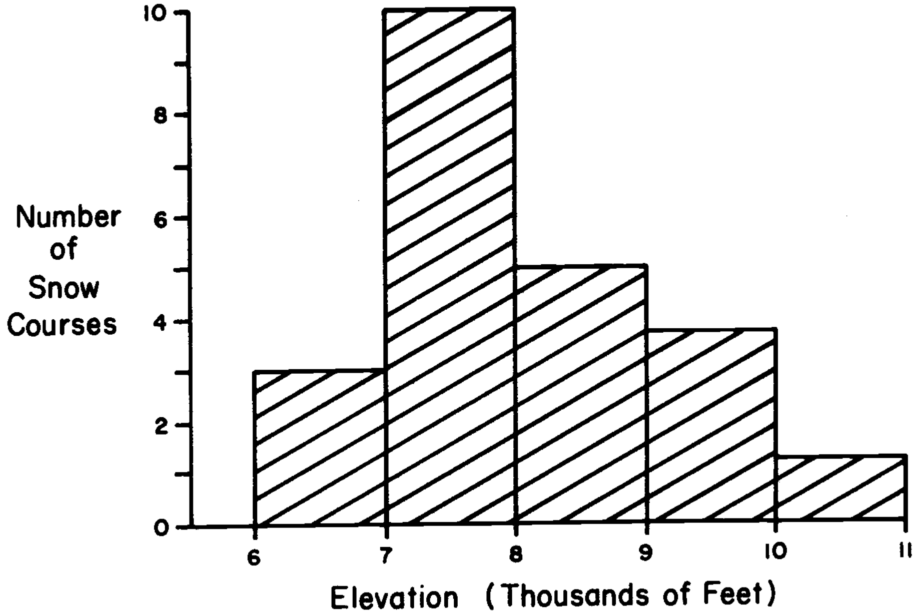

Figure 3. Histogram of the number of snow courses in relation to elevation.

values greater than or equal to zero. Thus, a distribution which is similarly bounded and which could preserve positive skew would be desirable.

The two parameter gamma family of distributions was selected for initial use as it possesses positive skew and two parameters which would facilitate fitting. In addition, a random variable which is gamma distributed is restricted to the interval 0 to $+\infty$. The form of the gamma distribution used in this study is as presented in Benjamin and Cornell (1970, p. 246).

The gamma shape and scale parameters were estimated by the method of moments for each snow course and measurement date. Parameter estimates varied erratically between snow courses for each given date. Approximately 50 percent of the fitted gamma distributions were rejected using a chi-square goodness-of-fit test. All statistical comparisons and tests of significance in this analysis utilized an $\alpha = 0.10$ level of confidence.

Although numerous reasons might explain the unstable nature of the gamma shape and scale parameters, two are thought to be of primary importance. First, the length of record may have been insufficient. Weisner (1970) recommended 50 years of record in order to obtain stable frequency distributions of precipitation. Studying precipitation in mountainous regions of India, Mooley and Crutcher (1968) similarly found that 50 years of record were required for stability of gamma parameters. In this present study, one-half of the snow courses had less than 25 years of record, and the maximum period of record was only 33 years (Figure 2).

Second, due to the infrequent nature of winter snowfall in Arizona and the relatively large number of snow courses below 8000 feet (Figure 3), many snow course records contained entries of zero water equivalent. This created a difficulty in applying a single distribution to those snow courses at which zero water equivalent occurred. One or more observations of zero water equivalent for a given course shortened the effective length of record and generated a "spike" at the origin of the probability distribution.

Inspection of the histograms revealed that the snow courses used in the analysis could be classified into three general types based upon the height of this "spike" in relation to the remainder of the histogram (Figure 4). The first type of histogram (I) consisted of those cases where the relative frequency of zero water equivalent exceeded the relative frequency of any other interval. Of the 132 histograms studied, 52 percent were classified as being of the first type. The second type of histogram (II) encountered was that in which the relative frequency of any one interval exceeded the relative frequency of zero water equivalent. Thirty-nine percent of the histograms studied were of the second type. Only nine percent of the histograms were of the third type (III) wherein snow was observed on the snow course at all times.

For types I and II the effect of the frequent occurrence of zero water equivalent was to "weight" the lower end of the probability distribution in favor of zero occurrence at the expense of small values of water equivalent. The probability of small values of water equivalent, as determined from a fitted distribution, would be exaggerated due to the influence of the zero water equivalent values and, thus, the gamma distribution did not adequately represent the distributions being modeled.

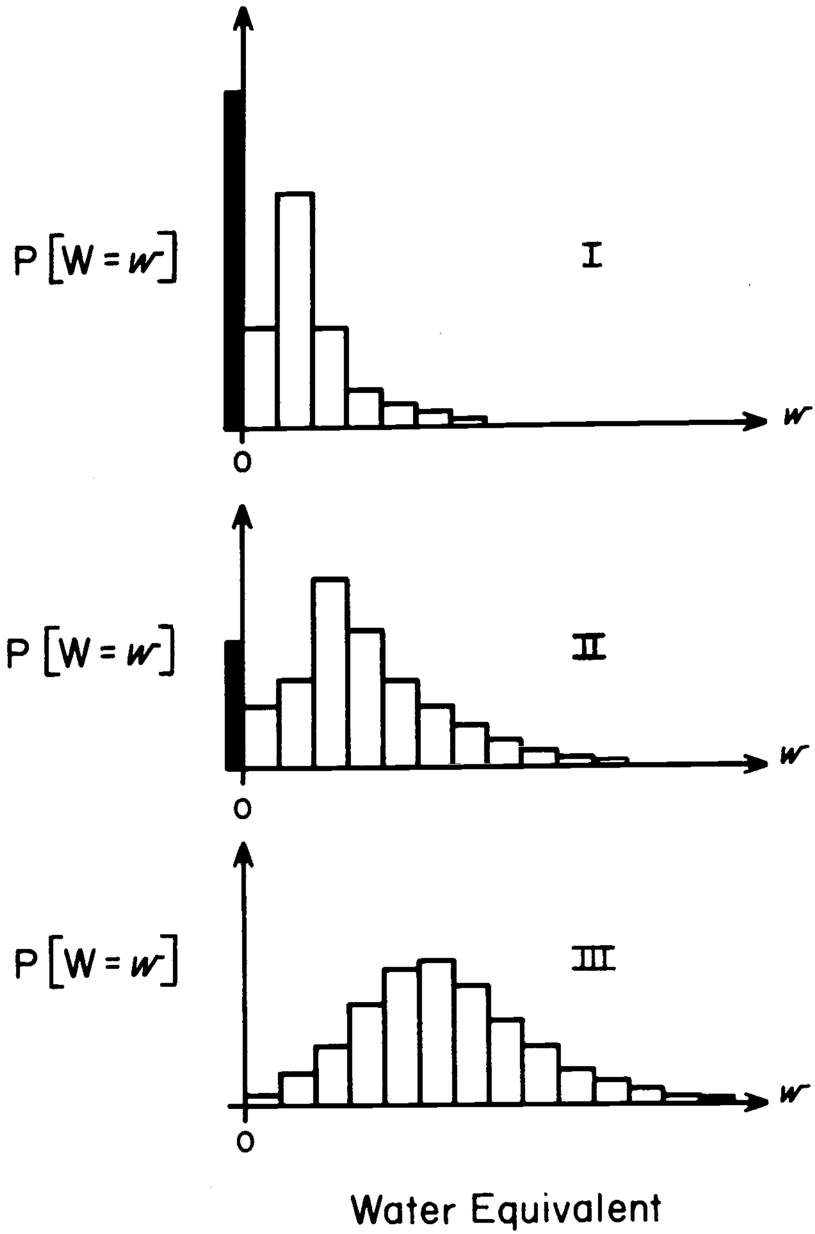

Figure 4. Probability distributions of snow course water equivalent illustrating type I, II, and III distributions.

Thom (1966), in studying the distribution of maximum snow water equivalent at National Weather Service stations, encountered a similar problem with southerly stations at which snow did not occur every year. His solution was to use a mixed distribution: a binomial distribution to describe the probability of the occurrence of snow and a lognormal distribution to describe the seasonal maximum water equivalent, given the probability of snow. Since this was intended to be a preliminary or "exploratory" study of probability distributions based on historical snow course records, and since the frequent occurrence of zero values in many of the histograms suggested it, similar distributions were considered.

In general, the probability of snow increases with elevation and may vary with geographic location. Therefore, it would be difficult to apply a single binomial distribution to all the snow courses. As was the case with the gamma distribution, a single binomial distribution was not applied. The binomial distribution parameter (P) for each snow course and each measurement date was estimated from the ratio of the number of years with snow to the total years of record. Thus, P represents the probability of the occurrence of snow for a given snow course and measurement date.

It was assumed that the probability of occurrence of snow remains constant from year to year at a particular snow course and for a particular measurement date. Such an assumption is reasonable provided the time series of climatologic variables (i.e., precipitation, temperature, solar radiation) associated with snow accumulation and ablation processes can be assumed stationary. As applied here, the second assumption underlying a binomial distribution, independence between events, implies independence between years.

Although a function of many variables, the probability of snow is perhaps most strongly influenced by elevation. There were no large differences in slope, exposure, or other site factors which could explain differences in the probability of snow between snow courses. Therefore, the probability of snow on each snow measurement date, for the 22 snow courses, was treated as a function of elevation. In order to study the relationship between the occurrence of snow and elevation, a linear regression of the probability of snow versus elevation was calculated for each of the six measurement dates (Table 1).

Table 1. Regression equations and associated statistics relating the probability of snow (P) to elevation (X) for each measurement date (n = 22).

Measurement Date	Regression Equations[1/]	Standard Error of Estimate	r^2	F
Jan. 15	$P = -0.08 + 0.11X$	0.09	0.57	26.2*
Feb. 1	$P = 0.25 + 0.08X$	0.08	0.43	15.3*
Feb. 15	$P = -0.03 + 0.11X$	0.10	0.55	24.0*
Mar. 1	$P = -0.39 + 0.15X$	0.09	0.70	45.8*

Table 1. (Continued)

Measurement Date	Regression Equations1/	Standard Error of Estimate	r^2	F
Mar. 15	$P = -0.74 + 0.17X$	0.13	0.62	32.6*
Apr. 1	$P = -1.06 + 0.19X$	0.18	0.51	21.0*

1/ Elevation in thousands of feet

* Significant at $\alpha = 0.10$

The regression for February 15 is illustrative of the amount of scatter of the points about the regression line which was encountered for all six regressions (Figure 5). All of the regression lines are presented in Figure 6. Differences between slopes of the regression equations were evaluated using t tests. Only the slopes for the February 1 and April 1 regression lines were significantly different. If it is assumed that the one significant value was a chance occurrence, then it could be concluded that the change in the probability of snow with elevation for all six measurement dates is described by a family of linear regression lines with the lines differing by a constant (the difference between the mean probabilities of any two lines).

T tests between the mean probabilities of snow for the six measurement dates were applied. Significant differences were found, and it was concluded that the mean probability of snow changes between measurement dates.

Following the development of the binomial distribution, a logarithmic transformation of the "non-zero" values of water equivalent was performed as follows:

$$Y = \log_{10} (W + K) \tag{1}$$

where Y = transformed variable
W = water equivalent, inches
K = constant

Only those snow courses with ten or more years of non-zero record were analyzed. The direct transformation of W without a constant would have resulted in negative values in the distribution of the transformed variables as well as causing relatively large variances about the mean. Thus, the addition of a constant was necessary to alleviate this problem. In addition, it was felt desirable to have the transformed data fall within the range of 0 to $+\infty$. For these reasons, $K = 1$ was used in this study.

As had occurred previously, the shape and scale parameters for the gamma distribution were highly variable and, apparently, not related to either elevation or geographical location. Again, the shortness of historical record may have been the cause of these instabilities, for now all zero values of water equivalent had been removed from the data. Histograms of the transformed data indicated that in many cases a normal distribution had been obtained. In addition, these distributions all had similar estimates of variance, averaging approximately 0.05.

To further explore the characteristics of the transformed distributions, both the coefficients of skew and kurtosis were subjected to t tests to evaluate the influence of elevation (low elevation vs. high elevation snow courses) or geographical location (courses located along the eastern portion of the central highland vs. those to the west). No significant differences were found in both cases. The range of values for the skew and kurtosis parameters is shown in the following tabulation:

Parameter	Range
Coefficient of Skew	-0.98 to 1.61
Coefficient of Kurtosis	2.09 to 7.38

If the transformed variables were normally distributed then these distributions should exhibit the characteristics of a normal population. The theoretical values of skew and kurtosis for a normal distribution are zero and three, respectively. The average skew and kurtosis values, considering all six measurement dates, were 0.08 and 3.04 (Table 2). The coefficients of skew and kurtosis for each measurement date were tested against the assumed values of a normal distribution, and no significant differences were found. Thus, it appears that the above transformation of the non-zero water equivalent data results in normally distributed variables with similar variances.

Table 2. Mean values of skew and kurtosis for each measurement date.

Measurement Date		Coefficient of Skew	Coefficient of Kurtosis
Jan. 15		.35	3.61
Feb. 1		.31	2.99
Feb. 15		.00	2.96
Mar. 1		-.06	2.66
Mar. 15		-.21	2.69
Apr. 1		.08	3.33
	Average over all measurement dates	.08	3.04

The mean ($\bar{Y}$) of the non-zero transformed water equivalents was computed for each snow course. The means for each measurement date were then regressed on elevation. The relationship of $\bar{Y}$ and elevation for the February 15 measurement date illustrates the scatter about the regression line (Figure 7). For each measurement date significant regressions were obtained (Table 3).

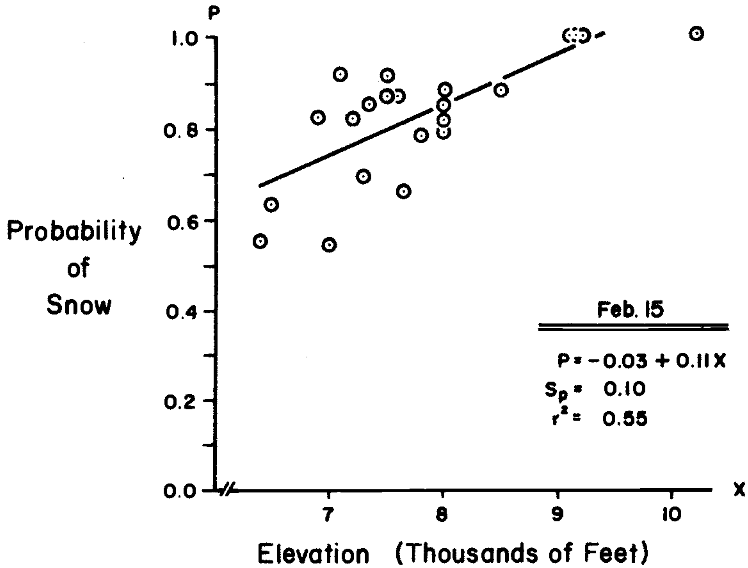

Figure 5. February 15 regression illustrating the relationship between the probability of snow (P) and elevation (X).

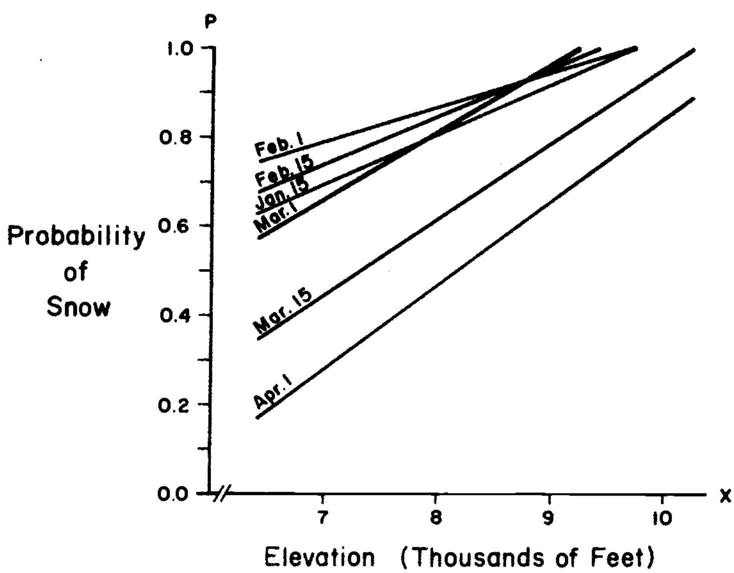

Figure 6. Regression lines illustrating the relationship between the probability of snow (P) and elevation (X) for each measurement date.

The regression lines for all six measurement dates are shown in Figure 8. Slopes of the regression lines were compared with no significant differences observed. The "vertical position" of the various lines were also tested with the results indicating significant differences present. The fact that significant differences occurred only in relation to the relative position of the regression lines and not the slopes of the curves indicates that a family of lines with the same slope, and differing only by constants, could be used to describe the relation of $\bar{Y}$ and elevation for the six measurement dates.

In order to make a probability statement as to the magnitude of water equivalent, consideration must be given to the probability of the occurrence of snow as follows:

$$F_Y(y) = Q + PN_y(y) \tag{2}$$

where
Y = log transform of water equivalent
P = probability of snow
Q = 1-P
$N_Y(y)$ = normal distribution function of Y
$F_Y(y)$ = cumulative distribution function of transformed water equivalent with the probability of snow considered

The probability that a transformed water equivalent will be less than or equal to a stated magnitude is weighted by the probability of occurrence of snow, plus the probability of zero water equivalent.

Table 3. Regression equations and associated statistics relating the mean ($\bar{Y}$) of the transformed variables at each snow course to elevation (X) for each measurement date.

Measurement Date	Regression Equations[1/]	Standard Error of Estimate	r^2	F	n
Jan. 15	$\bar{Y} = -0.11 + 0.08X$	0.07	0.47	17.1*	21
Feb. 1	$\bar{Y} = -0.22 + 0.11X$	0.09	0.59	28.3*	22
Feb. 15	$\bar{Y} = -0.23 + 0.11X$	0.10	0.53	22.4*	22
Mar. 1	$\bar{Y} = -0.35 + 0.12X$	0.11	0.54	23.3*	22
Mar. 15	$\bar{Y} = -0.60 + 0.16X$	0.11	0.65	29.7*	18
Apr. 1	$\bar{Y} = -0.67 + 0.16X$	0.13	0.50	9.2*	11

1/ Elevation in thousands of feet

* Significant at $\alpha = 0.10$

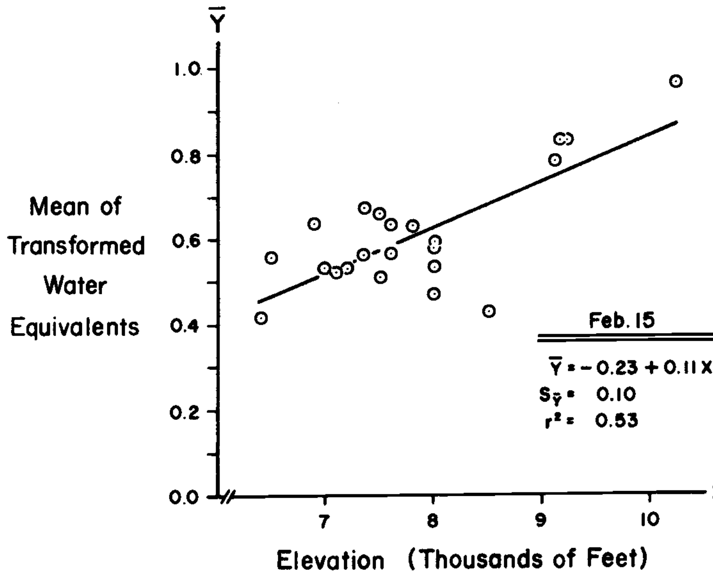

Figure 7. February 15 regression illustrating the relationship between the mean ($\bar{Y}$) of the transformed variables at each snow course and elevation (X).

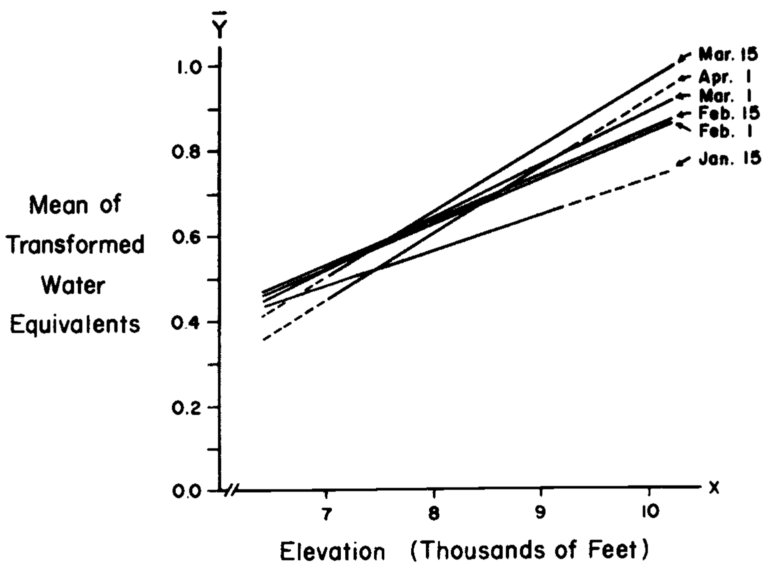

Figure 8. Regression_lines illustrating the relationship between the mean ($\bar{Y}$) of the transformed variables at each snow course and elevation (X), by measurement date.

CONCLUSIONS

In the location and placement of additional snow courses or snow measurement instruments in Arizona, use could be made of the regression equation of the probability of snow versus elevation. Since snow course information is used as an index of basin snow conditions it may be desirable to locate them at elevations where snow cover occurs with a stated probability. The probability could be predicted from the equations in Table 1 (or graphically from Figure 6). For example, if it were desired to locate new snow courses with at least a 0.5 probability of snow on all measurement dates then results indicate such courses should be established above 8200 feet elevation (Figure 6).

The results of this preliminary study could provide a basis for modeling snowpack water equivalent over the three month measurement period. To obtain an approximate distribution of water equivalent for a given measurement date and elevation, Figure 8 could be used to estimate the mean of the distribution.

After having selected the probability of snow and the appropriate distribution (as indexed by the mean) for a stated elevation, the probability of a given water equivalent magnitude could be calculated using Equation 2. Such information may have utility in snow load analysis and in evaluating the potential of various elevation zones for snow management by vegetation manipulation.

Attempts to describe snow course water equivalent data with a two parameter gamma distribution proved unsatisfactory. The frequent occurrence of zero water equivalent values precludes the application of any single continuous distribution to snow course data for the central highlands of Arizona. However, two distributions can be used to describe snowpack water equivalent; the probability of snow is described by a binomial distribution while the probability of water equivalent is described by a lognormal distribution.

This study indicates that future research is necessary for a more thorough understanding of distributions of snowpack water equivalent. Such research could, for example, consider the spatial dependency between stations as well as the temporal dependency for a given station.

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A PRELIMINARY ASSESSMENT OF SNOWFALL INTERCEPTION IN ARIZONA PONDEROSA PINE FOREST

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INTRODUCTION

In central Arizona, which includes the Salt and Verde River Basins, the maintenance of economic stability is dependent, in part, on the availability of useable water supplies that originate as snowmelt runoff from outlying forested areas. The forested region consists primarily of nearly 3 1/2 million acres of ponderosa pine (Pinus ponderosa Laws.). Thus, knowledge of interactions between snowpack dynamics and ponderosa pine forest is prerequisite to the proper implementation of water yield improvement programs involving the snow resource. The snowfall interception phenomenon should be more fully evaluated, since it may represent a potential water loss in terms of snowmelt runoff from forested areas (Harshbarger et al., 1966).

The disposition of snow on a forest canopy during and after snowfall involves five general processes that are affected by complex relationships between the tree canopy, intercepted snow and climatic parameters. These processes include wind erosion of canopy snow, snowslide from the canopy, stemflow, vapor transport from meltwater, and vapor transport of canopy snow (Miller, 1966). The first three processes do not necessarily represent a net water loss to the snowpack, only delayed delivery. The snow removal processes can be operative separately or in combination depending on wind, air temperature, available radiation and vapor pressure of the air and canopy snow (Miller, 1966).

DESCRIPTION OF STUDY

The purpose of this study is to make a preliminary assessment and ranking of the relative significance of the five processes that may cause snow removal from ponderosa pine forest canopies.

Data were collected from a study site located seven miles south of Alpine, Arizona on the Apache National Forest. Approximately 90 percent of the tree cover is uneven-aged ponderosa pine, with Gambel oak (Quercus gambelii Nutt) and quaking aspen (Populus tremuloides Michx) minor components. The estimated site index (Meyer, 1961) is 65 feet at 100 years.

The area is gently rolling, has few slopes exceeding 15 percent (Ffolliott and Thorud, 1972), and is 8100 feet above sea level. Annual precipitation averages 27 inches, almost half of which occurs between October 1 and May 3 (Rich, 1970).

The presence of snow in the canopy of a stand of trees ranging from 25 to 60 feet in height was documented with a super 8-mm time-lapse camera. The self-contained camera is activated during daylight hours by a photo-electric cell linked to a battery power source (Patton, Scott, and Boeker, 1972). The stand of trees was photographed at five-minute intervals.

Wind speed and direction were recorded at 30 feet above the ground with a continuous recording anemometer. Relative humidity and air temperature were recorded with a hygrothermograph which, along with maximum-minimum thermometers, was located in a standard instrument shelter, four feet above ground level. Incoming short wave radiation was recorded with an Eppley 180° pyrheliometer at Alpine. An index of incoming precipitation was obtained with standard rain gages located on the study site, and with a recording rain gage located on a nearby experimental watershed.

Photographs for January 1 through 4, 1973, which represented a post snow storm period, were analyzed by means of a movie projector and dot grid. Individual frames were projected onto the grid, and a snow load index was determined for hourly intervals.

The snow load index was expressed as a ratio of forest canopy area covered with snow to the total canopy area. Specifically, the number of dots for total canopy area (N), which included any above-ground tree parts hit by a dot, were counted for a clear day with no canopy snow. Then, during periods with canopy snow, total above-ground tree parts with no snow cover (n_1) were again determined; thus, $N-n_1$, or n_2, represents an estimate of the snow present on the trees comprising the stand. The snow load index, n_2/N, was then calculated and used to characterize the accumulation and disappearance of forest canopy snow.

Peak wind velocity (mph), air temperature (°C), atmospheric vapor pressure (mb), incoming short-wave radiation (ly), and cumulative precipitation (in.) were empirically correlated with snow load index values to help identify, assess, and rank in terms of magnitude, the five mechanisms by which snow is removed from the forest canopy.

RESULTS AND DISCUSSION

A six-hour snow storm occurred prior to activation of the time-lapse camera on January 1. The storm deposited approximately 0.5 inches of snowpack water equivalent on the ground; however, the total water equivalent of snow intercepted by the forest canopy is unknown. The snow load index as defined above, was determined for hourly intervals of camera operation from January 1 through January 4.

The changes in snow load index and the climatic data obtained for the evaluation period are graphically presented in Figure 1.

The snow load index changed from 83 to 64 between 0830 and 0930 on January 1 (Figure 1), suggesting immediate changes in snow storage after cessation of the storm. During this period, temperatures were below 0°C, peak wind velocity did not exceed 12 mph, and incoming short wave radiation was less than 3 lys per hour. The primary processes affecting snow removal appeared to be wind erosion, with the possibility of some snowslide. Considering the low air temperatures and short wave radiation load during this time interval, melt may not have been quantitatively significant; likewise, the relatively low energy load in combination with a short time interval suggests that vaporization may not have been a major removal process.

The snow load index change was minimal from 0930 on January 1 through 1500 on January 2. Although some change in snow storage may have occurred, intermittent snowfall during this period (Figure 1) could have offset loss of snow from the forest canopy.

Snowslide and wind erosion appeared to be a major cause of removal of canopy snow from 1500 on January 2 to 0800 on January 3, when the snow load index changed from 73 to 56 (Figure 1).

The greatest daily change in the snow load index occurred on January 3, when the index changed from 55 to 6 (Figure 1). Large masses of snow disappeared from the trees at a constant rate between 0900 and 1400 on this date. According to Miller (1966), a constant rate of snow removal is often associated with snowslide; consequently, this process may have been dominant at this time. Peak winds on this date were 25 mph, indicating that some wind erosion may also have occurred. However, the snow had been in the trees for two days and had possibly hardened as a result of metamorphism, which may tend to make it progressively less susceptible to wind erosion.

By 1300 on January 3, all canopy snow had disappeared from the upper 35 feet of the stand depicted by photography. Some snow remained on the smaller trees in the lower canopy until the following day. This small amount of residual snow persisted for more than one-half of the day on January 4 (Figure 1); melt and, possibly, vaporization may have been occurring at this time, as air temperatures were above 0°C and short wave radiation values were high.

Overall analyses of the behavior of the snow load index during the period January 1 through 4, 1973, suggests that snow removal by snowslide and wind erosion were of major importance. Conversely, stemflow, dripping melt water, and vapor transport of melt water and canopy snow appeared to be of comparatively minor importance. This preliminary assessment and ranking of the relative significance of the processes affecting the deposition of snow on a forest canopy agrees, generally, with findings observed in Colorado subalpine forests (Hoover and Leaf, 1966) and in northern Idaho (Satterlund and Haupt, 1970). However, additional analysis will be necessary before the tentative assessment and ranking of the processes that may cause snow removal from ponderosa pine canopies can be fully documented.

The potential loss of intercepted snow by vaporization in the water budget during this study period may have been minimal, as most of the canopy snow appeared to eventually reach the forest floor. Again, similar results have been reported elsewhere (Hoover and Leaf, 1966; Satterlund and Haupt, 1970).

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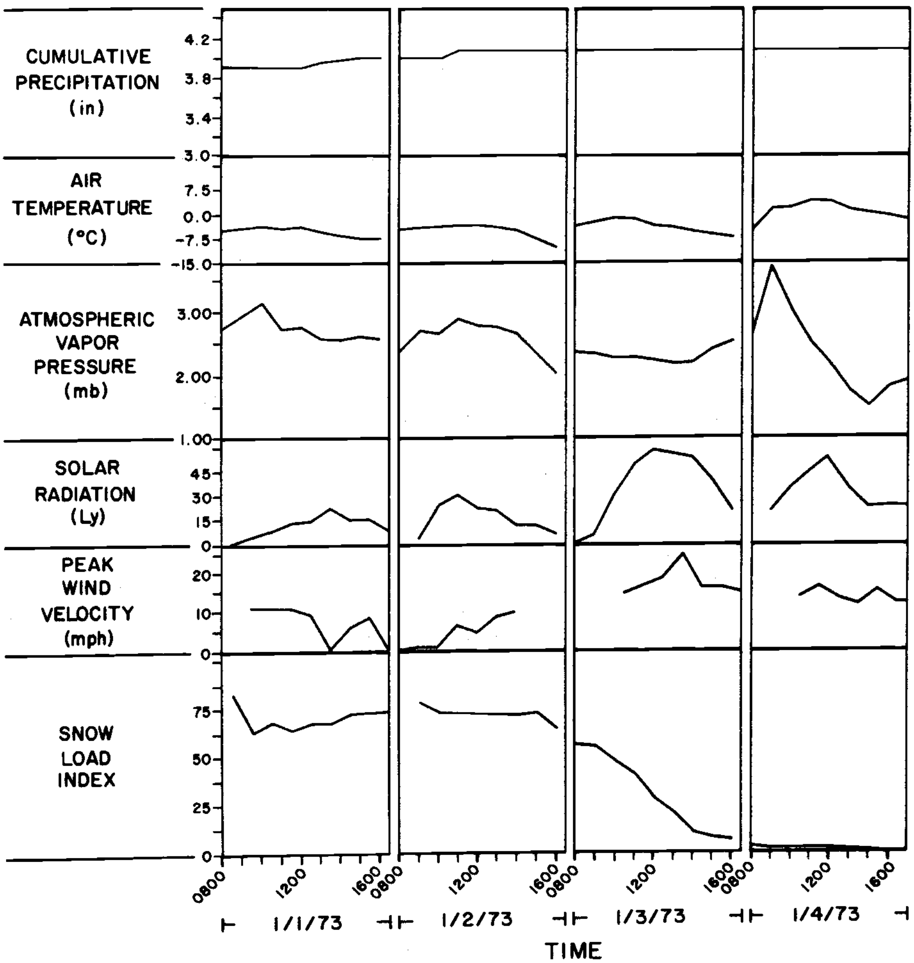

Fig. 1. Snow load index and climatic data for daylight hours of January 1 through 4, 1973.

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EFFECTS OF A WETTING AGENT ON THE INFILTRATION CHARACTERISTICS OF A PONDEROSA PINE SOIL

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ABSTRACT

An infiltration-wetting agent study, using the wetting agent "WATER-IN", was conducted in the ponderosa pine forest type of east central Arizona. An application rate of 10 gallons of wetting agent per acre was used on bare mineral soil and on ponderosa pine litter. The infiltration rate was measured by a modified North Fork infiltrometer. It was found that "WATER-IN" significantly increased water runoff when applied to litter, but, when applied to bare mineral soil, "WATER-IN" caused a significant increase in water infiltration. The wetting agent did not significantly affect antecedent moisture, soil particle distribution, litter water holding capacity, or litter bulk density. It is presently hypothesized that the increase in water infiltration on treated bare mineral soil is due to a decrease in the average bulk density of the surface inch of soil. The increase in runoff when litter is treated is probably due to an interaction, either physical, chemical, or both, between the humus layer and "WATER-IN", creating a hydrophobic condition where one did not exist before.

INTRODUCTION

Water repellent or hydrophobic soils are found in many parts of the world--from Australia to the United States (Debano, 1969). These soils have wide-spread implications in watershed management, particularly on steep slopes where they reduce infiltration of rainwater and cause serious erosion problems. Severe water repellency can also alter soil moisture relationships and impair vegetation growth.

Evidence of water repellent soils in Arizona was disclosed by Zwolinski (1971) following a series of infiltration measurements on ponderosa pine soils near McNary. In that study a number of infiltration curves plotted from field data exhibited a prominent depression shortly after the start of water application. Although the cause of this temporary resistance of surface soils to wetting is not known, it is suspected that organic substances from plant materials may play an important role. Fires are also reported to accentuate hydrophobic soil conditions (Debano, 1966; Debano and Krammes, 1966). Heat from a fire is known to vaporize volatile oils and resins contained in surface litter and brush. These substances subsequently condense on soil particles to form a water repellent layer.

The study described in this paper was designed to investigate some of the effects of a wetting agent on the water infiltration rates of hydrophobic soils in the White Mountains of Arizona.

WETTING AGENTS

Wetting agents (surfactants) have been used in southern California to reduce hydrophobic soil conditions following wildfire. By applying a wetting agent to several post-fire study areas in the San Dimas Experimental Forest, the Santa Ana and the San Gabriel Mountains, Krammes and Osborn (1969) were able to significantly reduce surface runoff and erosion.

Wetting agents used on water repellent soils are normally of the nonionic type, as opposed to the anionic and cationic types. Nonionic surface active agents are relatively unaffected by acids or alkalis and are less affected by strong electrolyte concentrations. The degree of solubilization, important to absorptive behavior, can be varied with more sensitivity using nonionic compounds (Black, 1969).

Several brand name wetting agents are available. Their addition to water results in a reduction of the water surface tension and a decrease in the water-soil contact angle. Soil water infiltration rates on water repellent soils are increased since the beneficial effect of lowering the water-soil contact angle normally overcomes the detrimental effect of decreasing the water surface tension.

The wetting agent "WATER-IN" was selected for this study because of its availability and relatively low cost compared to other commercially available surfactants. "WATER-IN" is a liquid wetting agent with a pH of 7 and a specific gravity of about 1.02 gm/cc at 21°C. Its chemical composition is alkyl polyethylene glycol ether, 95% by weight, and inert ingredients, 5% by weight. Treatment application was a rate of 10 gallons of active ingredient per acre.

TREATMENTS AND RESULTS

The study was conducted on the Fort Apache Indian Reservation in east central Arizona, 5 1/2 miles east of McNary. Soils in this region are derived from a mixture of basalt slag and volcanic cinders under ponderosa pine vegetation. They are silt loam in texture and belong to the Sponseller series.

Four infiltration plots were installed on each of three field sites. Three of the plots were randomly selected for treatment while the fourth was designated as a control. Treatment 1, and later Treatment 2, was applied to each of the three treated plots on all sites. Treatments 3 and 4 were subsequently applied to Site 3 only. Infiltration rates were measured by a modified sprinkling-type North Fork infiltrometer.

Treatment 1, consisting of an application of 100 ml of distilled water to the three treatment plots on each site, served as a calibration for subsequent infiltration determinations. Plots were undisturbed with litter and soil intact. The third and fourth infiltration runs on each plot showed consistent infiltration capacities. The mean infiltration rate for all sites with this treatment was 6.3 inches of water per hour.

The application of 100 ml of "WATER-IN" solution on each treatment plot on each site, in a manner similar to and under the same litter and soil conditions as Treatment 1, comprised Treatment 2. This treatment caused a significant increase in the amount of runoff from each treated plot. The mean infiltration rate following Treatment 2 over all three sites was 5.0 inches per hour.

Treatment 3 involved the removal of all litter from the three treatment plots on Site 3 and the spraying of 100 ml of distilled water on the exposed mineral soil within the treatment plots. This treatment resulted in a mean infiltration rate of 6.1 inches of water per hour. This value was not significantly different from the mean infiltration rate for the plots receiving Treatment 1 on Site 3.

Treatment 4 consisted of an application of 100 ml of "WATER-IN" solution in the same concentration as used for Treatment 2 to the bare soil of the three treatment plots on Site 3. This treatment increased the final infiltration rate to a mean of 8.1 inches per hour.

When comparing the mean infiltration rates following Treatments 1 and 3 on Site 3 (Figure 1), it appears that raindrop impact was negligible on bare mineral soil. Whether this was due to the small size of the water droplets, the failure of the droplets to reach terminal velocity, or an effect of the wetting agent is not clear.

The increase in the final infiltration rate caused by "WATER-IN" applied to bare soil on Site 3 (Treatment 4) appears to be related to a decrease in the soil bulk density for the surface one-inch of soil and possibly a reduction of the soil-water contact angle. How the bulk density is decreased is not known at this time.

There was no fungi mycelia growth to account for the reduction in infiltration rate with Treatment 2 on any of the three sites. Treatments 3 and 4 suggest that "WATER-IN" reacts upon the litter or is acted upon by the litter, probably in the H layer, creating a hydrophobic condition. Several possibilities exist on how these hydrophobic conditions can possibly be created.

First, a chemical reaction in the wettable litter may destroy the effectiveness of the wetting agent. Since the litter material was initially wettable, it is possible that the wetting agent was adsorbed on the litter in such a fashion that it made the litter water repellent. Chemically, the nonionic wetting agents have a hydrophilic (polar) group at one end and a hydrophobic (hydrocarbon) chain at the other. When added to a hydrophobic material, the hydrocarbon end of the wetting agent tends to be adsorbed onto the surface leaving the hydrophilic end in contact with water. Hence, good wetting properties are imparted to the water repellent substances. However, when the wetting agent is placed on a wettable surface, it is possible for the wettable end of the wetting agent to be adsorbed by the wettable surface leaving the hydrophobic end exposed to the water. This type of adsorption can leave a formerly wettable material water repellent.

Secondly, the dissolution of hydrophobic resins from the top litter layers may be redeposited in the H layer with its finer texture and more chemically active surfaces causing increased runoff. And, thirdly, a reduction by "WATER-IN" of the surface tension of the water passing through the litter may make it easier for the water to run off rather than infiltrate.

The interval between the start of rainfall application and the appearance of surface runoff is affected by the amount of surface litter. In general, the greater the amount of litter, the longer the period before runoff occurs. This is due to the water storage capacity of litter which must be satisfied before surface runoff can occur. Calculations using an average litter depth of 2.8 inches, a litter bulk density of 0.119 gm/cc and an average water holding capacity of 170% show that surface runoff is delayed until more than half an inch of rain has been absorbed by the litter. Therefore many rainstorms fail to wet the surface soil. It would be beneficial if a wetting agent could be found that could reduce the storage capacity of the litter without having a detrimental effect on the infiltration rate of the soil thus increasing substantially the amount of water entering the soil.

SUMMARY

The application of "WATER-IN" at the rate of 10 gallons of active ingredient per acre to soil covered with ponderosa pine litter significantly increased surface runoff. However, an application of "WATER-IN" at the same rate to bare mineral soil significantly increased the infiltration capacity from an average of 6.2 inches per hour to 8.1 inches per hour. This same treatment significantly decreased the average bulk density of the surface inch of soil from 1.17 gm/cc to 1.05 gm/cc, resulting in an increased total soil porosity from 56.0% to 60.4%. No other changes in soil physical properties were detected.

Average soil pH of 5.8 was not affected by the "WATER-IN" treatments nor were antecedent soil moisture contents and soil particle distribution.

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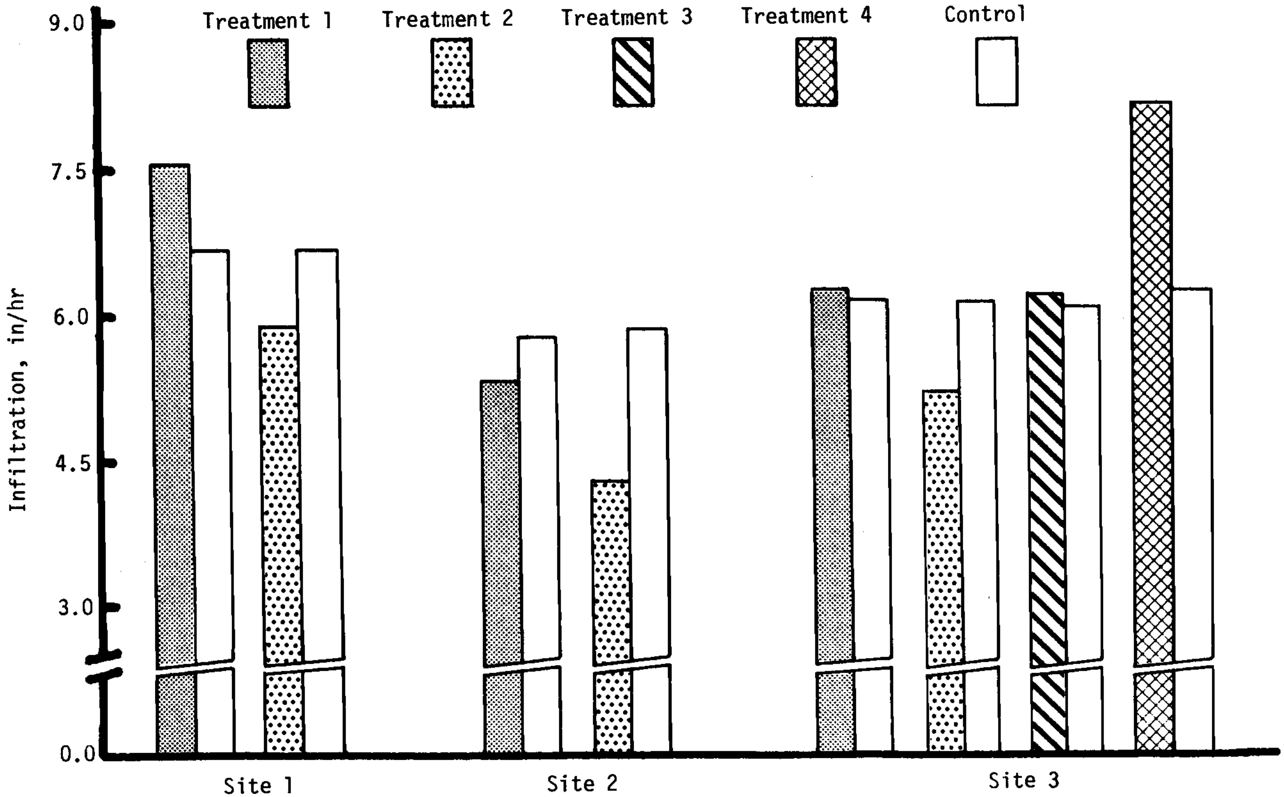

Figure 1. Mean final infiltration rates for treatment and control plots for three sites.

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STATIONARITY IN THUNDERSTORM RAINFALL IN THE SOUTHWEST[1]

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INTRODUCTION

Air-mass thunderstorm rainfall is a major source of water in the rangeland areas of the southwestern United States. These thunderstorms occur during the summer months and produce intense afternoon and evening rains of short duration and limited areal extent. For small (100 square miles and less) watersheds in the southwest, air-mass thunderstorms produce the major flood peaks (Osborn & Hickok, 1968), therefore, knowledge of occurrence frequencies of these thunderstorms is essential in predicting flood sediment transport for small watersheds in the southwest.

Air-mass thunderstorm rainfall appears to be randomly distributed in time and space. Thus, it can be considered as a stochastic process. Stochastic processes are classified as either stationary or nonstationary. If thunderstorm rainfall can be assumed to be stationary, it most likely can also be assumed to be ergodic. If ergodic, several independent short-term samples can be combined to provide a sample which is equivalent to a long-term record, thus providing greater opportunity for obtaining sample points in the extreme tails of the rainfall probability distribution. Such extreme sample points are needed to better define the rainfall probability tails for more accurate prediction of extreme events.

In this paper, several long-term U.S. Weather Bureau records from gages in southeastern Arizona are tested for stationarity. Both the maximum daily rainfall for each season and seasonal total rainfall are tested. Also, rainfall records from a dense raingage network on the ARS Walnut Gulch Experimental Watershed near Tombstone, Arizona are examined to determine the elevation range for which these records can be considered ergodic.

TESTS FOR STATIONARITY

A rigorous definition of a stationary process requires that statistical properties computed across the ensemble of all possible sample functions at any arbitrary time point be the same for any other time point. Verification of stationarity by this definition is

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1. Contribution of USDA, ARS, Soil, Water, and Air Sciences.

2. Research Hydraulic Engineers, USDA, ARS, Southwest Watershed Research Center, Tucson, Arizona.

not feasible since there are an infinite number of possible statistics, and a complete description of the random process by the ensemble of all sample functions would be required to compute them. Usually, long-term observations are available from only one sample function. Even so, practical tests for stationarity using individual sample functions can be devised by making some important assumptions, proposed by Bendat and Piersol (1966), which are "generally valid for the vast majority of random data in nature."

These assumptions are:

(1) If the data of interest are nonstationary, then the statistical properties computed over each of a sequence of short time intervals from a single sample record will vary significantly from one interval to the next.

(2) Verification of weak stationarity (time invariance of the mean value and autocorrelation function) is acceptable for the desired analyses and applications.

(3) The sample record of the data to be investigated is very long compared to the random fluctuations of the data time history.

(4) If the mean square value of the data is stationary, then the autocorrelation function for the data is also stationary. (This assumption is not necessary to test whether a single sample record is stationary, but it does simplify practical testing procedures.)

Under the above assumptions, stationarity of annual maximum 24-hour thunderstorm rainfalls and annual rainfall totals for the summer season for eight Weather Bureau stations in southeastern Arizona was tested. Figure 1 shows location and elevation of these gages. Figures 2 and 3 give annual maximum 24-hour thunderstorm rainfalls for the eight stations and 6-year means. Figures 4 and 5 show 6-year running means of summer seasonal rainfall.

To test for stationarity of the rainfall sequences, the following procedure was used.

Periods were divided into equal intervals of 6 years each, and a mean value and a mean square value were computed for each interval. The sequences of mean values and mean square values were then tested under the null hypothesis that they were each independent samples of a random variable with the same true mean value and the same true mean square value. The nonparametric run test as recommended by Bendat and Piersol (1966) was used.

In the run test procedure, the mean values and mean square values computed for a rainfall record are designated (+) or (-) depending upon whether they are above or below the average of all values for the record.

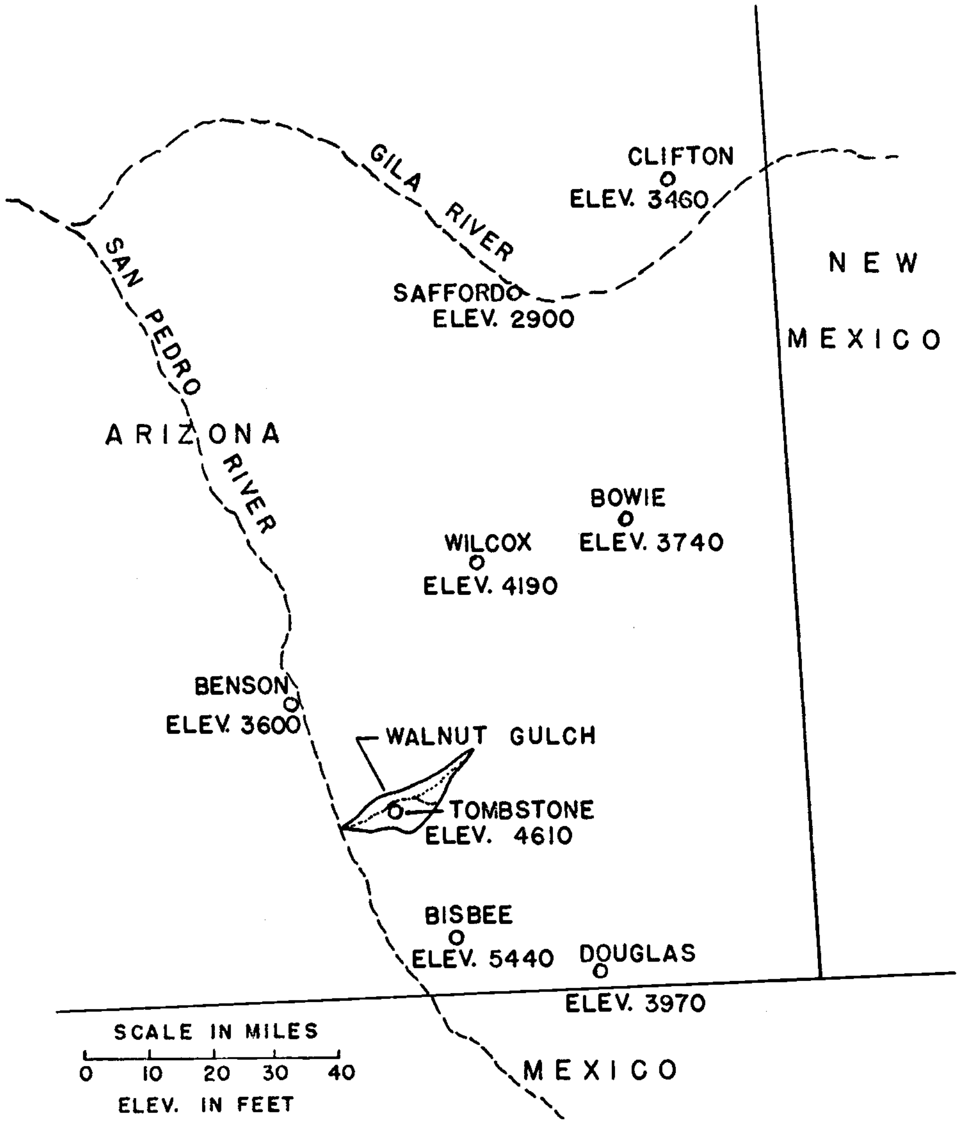

FIGURE 1. Location in southeastern Arizona of U. S. Weather Bureau rain gages used in tests for stationarity.

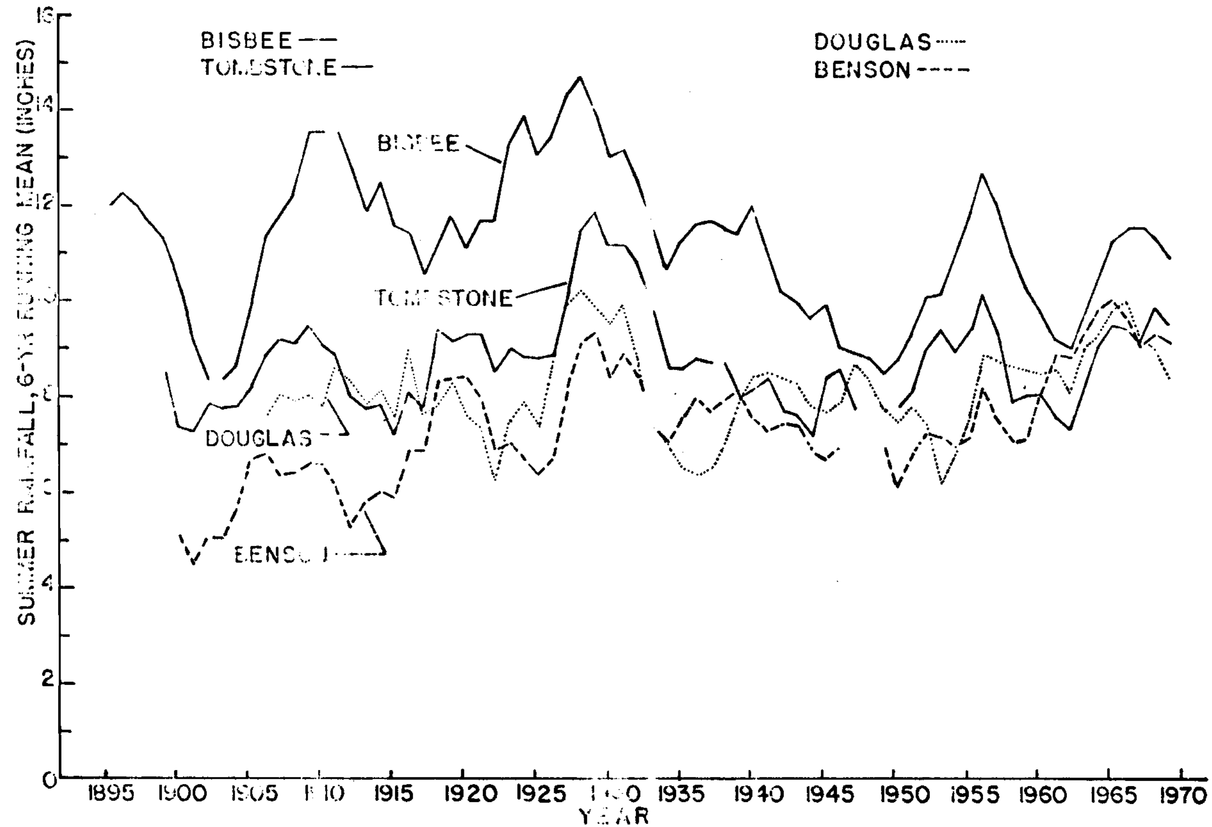

FIGURE 4. Six-year running means of summer seasonal rainfall recorded at Bisbee, Tombstone, Douglas and Benson.

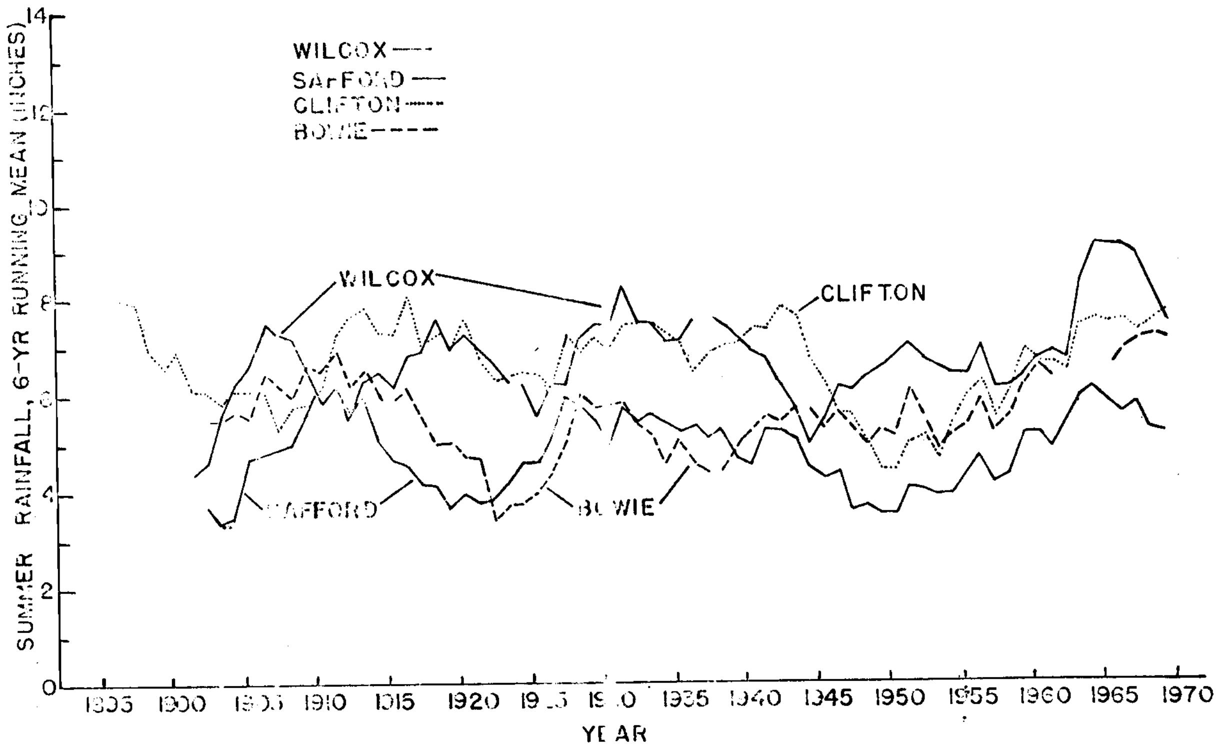

FIGURE 5. Six-year running means of summer seasonal rainfall recorded at Wilcox, Safford, Clifton, and Bowie.

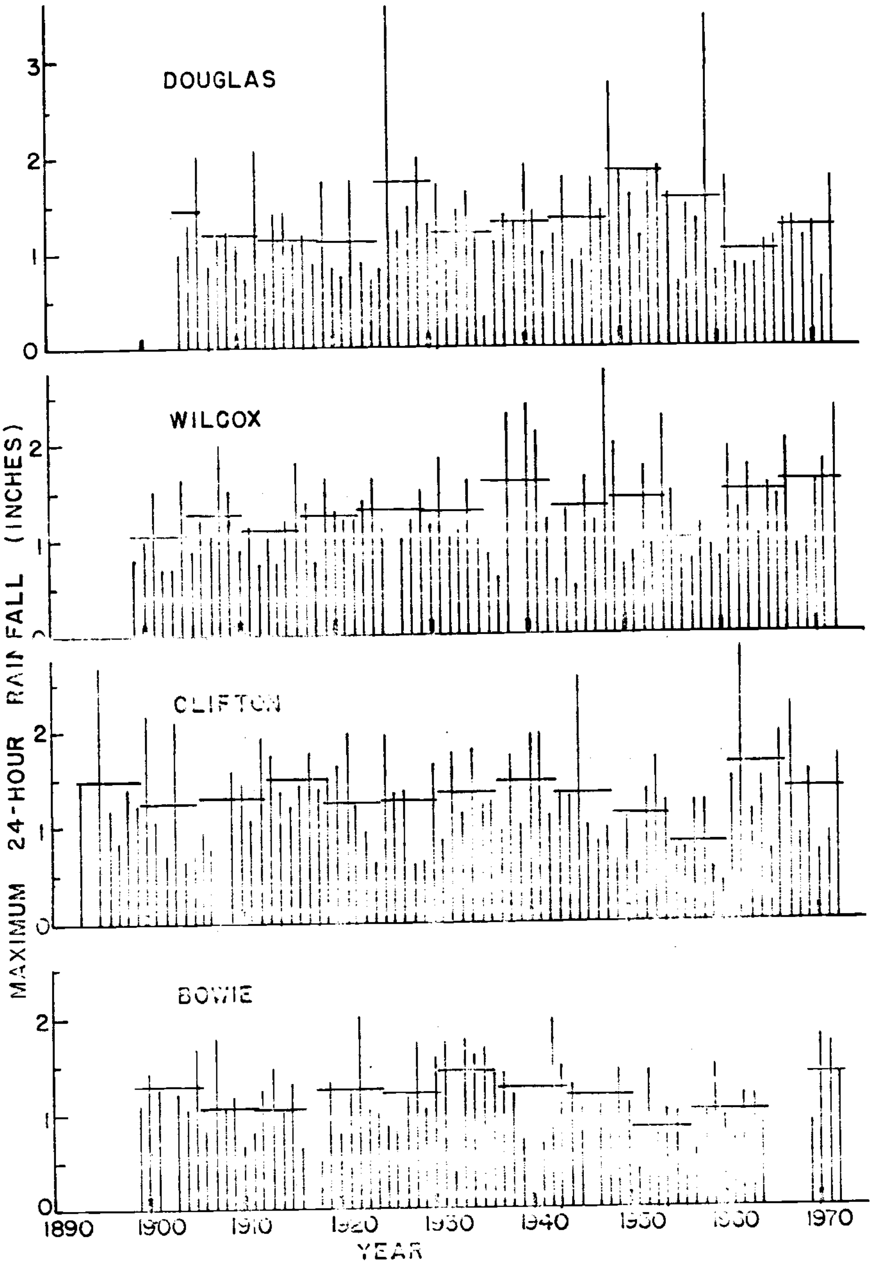

FIGURE 3. Annual maximum 24-hour summer rainfall recorded by U. S. Weather Bureau gages at Douglas, Wilcox, Clifton, and Bowie in southeastern Arizona.

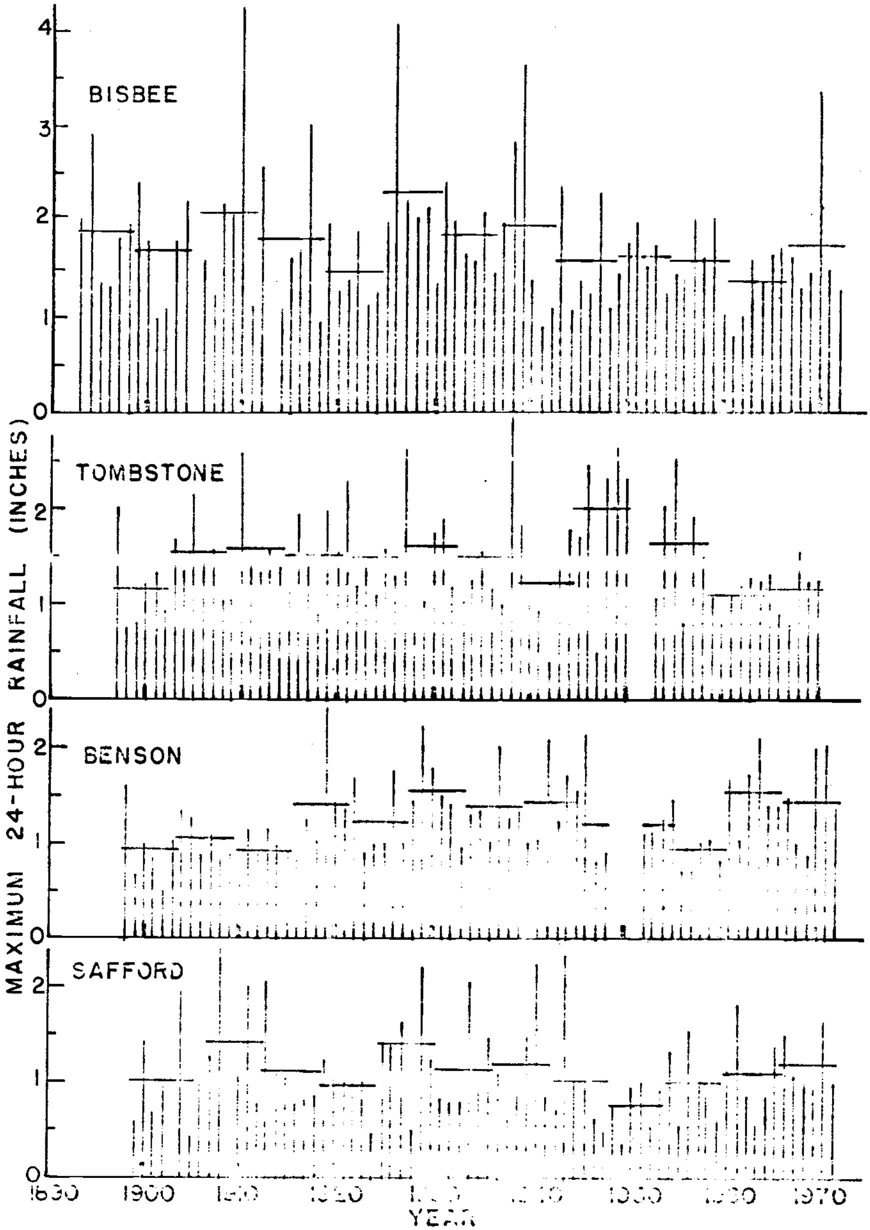

FIGURE 2. Annual maximum 24-hour summer rainfall recorded by U. S. Weather Bureau gages at Bisbee, Tombstone, Benson, and Safford in southeastern Arizona.

The number of runs, which is a sequence of like signs followed, preceded, or both by a different sign, is then determined for the record. The number of runs which occur in a sequence indicates whether or not results are independent random observations of the same random variables.

The number of runs determined for each of the eight Weather Bureau records are tabulated in Table 1. These are all within the 90% limits of the sampling distribution for runs, i.e., if A_i is the event that the number of runs is within these limits for record i, then $P(A_i) = .90$, where $P(A_i)$ is the probability that A_i occurs. Since air-mass thunderstorms are limited in areal extent and the minimum spacing is 25 miles between the Weather Bureau gages for which records were obtained, we assume the events A_i are independent. Therefore, the probability of the number of runs for all eight records falling within the 90% confidence limits of the runs sampling distribution is as follows:

$$\prod_{i=1}^{8} P(A_i) = (.90)^8 = 0.43$$

Thus, with these test results as a basis, the null hypothesis that the statistics for each interval were samples from the same population was accepted for all 8 records. That is, the sequences of annual maximum thunderstorm rainfall and summer seasonal rainfall for southeastern Arizona are assumed to be sample functions of stationary processes subject to the basic assumptions made in the above tests. It is recognized that, in general, thunderstorm rainfall is not a stationary process since the probability structure varies within the season. However, since in this paper we are interested only in sequences constructed from selected values from each season (maximum 24-hour thunderstorm rainfall and total summer seasonal rainfall), probability structure changes within the season would not be expected to affect these sequences.

CONSIDERATION OF ERGODICITY

For a stationary random process to be ergodic, the distribution of the variant must not differ when computed over different sample functions. This means that statistical properties of rainfall measured at different points must not differ significantly if the rainfall process is to be considered ergodic. Elevation may be the major factor in causing rainfall statistical properties to differ at different points in the rolling rangeland areas of southeastern Arizona. The ARS Walnut Gulch Experimental Watershed, which is instrumented with a dense raingage network, slopes gradually upward from west to east with the lowest and highest gages spaced about 15 miles and 1,400 feet in elevation apart. The lowest point on the watershed is about 4,000 feet; the highest about 6,300 feet.

TABLE 1. Number of runs obtained by the use of run test for stationarity on U. S. Weather Bureau thunderstorm rainfall records in southeastern Arizona.

Gage Location	Annual Maximum 24-hour Thunderstorm Rainfall		Summer Seasonal Rainfall			
	Mean X	Mean Square X^2	Mean X	Mean Square X^2	r_u 1/	r_L 2/
Benson	6	6	6	6	9	3
Bisbee	6	8	6	6	10	3
Bowie	6	6	3	4	7	3
Clifton	7	9	7	7	10	3
Douglas	5	5	8	8	9	3
Safford	5	5	5	5	8	3
Tombstone	7	7	8	6	10	3
Willcox	4	4	6	6	8	3

1) Number of runs r such that $P[R > r_u] = .05$ where R is random variable for number of runs.

2) Number of runs r such that $P[R < r_L] = .05$.

A regression analysis was run on the Walnut Gulch data in which average number of storms per year was regressed on elevation. Statistically, according to Student's "t" test with an α level of 0.05, elevation and occurrence of significant storms were not correlated. Furthermore, there was no suggestion of a significant increase in storm occurrence for the gages closest to the Dragoon Mountains. According to Student's "t" test, there was a correlation, although poor, between elevation and occurrence of storm rainfall greater than 1 inch. If the 3 gages at the lowest watershed elevations were eliminated, there would have been no significant correlation. Osborn, Lane, and Hundley (1972) found that mean summer rainfall was significantly greater on the upper half of Walnut Gulch than on the lower half. Also, analyses of records from 8, long-term (66 years or more) USWB gages in southeastern Arizona indicate differences in average summer rainfall and average maximum 24-hour rainfall between stations at different elevations (Figure 6). Therefore, for Walnut Gulch, an ergodic network for a range of 1,400 feet in elevation between gages may be too much to assume. In this particular case, records from gages located between 4,200 and 5,400 feet in elevation might be considered ergodic. There is no evidence that this restriction would be necessary for measuring or predicting major runoff-producing thunderstorm rainfalls, but it could be significant, as suggested above, in determining such things as average seasonal rainfall.

INDEPENDENCE OF SAMPLING POINTS

Thunderstorm rainfall records from 64 gages at various spacings on Walnut Gulch were examined to determine the minimum spacing for which gage records can be considered independent. Independence of records is necessary if they are to be combined as sample functions of an ergodic process. To check for independence, both conditional frequencies (estimates of conditional probabilities) and correlation coefficients were examined.

Conditional frequencies of measuring individual thunderstorm rainfall above thresholds of 1 and 2 inches were determined by tabulating total number of occurrences above the thresholds for selected gages and the number of simultaneous occurrences above these thresholds at other gages. The ratios of simultaneous (during the same event) occurrences to total occurrences gave the conditional frequencies (Figure 7). This relation indicates that the conditional frequency asymptotically approaches the unconditional frequencies as gage spacing increases.

To further investigate conditional frequencies, 10 years of summer rainfall records from 9 U.S. Weather Bureau standard raingages in the general vicinity of Walnut Gulch with spacings ranging from 10 to 67 miles were examined. Several stations with relatively short

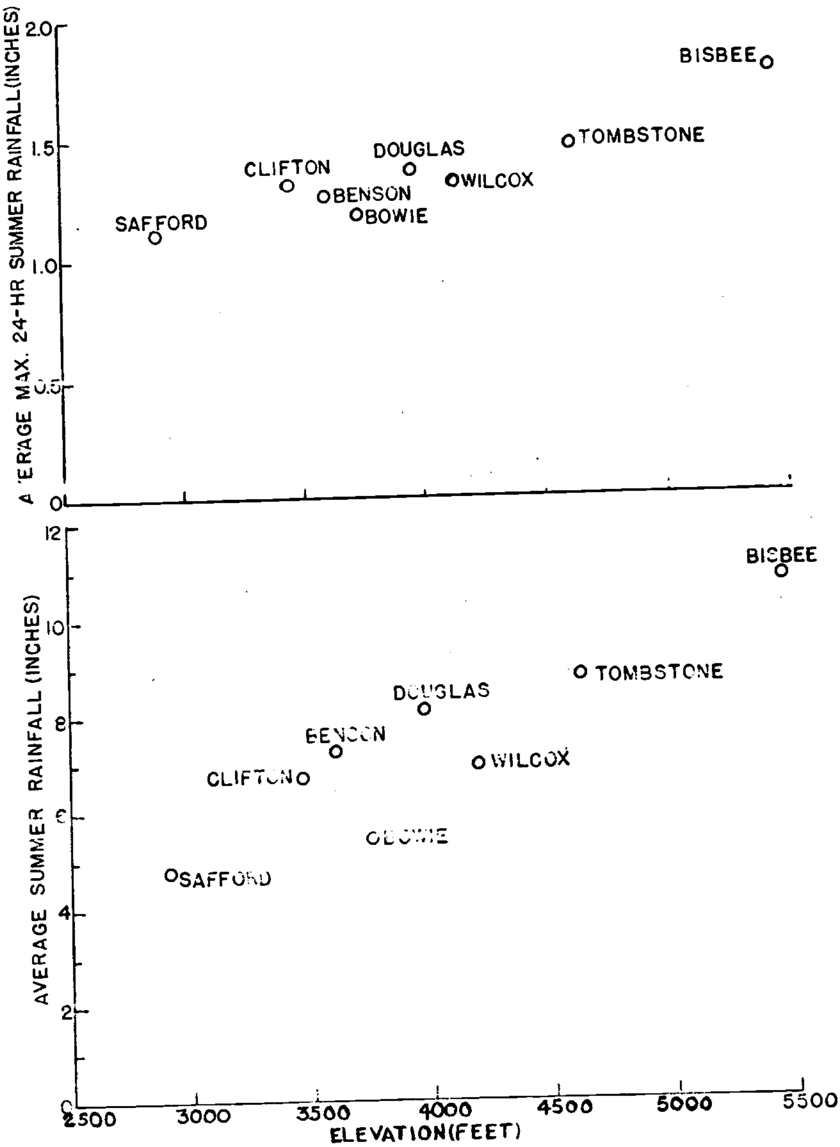

FIGURE 6. Variation of thunderstorm rainfall with elevation in southeastern Arizona.

records were included in this analysis to provide a denser network of gages, while some long-term stations were excluded because of spacing or because of missing records during the specific 10-year period. Generally, USWB gages are considered to provide independent rainfall measurements. Conditional frequencies were computed for 24-hour rainfall amounts in the same manner as for Walnut Gulch gages, on the assumption that for southeastern Arizona during the summer months 24-hour rainfall amounts essentially represent individual storm amounts. Figure 8 shows conditional frequencies plotted versus distance between gages. This plotting shows no relation between conditional frequency of 24-hour thunderstorm rainfall occurrence and distances between gages for spacings of 10 miles and more. It supports the interpretation of Figure 7 that conditional frequencies asymptotically approach the unconditional frequencies as distance between gages increases. Thus, paired gages for which conditional and unconditional frequencies are essentially equal are considered independent sampling points.

Correlation coefficients were computed for rainfall amounts recorded during thunderstorms for sets of 12 and 14 gages for a 12-year record from Walnut Gulch. The gages were chosen to provide as much variability in distances as possible without duplication and without having to compare all 64 gages. About 320 storms occurred during the 12-year record. Distance between the gages ranged from 0.5 to 14.5 miles. Figure 9 shows the relationship between correlation coefficient (r) and distance between gages. The upper 95% confidence limit for the null hypotheses that $r = 0$ was computed as 0.11 and is shown on the graph.

These studies indicate that gages spaced approximately 8 miles apart can be considered independent when air-mass thunderstorm rainfall is measured. However, independence may be assumed with much closer spacings for some purposes.

CONCLUSIONS

The following are major conclusions of this paper.

1) The hypothesis that sequences of annual maximum thunderstorm rainfall and summer seasonal rainfall for southeastern Arizona can be considered as stationary stochastic processes could not be rejected by run tests for stationarity recommended by Bendat and Piersol (1966).

2) Rainfall sequences that can be assumed stationary as recorded by gages located between elevations 4,200 ft. and 5,400 ft. on ARS Walnut Gulch Experimental Watershed may also be assumed ergodic.

3) Gages spaced approximately 8 miles apart on the rangeland areas of southeastern Arizona can be considered independent when air-mass thunderstorm rainfall is measured.

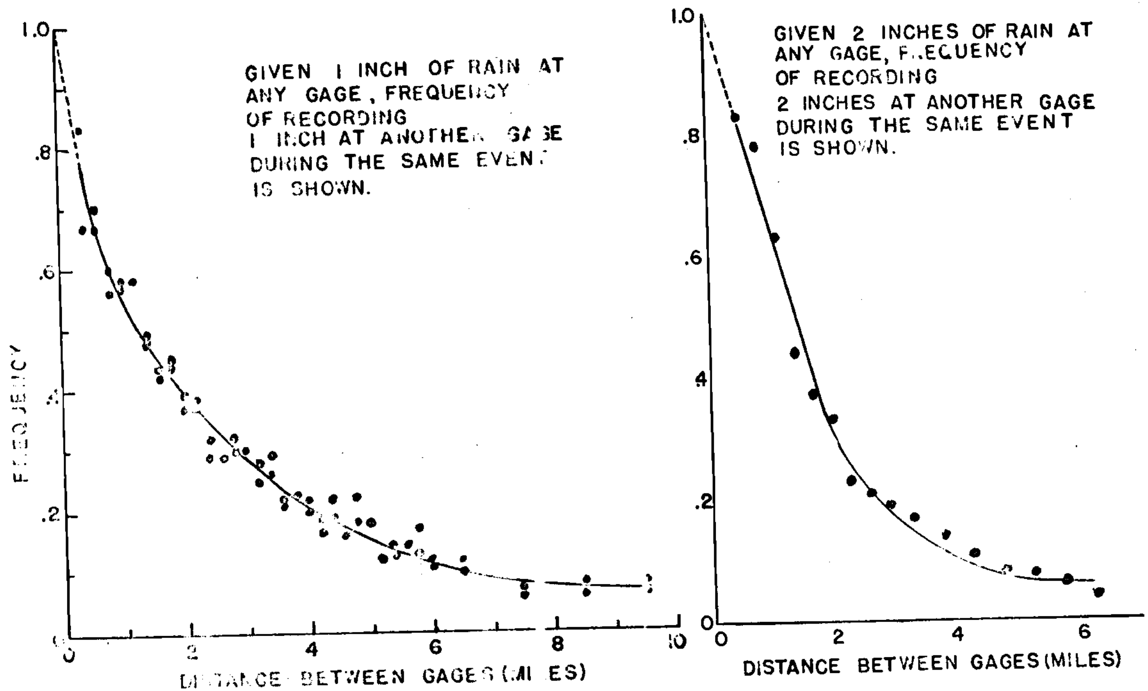

FIGURE 7 Conditional frequency of measuring thunderstorm rainfall above thresholds of 1 and 2 inches.

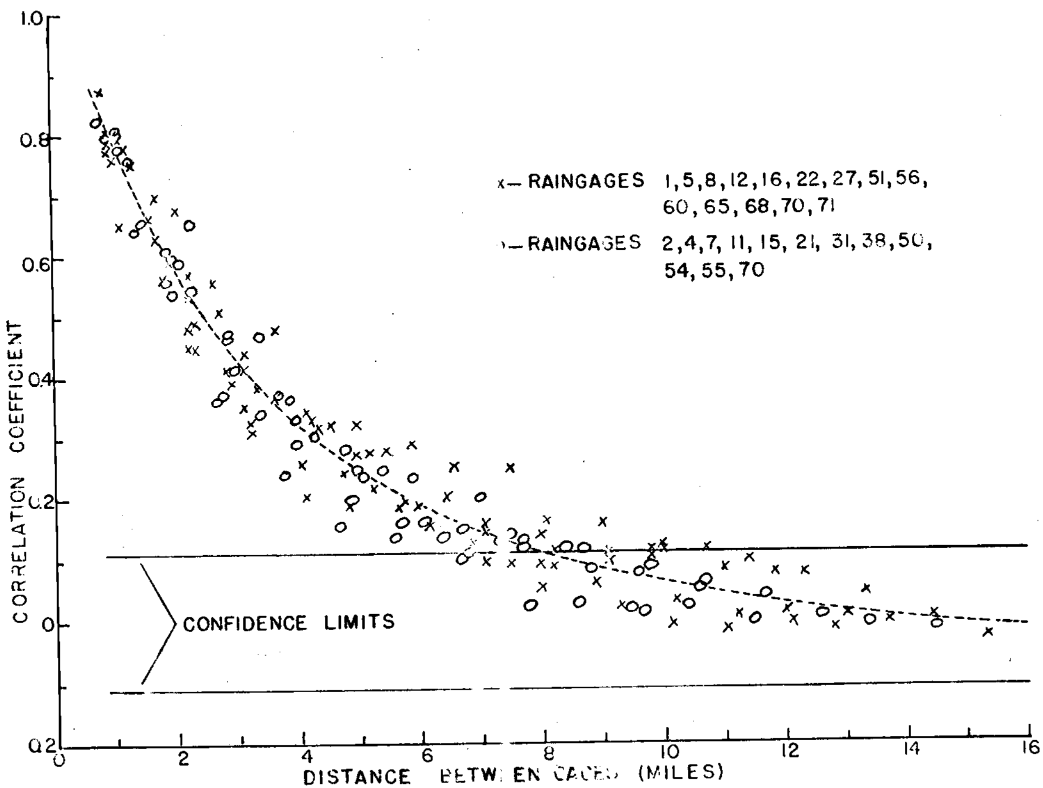

FIGURE 9. Correlation coefficients for rainfall amounts for selected pairs of gages on Walnut Gulch.

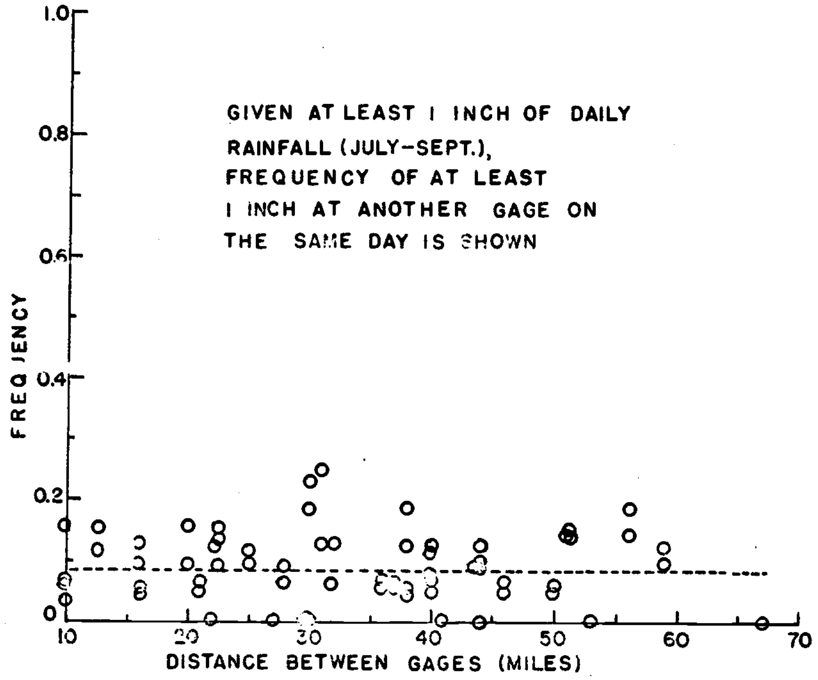

FIGURE 8. Conditional frequency of measuring 1 inch or more of rainfall at selected U. S. Weather Bureau stations in southeastern Arizona.

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SIMULATION OF GROUND WATER SYSTEMS WITH ANALOG MODELS[1/]

By E. P. Patten

ABSTRACT

Prior to the middle 1950's, analog modeling of ground-water systems was limited to the consideration of the steady-state response of aquifers whose permeability distribution was necessarily considered to be homogeneous and isotropic. Since that time, analog modeling capabilities have rapidly expanded to include the consideration of time dependent, non-homogeneous, non-isotropic systems with non-linear boundary conditions.

Today, a really extensive ground-water systems can be simulated with networks consisting of several tens of thousands of resistors and capacitors that approximate the layering of many aquifers and confining zones. Equipment used to impose pumping and recharge stresses to the modeled system has developed to the point where as many as 6,000 to 10,000 independent input functions can be economically imposed on the models. Non-linear boundary conditions can be simulated by using passive electronic devices whose electrical characteristics realistically simulate such hydrologic functions as recoverable evapotranspiration losses, the transition from confined to unconfined storage coefficient, and flow through the unsaturated zone.

A hybrid computing system using a relatively small digital computer in conjunction with the large multi-layer analog model is now being developed by the U.S. Geological Survey. That system is designed to select, measure, plot, and contour water-level data from the models, and to impose on the models pumping, recharge, and other stress functions. The hybrid system is designed to be interactive, with the hydrologist controlling the programming of the analog model with the digital computer, and appraising the output of the model through displays made available from the digital computer.

In the future, analog modeling undoubtedly will be of great value to the hydrologist in evaluating complex, multi-layered ground-water systems with non-linear boundaries: and with the development of hybrid capabilities, problems involving dispersion of mass transport can also be studied.

1/ Publication authorized by the Director, U.S. Geological Survey.

GROUNDWATER RECHARGE FROM A PORTION OF THE SANTA CATALINA MOUNTAINS

by

R. A. Belan and W. G. Matlock

INTRODUCTION

Tucson, Arizona's only source of water is from the underlying groundwater aquifer. For effective groundwater resource management all discharges and recharge sources of the aquifer must be known. Unfortunately, all have not been quantified; of particular interest is the mountain front recharge to the Tucson Basin. Very little research has been conducted to define the recharge of the various foothill regions surrounding the basin.

The basic objective of this study was to analyze the geohydrology of a portion of the Santa Catalina Mountains including definition of aquifer systems and their continuity throughout the foothills and to calculate groundwater recharge to the Tucson Basin from the foothills area. Figure 1 is a location map of the study area.

GENERAL DESCRIPTION OF AREA

PRECIPITATION, RUNOFF AND RECHARGE

Precipitation occurs in two seasons. Summer rainfall is characterized by high intensity, small areal extent and short duration, while the winter precipitation is of greater areal extent and longer duration. The annual precipitation in Tucson is about 11 inches, increasing with elevation to about 30 inches in the highest portion of the Santa Catalina Mountains (Sellers, 1960).

Much of the precipitation runs off the land surface because of shallow caliche zones that impede infiltration (Abuajamieh, 1966) or evaporates from the soil zone. The only significant areas of recharge are the stream channels and larger washes in the basin, and possibly faults and joints of the mountains and foothills. The greatest

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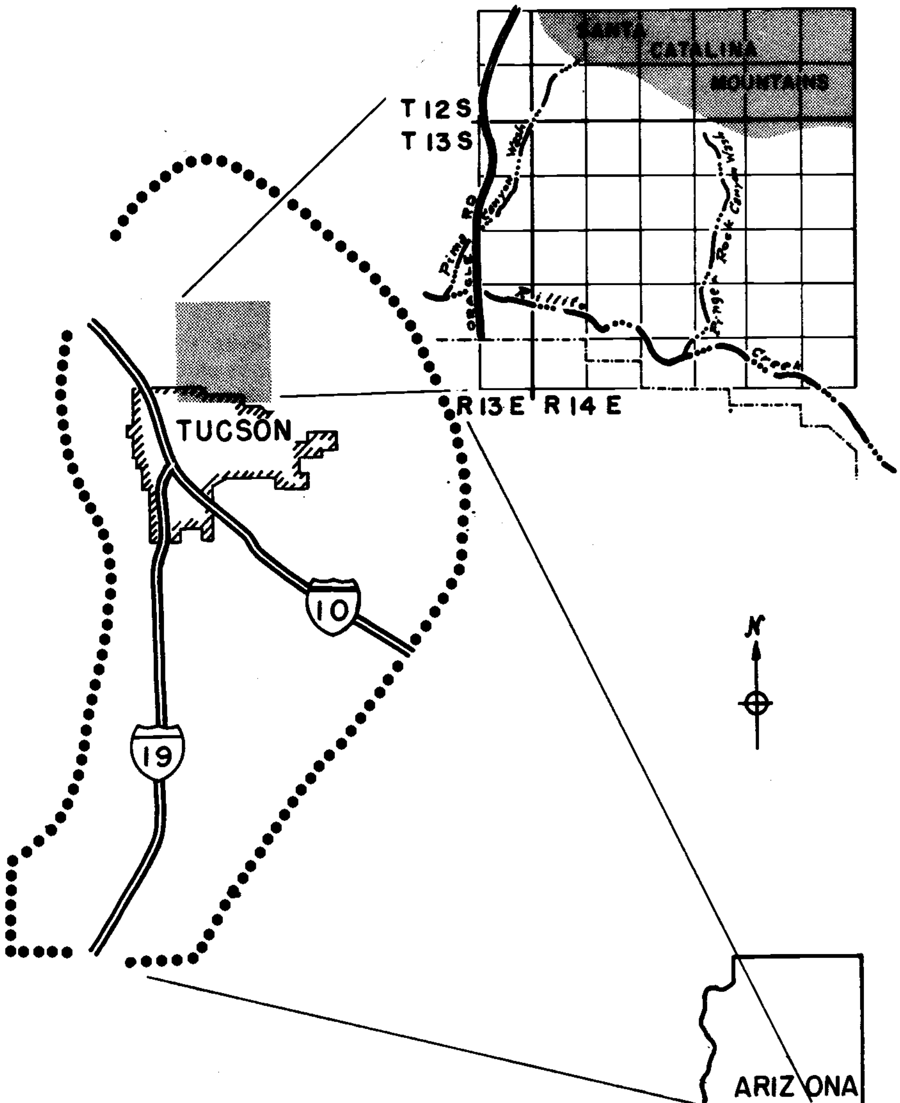

Figure 1 - Location map of study area (land grid) and drainage.

individual drainage areas in the study are Finger Rock and Pima Canyons.

Summer storms tend to produce sudden, often violent flows that may be heavily laden with silt which retards infiltration directly into the ground or in stream channels (Schwalen and Shaw, 1957). Winter precipitation often produces runoff which readily infiltrates in stream channels and recharges the groundwater system.

GEOLOGY

The Tucson Basin is surrounded by mountain ranges and was formed in the mid-Tertiary period (Maddox, 1960, Pashley, 1966). The Basin has since been filled with alluvium from the mountains which are composed chiefly of metamorphic gneisses and schists (Maddox, 1960, Medhi, 1964, Abuajamieh, 1966, and Pashley, 1966).

The study area is divided into four formations. First is the Santa Catalina gneiss, which forms the lowest boundary, and is composed of highly fractured, metamorphic granites (Medhi, 1964, Abuajamieh, 1966, and Pashley, 1966).

The Pantano Formation is the next oldest and was described by Brennan (1957), Streitz (1962) and Medhi (1964) as generally consolidated dark red or red-brown to purplish conglomerates and mudstones with some gypsum. Voelger (1953) and Pashley (1966) called this the Rillito Formation.

A "basin fill" unconformably overlying the Pantano consists of fine sands and silts or coarse sands and gravels weakly cemented (Pashley, 1966). This seems to correspond to the "deformed gravels" of Abuajamieh (1966) and forms the alluvial fans of the foothills.

Last is the Recent alluvium that fills the present wash and stream channels and flood plains (Schwalen and Shaw, 1957 and Pashley, 1966). This formation consists of unconsolidated silts, sands and gravels increasing in size towards the mountains to coarse sands, gravels and boulders.

The eastern half of the foothills is deformed by faults and joints. The largest fault is the Santa Catalina Fault, running east to west at the base of the mountains. It was accurately traced in the eastern half of the study area (Medhi, 1964, and Pashley, 1966), but in the western half the fault disappears under the alluvial fan and can only be inferred.

PROJECT DESCRIPTION

WELL NET

A well network was established in the study area by consulting well logs and drillers records in the files at The University of Arizona, United States Geological Survey and local well drillers' offices. The wells were drilled primarily for domestic supplies.

There were two main periods of well construction in the study area. The first was around 1930 when most of the wells along the northern portion were drilled or dug. The second period began in 1947 and continues to the present day. Wells drilled during the later period are located mostly along Rillito Creek and Oracle Road.

Some wells were abandoned and filled with rubble. In that case the location, bench mark elevation, and any recorded or estimated water levels were still useable. All other wells were measured in March, 1972. The base year 1930 was used for detailed study of the area as the water level information was more complete than that of recent years, and the water level was relatively undisturbed by pumping at that time.

WELL LOGS AND GEOLOGIC PROFILES

Geologic profiles based on well logs were made to correlate geohydrologic conditions and water level information. Well logs were dated from 1929 to 1971 with the older logs generally providing more descriptive geologic information. The well logs that contained the most details were used. In particular the interest was centered on water bearing horizon s.

The well logs and resulting profiles showed a high variability of subsurface geology and attemps to make stratigraphic correlations between wells were unsuccessful. Well depths range from tens of feet to over 500 feet. The shallowest wells are along Rillito Creek, and wells increase in depth going northward to the Santa Catalina Mountains. Apparent artesian conditions were found throughout the foothills with water table aquifers along Rillito Creek and a few areas next to the mountains. The current depth to water in the foothills is from 100 feet to 380 feet, and also tends to increase toward the mountains.

WATER LEVEL CONTOUR MAP

A water level contour map for 1930 shows the pattern of groundwater flow in the foothills. (Figure 2). The contours shown are the result of analysis of data from available wells, field observations, and study of the various geologic profiles. Contour lines are dashed in areas of uncertainty.

Interpretation in the eastern half of the area is complicated by faults and joints. The groundwater gradient varies from about 400 feet per mile in the northwest to about 60 feet per mile in the southeast. Contour lines along Rillito Creek were smoothed to reduce irregularities caused by local pumping and recharge from the flow in the creek. Comparing the 1930 and 1972 water level data showed that very little change has occurred in the water levels in the foothills themselves, but some lowering has occured along Rillito Creek because of heavier aquifer development.

DISCUSSION OF GEOHYDROLOGY

AQUIFERS

The well logs and water level information showed small sand and gravel aquifers primarily along the washes of the mountain front area which are perched relative to the Tucson Basin and local in nature. The only aquifers continuously traceable northward from the Tucson Basin aquifer were found along Pima and Finger Rock Canyon Washes in Section 12 of T13S R13E and in Section 22 and 23 of T13S R14E. These two aquifers which form the largest developed aquifers in the foothills were identified in troughs in the surface of consolidated sediments described by Maddox (1960).

Groundwater flow in the area between the defined aquifers is extremely small or nonexistent. This observation is consistent with the "dry" wells scattered throughout the area.

RECHARGE TO FOOTHILLS AQUIFERS

Water in the study area occurs as surface runoff, groundwater underflow and water which is, for the most part, locked in very low permeability materials. The surface runoff which occurs during times of rainfall and snowmelt gathers into the washes and drains into the lower basin area along Rillito Creek. Little of the surface runoff is thought to recharge local aquifers because of the tight materials under the Recent alluvium and limited duration of the flows. Recharge does occur in some sections of the washes and close to the mountains where the washes and faults cross or coincide. Significant recharge to the various sand and gravel aquifers also occurs directly through faults and joints in the mountains and higher foothills.

FLOW-NET ANALYSIS OF RECHARGE TO TUCSON BASIN

The study area was divided into 15 flow channels of which eight were considered to contribute significant groundwater recharge to the Tucson Basin (Figure 3). The Darcy equation was used in calculating the mountain front recharge with the total discharge computed as the sum of the discharges of the eight flow channels.

The widths and hydraulic gradients of the flow channels were found by flow net analysis of the 1930 water level contour map, by analysis of well logs and geologic profiles and by field observations. Transmissibility (T) values were computed from specific capacities (Cs) of wells representative of each channel based on the method of Thomason, et al, (1960):

$$T = C_s \times 2000$$

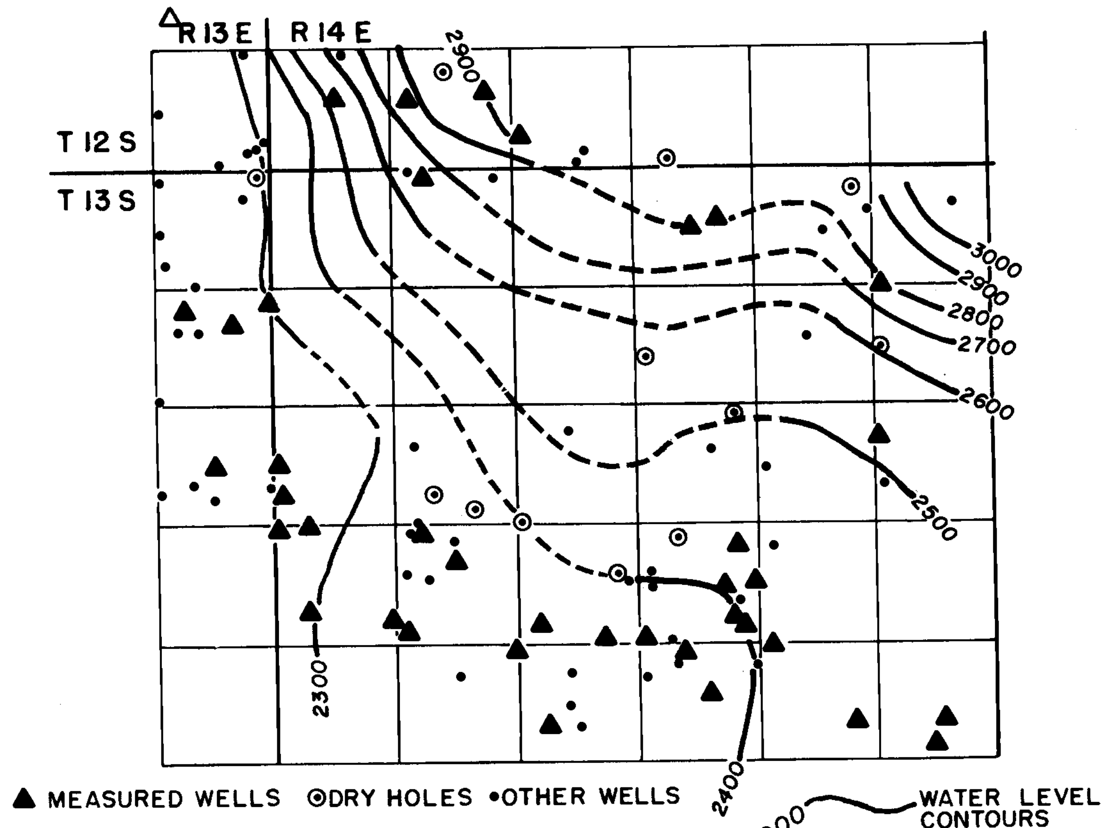

Figure 2- 1930 water level contour map.

All specific capacity values were determined from well log information except for two flow channels. Specific capacities for these channels were obtained from short term pumping tests.

The groundwater underflow across the 2400 foot water level elevation was then calculated for each lettered flow channel (Table 1). The areas between flow channels were considered to have near-zero transmissibility values, thus no significant flow in them. The resulting individual flows range from 4800 gallons per day (gpd) in channel G to 180,000 gpd in channel B. The total recharge was 300,000 gpd (336 acre-feet per year) or about 50 acre-feet per year per mile of mountain front as compared to the 325 acre-feet per mile per year obtained in an electric analog model study (Anderson, 1969). Total recharge from the study area represents less than one percent of the annual recharge to the Tucson Basin.

CORRELATION OF WATER QUALITY AND GROUNDWATER FLOW

A water sample was taken where possible from each well in the study area for chemical analysis including total dissolved solids, pH, hardness and common constituents found in local groundwaters such as sodium, calcium, chloride, sulfate, magnesium and nitrate.

Good correlations of water quality and groundwater flow were found for the flow channels of Figure 3. That is, water quality was uniform in a single flow channel but different from other channels. Anomalous qualities were explained by geologic variability, e.g., contact with the Pantano Formation, faults or mixing of groundwater flow. Excellent correlation was obtained in the far eastern portion of the study area in flow channels G. and H.

CORRELATION OF WATER TEMPERATURE AND GROUNDWATER FLOW

Groundwater temperatures ranged from 66.0°F to 78.8°F. The temperature readings were not _in situ_ measurements, and were used to show general trends with fairly good results. Differences of about 5°F or more were considered significant.

Water temperatures show the influence of recharge sources such as from snowmelt from higher elevations to the north. The generally lower temperature of groundwater in wells along Rillito Creek also is indicative of the snowmelt recharge from the Creek. Higher water temperatures found in two wells near Rillito Creek indicate minimal influence from Rillito Creek recharge, probably because of separation by low permeability materials.

Water from wells in flow channels B and C is warmer than that along Rillito Creek. There is a slight tendency toward increasing temperature away from the mountains in these channels. This may be because of longer contact with a warmer formation in the subsurface and possibly heat contribution from fault areas.

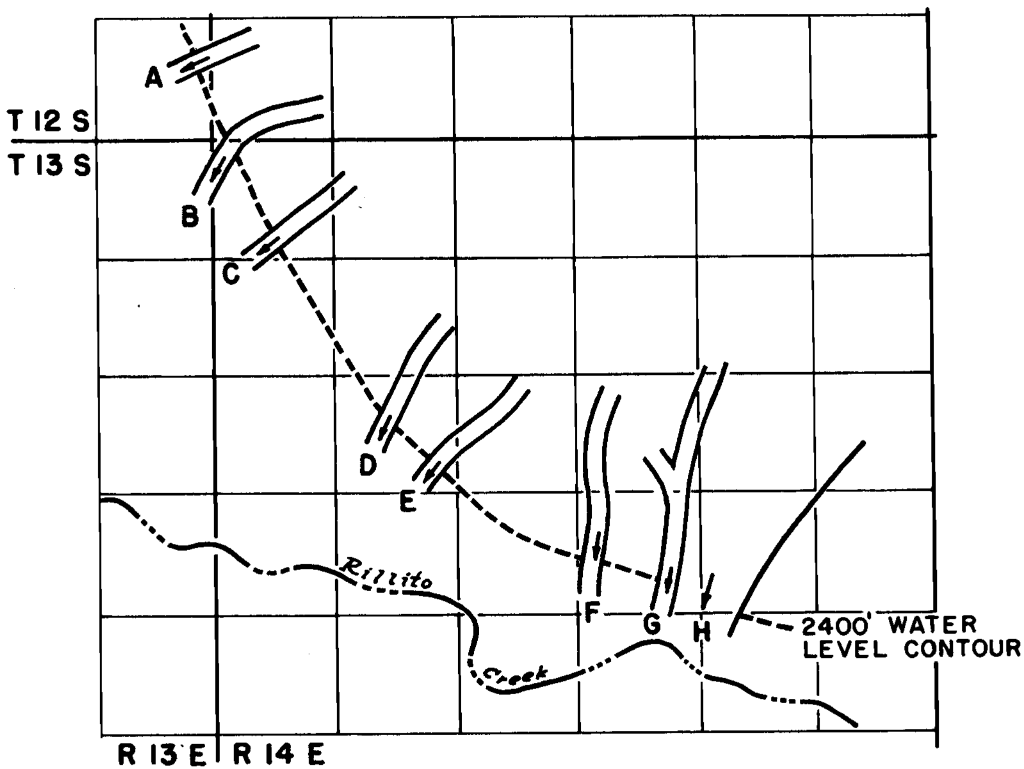

Figure 3 - Recharge flow channels.

Table 1. RECHARGE TO TUCSON BASIN

Flow Channel	Specific Capacity (gpm/ft)	Trans-missibility (gpd/ft)	Gradient (ft/mi)	Width (mi)	Discharge (gpd)
A	0.30	600	400	.04	9,600
B	2.70	5000	500	.09	180,000
C	2.60	5000	200	.04	40,000
D	0.70	1000	200	.06	12,000
E	0.70	1000	100	.07	7,000
F	0.90	2000	100	.07	14,000
G	0.40	800	60	.10	4,800
H	0.50	1000	50	.6	30,000
	TOTAL (ROUNDED)				300,000
					or
					336 AF/year

Water temperatures in flow channels G and H correlated well and confirmed other evidence of this groundwater flow area. Mixing with Rillito Creek groundwater occurs near the Creek.

CONCLUSIONS

The study area is an extremely complex transition zone between the mountains and the lower groundwater basin, and is representative of the foothills surrounding Tucson. Several aquifers showing unconfirmed signs of artesian pressure were located with the two largest developed aquifers in Section 12 of T13S R13E and in Sections 22 and 23 of T13S R14E. Water table aquifers are found near the mountains. All of the aquifers seem to be comprised of sediments that have filled old channels in the semi-consolidated Pantano formation which explains, in part, how extreme variations in quantity and quality of groundwater may occur in relatively short distances.

Groundwater gradients range from about 400 feet per mile in the northwest to about 60 feet per mile in the southeast. The depth to water varies from 100 feet to 380 feet.

Calculation of recharge to the Tucson Basin from the foothills study area by the Darcy equation is subject to considerable error. Groundwater gradients taken from the water level contour map were reasonably accurate but flow channel width and transmissibility values were subject to question. The use of specific capacity data to calculate transmissibility involves assumptions of aquifer uniformity obviously not met. Specific capacity is, itself, dependent on well construction and development and the effects of time on casing perforations. Flow channel widths were estimated from geologic profiles, flow net analysis of the water level contour map, and field observations.

Flow net analysis used to quantify groundwater recharge was limited by the scarcity of information and the complexity of the area. The total recharge was calculated to be 336 acre-feet per year. This represents about 50 acre-feet per mile of mountain front per year.

The study was limited by the low density of the wells or complete lack of wells in certain sections. Assuming transmissibility values of zero for inter-aquifer areas depicts conditions to the best of present knowledge. The true recharge is thought to be within the range of +100 percent to -50 percent of the value given but may be different by an order of magnitude.

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GROUNDWATER GEOLOGY OF FORT VALLEY, COCONINO COUNTY, ARIZONA

Ronald H. DeWitt

Fort Valley is an alluvial basin which lies approximately five miles northwest of Flagstaff. The basin is bounded by Tertiary volcanic features which are A-1 Mountain and Observatory Mesa on the south, Wing Mountain on the west, and the San Francisco Mountains on the north and on the east. Surface water drainage enters the area from the north, the northwest, and the south and leaves the valley to the southeast via Rio de Flag.

ROCK UNIT DESCRIPTIONS

PALEOZOIC AND MESOZOIC ROCKS

The subsurface sedimentary sequence consists of nearly flat-lying Paleozoic and Mesozoic rocks. The uppermost Paleozoic formation of interest in this study is the Supai Formation of Pennsylvanian and Permian age. The Supai Formation is composed, for the most part, of alternating siltstone and fine-grained sandstone. The Supai Formation is overlain by the Coconino Sandstone of Permian age. The Coconino Sandstone is very fine to fine-grained, is cross-bedded, and consists of well-sorted, rounded to sub-angular, clear, stained, and frosted quartz grains. The Coconino Sandstone is overlain by the Kaibab Limestone of Permian age. The Kaibab Limestone is a thick to thin-bedded, jointed, cherty, and sandy dolomitic limestone. In Fort Valley the Paleozoic rocks are penetrated by a single well which lies in the east central portion of the area. This well was drilled in 1938 for the City of Flagstaff but was unproductive and abandoned.

The Kaibab Limestone is discomformably overlain by the Moenkopi Formation of Triassic age. This Mesozoic rock unit is generally reddish-brown and consists of flat, very thin to thick layers of siltstone, mudstone, and sandstone. The Moenkopi Formation is encountered in all of the Fort Valley wells drilled to depths beyond 230 feet. The depth to the top of the Moenkopi ranges from 140 to 226 feet and the elevation at the top of the Moenkopi ranges from 7090 to 7180 feet. The 90 feet of relief on the top of the Moenkopi reflects an erosion surface and possibly structural offsets by faults now covered by the more recent volcanics.

VOLCANIC ROCKS AND INTERFLOW ZONES

The volcanics in the Fort Valley area range in age from late Miocene to Pleistocene (Cooley, 1960). Fort Valley is a focal point for volcanic flows which were derived from A-1 Mountain, Wing Mountain, and San Francisco Mountain. Interflow zones which comprise paleosoils, cinders, and conglomerates occur between individual lava flows in the subsurface.

ALLUVIAL ROCKS

Three alluvial units which overlie the volcanic rocks were distinguished by Cooley (1960) range in age from Pleistocene to Recent. The alluvial rocks comprise soil, cinders, clay, and boulders.

HYDROGEOLOGIC FEATURES

FENCE DIAGRAM

At present (1973) data are available for 24 wells which are located in the alluvial portion of the valley. The wells throughout the valley yield from 0-20 gallons per minute. An analysis of drillers logs of 10 of these wells was used to construct Figure 1, a hydrogeological fence diagram of Fort Valley. The rock units shown in Figure 1 are generalized.

Alluvial rocks. The unit at the top of each section represents alluvium which ranges in thickness from 7 to 123 feet.

Volcanic rocks and interflow zones. The second units in descending order represents volcanic rocks which range in thickness from 4 to 70 feet. The wells penetrate from 1 to 4 volcanic units. Correlation of the volcanic units is difficult due to similar lithology, multiple source areas, and abrupt pinch outs. The volcanic rocks encountered in wells in the southeast portion of Fort Valley may be derived from A-1 Mountain, those in the north from San Francisco Mountain, and those in the west from Wing Mountain. Interflow zones are also shown on Figure 1. The thickness of these zones ranges from 6 to 54 feet.

Moenkopi Formation. The lowermost unit shown on Figure 1 represents the Moenkopi Formation which is penetrated by 6 of the 10 wells used in construction of the diagram. Wells in the western portion of Fort Valley are of insufficient depth to penetrate the Moenkopi Formation. The drillers log from the City of Flagstaff well drilled in 1938 indicates that the total thickness of the Moenkopi is 270 feet.

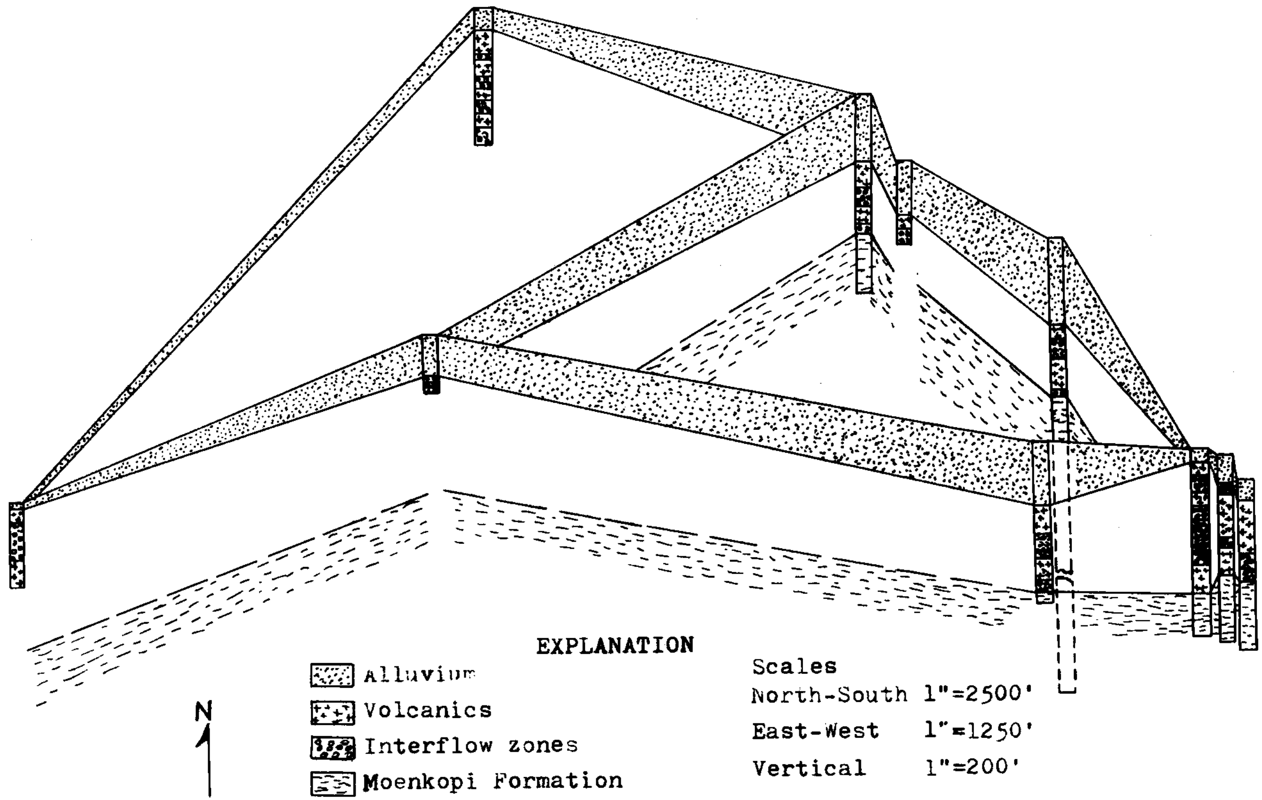

Figure 1 HYDROGEOLOGICAL FENCE DIAGRAM OF FORT VALLEY

GROUNDWATER FEATURES

All of the groundwater presently developed in Fort Valley is found in perched aquifers. The regional water table in the Fort Valley area is believed to lie at a depth of approximately 1750 feet, a depth which is below the base of the Coconino Sandstone (McGavock, 1968).

Occurrence and flow pattern. Groundwater reservoirs found in the alluvial rocks are perched on impermeable clay zones located at the base of the alluvial units. Groundwater is also found in the volcanics which are locally highly fractured and which also overlie impermeable clay zones. Perched aquifers also occur in interflow zones perched on impermeable clays or on unfractured volcanics. Significant quantities of water are perched on top of the Moenkopi or in sandstone layers in the upper portion of the Moenkopi Formation.

Recharge relations. Groundwater in Fort Valley is derived from infiltration of runoff and from precipitation. Chief sources of runoff are watersheds on San Francisco Mountain, Wing Mountain, and A-1 Mountain. This recharge water infiltrates alluvium or fractured volcanics and percolates downward until it reaches an impermeable zone where it becomes perched groundwater. Under natural conditions water in the perched reservoirs slowly drains into the underlying Paleozoic rocks and is lost to the shallow groundwater system.

CONCLUSIONS

The results of this study indicate the following:

1. Local perched groundwater reservoirs occur where an impermeable zone is provided by unfractured basalt or by an impermeable clay zone or by the Moenkopi Formation.
2. Greatest well yields are derived from fractured volcanic rocks, from alluvial units, and from interflow zones where these rock units are underlain by suitable perching zones.
3. The reliability of wells in Fort Valley is dependent largely on seasonal precipitation and runoff.
4. Most wells in Fort Valley supply adequate amounts of water for domestic purposes.

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WATER-QUALITY STUDIES TODAY AND TOMORROW[1]

By John D. Hem[2]

ABSTRACT

Development of better instruments for analysis and automation have greatly increased the available information on quality of water during the past decade. There remains a need for further research on relationships between dissolved material and the solids in contact with water in order to cope with existing or potential problems in water quality such as the extent to which lead from automobile exhausts may contaminate water supplies, or the safety of disposal of toxic wastes into deep saline aquifers.

IMPORTANCE OF WATER QUALITY

"Aquatic chemistry is of great practical importance because water is a necessary resource for man. We are not concerned with the quantity--water as a substance is abundant--but with the quality of water and its distribution."

Strumm, W., and Morgan, J. J., 1970, "Aquatic Chemistry," New York, Wiley-Interscience, p.3.

It may seem a bit strange to an Arizonan to hear someone say water is an abundant substance, but certainly few would argue about the importance of quality and distribution of water resources. It is hardly necessary to point out the importance of water quality to hydrologic science.

As a nation we have become much more sensitive to the quality of our water resources during the past decade. There is little doubt that man and his works often are mainly, if not entirely, responsible for the deterioration of the quality of the water in the old swimming hole, or the fishing stream we may remember from our youth. But some deterioration had probably occurred even then, and nature itself produces water of very bad quality, as any of you who know Lemonade Spring in New Mexico, or Salt Banks on Salt River, Arizona, can well attest.

Part of the present day concern over water quality results from

1. Publication authorized by the Director, U.S. Geological Survey.

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an increased awareness of our environment and how we relate to it. But in another sense the concern is heightened by rapid advances in technology that have enabled us to look more closely and carefully at details of water quality than was possible a few years ago. We can determine concentrations of specific organic compounds, synthesized so far we know only by man, down to very low concentrations. Something is known about toxic levels and effects of some of these substances, such as the chlorinated hydrocarbons PCB and DDT. Small amounts of other exotic but chemically stable organics such as the phthalate esters, which are widely used as plasticizers in the plastics industry, are now known to be widespread in water but their effects on life forms are almost entirely unknown. One is left with a nagging feeling of uncertainty about the possible effects of such pollutants. A more familiar hazard--pollution by harmful bacteria--has received emphasis for many years. Water supplies in our municipalities are bacteriologically monitored and owing to effective treatment methods these supplies have an excellent safety record. Waterborn disease is not a significant public health problem in the U.S., and has not been for many years, thanks to controls and monitoring on this aspect of water quality. Treatment for some of these other pollutants may turn out to be a necessity also.

Because my background and qualifications are in the area of inorganic chemistry this discussion will emphasize the inorganic side of water quality.

INSTRUMENTATION

One can discern several lines of instrumental development that are greatly increasing our knowledge of the chemical composition of natural water. Instruments such as the atomic absorption flame-spectrophotometer enable us to determine low concentrations of a greater number of elements in solution about which we have had little knowledge previously. Gas chromatography coupled to mass spectroscopy has enabled us to determine specific organic compounds at very low concentrations.

A second line of instrument development is the automated analysis laboratory, capable of determining in considerable detail the chemical composition of large numbers of water samples with minimal human attention and minimum cost.

A third line of development concerns instruments that can be used in the field at the actual sampling site, to detect and record changes in the actual water body or stream. Thus one can learn more about possible artifacts that result from sampling and minimize the uncertainties inherent in sampling and possible changes within the sample before it can be analyzed. Good examples are the various electrodes that make possible the portable DO meter and fixed installations capable of continuous monitoring of pH, chloride, DO, temperature, and other parameters of stream water at the site.

An obvious result of these development, and of the general interest in and desire to improve the environment, is an enormous outpouring of data on water quality, as we endeavor to find out how bad (or good) things really are, and to monitor change. To cope with this flow of information requires the services of electronic computers for data storage and recovery.

FUTURE NEEDS

Although these developments of instruments and technology have greatly increased our ability to evaluate water quality, there remains a great deal of fundamental work to fit the measurements together in coherent ways that will help to attain a better understanding of processes and interactions within natural systems. With the present level of knowledge we can accumulate large amounts of data to show water-quality conditions now, and as time passes we can observe changes occurring. This makes us archivists, or historians, but not truly scientists, until we can logically explain why and how the changes occur. Hopefully a better understanding of processes would permit development of predictive models, and enable us to know in advance something aboutthe final result of a specific action directly or indirectly altering water quality. Many of the man-caused changes in water quality that have occurred in the past resulted from applications of technology that had unanticipated effects. It might also be worth noting that current efforts to improve water quality by applied technology may also have unanticipated effects and may not be very successful until we understand better the total impact of our well-meant efforts.

Subjects that I believe are of primary importance for more intensive applied research include:

1. Development of a more complete understanding of chemical relationships among dissolved and solid-state or sorbed materials in natural systems.
2. Development of equilibrium and transport models for water quality that take into account the relationships developed under 1.

Two specific environmental problems bearing on water quality will be cited to show more definitely where we lack certain kinds of information and understanding.

In the mid 1920's it was found that the addition of lead tetraethyl to gasoline permitted higher compression ratios and better performing automobile engines. Even at that time serious questions were raised concerning the safety of leaded gasoline and after several government-sponsored investigations it was decided that hazards associated with the extent of use of leaded gasoline then foreseen would not be important enough to ban its use. A recommendation apparently was made, however, that the subject be restudied in future years, but this seems to have been forgotten. By the 1950's and 60's the amount of lead per gallon had been greatly increased, to an average of about 3 grams per gallon of gasoline. And nearly all gasoline sold was leaded. The amount of lead thus introduced into the environment within the United States has been around 250,000 tons a year in recent years--an amount far greater than could ever have been forseen in 1925. Our knowledge of where this large amount of lead has gone is inadequate--we do know at least some of it has accumulated somewhere, probably mostly in the more thickly populated regions of the United States. In the meantime, however, the possibility of environmental hazard from this inadvertent lead stockpile had been forgotten. It was not until the mid 1960's that interest in the subject revived, after publication of papers such as that of Tatsumoto and Patterson (1963) that showed lead at low concentrations was a widespread environmental pollutant. About this time analytical methods of better sensitivity and convenience for determining these concentrations of lead became available. Many studies of the occurence of lead in the air, in rain and snow, in the ocean, in river water, and in plants and soil near highways, have now been published, but whether the amounts found constitute a major hazard or not is still not agreed upon. The amounts can be significant in water-quality terms, however. Studies by Lazrus, Lorange, and Lodge (1970) and by others have shown, for example, that rainfall in urbanized areas commonly contains lead in concentrations above the U.S. Public Health Service upper limit for drinking water (50 micrograms per liter) although some of this is in particulate rather than dissolved form. The runoff from such areas seems commonly to contains much larger amounts of lead; mostly in suspended particulate form, however, and therefore not observed by most water-quality investigators, who are inclined to look only at dissolved fractions of runoff load. It is not known whether the particulate lead is mostly composed of actual solid lead compounds or if the lead is adsorbed on other kinds of particulate material. Furthermore, our understanding of the stability of the particulate lead is inadequate--we cannot predict what conditions in the runoff water, or soil moisture, might tend to release lead to solution. If such releases did occur on a substantial scale, lead contamination of many water supplies obviously would result. Research on form and stability of lead in soil and stream sediment and the behavior of such materials under different conditions is needed. Lead in rainfall reaches high regional levels in the northeastern United States and low soil pH and relatively strong solvent power of runoff in that region may already be adding dissolved lead to river water. In any event more attention needs to be given to the occurence of lead in water supplies in susceptible areas. Roadside lead also may tend to be mobilized by the salt (sodium chloride) used in snow removal. Research results available now can serve as a guide to selecting the most critical areas for monitoring.

Another kind of environmental problem related to water quality is raised by the disposal of obnoxious wastes through deep wells. This approach to solution of waste-disposal problems is rapidly becoming more attractive to industry because it often promises to be less costly than treating such wastes to a level that would permit their release into a surface stream. Wastes placed in deep saline-water aquifers may nevertheless return to cause trouble, or bring about undesirable changes in other parts of the aquifer than that into which they are introduced. Both the hydraulic and chemical aspects of this topic are matters for serious concern. The injected waste must first displace whatever fluid was initially present, perhaps with undesirable effects elsewhere in the hydraulic system. The ways in which the injected material may react with dissolved solutes or with rock materials are almost impossible to predict. Hopefully such reactions might tend to prevent the wastes from travelling very far, but for specific minerals and wastes the present state of knowledge will not generally permit one to draw any firm conclusions. An extensive research program aimed at devising ways of predicting how injected wastes will behave is obviously needed. In the meantime, however, there will be an increasing pressure on regulating agencies to permit an ever larger amount of subsurface waste injection.

I have cited these two areas of water-quality concern to illustrate that collection and analysis of water samples does not always provide a sufficient base for understanding the factors that control water composition. Nor do such data help much in deciding which factors are the most important ones. It happens that both the lead-circulation and the underground waste-disposal problems are made especially difficult because of lack of understanding of chemical processes that take place at solid-liquid interface zones, and because we have not adequately explored the composition of suspended loads in streams. Other poorly understood factors controlling water quality in hydrologic systems could be cited. Pollution control and environmental improvement will be difficult to attain until the systems are better understood. Now that we have better analytical tools and facilities for making quality measurements, it is time to put them to use to help gain the needed understanding.

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THE USE OF A COMPUTER MODEL TO PREDICT WATER QUALITY TRANSFORMATIONS DURING SUBSURFACE MOVEMENT OF OXIDATION POND EFFLUENT

G. G. Small and L. G. Wilson
WATER RESOURCES RESEARCH CENTER

Oxidation lagoons are increasing in popularity in the southwestern United States for the economical treatment of small quantities of domestic sewage. However, seepage may occur from the bottom of these lagoons, possibly endangering the quality of the groundwater.

This study was devised to 1) estimate seepage losses during the inception of a new oxidation lagoon; 2) monitor the subsurface movement of seepage water; 3) study the transformations in nitrogen and certain other quality characteristics during movement of effluent through the sediments above the water table and during mixing with the native groundwater; and 4) compare the field data on nitrogen transformations during vertical flow from the lagoon with parallel data from a predictive computer model.

The lagoon study site was located northwest of Tucson on Interstate 10 and Ina Road. The lagoon was constructed in 1970-1971 for an additional treatment area for the increasing amount of raw sewage from northwest Tucson. The lagoon, managed by the Pima County Sanitation Department, has a surface area of 10 acres (Fig. 1).

Three existing wells were encompassed by the lagoon after final construction of the dikes. These wells include 2 PVC sampling wells terminating within the zone of aeration at depths of 40 and 60 ft and a 100 ft steel access well designed to be used in conjunction with a neutron probe for obtaining moisture profiles.

A nearby irrigation well provided samples of the native groundwater before and during the operation of the lagoon. A critical depth flume located in the inlet line of the lagoon coupled with an automatic water stage recorder was used to meter the daily sewage flow into the lagoon (Fig. 2).

Raw sewage inflow averaged 0.80 mgd over the approximate 3 months of monitoring. Seepage rates were calculated from data on inflow, evaporation, and change in storage of the lagoon. The seepage rates ranged from 0.40 ft/d to 0.03 ft/d. Although the seepage rates were low, the total infiltration of the lagoon was high. Approximately 76 percent of all sewage entering the lagoon was lost as seepage.

Water samples were obtained from 40, 60, and 100 ft below the lagoon. These water samples together with samples of the lagoon water, were examined for coliform organisms, plus various physical and chemical constituents. After several days of lagoon operation, samples from the PVC wells illustrated a reduction in COD and coliforms due to the percolation of the effluent through the zone of aeration as compared to the COD and coliform values for the raw sewage. High nitrate levels, composed of indigenous nitrate and nitrate introduced into the soil from the effluent, appeared in the PVC wells.

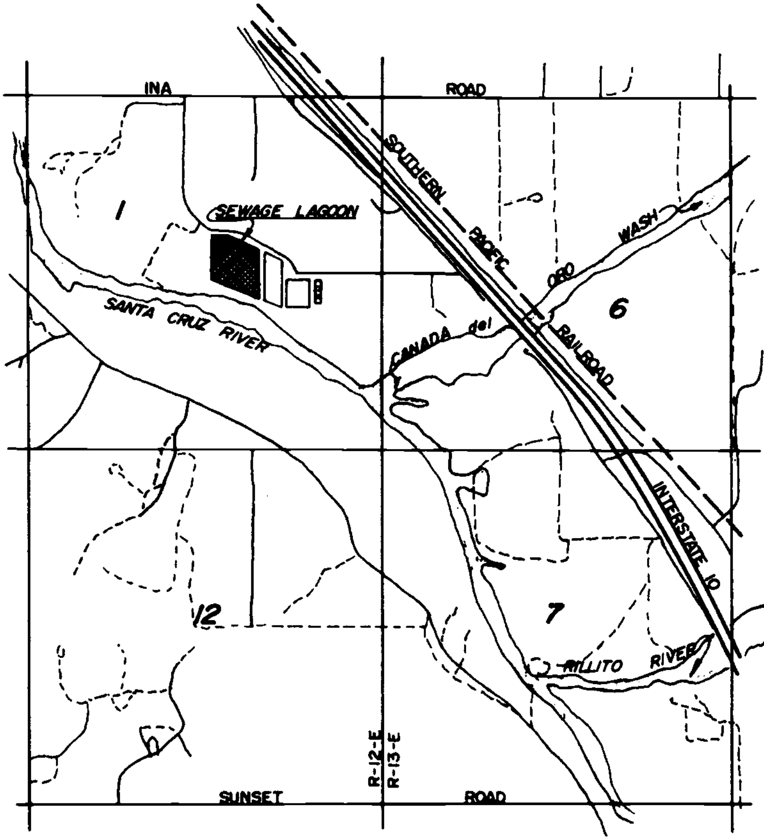

Figure 1. Location of Sewage Lagoon

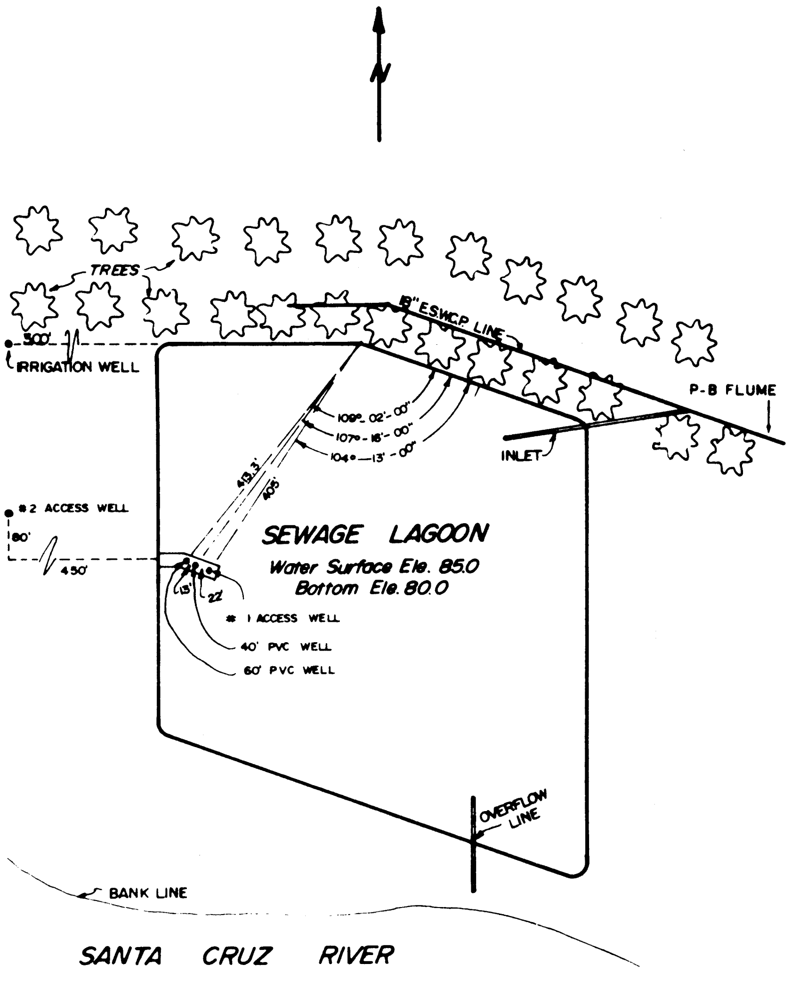

Figure 2. Location of Existing Wells in the Lagoon Vicinity

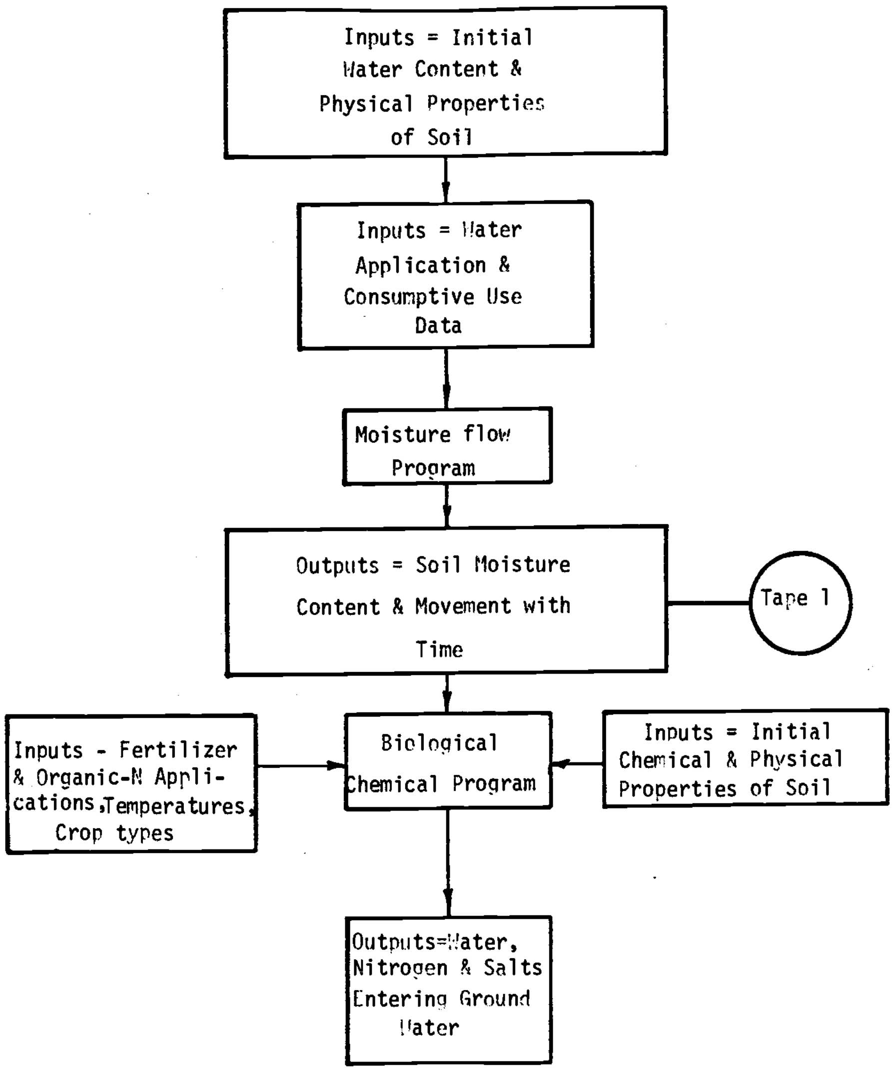

Figure 3. Generalized Block Diagram of the Computer Model (after Dutt et al., 1970)

Dissolved oxygen concentrations of the lagoon surface water reached supersaturation levels during the daylight hours although the benthic deposits on the bottom of the lagoon were approaching anaerobic conditions. This resulted in nitrification taking place in the upper lagoon waters and denitrification occurring in the benthic deposits.

Moisture logs showed substantial water content changes in the first 10 ft of sediment with only a slight water content change occurring at 40 and 60 ft. Even though the moisture logs indicate saturation in the first 10 ft of sediments enough air apparently remained trapped within the soil pores to permit nitrification of the infiltrated effluent and the indigenous nitrogen. Additional results were presented by Small (1973).

A computer model developed by Dutt, et al (1970) was used to approximate the change in nitrogen concentrations of the groundwater, employing the soil segment concept. This concept was adopted to approximate field variations, using mathematical relationships developed for homogeneous soil systems. The basis of the soil segment concept is the use of a finite number of equal length units of soil along the flow lines. Each soil segment is considered to be homogeneous and the same segment may or may not be considered to be homogeneous with its connecting segments.

The model assumes that the effects of solute-soil interaction on soil moisture flux are negligible in comparison with physical and biological properties effecting water movement, independent of chemical changes in the soil solution. Therefore the model is divided into two independent parts; the first deals with moisture movement, Moisture Flow Program, and the second deals with changes in chemical composition and distribution, Biological-Chemical Program.

In the execution of the model, the initial moisture contents and physical properties of the soil are inputs to the Moisture Flow Program as well as the amount and time of water applications and consumptive use data (Fig. 3). Using these inputs the Moisture Flow Program calculates the moisture content and movement at stipulated time intervals for each soil segment.

The output from the Moisture Flow Program is then used as input for the Biological-Chemical Program, along with temperature at various stipulated times and depths plus initial chemical parameters for the solutes being considered in each soil segment. Also serving as inputs are the amount and time of fertilizer applications plus the organic matter residues and the concentrations of solutes in the applied water. The Biological-Chemical Program calculates the chemical composition of the soil segments and the chemical composition of the water entering the water table.

The following list of assumptions was devised to simplify the modeling of the soil-water system [Dutt, et al (1970)].

1. No gaseous losses of nitrogen occurs. This assumption is valid when aerobic conditions exist in the soil, and urea and ammonia fertilizers are not applied on or near the land surface. The assumption would not hold in cases such as bog soils where restricted aeration exists, nor in cases where ammonia is easily lost as a gas.

2. The soil pH remains in the range 7.0 to 8.5. The effect of hydrogen ion activity on soil nitrogen transformations is approximately constant in this interval.

3. Symbiotic and non-symbiotic nitrogen fixation and fixation of ammonia in clay crystal lattices are small in magnitude by comparison with other nitrogen transformations considered in this research.

4. The ammonia-N mentioned in this study is total soil ammonia-N less that fixed in clay lattices and any ammonia gas.

5. Nitrites do not accumulate in the soil beyond trace amounts.

6. Fertilizers and other nitrogen additions are applied uniformly and thoroughly mixed with the soil.

7. The microbial populations of different soils are approximately equivalent in their responses to pertinent parameters associated with nitrogen transformations.

8. The upward movement of nitrogen species in the soil during evaporation of moisture is not significant.

9. The chemical composition of the soil (other than nitrogen species) has little effect on nitrogen transformations.

The following assumptions were necessary in order to model the nitrogen transformation and movement beneath an oxidation lagoon.

1. Initial-nitrogen concentrations in the first 10 ft of soil may be approximated by an average of the first 5 ft (NH_4-N, NO_3-N, and organic-N).

2. Initial-nitrogen from 10-60 ft may be approximated by well analysis at 60 ft (organic-nitrogen taken from soil analysis at the lower part of the first 5 ft).

3. The lateral and upward movement of water is negligible.

4. The temperature profile is independent of time.

5. The system remains aerobic at all times.

6. The infiltration rate of water is constant with respect to any one day.

7. Complete mixing occurs in each soil segment with respect to time and space.

8. Soil organic-nitrogen does not move.

9. Exchangeable NH_4-N does not move.

10. Soluble NH_4-N and NO_3-N moves with the water (no hold back volumes).

11. Local infiltration (seepage) rates can be approximated by average rates for the entire lagoon.

12. The NH_4-N and the NO_3-N concentrations of the input (lagoon) water remains constant. (Numbers are an average of several values).

13. The organic-nitrogen entering the soil with the lagoon water is insignificant.

14. (NO_2-N + NO_3-N) $\cong$ NO_3-N at all times.

15. No losses as NH_3 gas.

16. The carbon: nitrogen ratio of the soil's organic matter equals 10.0.

17. No water flow occurred on day one.

Water analyses from the 60 ft PVC well were compared with the results of the computer model (Table 1). A similar comparison of computer values with field values taken from a depth of 5 ft was not possible because of insufficient field data. The computer model predicted values for the complete nitrogen series: however, only the NO_3-N values could be compared. Deviation in the results were apparently due to river seepage which moved through a nearby sanitary landfill and into the 60 ft zone. The model assumed strictly vertical flow and did not account for horizontal flow of water through the soil profile. Furthermore, the model assumes aerobic conditions throughout the soil profile. The lagoon during filling was observed to shift from aerobic to the anaerobic state.

Table 1. Computer Model Prediction of NO_3-N Concentrations at the Water Table

Date	NO_3-N (mg/l) MEASURED	NO_3-N (mg/l) PREDICTED
8-12-71	39.0	40.9
8-19-71	55.0	41.0
8-26-71	55.0	44.4
9-2-71	57.5	45.3
9-15-71	18.5	45.3
9-22-71	32.0	41.4
9-29-71	21.0	35.6
10-13-71	26.0	22.9

Sealing of the lagoon did not occur in the first few months of operation. The combined effect of compaction by heavy equipment moving across the base of the lagoon during construction and the accumulation of sludge during filling did not promote sealing. The total volume of sewage infiltrated during the approximate 3 months of operation was 152 ac-ft. However, to date, no known contamination has occurred in the native groundwater from lagoon seepage in this area (Small, 1973).

ACKNOWLEDGMENTS

The authors are particularly grateful to the Pima County Department of Sanitation for their cooperation and assistance during the construction and operational phases of this project. The work upon which this paper is based was supported by funds from the United States Department of the Interior, Office of Water Resources Research, as authorized under the Water Resources Research Act of 1964.

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PENETRABILITY AND HYDRAULIC CONDUCTIVITY OF DILUTE SULFURIC ACID SOLUTIONS IN SELECTED ARIZONA SOILS[1]

S. Miyamoto, J. Ryan and H. L. Bohn[2]

INTRODUCTION

Sulfuric acid (H_2SO_4) is the common oxidation product of the sulfur liberated in large amounts from smelting, oil and coal burning, and mining processes. In the Southwest, the H_2SO_4 comes mostly from the oxidation of SO_2 produced from copper smelters in the amount of 5 million tons H_2SO_4/year (McKee, 1965). Waters draining from coal and metal mining sites contain H_2SO_4 from the natural oxidation of sulfides and sulfhydryl groups are known as acid mine wash and cause contamination of streams and sometimes soils (Blevins, et al., 1970; Terkeltoub, 1971).

The application of H_2SO_4 to soils and water can be harmful or beneficial for plant growth depending primarily upon application rates, methods and the buffering capacities of the soil and water against acidification. In southern Arizona, soils are generally calcareous and the water contains substantial amounts of bicarbonate. Both are basic buffering constituents against acidification. The proper application of H_2SO_4 to such soils and water is not only safe but also beneficial for plant growth mainly through increasing micronutrient availability (e.g., Ryan, et al., 1973) and decreasing sodium hazard (e.g., Tisdale, 1970).

For deciding better H_2SO_4 application practices, the flow properties of H_2SO_4 solutions in soils must be know. This paper presents the effect of H_2SO_4 concentrations on the solution penetrability, (the rate of solution entry into dry soil, Mustafa, et al., 1970) and the hydraulic conductivity of the solutions in selected Arizona soils.

EXPERIMENTAL

Surface soils collected in southern Arizona were air dried and sieved through a 1 mm screen. Some of their properties are listed in Table 1. The soils, except Mohave, have high carbonate contents as indicated by the acid-titratable basicity (Miyamoto, et al., 1973). The acid-titratable basicity is the measure of readily-reactive basic constituents and is slightly greater than the carbonate content.

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[1]The work supported in part by Rockefeller Foundation Grant 70073 and a grant from the Arizona Mining Association.

[2]Research Associates and Associate Professor in the Department of Soils, Water and Engineering, College of Agriculture, The University of Arizona, Tucson 85721.

Table 1. Some properties of the selected Arizona soils (<1 mm fraction).

Soil Series	Acid-titratable Basicity	Silt and Clay	Saturation Extract Na	Saturation Extract Ca + Mg	Clay Minerals
Neutral	eq/kg	%	ppm	ppm	
Mohave	0.08	29	4	65	Mica, Kaol.
Calcareous					
Elfrida	.54	76	230	1190	Mont.,Mica,Kaol.
Anthony	.72	36	13	313	Mica,Mont.
Perriville	1.04	59	95	1870	Mon.,Mica
Cogswell	1.36	79	42	341	Mica,Mont.
Cave	1.92	53	13	61	Mica,Kaol.
Karro	2.48	57	55	410	Mont.Mica
Sodic					
Stewart	0.99	79	1500	4	Mica,Analcine

The synthetic irrigation water contained 3.1 meq/l total salts ($NaCl$, $CaCl_2$ and $MgCl_2$) with a sodium adsorption ratio of 30. No bicarbonate was present. Streptomycin (50 ppm) was added to suppress microbial growth. The water was not deaerated. The H_2SO_4 concentrations were 0, 500, 1,000, 5,000, 10,000 and 20,000 ppm by weight, having pH values 6.8, 2.2, 2.0, 1.6, 1.4, 1.0 and 0.7, respectively.

The soil samples were packed with a vibrator into a plastic column (7.5 cm ID) to depths of 10 cm and 30 cm. The depth of the solution penetration front was measured with time in 10 cm and 30 cm columns at a constant 2 cm water-head to determine penetrability. The hydraulic conductivity was measured periodically for a week or more in the 10 cm soil column at a constant 10 cm water-head. The differences in duplicate penetrability and conductivity values were approximately within 5 and 15% of the average values, respectively.

Some soil columns were divided into 1 cm sections and analyzed for the changes in acid-titratable basicity after penetration of the 1000 ppm H_2SO_4 solution.

RESULTS

Penetrability. The depth of the solution penetration front is plotted vs. the square root of penetration time for three soils in Fig. 1. This plot gave a good linear relation for all the soils, including the 30 cm column, but did not pass through the origin when extrapolated back to zero time. This positive intercept is probably due to the positive water-head applied. The penetrability values, more strictly the vertical penetrability, is the slope of the lines in Fig. 1. Corrections for the density, viscosity and surface tension of the solutions were neglected because their changes were insignificant under the experimental conditions used here. Since the positive intercepts are less than 0.5 cm, the penetrability value alone should approximate the infiltration under shallow pondings.

The penetrability values are plotted vs. the log of the H_2SO_4 concentration in Fig. 2 for the three types of soils. In the neutral Mohave soil, the penetrability decreased gradually with the concentration, and in the sodic Stewart soil it increased with H_2SO_4 concentration up to 10,000 ppm, then decreased. In the calcareous Anthony soil, little change occurred up to 10,000 ppm. The identical trend was observed for all calcareous soils tested.

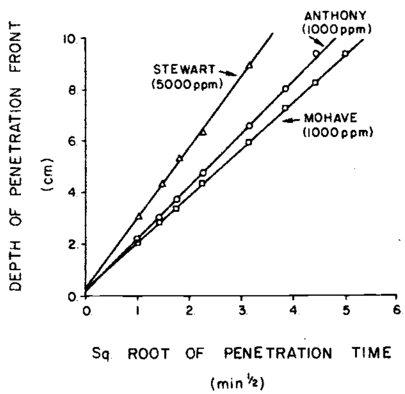

Figure 1. Depth of penetration front vs. square root of penetration time in three selected soils. H_2SO_4 concentrations are shown in parenthesis.

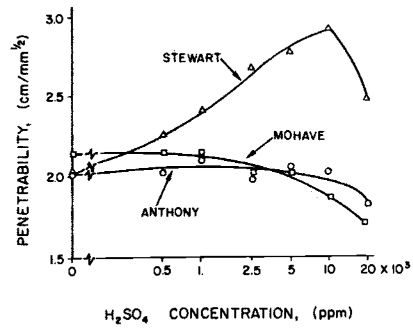

Figure 2. Effect of H_2SO_4 concentration on the penetrability of three types of soils; Mohave (neutral), Anthony (calcareous) and Stewart (sodic).

Hydraulic Conductivity

The conductivity changed with time or the volume of solutions passed. At 0 and 500 ppm solutions, it approached a stable value after 2 to 8 days. Those stable conductivities are listed in Table 2.

Table 2. Penetrability and hydraulic conductivity of selected Arizona soils.

Soil Series	Penetrability Water	Hydraulic Conductivity Water	Hydraulic Conductivity 500 H_2SO_4	Increase	Average Conductivity 500 ppm H_2SO_4
	$cm/min^{1/2}$	cm/hr	cm/hr		cm/hr
Mohave	2.13	0.75	2.40	220	--
Elfrida	1.08	0.05	0.24	380	0.25
Anthony	2.00	1.63	1.82	11	1.80
Perryville	1.50	0.22	0.48	118	0.42
Cogswell	1.46	0.60	0.51	-15	0.55
Cave	2.79	2.10	2.14	19	2.11
Karro	1.91	1.00	1.70	70	1.64
Stewart	2.02	0.42	0.42	0	0.75

In the calcareous soils, the conductivities of 500 ppm solution were significantly larger than those of water only in montmorillonitic ones; Elfrida, Perryville and Karro soils. In the neutral Mohave soil, the significant increase was also observed. In the sodic Stewart soil, the conductivity increased significantly at 1000 ppm. Also presented in Table 2 is the average hydraulic conductivity until the applied H_2SO_4 solution (500 ppm) neutralizes the basicity to a 10 cm soil layer. Those values are approximately the same as the stable conductivities at 500 ppm.

At higher concentrations, the conductivity in all calcareous soils decreased at first, then increased with time. The time of the onset of flow reduction increased with higher acid-titratable basicity of the soil and lower flow rates. The initial reduction was severe at the higher H_2SO_4 concentrations; flow ceased temporarily at 10,000 and 20,000 ppm H_2SO_4, then increased erratically. When flow ceased, the CO_2 was observed as a distinct layer in the soil column. This mechanism is different from air entrapment (Christiansen, 1944). The smoothed data obtained from the Anthony soil are shown in Fig. 3 as an example of all the calcareous soils. The erratic recovery of flow, indicated by the dashed arrows, usually occurred when the amount of added H_2SO_4 equalled the basicity of the soil in the column. At this time, of course, CO_2 production ceased. This equivalence of H_2SO_4 and acid titratable basicity is shown by the saturation point in Fig. 3.

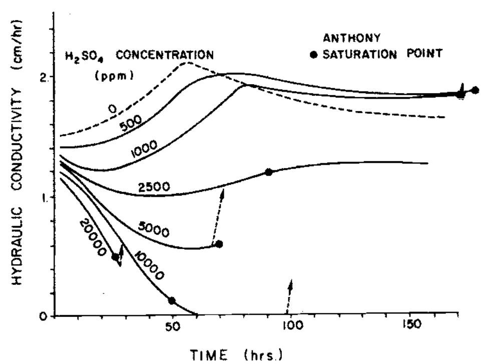

Figure 3. Changes in hydraulic conductivity of H_2SO_4 solutions with the percolation time in a calcareous soil. Saturation point referred to the neutralization of soil's basicity by applied H_2SO_4.

In the sodic Stewart soil, the hydraulic conductivity changed with time similar to the calcareous soils, but the initial reduction was less. The conductivity increased after a small reduction to large values such as over 5 cm/hr when the concentrations were between 1,000 and 10,000 ppm. At 20,000 ppm, the conductivity decreased continuously and approached a smaller value. In the neutral Mohave soil, the conductivity of all solutions did not decrease initially, but increased to stable values which are presented in Figure 4.

For calcareous and sodic soils, the average conductivity of each solution to neutralize the basicity to a depth of 10 cm layer was determined. The results are shown in Fig. 4 for the Anthony and the Stewart soils. The average conductivities of other calcareous soils are presented in Fig. 5 as a relative value to those at 500 ppm. (The average conductivities at 500 ppm are listed in Table 2). The symbols in Fig. 5 are not identified because the relative values are about the same for all calcareous soils tested.

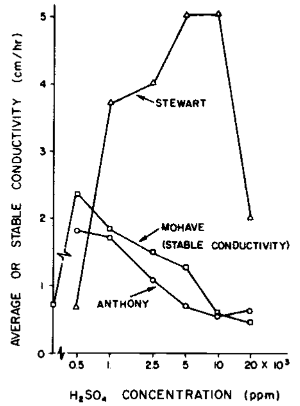

Figure 4. Effect of H_2SO_4 concentration on the average hydraulic conductivity in Stewart and Anthony soils, and on the stable conductivity in Mohave soil.

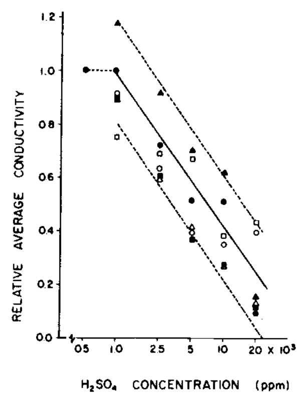

Figure 5. Effect of H_2SO_4 concentration on the average conductivity relative to 500 ppm solution in calcareous soils. Dashed lines are ± 20% variance.

Changes in Acid-titratable Basicity

The changes in the acid-titratable basicity with depth after penetration of the H_2SO_4 solutions (1000 ppm) to the designated soil depth are shown in Fig. 6 for the Anthony and the Cave soils. The H_2SO_4 reacted at the soil surface and lagged behind the solution penetration front, indicating the faster reaction rate than the rate of water entry. As the volume of the solution applied increased, the reacting front moved down. During the penetration of the H_2SO_4 solutions, CO_2 evolved extensively upon the reaction with carbonates in the calcareous and the sodic soils.

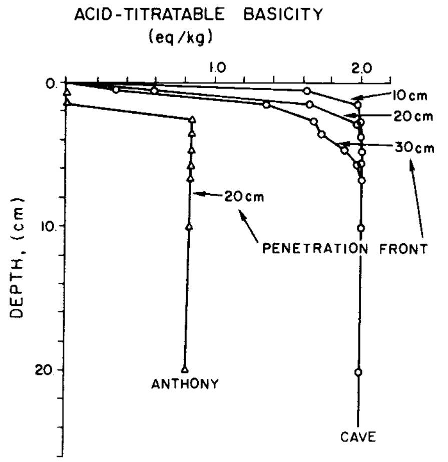

Figure 6. Distribution of acid-titratable basicity after penetration of 10,000 ppm H_2SO_4 solution to the designated depth.

DISCUSSION

The most important findings of these preliminary experiments are probably the severe decrease in hydraulic conductivity in calcareous soils with increasing concentrations over 1000 ppm, and that little change occurred in penetrability. The applied H_2SO_4 reacts with the basicity, mostly carbonates, and generates CO_2. Based on the acid-titratable basicity of the soils, the volume of CO_2 evolved should be about 10 to 50 times the reacted soil volume. The reacted layer during the penetration is limited in a few centimeters in the soil surface (Fig. 6). The CO_2 generated probably escaped through this thin layer without interfering with the penetration. As the volume of H_2SO_4 solution passing through the soil increases, the reacting front moves to depths from which CO_2 may not readily escape. The conductivity, then should decrease with time as shown in Fig. 3. At the peak of CO_2 "build-up", it forms a distinct layered space in the soil even though both sides of the 10 cm soil column are open to the atmosphere. Upon completion of the neutralization reaction, the CO_2 evolution ceases. Then the conductivity increases to some large values due to leaking through channels formed by the escaping CO_2.

This finding suggests that the disposal of H_2SO_4 solutions into calcareous soils will result in plugging by the CO_2 at concentrations of 1 or 2%. Even at lower concentrations, the solution movement will decrease with time if discharged continuously. For the purpose of maintaining good solution movement, the H_2SO_4 concentrations less than 1000 ppm would be safe for disposal (Fig. 5). Approximately 3,500 tons H_2SO_4/hectare/meter (or 3.5×10^6 tons of 1,000 ppm H_2SO_4 soltuion) can be disposed without lowering the conductivity and the pH of leached water for the soils with a basicity of 5 eq/kg. The increase in salt concentration of the leached water, however, requires the special attention. When solutions with higher concentrations of H_2SO_4 are applied to the soil surface for improving micro-nutrient availability in calcareous soils, the effect on solution penetrability is of little significance.

In the sodic Stewart Soil, both penetrability and conductivity increased with increasing H_2SO_4 concentrations up to 10,000 ppm, but decreased at 20,000 ppm. At least two opposing factors are involved; increase in salts, especially divalent cations which increase the conductivity (McNeal, et al., 1966), and increase in the CO_2 evolution which decreases the conductivity. The significant increase in conductivity of the H_2SO_4 solutions over the water indicates the possibility of using dilute H_2SO_4 for reclaiming salt and sodium affected soils. It would be especially effective in decreasing the time required for completing leaching and presumably in replacing sodium. Further studies are needed to determine the amounts, methods of application and subsequent changes in water movement.

The stable hydraulic conductivity of 500 ppm H_2SO_4 solutions was significantly higher than that of water alone in montmorillonitic calcareous soils. Similar results were also reported in some California soils (Mohammed, 1972). The disposal of H_2SO_4, SO_2, and acid mine wash into the irrigation water (Bohn and Westerman, 1971; Terkeltoub, 1971) will promote water movement in such soils. In the Elfrida and Perryville soils which tend to accumulate salts, the improvement of water movement will be especially effective for maintaining the salt balance in the plant root zone. The addition of H_2SO_4 will be more effective in increasing water movement when the irrigation water contains relatively high sodium and bicarbonate yet low total salts (McNeal, et al., 1966). The acidity of water will remain above pH 5 if the application rates of H_2SO_4 do not exceed the bicarbonate of the water content (Bohn and Westerman, 1971). In sodic soils, such a practice will not improve water movement within a short time. Heavy pre-treatments with higher concentrations of H_2SO_4 should accomplish a faster response in improving water movement.

SUMMARY

The penetrability and hydraulic conductivity of dilute H_2SO_4 solutions in selected Arizona soils were measured in preliminary laboratory experiments. In six calcareous soils, the penetrability decreased slightly with increasing H_2SO_4 concentration. The conductivity decreased at concentrations greater than 1000 ppm and flow ceased temporarily at 10,000 and 20,000 ppm. CO_2 evolution appeared to be responsible for the flow reduction and cessation at higher H_2SO_4 concentrations. The stable values of hydraulic conductivity for 500 ppm H_2SO_4 were significantly higher than that for water in montmorillonitic calcareous soils. In a sodic soil, the penetrability and conductivity increased markedly with H_2SO_4 concentration between 1000 to 10,000 ppm. In a neutral soil the penetrability decreased with increasing H_2SO_4 concentration but the stable conductivity for 500 to 5,000 ppm was higher than for water alone.

Acknowledgment. We thank Mr. J. Renthal for laboratory assistance and Dr. D. M. Hendricks for the mineralogical analyses.

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CHEMICAL AND BIOLOGICAL PROBLEMS

IN THE GRAND CANYON

by

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and
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ABSTRACT

A survey of chemical and bacteriological water quality in the Grand Canyon was undertaken to assess possible health hazards to river travelers. The water quality of the main Colorado River channel is relatively stable with only slight increases in ionic concentration and bacteriological load with respect to distance from Lee Ferry and time over the summer season. The tributary streams show extreme temporal variability in chemical water quality and bacteriological contamination as a result of the summer rain and flood patterns in the tributary canyons. These side streams pose a definite health hazard to unwary river travelers. More extensive sampling is called for to determine the sources of this contamination and to protect the quality of the Grand Canyon experience.

INTRODUCTION

In recent years, the recreational use of the Grand Canyon National Park has increased fantastically. It has become necessary for the National Park Service to limit the number of individuals that can travel on the Colorado River through the park in order to avoid deterioration of the area and the experience. One aspect of this preservation effort is to monitor the water quality of the main river channel, tributary streams and springs in which river travelers drink, swim and bathe.

During the summer of 1972, outbreaks of acute gastroenteritis were reported among the boatmen and passengers of the river raft trips in the Grand Canyon. The cause of some of the illness has been identified as Shigella sonnei and it is strongly suspected that this organism was the predominant agent in the outbreak. The mode of transmission of the disease, whether by contaminated food and water sources or person-to-person contact, is still a matter of conjecture.

These events support the conclusions of the authors and others who conducted a series of water quality surveys of the Colorado River in the Grand Canyon during 1971 and 1972 (Slawson and Everett, 1972a,b; Everett et al., 1971). These studies indicate the existence of potential health hazards in this section of the Grand Canyon National Park. This is a report of the 1972 chemical and biological investigations of the Grand Canyon from Lee Ferry to Diamond Creek (see Figure 1).

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The location of sampling stations will be given as river miles with Lee Ferry at mile 0 and Diamond Creek at mile 226. Samples taken in tributary streams were taken upstream from the confluence to eliminate the influence of the river water. Three surveys were conducted during the summer of 1972. In the report, the individual trips will be referred to as the June, July and August surveys. Actual survey days are given below:

Survey	Dates
June	June 13-19, 1972
July	July 25-29, 1972
August	August 28, 1972

Because of the infrequency of the sampling, the occurrence of rapid changes in the character of the system during the summer rainy season, and the technical difficulties of performing bacteriological analysis in the field, it is difficult to make specific conclusions concerning the chemical and bacteriological quality of the main channel and side streams. But it is possible to draw general inferences concerning the general spatial and temporal pattern of these water quality parameters.

CHEMICAL WATER QUALITY

Water chemistry samples were taken in the main river channel and in many of the streams and springs entering the river. Chemistry samples were collected and stored in prewashed one-liter polyethylene bottles. Chemical analysis was performed by the Soil and Water Testing Laboratory of the University of Arizona using the techniques outlined in Standard Methods. The results of this analysis are given in Table 1.

Table 1
Chemical Standards for Drinking Water (WHO, 1963).

Ion	Permissible Level (mg/l)	Excessive Level (mg/l)
Ca	75	200
Mg	50	150
SO_4	200	400
Cl	200	600
pH	7.0-8.5	<6.5 or >9.2

In order to evaluate the chemical water quality in the main river channel, tributary streams, and spring sampled, the concentrations shown in Table 1 were used as the standards for surface waters to be used as a public water supply: these standards have been set by the World Health Organization (1963). The "permissible" concentrations listed indicate levels that are generally acceptable for human consumption. The "excessive" concentrations indicate levels at which the potability of the water has become markedly impaired.

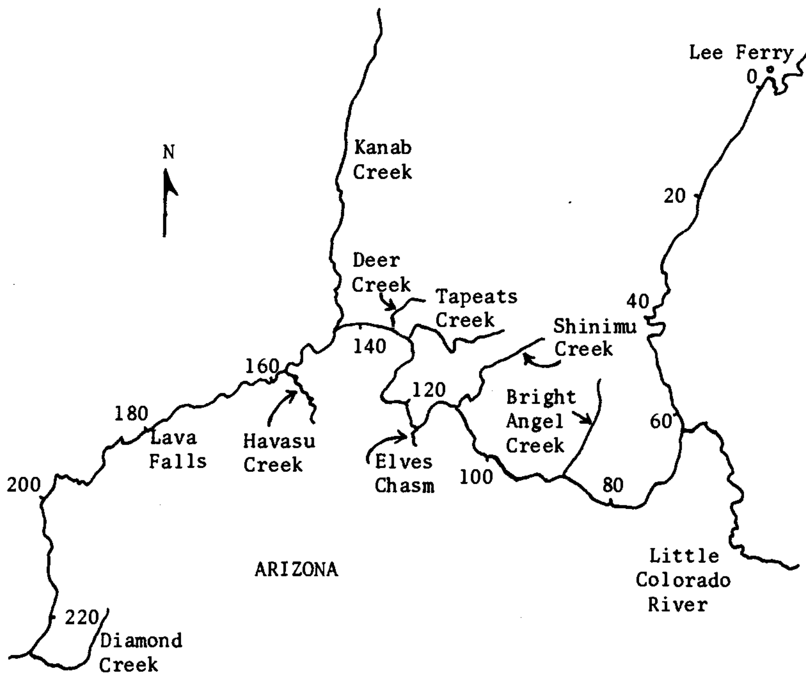

Figure 1. Main Colorado River channel and principle tributaries in the Grand Canyon area between Lee Ferry and Diamond Creek. Numbers indicate river miles from Lee Ferry.

In addition, the following standards are recommended by the U. S. Public Health Service (1962). The level listed for fluoride concentration is for a maximum daily air temperature in the range of 70-90°F.

Table 2
Chemical Standards for Drinking Water (PHS, 1962)

Ion	Recommended Limit (mg/l)
Total Dissolved Solids (TDS)	500
Fl	0.8-1.0
NO_3	2.0-4.0

Table 3 summarizes the results of the chemical analysis of samples taken in the main Colorado River channel. Table 4 summarizes the chemical analysis of tributary stream water.

Table 3
Summary of Chemical Water Quality Surveys of the Main Colorado River Channel. -- Samples taken at tributary streams were taken in main channel above the confluence.

Ion			TDS	Ca	Mg	Cl	SO_4	Fl	NO_3	PO_4
Permissible Level (mg/l)			500	75	50	200	200	1	2-4	-
Location	River Mile	Survey								
Lee Ferry	0	June	585	80	24	16	235	<.2	1.8	-
		July	641	90	28	54	230	.21	2.3	0.0
		August	639	84	27	54	240	.23	2.6	0.02
Mile 41	41	July	654	92	28	54	240	<.2	2.6	0.0
Little Colorado	60	June	572	76	24	16	220	<.2	1.8	-
		August	645	94	24	56	230	<.2	2.4	0.01
Hance Rapid	78	July	629	78	28	28	240	.21	3.2	0.02
Bright Angel Creek	89	August	683	84	27	80	230	.21	2.4	0.07
Elves Chasm	115	August	665	84	27	72	230	.20	2.4	0.05
Deer Creek	136	June	672	74	27	76	240	.20	1.8	-
Kanab Creek	140	August	675	88	26	64	250	.21	2.6	0.04
Lava Falls	180	June	677	78	28	80	235	.20	1.8	-
Diamond Creek	226	June	611	78	26	34	230	.26	2.4	-
		July	655	78	22	64	240	.30	2.3	0.05

Table 4
Comparison of Chemical Analysis of Selected Ions at Lee Ferry and Main Tributaries in Grand Canyon, Summer, 1972.

Location	Survey	TDS	Ca	Mg	Cl	SO_4	NO_3	PO_4
Lee Ferry	June	585	80	24	16	235	1.8	-
	July	641	90	28	54	230	2.3	0.0
	August	639	84	27	54	240	2.6	0.02
Little Colorado River	June	2787	120	80	1214	180	3.8	-
	July	2731	154	41	1060	250	1.5	0.10
	August	1207	104	17	48	200	4.6	0.19
Bright Angel Creek	June	302	38	21	14	18	0.0	-
	July	315	40	21	14	12	0.1	0.01
	August	305	45	24	12	8	0.2	0.0
Elves Chasm	June	521	70	45	28	205	2.2	-
	July	545	76	40	28	200	2.8	0.06
	August	525	98	29	24	200	3.3	0.05
Tapeats Creek	June	247	19	22	12	15	0.2	-
	July	270	40	15	8	10	0.0	0.01
	August	341	59	19	8	40	0.4	0.06
Deer Creek	June	385	42	21	10	90	0.0	-
	July	335	44	22	12	20	0.1	0.01
	August	382	55	22	12	40	1.0	0.16
Kanab Creek	June	1123	176	80	28	680	0.2	-
	July	1142	196	69	40	650	0.5	0.01
	August	1007	228	45	20	570	3.2	0.05
Havasu Creek	June	528	38	44	52	60	0.6	-
	July	500	40	45	56	70	1.0	0.02
	August	500	50	49	52	55	0.8	0.01

An examination of Table 3 shows that the chemical quality in the main river channel is relatively uniform with respect to both time and distance during the summer season. The general spatial pattern is a nearly constant concentration from Lee Ferry (mile 0) to the confluence of the Colorado River and the Little Colorado River (mile 60). At this point, due to the inflow of the Little Colorado River, the salinity (TDS) increases by 50-70 mg/l. The high sodium and chloride concentrations in the Little Colorado River cause this increase. There was also an increase of about 60 mg/l from June to August at Lee Ferry. This reflects the seasonal fluctuations in water quality in the Colorado River. The spatial and temporal water chemistry variations result predominantly from variations in the chloride and sodium ion concentrations.

The chemical quality of the tributary streams (Table 4) is capable of more significant changes throughout the summer season. These changes are due to the cyclic patterns in salinity caused by the periodic storms in the side canyons. These rains have two effects on the water chemistries of the tributary streams. Decreases in the concentration of some ions can be caused by the dilution effect of the rain water. Such changes are usually noted in the concentrations of sodium (Na), chloride (Cl) and total dissolved solids (TDS).

The second effects of the summer rains and resultant floods in the tributary canyons is due to the washing or leaching effect of the rains on the watersheds of the tributary basins. This effect is to increase the concentrations of certain ions by increasing the salt load carried by the streams. Such increases are most notable in the concentrations of nitrate (NO_3) and orthophosphate (PO_4) ions. These concentrations can be related to increases in bacteriological activity which have been noted during periods of flooding in the side streams (Slawson and Everett, 1972a,b,c; Everett et al., 1971). Flooding in the tributary basins would increase the organic matter content of the stream waters. The biological decomposition of this organic matter leads to an increase in microbiological populations and the release of nitrates and phosphates into solution.

The recommended limit for nitrate concentration is related to the danger of infantile methanemoglobinaemia if the water is consumed by infants. Thus, nitrates present no health problem to river travelers in the Grand Canyon. The impact of increases in the nitrate and phosphate concentrations may be important downstream in Lake Mead. Here, this nutrient input, accompanied by increases in water temperature, result in significant and possibly deliterious increases in alge populations.

The recommended salinity (TDS) level of 500 mg/l for drinking water is exceeded at all river locations sampled, as well as several tributary streams. Since waters of the concentrations found in the main channel are commonly used for domestic purposes in the Lower Colorado River Basin, there is probably no health hazard caused by the salinity levels in the main river channel and most of the tributary streams. Extremely high TDS concentrations have been noted in the Little Colorado River, Kanab Creek, and several small streams and spring not listed in Table 4 (Crystal Creek, about 2000 mg/l; Pumpkin Bowl, about 7000 mg/l). These locations pose potential health problems to river travelers who should be warned against drinking from these sources.

In the main river channel, and most of the tributary streams, concentrations of calcium (Ca), magnesium (Mg), chloride (Cl), and sulphate (SO_4) ions are generally below or slightly above the permissible levels listed in Table 1. It is difficult to determine the significance of those samples which showed concentrations slightly above the permissible level because different individuals will show varying sensitivity to these ions. In general, these ions probably present no great danger to river travelers in most of the side streams and in the main river channel within the Grand Canyon. Notable exceptions include the Little Colorado River with respect to chloride, calcium and possibly magnesium ion concentrations; Elves Chasm with respect to calcium concentration; and Kanab Creek with respect to calcium, magnesium, and sulphate ion concentrations. Drinking from these streams should be limited.

Fluoride (Fl) ion concentrations were generally found to be at or below 0.25 mg/l in the main channel and tributary streams. Concentrations in excess of 2.0 mg/l were found in the Pumpkin Bowl spring and concentrations of about 0.9 mg/l were noted in Diamond Creek (mile 226). Because of the limited exposure time, there is probably no danger to river travelers at these two locations.

BACTERIAL ANALYSIS

The presence of coliform bacteria is commonly used to indicate the presence of pollution. Members of the coliform group are associated with the excretia of warm-blooded animals. The possible presence of pathogenic organisms such as those known to cause typhoid fever, cholera, and various types of dysentery, is shown by the presence of these indicator organisms.

There are millions of coliforms for every enteric pathogen, but both types of organisms exist under similar environmental conditions. The isolating and culturing of pathogenic bacteris is expensive, time-consuming, difficult and dangerous. For these reasons, indicator organisms are used. Routine procedures have been developed for the culturing and enumeration of coliform bacteria.

Bacteriological analysis was undertaken using the membrane filtration techniques described by Standard Methods. Bacteria were collected on 0.45 micron membrane filters. Because of the difficulty of culturing fecal coliforms in the field during the June and July surveys, total coliform counts (i.e., including both fecal and soil groups) were made. Measured volumes of water samples were aspirated through the filters which were then placed on blotters containing 2 cubic centimeters of Difco-M-Endo MF media. The media contained a 95% ethanol solution to suppress the growth of non-coliform groups. The plates were then incubated at 35 $\pm$ 1°C for 48 hours.

The August samples were collected, kept cool and driven the same day to Phoenix, Arizona. Here, at the U. S. Public Health Service laboratories, the water samples were filtered as above and placed on specific media used to culture fecal coliforms and fecal streptococci. An effort was also made to detect the presence of Shigella by first placing filtered samples in GN (gram negative) broth to stimulate growth and then steak-plating these samples on differential growth media.

The results of the June and July surveys are summarized in Table 5. It should be noted that only one main channel sample location is listed in this table. As with the chemical quality, the bacteriological quality of the main Colorado River channel in the Grand Canyon is relatively uniform. The several other samples taken in the main channel during the June and July surveys yielded colony counts similar to or less than the Lee Ferry samples.

The desired quality criteria for surface waters to be used for drinking is that there be less than 100 total coliforms (TC) per 100 ml (milliliter) and less than 20 fecal coliforms (FC) per 100 ml. The maximum permissible level is 10,000 TC per 100 ml and 2000 FC per 100 ml (Clark, Viessman and Hammer, 1971). These criteria were issued by the Federal Water Pollution Control Administration (FWPCA) in 1968. The levels were set as the quality of surface waters that could be made potable using available technology. The Arizona water quality standard for primary contact (recreation) is a mean fecal coliform count of less than 200 per 100 ml with 10% of the samples having less than 400 per 100 ml sample. Nevada has set the upper limit at 100 fecal coliforms per 100 ml for recreational use. Using the rule-of-thumb ratio of 20 total coliforms per fecal coliform, the following criteria were

Table 5
Summary of Bacteriological Analysis for June and July Surveys.

Location	River Mile	Survey	Sample Size (ml)	Colony Count or Description
Lee Ferry[1]	0	June	50	TNTC[2]
			25	190
			10	100
		July	10	428
			5	142
Little Colorado River	60	June	25	49
			10	21
Bright Angel Creek	89	June	20	TNTC
			10	Colonies spread by excess water
		July	5	Packed[3]
			10	Packed
Pipe Creek	90	June	10	TNTC
		July	10	Packed
Elves Chasm	118	June	25	Packed
			10	TNTC
		July	10	Packed
Deer Creek	136	June	20	Colonies spread by excess water
			10	120
		July	10	15
			5	36
Kanab Creek	140	June	20	226
			10	118
			5	49
		July	5	Packed
			10	Packed
Havasu Creek	155	June	20	Packed
			10	Packed
			5	Packed
		July	10	Packed
Diamond Creek	226	June	20	TNTC
			10	TNTC

[1] Main Colorado River channel sample; other samples taken in tributary streams.

[2] Too numerous to count.

[3] Colonies overgrowing one another with no clear definition of colonies.

used to evaluate the bacteriological quality (with respect to TC) of the samples taken during the June and July surveys.

Use	Desired Criteria	Maximum Permissible Criteria
Drinking	100/100 ml	10,000/100 ml
Recreational		
Arizona		4,000/100 ml
Nevada		2,000/100 ml

In general, the colony counts were less in the main Colorado River channel than in the side streams. It should be noted that it is in these tributary streams that most of the drinking and recreational activity of river travelers occurs. Thus, these sites are of predominant importance in assessing the health hazards to the river runners.

The bacteriological contamination in the main river channel is normally at or below the criteria for primary contact and is below the maximum permissible level for drinking water. This does not preclude the necessity of treating water taken from the main channel for drinking purposes. It is indicated that proper chlorination or other treatment should make the water quite safe for drinking.

Many of the side streams present quite another picture, at least with respect to primary contact. With the possible exception of Deer Creek, the bacteriological contamination in most of the popular streams and swimming pools is in excess of the levels recommended for primary contact. Because of the failure to obtain countable plates with even 5-ml samples in some cases, it is difficult to make a quantitative evaluation. But the evidence is strong enough to indicate the presence of a very real hazard.

From Table 5 it is also evident that great variations can occur in the bacteriological water quality of the tributary streams and the main river channel within the Grand Canyon. The variations in the side streams appear to be related to summer storm and flooding cycles in the side canyons of the Grand Canyon. The impact of this flooding with respect to water chemistry was discussed earlier. The variations in the main channel may be related to the slight warming of the water observed as the summer progressed.

Because of the sporatic nature of these summer storms and the incomplete knowledge of the effect of these storms on water quality in the tributary streams, it is difficult to quantitatively predict the water quality status of these streams. Qualitatively, it is shown that flooding in the side canyons causes a tremendous increase in the bacterial population of the waters entering the Colorado River. Such contamination poses a definite health hazard to river travelers.

The results of the August bacteriological survey are shown in Table 6. Analysis for fecal coliforms in the main river channel showed much greater variability than had been observed for total coliforms in the earlier surveys. Some of this variability may be due to local effects as the samples were taken near the shoreline in all cases. Of the main channel samples, only the Lee Ferry sample showed a reasonable possibility of human fecal contamination. This possibilty is shown by the fecal coliform (FC) to fecal streptococci (FS) ratio. Ratios less than unity indicate contamination from non-human sources.

Ratios from 2 to 4 indicate human contamination. The FC/FS ratio is not an exact indication of the source of fecal contamination. Therefore, it is necessary to consider all fecal coliform organisms as indicative of dangerous contamination.

Table 6
Fecal Coliform Counts from August Survey.

Location	River Mile	Fecal Coliforms (per 100 ml)	FC/FS Ratio
Lee Ferry[1]	0	20	2
Above Little Colorado River[1]	60	60	0.5
Little Colorado River	60	30	0.2
Bright Angel Creek[2]	89	10	0.1
Bright Angel Creek[3]	89	40	0.1
Above Elves Chasm[1]	118	10	1
Elves Chasm	118	200	0.1
Deer Creek	136	0	0.0
Above Kanab Creek	136	60	1
Kanab Creek	140	1600	4
Havasu	155	50	2

[1]Main Colorado River channel sample; other samples were taken in tributary streams.

[2]Above Phantom Ranch.

[3]Below Phantom Ranch.

As was the case with the total coliform analysis of the June and July surveys, all of the tributary streams samples, except for Deer Creek, showed signs of contamination. The FC/FS ratios indicate the possibility of human fecal contamination in Kanab Creek and in Havasu Creek. The State of Utah requires that Kanab Creek have a <u>total</u> coliform monthly arithmetic mean of 5000 per 100-ml sample "with exceptions." These "exceptions" are ill-defined but apparently August must be one of them.

Another factor which should be mentioned with this bacteriological analysis is sediment. The July and August surveys were conducted after the commencement of the summer rains. Therefore, most of the tributary streams and the main channel below the Little Colorado River were carrying considerable amounts of sediment. Sediment hinders the membrane filter technique by masking the presence of growing colonies and clogging the filters. Sediment in the samples taken by the U. S. Public Health Service (in August) was allowed to settle out to some degree before the samples for analysis were extracted. Obviously there was opportunity for bacteria to settle out with the sediment. The general effect of sediment was to reduce the numbers of colonies observed and any contamination level observed is minimized. Thus, where any fecal contamination was observed there is a great possibility of the actual presence of significantly greater levels of contamination.

The effort to culture and detect Shigella yielded negative results. This fact by itself does not, however, eliminate water as the source of the epidemic. There are several reasons why Shigella may not have been found:

1. The samples may have been taken at a low point in the cyclic pattern of water quality in the side streams.

2. The initial source of contamination may have been transient with the ensuing epidemic being spread by person-to-person contact among river travelers and boatmen.

3. The difficulties in analysis caused by large amounts of sediment in the water samples may have rendered undetectable any Shigella present. Most of the tributary streams were flooding and filled with sediment when the August survey was made.

Even though Shigella organisms could not be isolated in the water samples, Salmonella were found at several sites: in the swimming pool at Elves Chasm, below Deer Creek falls, in Kanab Creek, in the Colorado River below Kanab Creek and Havasu Creek. Salmonella groups B, C, and D were detected. This is further evidence of potentially hazardous fecal pollution within the Grand Canyon.

DISCUSSION

From the data presented in the previous sections, it is evident that the water quality in the Grand Canyon reach of the Colorado River is a function of both time and location. In general, the chemical and bacteriological quality of the main Colorado River channel appears to be within acceptable limits. Potential health hazards to river travelers have been noted in several tributary streams and springs within the canyon. The infrequency of the sampling program followed in these studies makes it virtually impossible to reach quantitative and specific conclusions about the chemical and biological quality of the main channel and side streams. The summer rains can cause rapid and significant changes in water quality. It is evident that a more intense monitoring program is required to assure the accurate characterization of this water quality system and the enumeration of natural and man-caused pollution sources.

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ABSTRACT

CALCITE PRECIPITATION IN LAKE POWELL

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Field and laboratory studies of Lake Powell suggest that precipitation of calcite is the most quantitatively important chemical process that alters water quality as a result of impoundment. Evidence for calcite deposition is based on (1) salt flux data at Lee's Ferry before and after construction of the dam, (2) computer processing of analytical data on water samples which indicates that the surface waters are oversaturated in calcium carbonate, and (3) x-ray diffraction determinations of calcite coating on instruments that were suspended in the lake for a period of months.

Calcite precipitation is believed to be caused by photosynthesis. The Colorado River is somewhat oversaturated in CO_2. This condition persists because heavy loads of suspended sediment severely limit light penetration and photosynthesis. Impoundment in Lake Powell allows the settling of suspended sediment and the resulting clarification of the water favors the development of low but significant densities of phytoplankton in the upper levels of the lake. Photosynthetic withdrawal of carbon dioxide causes an increase in pH and a concomitant precipitation of calcite. Unlike marine waters, oversaturation in calcite need not be extreme in order to initiate nucleation and promote crystallization of calcium carbonate from typical Lake Powell waters.

URBAN HYDROLOGY--STATE OF THE ART[1]

by

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INTRODUCTION

Fast removal of storm water, a luxury in the early nineteenth century, is essential to modern living. Urban drainage systems have grown from primitive ditches and stepping stones to today's complex networks of curbs, gutters and miles of underground conduits. Along with the increasing complexity of these systems has come the need for a more thorough understanding of basic hydrologic processes. Simple rules-of-thumb and crude empirical formulas are inadequate. The approximation of maximum rates of flow to be expected with some relative frequency is no longer sufficient for modern design. Management of urban waters on a day-to-day basis requires continuous time histories. Accordingly, we must represent all key hydrologic processes and unite them in composite models which can determine outputs at points of interest in time and space. In addition, demands by society for better environmental control require that quality be superimposed on estimates of quantity so management of the total water resource can be effected.

SOME HISTORY

Methods used for estimating quantities of storm water runoff from urban drainage areas may be classified as: the rule-of-thumb approach; the macroscopic approach; the microscopic approach; and continuous simulation.

The Rule-of-Thumb Approach

An early statement about urban rainfall-runoff was precipitated by the storm of June 20, 1857 on the Savoy Street sewer in London. One inch of rain fell in 75 minutes producing a maximum flow of 0.34 cubic feet per second per acre. Based upon information then available, the distinguished engineers Bidder, Hawksley and Bazalgette concluded that 0.25 inches of rainfall would contribute about 0.125 inches to the sewer, and 0.40 inches would yield approximately 0.25 inches to the sewer. At this time, a general English rule-of-thumb was that about 50 percent of rainfall would appear as runoff from urban surfaces. These early guidelines were forerunners of modern urban hydrologic models.

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1. Presented at the Joint Annual Meeting of the Arizona Section of the American Water Resources Association and the Hydrology Section of the Arizona Academy of Science, Tucson, Arizona, May 4, 1973.

Macroscopic Approach

Following the early rules-of-thumb, empirical formulae became the principal mechanism for determining amounts of urban runoff. Most second-generation approaches were macroscopic. They are characterized by : (1) consideration of the entire drainage area as a single unit; (2) estimation of flow at only the most downstream point; and (3) the assumption that rainfall is uniformly distributed over the drainage area. The foremost example of this approach is the Rational Method. It was introduced by Emil Kuichling in 1889 and was based on four years of rainfall data using non-recording raingages and one year of runoff data estimated from pairs of white-washed sticks. Five open ditches were used for flow determination.

The Rational Method is described by the statement $Q = ciA$, where Q equals peak runoff rate in cfs, c is a runoff coefficient (the ratio of an instantaneous peak runoff rate and a rainfall rate averaged over a time of concentration), and A is the drainage area in acres. The Rational Method has been used for over half a century with little change in its original form. It is the standard method of urban storm drainage design today. Persistence in the use of this formula can be attributed to its simplicity. The present analytical effort in urban hydrology should bring about some change in design concepts, but new techniques should not sacrifice the practicing profession's desire for easy-to-apply procedures. In fact, sophisticated models might serve a useful purpose in evaluating parameters for simpler procedures which can be applied to routine problems.

A second example of the macroscopic approach is the unit hydrograph method developed by Leroy K. Sherman in 1932. The unit hydrograph is the hydrograph of one inch of runoff from a drainage area produced by a uniform rainfall lasting one unit of time. Once determined, the unit graph can be used to construct the hydrograph for a storm of any magnitude and duration. Until recent times, the unit hydrograph concept was applied mainly to river basins, but it is now being adapted to urban networks as well. The concept of the instantaneous unit hydrograph has been a major factor in this expansion to use in urban hydrology. The instantaneous unit hydrograph operates on an effective precipitation applied in zero time. This is unreal but the assumption makes the hydrograph independent of duration of effective precipitation and eliminates one variable from hydrograph analysis. A number of models using an instantaneous unit hydrograph or an approximation of it have been reported in the literature.2, 11, 13/ One of the pioneers was J.E. Nash.8/

Microscopic Approach

The microscopic approach is characterized by an attempt to quantify all pertinent physical phenomena from the input (rainfall) to the output (runoff).4, 6, 9/ This usually involves the following steps: (1) determine a design storm; (2) deduct losses from the design storm to arrive at an excess rainfall rate; (3) determine the flow to the gutter by overland flow equations; (4) route the gutter flow; (5) route the flow in pipes; and (6) determine the outflow hydrograph. The result obtained is affected by the accuracy of determination of losses and hydraulic phenomena and the validity of simplifying assumptions. If errors are small and non-cumulative, the prediction of runoff is valid.

Tholin's Hydrograph Method (Chicago, 1957) is a good example of the microscopic approach.14/ The procedure is to: (1) develop a design storm pattern from local intensity duration-frequency curves and an average chronological storm pattern; (2) compute overland flow using selected infiltration capacity curves, estimated depth of rainfall retained in surface depressions, and Izzard's overland flow equations; (3) route overland flow through gutters using the storage equation to obtain the runoff into catchbasins; (4) route sewer supply hydrographs from roofs and street inlets along a typical headwater sewer lateral to produce a lateral outflow hydrograph; and (5) route the lateral outflow hydrograph by a time offset method along sub-mains and the main sewer to a point of discharge. Tholin provides charts to facilitate computation.14/

Until recently, most microscopic procedures delt solely with individual storm events. With the advent of modern computers, the trend has been more toward continuous simulation of hydrologic processes.3, 12/

SIMULATION

Crawford and Linsley state that simulation is an indirect approach to the study of the behavior or response of a system. Systems may be simulated by physical models, analog models or digital models. Digital simulation is a more recent method used to analyze large and complex systems. It has the advantage of high speed and does not require extensive hardware often needed for physical or analog models. The digital program itself becomes the model, and its parameters can be changed to allow for experimentation or to represent any particular condition of interest. Simulation of the hydrologic system through use of models has many virtues. The model can be much more easily operated and observed than the real system. Another important advantage is that it is possible to compress real time scales on the order of years to time scales on the order of minutes. As a result, long periods of time can be successfully studied.

Digital simulation expresses physical systems in mathematical terms which involve various parameters. These mathematical models are improved and verified by simulating the systems reaction to known inputs and outputs, continuing until the model is considered to be a reliable representation of the prototype. The procedure is analogous to that of verifying a physical model. Once this has been accomplished, the model can be used for a variety of purposes including project planning and operation. Seasonal effects, impact of land management practices, and many other situations of practical interest can be modeled.

Simulation Using Equations of Gradually Varied Unsteady Flow

Equations for gradually varied unsteady flow in open channels have been used to describe relationships between rainfall and area, depth, velocity and rate of flow of surface runoff since about 1960.10,15/ Conservation of mass and momentum of surface runoff at any point in space and time are accounted for. Because the equations are complex, numerical methods are resorted to for solutions.

The first descriptive equation is a continuity equation written as:10/

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q$$

This equation is derived by considering the water entering and leaving an infinitesimal section of channel (Figure 1). The term $\partial A/\partial t$ accounts for the change in storage with time in the infinitesimal section; the term $\partial Q/\partial x$ accounts for the difference between the outflow and the inflow to the infinitesimal section; and the term q is the lateral inflow in cfs/ft along the channel.

The second equation (the momentum equation) refers to the dynamic behavior of the flow. It is written as:

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial H}{\partial x} + g(S_f - S_o) + \frac{Vq}{A} = 0$$

and derived by considering all of the forces acting on a fluid element. Neglecting the first and last terms, the remaining terms are commonly used to compute backwater profiles for steady flow in reservoirs and stream channels. The first term, $\partial V/\partial t$, accounts for the local acceleration of the fluid. The convective terms, $V\frac{\partial V}{\partial x}$ and $g\frac{\partial H}{\partial x}$, relate to changes in kinetic and potential energy respectively. The terms gS_f and gS_o account for friction along the channel and the component of gravitational force in the direction of flow, respectively. The last term, qV/A, accounts for momentum that must be imparted to the lateral inflow by the water flowing in the channel.

The most important term in the dynamic equation is the friction term S_f. Since its magnitude is usually larger than any of the other terms except S_o, the method of evaluating S_f is important. A uniform flow formula is generally used to determine S_f. Where flows are laminar, the Darcy-Weisbach formula is suitable.

$$S_f = \frac{f}{H}\frac{V^2}{2g}$$

Manning's equation is used for turbulent flows.

$$S_f = \left(\frac{nV}{1.486\ r^{2/3}}\right)^2$$

It is generally assumed that the value of S_f occurring for gradually varied unsteady flow anywhere in a drainage basin is the same as the value of S_f at the same velocity and hydraulic radius for uniform steady flow at that point. The error introduced by this assumption is probably less than the error involved in selecting proper values for the coefficients in the foregoing equations for S_f.10/

Assumptions used in deriving the equations of flow normally include: (1) Accelerations normal to the direction of flow have been neglected. (2) Velocities normal to the direction of flow have been neglected. (3) Velocities are assumed uniform throughout a section normal to the direction of flow. This assumption is more restrictive than necessary since coefficients can be introduced into the dynamic equation to account for a variable velocity profile. (4) Frictional resistance is assumed to be the same as for steady uniform flow at the same velocity and depth of flow. The importance of these assumptions has been thoroughly studied.3, 10, 15/

The equations of flow written in finite form become

$$\frac{\Delta A}{\Delta t} + \frac{\Delta Q}{\Delta x} = q$$

$$\frac{\Delta V}{\Delta t} + \frac{\Delta V}{\Delta x} + g\frac{\Delta H}{\Delta x} + g(S_f - S_o) + \frac{qV}{A} = 0$$

To be computationally useful, the solution of the finite difference equations should approach the solution of the partial differential equations as $\Delta x \rightarrow 0$ and $\Delta t \rightarrow 0$. The strategy followed is to divide the channel into a number of intervals of length, Δx, and then solve the difference equations for $A(x,t + \Delta t)$ and $V(x,t + \Delta t)$ at successive intervals of time. Examples of the application of such equations to modeling urban runoff are well documented.10,15/

Stanford Watershed Model

This model, or modifications of it, has had more applications in recent years than other deterministic ones because of its versatility. Basically, the model simulates snow pack and soil profile processes and calculates continuous soil moisture, evapotranspiration, groundwater accretion and inflow to stream channels. It assembles and routes channel inflows through the channel network and reservoirs. The model includes all major hydrologic processes and considers the surfacewater-groundwater linkage. The model must be calibrated to each watershed from existing records of rainfall and streamflow. Input parameters required are rainfall, potential evapotranspiration and physical descriptors of the watershed and hydrologic properties. If snow melt is involved, additional input parameters are required.

Three zones of moisture regulate soil moisture profiles and groundwater conditions. The rapid response encountered in smaller watersheds is accounted for in an upper zone, while upper and lower zones control such factors as overland flow, infiltration and groundwater storage. A lower zone is responsible for long-term infiltration and groundwater storage which later becomes base flow. Computed stream flow includes components of overland flow, groundwater flow and interflow. The model has been successfully applied to urban drainage systems.3/

The Stanford Model routes moisture entering the watershed through various storage categories until it becomes stream flow, evaportanspiration or subsurface outflow. The path followed by incoming moisture is determined by antecedent moisture storage, assigned parameter values, entry rate and season. A system of empirical and theoretical equations describes specific hydrologic processes and estimates rates and volumes of moisture movement through the various storages. Major storage categories are surface (upper zone including interception and depression), soil moisture (lower zone), groundwater, interflow, overland flow (divided between impervious surfaces and soil surfaces) and channel. Moisture movement between storages occurs through infiltration, deep percolation, overland flow and channel flow processes. The model routes channel inflow from the point it enters a tributary channel to the location where a hydrograph is required. A subroutine is available for handling snow.

To apply the model to flows from a given watershed, a trial and error process is used to assign values to watershed parameters. The synthesized and recorded flows are compared by inspecting pairs of synthesized and recorded numerical values and hydrographs. Parameters are then reviewed and adjusted if necessary to bring synthesized flows closer to recorded flows. Another run is made and the cycle repeated until a satisfactory verification results.

Quantity-Quality Models

The problem of urban water quality has been neglected.1,15/ Few useful data are available and most of these are not adequate for continuous simulation. Variations in urban runoff quality result from changes in season and geographical area. Practices such as lawn watering also affect the ultimate destination of lawn fertilizers and other chemicals.16, 17, 18/

While a better understanding of the complex nature of physical, chemical and biological processes which affect urban water quality is needed for structuring urban water quality models, progress is being made. Initial efforts have been few and spotty but some examples exist.

EPA Storm Water Management Model

The EPA Storm Water Management Model was developed by Metcalf and Eddy, Inc., the University of Florida and Water Resources Engineers, Inc.7/ The objective was to produce a combined sewer system model which would simulate the quantity and quality of storm water flows from urbanized areas. The program consists of a control segment and five computational blocks: (1) Executive; (2) Runoff; (3) Transport; (4) Storage; and (5) Receiving Water.

The Executive block is the control for computational blocks with all access and transfers between the other blocks routed through its subroutine MAIN. The Runoff block accepts rainfall data and a description of the drainage system as inputs and provides both hydrographs and time-dependent pollutional graphs (pollutographs). The Transport block routes flows which are corrected for both dry weather flow and infiltration through the distribution system. It is also capable of producing hydrographs and pollutographs at specified points. In the Storage segment, flows from the Transport block are received and the effect of any treatment provided is considered. Effects of the discharge on a receiving stream are determined in the Receiving Water block. Segments may be run separately to facilitate adjustments.

The drainage area is divided into subcatchments, gutters and pipes. Subcatchments are divided into three parts: pervious, impervious with surface detention, and impervious without surface detention. Subcatchments are defined by area, width, slope and ground cover, while gutters and pipes are described by slope, length and Manning's roughness coefficient. This information and rainfall data in the form of hyetographs are read into the Runoff block. A step-by-step computation of runoff volume is then initiated. The mathematical procedure is as follows:

(1) The water depth on the subcatchment is found from the hyetograph as follows:

$$D_1 = D_t + R_r \Delta t$$

Where D_1 = Water depth after rainfall

D_t = Water depth at time t

R_t = Intentisty of rain, time interval Δt

(2) Horton's exponential function is used to account for infiltration. This is:

$$I_t = f_o + (f_i - f_o)\ e^{-at}$$

where I_t = Infiltration

f_o, f_i and a = Hortin's coefficients

(3) Infiltration is subtracted from water depth according to the following equation:

$$D_2 = D_1 - I_t\ \Delta t$$

where D_2 = Depth after infiltration

I_t = Infiltration rate at time t

(4) The water depth D_2 is compared with a specified detention value D_d and, if found greater, the outflow for the catchment is found by using a form of Manning's equation.

$$V = 1.49/n\ (D_2 - D_d)^{2/3}\ s^{\ 1/2}$$

where V = Velocity
n = Manning's coefficient
D_d = Detention requirement

$$Q_w = VW\ (D_2 - D_d)$$

where s = Ground slope
W = Width of area
Q_w = Outflow

(5) Water depths on the subcatchments are computed by the continuity equation:

$$D_{t + \Delta t} = D_2 - (Q_w/A)\Delta t$$

where A = Surface area of subcatchment

(6) The preceding steps are continued for all subcatchments.

(7) Gutter inflow is found by adding the outflow of the subcatchments tributary to it and the flow from all upstream gutters.

$$Q_{in} = \Sigma Q_{w,o} + \Sigma Q_{g,i}$$

where $\Sigma Q_{w,i}$ = Sum of flow from subcatchments
$\Sigma Q_{g,i}$ = Sum of flow from upstream gutters

(8) Depth of flow in gutters is calculated as follows:

$$Y_1 = Y_t + (Q_{in}/A_s)\Delta t$$

where Y_1, Y_t = Water depths in gutter
A_s = Mean water surface area between Y_1 and Y_t

(9) Outflow from the gutters is computed from Manning's Equation:

$$V = 1.49/n \ (R)^{2/3} \ (S_i)^{1/2}$$

where R = Hydraulic radius
S_i = Invert slope

$$Q_g = VA_c$$

where A_c = Cross-sectional area at Y_1

(10) Water depth in gutters is found using the continuity equation:

$$Y_t + \Delta t = Y_1 + (Q_{in} - Q_g) \ \Delta t/A_s$$

where all symbols are as defined previously.

(11) Gutter computations are carried out for all gutters in the system and summed to yield runoff.

Input to the runoff quality model consists of flow hydrographs develoepd in the quantity model. Output takes the form of pollutographs for each pollutant modeled. Pollutographs and hydrographs are introduced into the Transport block where they are summed and modified by addition of dry weather flow and infiltration to produce final outfall characteristics. At present, due to lack of sufficient data, only a few water quality parameters can be modeled.

It is assumed that the amount of pollutant which can be removed during a rainfall is dependent on storm duration and initial quantity of pollutant. This can be modeled by a first-order differential equation of the form:

$$-dp/dt = dP$$

or

$$P_o - P = P_o(1-e_{-kt})$$

where P_o = Pollutant originally on ground, lbs.
P = Pollutant after time t, lbs.
k = A constant

The value of k is assumed to be directly proportional to the rate of runoff. Therefore, k = br, where b is a constant and r is the runoff intensity. Based upon available data, a value of 4.6 is assigned b. For each time step, the runoff rate is determined from the hydrograph and a value of P, which becomes the new value of P_o for the next step, is calculated.

Cincinnnati Urban Runoff Model

The Cincinnati Urban Runoff Model (CURM) was developed by the Division of Water Resources, Department of Civil Engineering, University of Cincinnati.5/ The program consists of three sections:

(1) MAIN-infiltration and depression storage and two subroutines; (2) GUTFL-gutter flow; and (3) PIROU-pipe routing. It is similar to the EPA model and divides the drainage basin into subcatchments whose flows are routed overland into gutters and sewer pipes. Rainfall is read in as a hyetograph. Infiltration and depression storage are summed and subtracted from rainfall to give overland flow. This is routed through the gutter system to storm water inlets and the pipe network. Starting at the upstream inlet, flows are calculated in successive segments of the sewer system, including discharges from inlets, to produce the total outflow.

The drainage area is divided into small subcatchments with closely matched characteristics. Rainfall data are introduced and infiltration is computed for each subcatchment. Principal elements of the modeling process follow.

(1) It is assumed that runoff begins when the rainfall rate equals infiltration rate and the mass of precipitation balances infiltration. The equations representing these conditions are:

$$t = \frac{-1}{k}\ \ln((i(I) + x/DT(i(I+i)-i(I))-f_c)/f_o-f_c)$$

and

$$\frac{f_c}{60}\, t + \frac{f_o - f_c}{60k}\ (1-e^{-Kt}) = mi(I)+(i(I) + \frac{x}{2DT}\ (i(I+1) - i(I)))\frac{x}{60}$$

where mi(I) = Mass precipitated until time t, in.
i(I) = Ordinates of rainfall intensity curve
K = Decay rate of infiltration (units/min.)
f_o = Initial infiltration capacity, in./hr.
f_c = Ultimate infiltration capacity
DT = Time increment of rainfall intensity curve
t = Time to intersection of rainfall curve and infiltration curve
x = Increment of DT

The infiltration curve is computed from the equations and t, I and x are stored.

(2) Surface retention is related to depression storage by an equation derived by Linsley, Kohler and Paulhus:

$$s = (i-f)\ e^{\frac{-P-F}{S_d}}$$

where S_d = Total depression storage, inches
P = Accumulated rainfall in storage, inches
F = Accumulated infiltration, inches
i = Rainfall intensity, inches/hour
f = Infiltration, inches/hour
s = Surface retention, inches/hour

Infiltration and surface retention are subtracted from rainfall intensity to yield runoff.

(3) The hydrograph of overland flow is derived by solving

$$\frac{r_1 + r_2}{2} - \frac{q_1}{2} + 60\ D_1/t = \frac{510.35}{n1}\ s^{1/2}\ D_2{}^{5/3}\ (1+.6(D_1/D_{2e})^3)5/3 + 60D_2/t$$

where $D_e = (.0097n^{0.6}r^{0.6}L^{0.6})/s^{0.3}$

$D_{1,2}$ = Depresseion storage at beginning and end of time interval t, inches/unit area

r_1, r_2 = Overland flow supply at beginning and end of time interval, t, cfs/min.

n = Manning's coefficient

L = Length of overland flow

s = Slope, ft/ft

q = Discharge, in./hr./unit area

(4) For the initial time increment, values of $q_1 = 0$ and $D_1 = 0$ are substituted, D_2 is calculated, and q_2 is found from:

$$q = 1020.7/nL\ (s^{1/2})D^{5/3}(1 + .6(D/D_e)^3)^{5/3}$$

where the symbols are as previously defined.

The determined values of D_2 and q_2 become new values of D_1 and q_1. The overland flow hydrograph is derived by repeating this cycle.

(5) Gutter flow is computed using the continuity equation:

$$\frac{\partial q}{\partial x} + \frac{\partial y}{\partial t}\ T = q_2$$

where T is the width of the water surface.
The term $\frac{\partial y}{\partial t}$ T is neglected because change in depth of gutter flow is very small with respect to time. After integration, the equation becomes:

$$Q = q_L(L) + Q_o$$

where Q_o = Upstream gutter contributions

L = Length of gutter, feet

q_L = Overland flow from hydrograph

Q = Flow from gutter system

Inlet flows are routed through the pipe network by delaying the hydrographs by the flow time required to reach the next inlet and summing values at a terminal point in the network. Manning's equation is used to find the velocity of flow in the sewer and the corresponding time delay. Provision is made for sewers of varying cross section. The model has been found to closely simulate gaged flows.5/

Calculation of pollutant concentrations in runoff is based on the assumption that the rate of pollutant removal depends on the amount of pollutant initially in the drainage area and the rainfall intensity. The equation used is:

$$\frac{dP}{dt} = KqP$$

or $$\frac{-dP}{P} = Kq\,dt$$

After integration, this becomes

$$P = P_o e^{-KV_t}$$

where P = amount of pollutant remaining on the ground at time t, lbs.
P_o = Initial amount of pollutant
K = Constant
V_t = Volume of runoff up to time t, ft^3

During a storm, the amount of pollutant washed into the sewer system in a time interval t, is:

$$P_1 - P_2 = P_o(e^{-KV_1} - e^{-KV_2})$$

The amount of solids washed into a storm sewer during a rainstorm is assumed proportional to the square of the runoff intensity. This equation is as follows:

$$r = \lambda q^2$$

where r = Fraction of solids carried off
λ = Proportionality factor
q = Runoff intensity, inches/hour

The amount of solids brought into the system in a time interval may be expressed as:

$$P_1 - P_2 = P_o\ \lambda\bar{q}^2\ (e^{-KV_1} - e^{-KV_2})$$

where $\bar{q}$ = Mean runoff intensity
V_1, V_2 = Runoff volumes at time t_1, t_2, ft^3

Soluble pollutants are routed downstream at the same velocity as the flow, and are summed in the same manner as flows to determine final values. Provision is made for sediment transport.

DATA NEEDS

A good deal is known about analyzing the hydrology of natural basins. Much of this knowledge should be transferrable to urban watersheds but more data are needed to prove this point. The present data base is inadequate. Mathematical modeling requires data for calibration. Rainfall-runoff-quality data are needed on a variety of urban watersheds exhibiting a range of climatic and watershed parameters.

Several fully instrumented urban watersheds should be developed to provide data of research quality. These data should be on a time and space grid at least as small as that required for finite differencing solutions to hydrologic problems.15/ Enough data should be obtained to provide for split-sample testing. Perhaps the most serious data deficiency is in the area of urban water quality. Fully instrumented research watersheds should include an extensive quality sampling program designed to provide insight into time and space variations in quality loadings for the major constituents considered important in urban runoff. Of special concern is the variability of quality loadings in response to environmental and climatic variations, seasonal effects and the effects of flushing by antecedent rainfall.

SUMMARY

The state-of-the-art in modeling urban runoff quantity and quality has progressed from simple rules-of-thumb to complex simulation models incorporating all fundamental hydrologic processes. We are not totally aware of the mechanics of all of these processes and some empiricism remains. In fact, urban hydrologic modeling is still part art and part science, but as more data become available and we become better equipped to evaluate the tools being developed, a greater degree of sophistication and reliability will be the product. Perhaps the greatest underlying need is for more data characterizing the water cycle on urban areas. Information on both the quantity and quality of urban runoff is in critically short supply.

EFFECT OF URBANIZATION ON RUNOFF FROM SMALL WATERSHEDS

by

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ABSTRACT: Hydrologic data collected from three small urban watersheds and one rural watershed were analyzed for the purpose of investigating the effect of urbanization on runoff. A procedure developed by the Soil Conservation Service was used to explain the relationship between the amount of rainfall and runoff. It was noted that the runoff curve number, a parameter of the method, increased as the percentage of impervious area increased. Also, there was evidence that a linear relationship existed between the runoff volume and its corresponding peak rate.

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EFFECT OF URBANIZATION ON RUNOFF
FROM SMALL WATERSHEDS

INTRODUCTION

Most of the cities in the United States have been expanded laterally during the past two decades. As urban development takes place, the response of a watershed to the rainfall input is departing from its natural condition. Usually, the infiltration is reduced sharply due to the increase of impervious areas. As a result, runoff becomes less dependent on evapotranspiration and infiltration into the soil. In most areas, however, the quantified information about the change of runoff volume and peak rate due to urbanization have not been investigated yet. As the trend toward more intensive urbanization is expected to continue through the remainder of this century, the need for a better knowledge of the effects of urbanization on runoff has become more urgent than ever before. In an effort to fill the above void, hydrologic data collected from three small urban watersheds and one rural watershed have been analyzed for the purpose of investigating the effect of urbanization on rainfall-runoff relationships.

In general, there are two distinct periods of precipitation in southern Arizona, a summer rainy season beginning from July through September and a winter rainy season extending from December through the middle of March. Rainfall-runoff relationships in those two periods are so different that they should be considered separately. The runoff data collected for this study are strictly associated with summer convective storms. These storms occur as high intensity, short duration, widely scattered thundershowers. Severe local flooding, which causes heavy damage and may include loss of life, is often the result of such storms. This paper describes a procedure for determining the hydrologic effects of urbanization of small semiarid watersheds.

EXPERIMENTAL WATERSHEDS

As shown in Figure 1, all three urban watersheds are located in the city of Tucson, Arizona. The rural watershed is located about five miles southwest of Tucson. Each watershed has one or two unlined channels passing through and collecting runoff from the land surface and/or streets. As determined from aerial photographs, the land use on the urban watersheds varies from residential to commercial and industrial. The photographs have been made to a scale of one inch equivalent to 400 feet. The description of the characteristics of the watersheds is summarized in Table 1. The term "impervious area" refers to the sum of paved streets and parking lots, institutions, industrial and commercial areas, and unpaved but considerably compacted alleys.

The average rainfall on the watersheds was computed by means of the Thiessen method. The runoff, on the other had, was measured at each outlet by a critical-depth flume, developed and rated by U.S. Water Conservation Laboratory, Tempe, Arizona.

A storm event was defined as one which yielded an amount of mean areal rainfall equal to or greater than a specific threshold value. Huff (1967) has defined a threshold value as 0.50 inch. In our analysis, however, as available data were very limited, all events in which the mean rainfall exceeded 0.25 inch were included in the analysis.

RELATING RUNOFF VOLUME TO RAINFALL

The plots of the data from all watersheds suggest that the procedure developed by the Soil Conservation Service (SCS) might be appropriate for the study of runoff (U.S. Soil Conservation Service, 1972). This method is used extensively for agricultural watersheds with a size from a few acres to several thousand acres. The basic equation of the method is

$$Q = \frac{(P - kS)^2}{[\ P + (1-k)\ S\ ]} \tag{1}$$

where Q is the runoff in inches, P is the mean rainfall in inches, S is the potential infiltration in inches, and k is a coefficient. In the SCS procedure, the initial abstractions prior to the beginning of runoff are generally estimated to be 0.20 S. In other words, the coefficient k is equal to 0.20. However, since the initial abstractions in an urban or cleared surface area should be less than that on agricultural land, a sensitivity analysis was conducted. The results indicate that the estimation error is at a minimum for a k value of 0.15. A runoff curve number or hydrologic soil-cover complex number is a transformation of S and is defined as

$$N = 1000/(S + 10) \tag{2}$$

Substituting Eq.(2) into Eq.(1),

$$Q = \frac{\left[\ P - \dfrac{1.5\ (100-N)}{N}\ \right]^2}{P + \dfrac{8.5\ (100-N)}{N}} \tag{3}$$

Therefore, the only estimate required for the model is the runoff curve number.

The estimated runoff curve numbers for the different watersheds were obtained by a regression analysis which related rainfall to runoff through equation 3. The results are tabulated in Table 2 and are also shown in Figure 2. As expected, the watershed with larger impervious areas has a higher runoff curve number. It is significant to note that the SCS procedure explained at least 75 percent of the variance in all cases. As shown in Figure 3, the plot of percentage of impervious area versus curve number indicates that the curve number increases rapidly, as the impervious area increases from zero to twenty percent. After passing this range, the relationship between the percentage of impervious area and the curve number approaches one that is linear.

Quite often, the result of a direct plotting of rainfall against runoff for individual storms may be a wide scatter of points. Hydrologists usually attempt to introduce a third variable, such as antecedent soil moisture in order to explain the departures from the simple relation. Miller and Viessman (1972) classified antecedent soil moisture conditions into three categories, dry, average and wet, respectively. The curve numbers are adjusted according to the antecedent soil moisture conditions. As the other factors remain constant, the curve number for wet antecedent moisture condition is greater than that for dry condition, as illustrated in Figure 4. Nevertheless, many authors discovered that antecedent soil moisture gave little improvement on the rainfall-runoff relation for the convective storms over the southwest watersheds (Keppel, 1965; Schreiber and Kincaid, 1967; Fogel, 1969: Osborn and Lane, 1969). In addition, Feddes et al (1970) obtained the same conclusion for the urban watersheds in Texas. Our results have indicated that none of those third variables was useful to improve the relationship between rainfall and runoff. It is not to say that the antecedent soil moisture has no effect on runoff but rather that the likelihood of two or more storms occurring on the same area within a short time span is remote.

VOLUME-PEAK RELATIONSHIPS

The volume-peak relationships for three urban watersheds have been examined. The plots of the data (Figure 5) indicate that the peak rate of the flow is linearly proportional to its total runoff volume. Thus, a linear regression model with one dependent variable was used to describe the relationship between the peak rate and its corresponding volume. The prediction equation is

$$q_p = a + b\ Q \tag{4}$$

where q_p is the peak rate in cfs and a and b are constants. With R^2 values in excess of 90 percent, the principle of linearity appears to be sustained. The results, however, do not appear to be consistent in that the watershed with the highest percentage of impervious area produced the lowest peak rate for a given runoff volume. Possible explanations are that street patterns may have an overriding effect on the peak rates and that the variability of storm durations were not considered. The estimated coefficients are listed in Table 3. Further research will include the investigation of the effect of the pattern of urbanization on the relationship between runoff peaks and rainfall intensities.

CONCLUSIONS

1. The Soil Conservation Service procedure can be used to relate convective storm rainfall to the volume of runoff from small, semiarid watersheds.

2. The SCS method appears to be sufficiently sensitive to determine the effect of urbanization on the volume of runoff.

3. The relationship between the peak flow rate and the corresponding total runoff volume from small urban watersheds is linear.

4. Urban watersheds with the highest percentage of impervious areas do not necessarily produce the highest peak rates for a given volume of runoff.

ACKNOWLEDGMENTS

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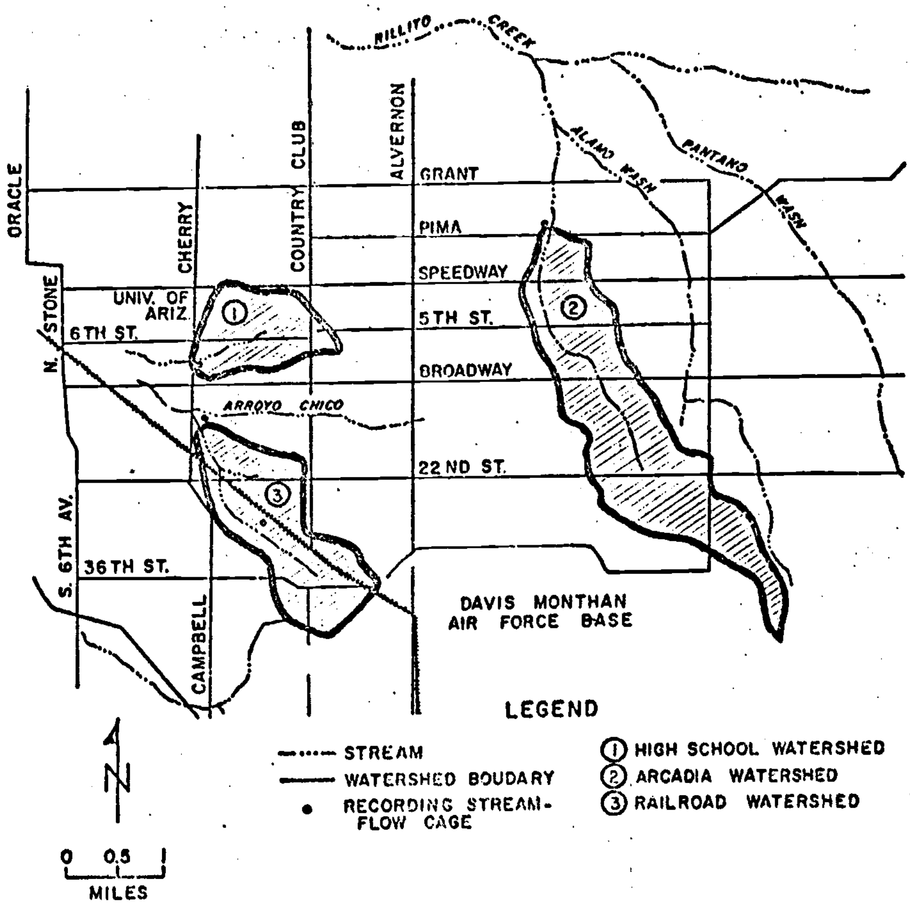

Figure 1. Urban hydrology experimental watersheds.

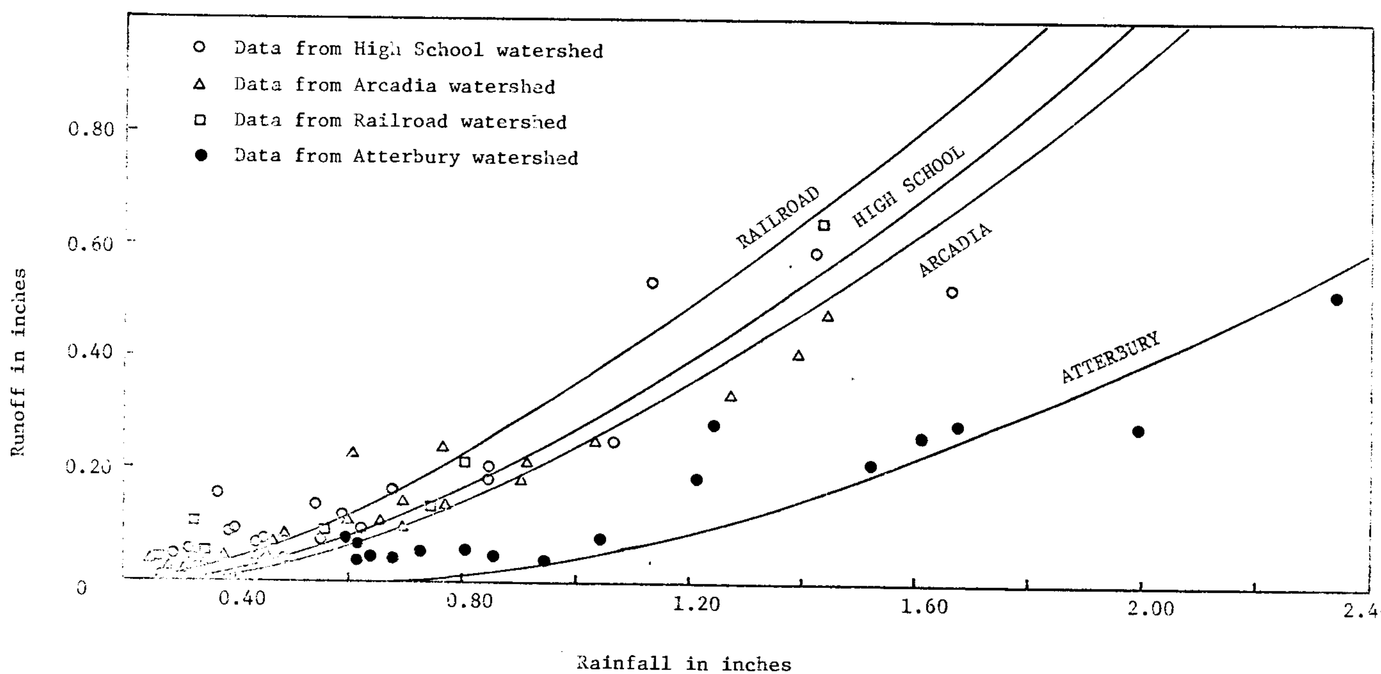

Figure 2. Rainfall-runoff relationships for small urban and rural watersheds

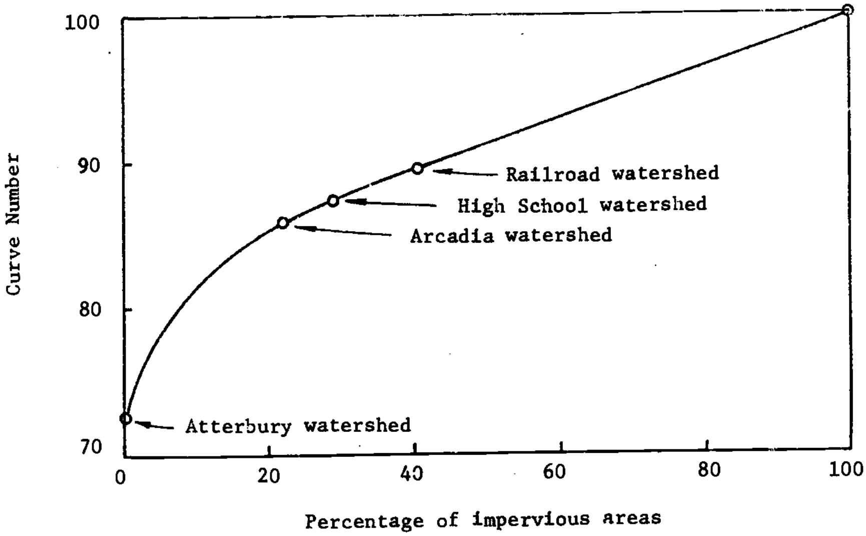

Figure 3. Effect of urbanization on curve number

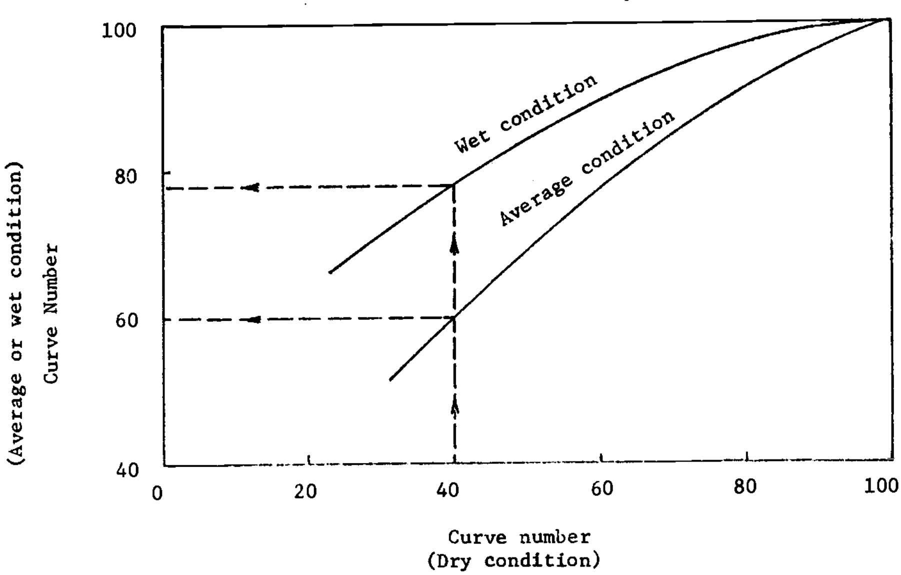

Figure 4. Adjustment of curve number for antecedent moisture conditions

Table 1. DESCRIPTION OF WATERSHED

Watersheds	High School	Arcadia	Railroad	Atterbury
Area, square miles	0.90	3.50	1.90	0.45
No. of raingages				
(recording)	4	6	4	2
(non-recording)	2	3	3	2
Residential area, %	65.5	60.4	38.7	0.0
Commercial area, %	3.5	6.1	1.5	0.0
Industrial area, %	0.0	0.0	26.3	0.0
Impervious area, %	28.8	21.9	40.3	0.0

Table 2. THE ESTIMATED CURVE NUMBERS FOR DIFFERENT WATERSHEDS

Watersheds	High School	Arcadia	Railroad	Atterbury
Estimated curve number	87.4	85.9	89.7	72.7
% of variance explained by the model	76.3	84.1	95.6	76.2
Number of storm events analyzed	20	27	8	18

Table 3. VARIABLES DETERMINING FLOW VOLUMES AND PEAK RATES

Watersheds	High School	Arcadia	Railroad
Coeff. a	-23.10	-31.99	24.26
Coeff. b	996.08	2494.13	926.61
% of error explained	0.94	0.93	0.98

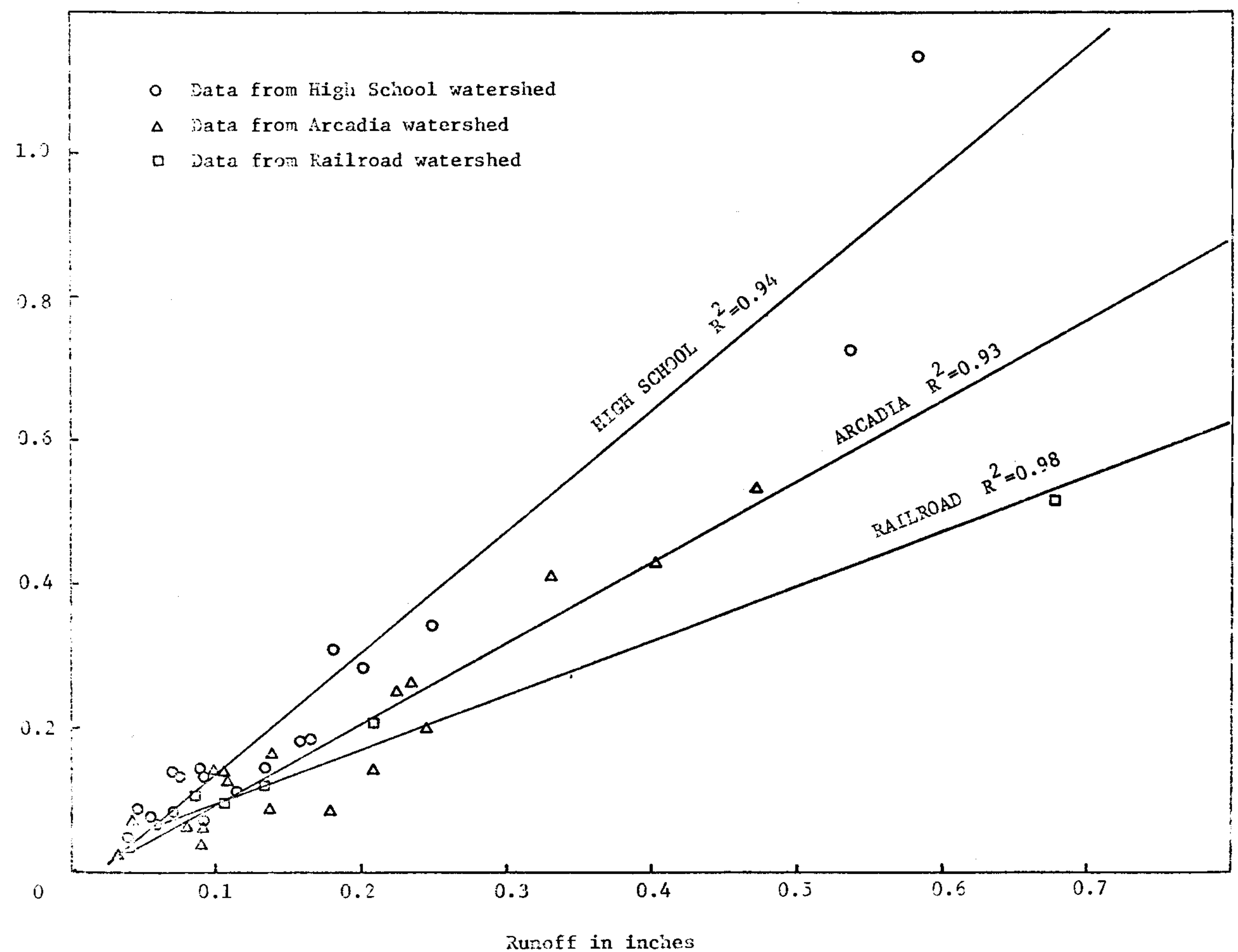

Figure 5. Volume - peak relationships

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LAND TREATMENT FOR URBAN WASTE WATER MANAGEMENT

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ABSTRACT

The increased demands of the public for better water quality in streams and lakes, and the establishment of national water quality goals for 1983 and 1985 by Congress points towards an innovative approach to treatment of municipal sewage and industrial waste water. One of the most promising advanced wastewater treatment methods is land treatment by irrigation.

As we become more aware of the resource limitation in our environment, we must consider ways of conserving our natural resources.

Land treatment best meets the environmental planning principles related to recycling and wise use of our resources.

LAND TREATMENT FOR URBAN WASTE WATER MANAGEMENT

A broad concensus that the quality of the Nation's waters must be improved found expression in the Federal Water pollution Control Act Amendments of 1972. The role of the urban water resources manager and hydrologist in this water quality improvement effort will be a significant one.

The basic intent of the Act is contained in a series of national goals and policies. Three key statements from the Act are:

1. The discharge of pollutants into the navigable waters will be eliminated by 1985.
2. Wherever attainable, water quality which provides for the protection and propagation of fish, shellfish, and wildlife and provides for recreation in and on the water will be achieved by July 1, 1983.
3. The discharge of toxic pollutants in toxic amounts will be prohibited.

The national policy on pollution control includes federal financial assistance for publicly owned waste treatment works and encourages waste treatment planning on an area-wide basis. It emphasized the need for research and demonstration efforts to develop the necessary technology to eliminate the discharge of pollutants. Public participation in the fight against pollution is to be encouraged and assisted.

The Water Pollution Control Act Amendments of 1972, under Title II, states that encouragement should be given to waste treatment management plans and practices which include reclaiming and recycling of water and confined disposal of pollutants so that they will not migrate, and also to that treatment which results in the construction of revenue producing facilities, including:

a. The recycling of potential sewage pollutants through the production of agricultural products,

b. The confined and contained disposal of pollutants not reclaimed,

c. The reclamation of waste water, and

d. The ultimate disposal of sludge in a manner that will not result in environmental hazards.

Furthermore, waste treatment management which integrates sewage treatment facilities and recycling with other urban subsystems, shall be encouraged, such as solid waste, waste heat and thermal discharges. Revenues produced are to aid in financing other environmental improvement programs. The law goes on to state that waste treatment management should incorporate open space and recreation considerations. In regard to the Great Basin, the "Clean Water Bill" (1) states that one or more projects should be undertaken to demonstrate new methods and techniques for the elimination or control of pollution within the watersheds of the Great Lakes, and that the Chief of Engineers is directed to design and develop a demonstration wastewater management program for the rehabilitation and environmental repair of Lake Erie. The program must set forth alternative systems for managing wastewater on a regional basis including land treatment disposal systems with "aerated treatment-spray irrigation technology," provisions for the disposal of solid wastes, and advanced wastewater treatment technology. The law states that wastewater treatment management should encourage construction of revenue-producing facilities through the use of sewage effluent recycling to agricultural lands; the storage of pollutants not recycled; the reclamation of wastewater; and an environmental sound method of handling sewage sludge. Wastewater treatment management should take advantage of opportunities related to sewage treatment and sewage recycling for industrial and municipal wastes, including waste heat and thermal discharges, and open space and recreational possibilities.

The Water Pollution Control Act Amendments of 1972 provide for effluent limitations in Title III - Standards and Enforcement:

" (B) In order to carry out the objective of this act there shall be achieved --

"(1) (A) not later than July 1, 1977, effluent limitations for point sources, other than publicly owned treatment works, which shall require the application of the best practicable control technology currently available . . . and

"(B) for publicly owned treatment works in existence on July 1, 1977, . . . effluent limitations based upon secondary treatment . . . or

"(C) not later than July 1, 1977, any more stringent limitations, including those necessary to meet water quality standards, treatment standards, or schedules of compliance established pursuant to any State law or regulation . . .

"(2) (A) not later than July 1, 1983, effluent limitations for categories and classes of point sources, other than publicly owned treatment works, which shall require application of the best available technology economically achievable which will result in reasonable further progress toward the national goal of eliminating the discharge of all pollutants . . . and

"(B) not later than July 1, 1983, compliance by all publicly owned treatment works . . ."

There is a clear mandate from Congress to eliminate the discharge of pollutants into the Nation's waterways. Far-reaching goals and policies relative to pollution control are now the law of the land.

LAND TREATMENT - A VIABLE ALTERNATIVE

The application of secondarily treated sewage effluent to agricultural or open space lands, using irrigation engineering methods and techniques, is known as land treatment of sewage effluent (2, 3, 4, 5, 6). Land treatment provides for reclaiming and recycling of certain pollutants in water, and the confined disposal of other pollutants not recycled. The land treatment technology provides the opportunity for revenue producing facilities through the production of agricultural goods. For instance, a Corps of Engineers study has shown that the readily identifiable equivalent crop fertilizer 1973 market value of the nutrients in the Cleveland-Akron area will be approximately $10,000,000 per year for the year 2020 projected loadings.

Planning for land treatment alternative systems should incorporate the opportunity for waste heat dissipation from power plants on a revenue producing basis. Open space and recreational considerations should be integrated into the waste water management planning, and recharge of ground water aquifers should be undertaken where practical.

In essence, land treatment is the application of controlled amounts of secondarily treated wastewater to the land surface for final treatment of water and beneficial reuse of many of the pollutants. The treated water is then recovered for discharge to a natural body of water or recycled to town and industries.

The application of a secondary sewage plant effluent to agricultural or open space lands will result in a highly treated effluent which will meet the 1985 goals and objectives of the Water Pollution Control Amendments of 1972.

WASTEWATER TREATMENT TECHNOLOGY

Wastewater treatment depends upon two factors: the type of effluent desired and type of process of technology used to achieve it.

There are, generally, speaking, three types of effluent -- primary, secondary and advanced treatment effluent. Primary treatment removes approximately 40 percent of the pollutants from wastewater by a settling process. Under secondary treatment, water is treated by a bacteriological process, such as trickling filters, activated sludge, or aerated lagoons and settling. this is generally the highest level of treatment currently provided in the United States and typically removes 80 to 85 percent of the pollutants. The third level of treatment is advanced treatment, which can remove nearly all of the pollutants (99 percent) from wastewater, concentrating on nutrients which have not been removed by primary and/or secondary treatment. However, some advanced wastewater treatment plants are designed to only do a partial job of removing pollutants, there being many levels of efficiency.

The technologies of advanced treatment can be classified as advanced biological, physical-chemical, and land treatment. Biological treatment digests the organic pollutants in wastewater by bacteria, and chemical action coupled with settling completes the process. Physical-chemical technology functions by removing pollutants by chemical action, gravity settling. The land treatment technology utilizes biological processes to achieve secondary treatment; the effluent is then applied to agricultural or open space land where the water receives final treatment by flowing through the "living filter" of the soil. Diagrams of the three types of advanced treatment are given in Figure 1.

ADVANCED BIOLOGICAL TREATMENT

Paralleling nature, the advanced biological treatment process provides an environment for the growth of biological organisms as in the secondary treatment process. These organisms use the organic matter contained in wastewater as a source of food and are supplied with sufficient oxygen to complete the digestive process. Settling is then required to separate solids and bacteria from the water.

Removal of nutrients, taste and odor, and remaining suspended solids is the aim of the advanced treatment. Nitrogen and phosphorus are nutrients found in wastewater, which are harmful to bodies of water by causing accelerated eutrophication. Nitrifying bacteria are used to convert ammonia-nitrogen to a gaseous form which escapes into the atmosphere. Phosphorous is removed through biological uptakes, chemicals and filtration processes. As shown in Figure 1, the treatment process includes filtration to remove fine particles.

This treatment produces solid by-products (sludge) suitable for agricultural applications. Properly controlled, sludge is a valuable resource for its fertilizer (nitrogen, phosphorous) use.

Operation and maintenace costs are lower than those incurred by physical-chemical treatment but higher than those for land treatment. It requires a relatively high capital investment. Load fluctuations reduce treatment effectiveness and industrial spills can easily upset the biological processes. Plant operation and maintenance is farily complex.

PHYSICAL-CHEMICAL TREATMENT

Physical-chemical treatment evolved as a sophisticated method to remove complex wastewater constitutents, but deviates from the processes we think of as "biological" or "natural". There is still a reliance upon physical separation of solids and water, althought chemicals are used to stimulate and greatly enhance the efficiency of separation. In addition, the use of chemical treatment processes removes organic and inorganic material ordinarily unaffected by biological secondary treatment. Components may be added to remove specific pollutants.

This technology usually requires a fairly small land area and lower capital costs than biological or land treatment. It can withstand load fluctuations well and is flexible for growth and operation. Certain chemicals can be reclaimed; however, incineration of the sludge is required for this process. Large chemical supply needs and high operation and maintenance costs, coupled with complex plant operation, make the physical-chemical plant less attractive. The beneficial use of the soil by-products may also be questionable, depending upon local soil conditions.

LAND TREATMENT

This method of wastewater treatment utilizes the natural processes of the earth's soil zone and growing crops. Wastewater first undergoes secondary biological treatment in either an aerated lagoon or conventional activated sludge plant; the secondary effluent is then applied to agricultural or other suitable land by any of several irrigation techniques.

The soil biota, the filtration and chemical exchange capacities of the soil and its living organisms are collectively referred to as the "living filter". As the wastewater passes through the soil, the organic matter, bacteria and plant nutrients are removed. Within a few feet of the soil surface the water is clear, being similar to water obtained from wells.

The effect of land treatment is to recycle to the environment elements such as nitrogen and phosphorous discarded as pollutants by man but readily utilized by nature as nutrients.

The removal and storage capacity of the soil for heavy metals is great enough that a site can usually be used for hundreds of years, thus making it less important for industry to remove its metals in-house prior to secondary treatment. Land treatment provides a high degree of water purification including effective virus removal (7), (8).

Operation and maintenance costs are generally lower than those incurred with advanced biological or physical-chemical plants, although power demands are higher. Areas of land are committed to open space agriculture for long periods of time, winter storage reservoirs are required for the secondary effluent, present agricultural practices may have to be altered, and land may not always be readily available near the cities. Aerated lagoons require larger land areas and higher power use than would conventional activated sludge plants.

The overall land treatment system has eight basic components:*

1. A collection system, which may be expanded to serve growing areas;
2. Biological treatment cells or conventional secondary treatment plants;
3. Transmission pipeline or tunnel to convey the water to storage;
4. Storage reservoir to provide for winter carry-over and storage during rainy weather;
5. Irrigation system similar to those used by farmers, and agricultural or open-space land;
6. Soil zone consisting of the upper one or two feet of the earth;
7. Underground drainage system which collects the percolating water from beneath the soil zone and transmits it to a natural or artificial channel;
8. Sludge disposal land and facilities which apply the settled solids to land at predetermined rates to increase the nutrient content of the soil and to provide a soil conditioner.

* These components vary from system to system. For instance, in warm climates the winter reservoir storage is not necessary, and in some soils, artificial drainage is not required.

The key component of a land treatment system is the "living filter", and the proper selection of the soil association to be utilized, coupled with the proper design of the irrigation and farm management system to provide the desired results.

LAND TREATMENT SOIL CRITERIA

The final soil selection criteria are based upon testing the overall soil renovative capacity and character against specific objective and subjective measuring constraints, including:

1. Reasonably high and suitably safe hydraulic capacity, as related to specific irrigation application and management methods, or more than 50 inches per year.

2. A high degree of heavy metal removal and storage capacity approaching 100 percent.

3. Nitrogen removal, with reuse, temporary storage, and atmospheric loss potential without significant leaching, of between 80 and 99 percent.

4. Phosphorous and calcium removal, with reuse or storage potential without significant leaching, approaching 99 percent.

5. Suitability for economically reasonable artificial and natural drainage control.

The three basic techniques now used in the United States for land treatment of wastewater are rapid infiltration, spray irrigation, and overland runoff. The first requires very permeable soil and subsurface conditions, the second requires moderate permeabilities with few constrains on subsurface geology, and the third technique operates best with very tight and impermeable soils. The techniques are defined as follows:

> Spray irrigation is the controlled spraying of liquid onto the land, with the flow path being infiltration and percolation within the boundaries of the site. Overland runoff is the controlled discharge of liquid onto the land, with the flow path being downslope sheet flow. Rapid infiltration is the controlled discharge, by spreading or other means, of liquid onto the land, at a rate measured in feet per week, with the flow path being high-rate infiltration and percolation. (2)

These techniques provide for a range of soil characteristics suitable for effluent treatment which includes very tight soils with almost no permeability or infiltration capacity, to extremely coarse soils such as one would find in sand and gravel pit areas. As the type of soil criteria varies, the land treatment management and farming practices should significantly vary.

It follows that an almost infinite number of variations could be developed to suit soils falling between the coarse and fine-grained soil extremes. A soil selection philosophy should be used which is based on tailoring of farm management techniques to best suit specific and unique soils.

EXISTING LAND TREATMENT SYSTEMS

The tailoring of farm management techniques to suit specific soils available is best illustrated by reviewing some of the 1300 or more land treatment installations in the United States.

The Paris, Texas, installation of the Campbell Soup Company best illustrates the adoption of a unique land treatment farm management technique to soils which ordinarily would never be considered for irrigation purposes. At Paris, Texas, the company uses the overland runoff method, which was discovered in 1954 by the Campbell Soup Company at Napoleon, Ohio, just north of Columbus. (4) At Paris Reed Canarygrass is grown on soil having an infiltration capacity of 0.1 inches per day. Sewage effluent containing 550 to 900 milligrams per liter of BOD, discharged by the plant at a rate of 3.6 million gallons per day, is applied at a unit rate of one mgd per 139 acres. The hydraulic loadings at Paris is based upon a design rate of 3.5 inches of irrigation water per week during the summer and about one-half that much during the winter; however, as it turned out, the actual applications substantially exceeded the design rate. This land treatment system has been in operation for ten years and has been performing with a very high degree of efficiency with 99 percent reduction in BOD, total color removal, and up to 90 percent reduction in nitrogen and phosphorous.

The wastewater renovation at low cost for industry to standards meeting or approaching those goals of the 1972 Clean Water Bill has been demonstrated by the Campbell Soup Company at Paris; however, the crop value is of particular significance to the agricultural fraternity. The attached Table 1 provides a summary of nutritional and chemical analyses of hay samples from the Paris, Texas, site. The fodder analysis data for crops irrigated with sewage effluent should be compared with nutritional analyses of selected animal feeds grown with ordinary irrigation water as presented in Table 2. The following is a quote from the Campbell Soup Company's wastewater renovation report:

> "It is a well-established agricultural fact that when an excess of nutrient elements are present in the soil, plants will uptake in greater quantity than normal. In wastewater disposal, this phenominon is of special significance in that those plant nutrients removed in the hay are totally unavailable for algae growth in the receiving stream. The phosphorus thus removed amounts to ten pounds of P per ton of hay, or 8,000 pounds removed in a single cutting of 32,000 bales in Paris in 1968 (24,000 lbs. of PO_4).

TABLE 1

NUTRITIONAL AND CHEMICAL ANALYSIS
OF
REED CANARY HAY SAMPLES

Harvest Date	Sample No.	Growth Units	Protein %	Fiber %	TDN* %	Phos. %	Nits. %	Dry Matter %
5/ 2/68	s- 1	1122	21.6	24.4	61.7	0.5	3.7	89.8
5/ 9/68	s- 2	1300	14.7	28.7	54.1	0.6	2.4	89.5
5/16/68	s- 3	1549	16.6	27.4	57.3	0.5	2.7	90.6
5/23/68	s- 4	1721	19.8	29.3	55.7	0.6	3.1	90.2
7/ 1/68	s- 6	3508	22.2	27.7	61.6	0.4	3.6	93.4
7/22/68	s- 9	4615	16.4	30.3	56.6	0.4	2.6	93.5
8/12/68	s-12	5786	16.3	27.6	59.6	0.3	2.6	93.6
9/ 3/68	s-15	6844	18.1	27.0	60.5	0.5	2.9	93.1
9/23/68	s-18	7505	18.8	28.3	59.0	0.6	3.1	92.7
10/ 7/68	s-19	7845	22.0	25.0	64.2	0.5	3.6	93.2
10/28/68	s-22	8270	23.0	24.2	65.5	0.6	3.7	93.2

*Total Digestible Nutrients

TABLE 2

NUTRITIONAL ANALYSES OF SELECTED ANIMAL FEEDS

Dry Roughage	Protein %	Fiber %	TDN* %	Phos. %	Nit. %	Dry Matter %
Alfalfa hay	14.7	29.0	50.3	.21	2.35	90.4
Bermuda grass hay	7.3	25.6	43.0	.20	1.17	90.7
Clover hay	12.0	27.1	49.0	.23	1.92	89.0
Johnson grass hay	6.5	20.4	50.3	.26	1.04	90.1
Prairie hay	5.7	30.3	49.2	.10	.91	90.4
Reed canary hay	7.5	29.1	46.6	.23	1.20	90.8
Timothy hay	6.2	30.1	46.9	.16	.99	88.7

*Total Digestible Nutrients

Studies indicate that Reed canary hay can equal or exceed the nutritional value of alfalfa and clover hay, and feeding tests indicate a superior potential market value when compared with other good quality hay.

In the table above (Table 1) the fodder analysis data from the Paris, Texas, land treatment area are presented for comparison with information in Table 2.

Even more impressive is the uptake of nitrogen which averages 3.1 percent of dry weight of the Reed Canary hay or 62 lbs. per ton -- nearly 50,000 lbs. of N removed in the single harvest of 1968".

Probably the best known land treatment installation demonstrating rapid infiltration is the Flushing Meadows operation near Phoenix, Arizona, which discharges 235 million gallons per year into six basins, each basin having a bottom area of less than two acres. The annual application rate amounts to 400 AF per year.

At Flushing Meadows the system operates with depths of six inches and one foot of standing water, the soil having approximately three feet of fine loamy sand underlain by coarse sand and gravel to a depth of 240 feet. The loamy sand soil is estimated to contain approximately 85 percent sand, ten percent silt and five percent clay. They hydraulic conductivity of the loamy sand was originally four feet per day and of the underlying materials 28 feet per day horizontally and 17.6 feet per day vertically. The static ground water table is generally a depth of about ten feet. The water isapplied on a year round basis, the optimum cycle being two weeks of inundation followed by a ten-day drying period and a 20-day drying period for the summer and winter respectively. Wastewater renovation using the rapid infiltration method is remarkably high (3).

One of the most well known rapid infiltration sites is at the Seabrook Farms in New Jersey. The Whittier Narrows plant near Los Angeles is a typical example for ground water recharge where disinfected sanitary effluent is applied. The Whittier Narrows effluent from a 15 million gallon per day activated sludge plant is chlorinated and pumped to the infiltration site for ground water recharge.

The most talked about and most highly monitored land treatment system is at Muskegon, Michigan, which is sized for 1990 design flows of approximately 43 million gallons per day. The 10,000-acre land treatment site includes five aerated lagoons, a three square mile winter storage reservoir, drainage wells, drainage canals and drain tiles. This system, operated by the County Public Works Department, is expected to widen the economic base of Muskegon County, expanding the primarily industrial economy to include an agriculturally oriented base. Investigations are presently under way concerning the utilization of a three square mile reservoir as a colling pond for an electrical power plant. This system at Muskegon was put into initial operation in April of 1973.

The City of Melbourne Australia, having a population of about 1.8 million and with heavy industry has been using the land treatment system for treatment of sewage effluent since about 1894 (8). At Melbourne the Public Works Department which operates the "Sewage Farm" is extremely pleased with the lush vegetation, the high quality hay and pasture, and their record in the market place in the production of high quality beef and lamb. At Melbourne the land treatment system is a combination of overland runoff and soil infiltration.

The economics of land treatment are self evident. With over 1300 installations in the United States, many of which are operated by private industry who are interested in economy and dependability, the systems generally are lower cost than competing alternatives for those particular locations and loadings. However, studies of land treatment systems for major metropolitan areas such as Chicago and Cleveland indicate that overall costs are less for the land treatment systems than advanced wastewater treatment systems using advanced biological or physical-chemical treatment, to meet equivalent 1985 anticipated effluent standards. Similar conclusions were reached at Muskegon, Michigan, where the economy of land treatment was matched against present state effluent standards which includes phosphorous and nitrogen removal.

ENVIRONMENTAL ASPECTS

The use of the land surface and the living filter for final treatment of sewage effluent via agricultural engineering techniques whereby the effluent is used as irrigation water has an inherent strong appeal to the environmentally concerned citizen. The environmental community in the United States, which has gained wide-spread support since 1969, generally views the land treatment method for wastewater management as an approach which meets the three environmental planning principles:

1. The environment is a single system.
2. The environment is a closed system.
3. Pollutants are resources out of place.

In the "Documentary of Revolution in a Chinese Village" William Hinton, in describing a Shansi Privince village, states that "the crops grew only on what was put into the soil each year . . . This need to conserve every kind of waste and return it to the land was responsible for the tidy appearance of the streets and courtyards." (9)

Concerned citizens in the United States recognize the problems civilization faces with geometric growth and geometric consumption of our natural resources, as so clearly described in "Limits to Growth," (10)

Typical metropolitan area wastewater can be expected to have the following pounds of macro-nutrients discharged each year for every one million population:

Macro-nutrient	Pounds of Nutrient per Year per One Million Population
Nitrogen	6,000,000
Phosphorous	3,000,000
Potassium	7,000,000

The matter of heavy metals in wastewater is represented by significant washes of contaminants into receiving waters during rain storms. As an example, the heavy metals washed into receiving waters during a 0.5-inch per hour rainstorm for an urban area of one million people are shown below: (11)

STORM RUNOFF FLUSHING OF HEAVY METALS

Element	Runoff Load in Pounds per Hour
Zinc	2,600
Copper	800
Lead	2,300
Nickel	200
Mercury	290

Heavy metals are also a serious question in municipal/industrial wastewater as represented by the following estimate of current discharges from the Cleveland and Akron area into Lake Erie:

DISCHARGE OF HEAVY METALS INTO LAKE ERIE

Heavy Metal	Pounds per Year
Silver	3,600
Copper	656,000
Chromium	1,050,000
Cadmium	24,000
Nickel	650,000
Lead	20,000
Zinc	813,000
Iron	5,000,000

Current wastewater treatment practices do not specifically treat for heavy metal removal, though perhaps 60 percent can be collected in the sludge in the secondary treatment plants. A unique aspect of land treatment, however, is that the soil has a very high removal efficiency for heavy metals, approaching 100 percent, and the capability to accumulate these heavy metals in the soil zone for hundreds of years before toxic concentrations are reached. Deep plowing, selective removal by industry, recycling, selection of crops, and rotation of irrigated lands provide ample opportunity for management of heavy metals with land treatment.

Corn grain crops in the corn belt having annual yields of 100 bushels per acre with current farming practices can be expected to increase to 200 bushels per acre upon receiving nutrient-laden sewage effluent with a well managed operation, including artificial drainage.

Reed Canarygrass has been shown to be a desirable and good cattle feed after receiving irrigation with sewage effluent. The protein and total digestible nutrients of such fodder has been shown to exceed that of alfalfa hay.

The irrigation of open space and agricultural land near urban areas with sludge effluent provides a commitment to open space for a long period of time. In many communities across the country this is a welcome feature as one recognizes the scattered urbanization of our rural lands surrounding our cities.

In the northern climates where winter storage of sewage effluent is desirable, or often required, the large surface area of the reservoirs is well suited for thermal power heat dissipation, such as that required for nuclear power plants. A reservoir surrounded by several miles of irrigated farm land or open space under control of the local government for sewage treatment often provides an ideal site for a nuclear power plant.

Land treatment of sewage effluent provides certain opportunities to the sanitary engineer, water resources manager, and municipal administrator to take a major step forward in making our cities more compatible with the land and to reverse the every increasing trend towards wasteful consumption of our natural resources.

SUMMARY

The insistence by the people that the quality of the nation's water must be improved and that we must stop using our rivers and lakes as final disposal points and for their assimilative capacity was expressed by Congress in the Clean Water Bill of 1972. The goals and policies which were articulated in this bill are environmentally sound and represent a major step towards recognizing the physical limits of growth and resource consumption. The review and analysis of sewage treatment practices and technology in the United States today points towards the fact that a rethinking of our approach to wastewater management is essential.

The more than 1300 land treatment installations in this country today, many operated by private industry, show that land treatment is a viable alternative to conventional technology using advanced biological and physical-chemical treatment.

The environmentally concerned citizen has been found to respond well to the basic concept of recycling sewage effluent back to the land for the production of crops, using the pollutants as resources, turning waste into wealth, and meeting long range goals for clean water. This concept represents a major opportnity for our society to reorient itself from a consumptive society to one that recycles its resources.

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A JEEP-MOUNTED RAINFALL SIMULATING INFILTROMETER

William R. Henkle[1]

Runoff studies done at Northern Arizona University have shown the need for a portable infiltration and runoff measuring device. An instrument designed by R.O. Meeuwig of the U. S. Forest Service was examined and an extensive literature review was made to determine the most suitable instrument for use in these studies. Subsequently a hybrid device incorporating features of ring-type flooding-infiltrometers, portable rainfall-simulating infiltrometers and a laboratory device used by V. T. Chow was designed.

REQUIRED DESIGN CHARACTERISTICS

Runoff and infiltration studies are usually made to predict runoff resulting from natural storms. Predictions are more reliable when the instruments used closely simulate natural rainfall and runoff conditions. Portable instruments can neither exactly duplicate storm events nor can they sample watershed areas in sufficient detail to give highly reliable runoff and infiltration data. Runoff and infiltration devices must be designed to duplicate natural events as nearly as possible.

PORTABILITY

The instrument and an adequate quantity of water for a measurement test must be sufficiently portable to be moved in the field to various test sites. Many instruments have been designed with portability as an important factor. The most portable instruments may be hand carried to reach isolated sites within a study area. Maximizing the portability aspects of the instrument requires that the amount of water and the size of the instrument carried be small. These practical considerations severely limit the size of the test plot and the duration of the test. Less portable instruments sample larger plots and give a better statistical representation of the watershed, however water requirements make them expensive to operate. These devices are generally limited to gentle terrain due to thier size.

WATER DROP CHARACTERISTICS

Infiltrometers which simulate natural storms must produce artifical rainfall which is similar in intensity, drop size, and drop kinetic energy at ground surface to natural events. Rainfall intensity in inches per hour is usually designed to approximate intensities of near maximum storms. In the Flagstaff region storms having an intensity of two to six inches per hour are of chief interest. The rainfall simulating device must be designed to produce rainfall in the range required in the study.

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The size of the drop produced by the simulator should approximate the size of natural rain drops. Because of the relationship between drop size and terminal velocity, drop size is usually modified to give an appropriate terminal velocity. This modification usually causes the drop size produced by the simulator to be larger than the drop size in natural storms.

In natural storms water drops reach terminal velocity prior to striking the ground. The height of fall in rainfall simulators is limited and drops do not reach terminal velocity. A drop height of one meter or less is usually used in the simulators. Because kenetic energy at impact is considered to be more important than drop size, a larger drop is used in the simulator to give kenetic energy comparable to natural rain drops falling at terminal velocities.

TEST AREA

Many hand-portable infiltrometers use a test plot area of several square inches to a few square feet. A larger test area is desirable to give a more valid statistical representation of the watershed characteristics. The shape of the test area should be circular to minimize edge effects. The watershed being studied may have a variety of slopes. The test apparatus should be designed so that data may be collected on sloping test areas.

NON-DISTURBANCE OF SOIL

Soils are disturbed by water drop impact from both natural storms and simulated storms. Kenetic energy of artificial drops is regulated to reach levels of soil disturbance which are comparable to natural events. Soils are also disturbed if disruption is necessary in the installation and operation of the test apparatus. Design and placement of the test apparatus should provide for minimum disturbance of soil.

LATERAL FLOW

During natural storms the soil is wetted in large areas. Water which enters the soil surface moves downward and horizontal components of movement are minimal. Infiltrometers usually wet a limited test plot and significant edge effects may occur through lateral movement of water from the wetted area to adjacent dry areas. Lateral flow has been minimized by using infiltrometers which are designed with double infiltration rings. In the double ring infiltrometers, the outer ring is used to supply water which retards lateral flow from the inner ring. Other designs use peripheral wetting which provides water to retard lateral flow from the test plot. Significant errors may result if lateral flow is not retarded. Design of a simulator must include a means of minimizing this loss.

METHOD OF MEASUREMENT

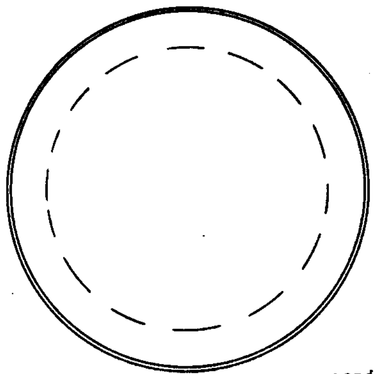
The information required from an infiltration test is the amount of infiltration or runoff as a function of time during the test. The rate of

FIGURE I

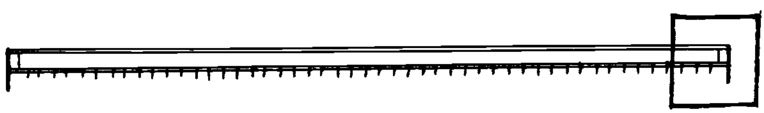
Sprinkler Head:

scale: 1 in = 0.5 meters

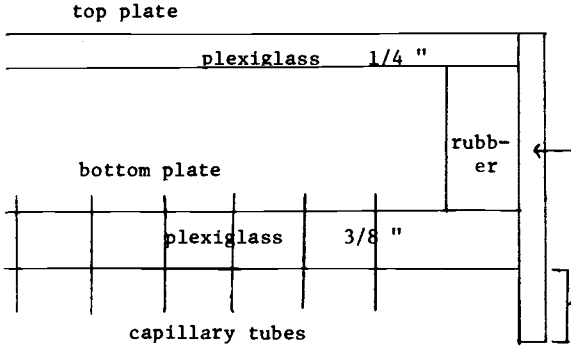
PLAN



CROSS SECTION X2



BLOW UP - AREA B - ACTUAL SIZE



infiltration is often measured by subtracting amounts of runoff from quantities of water supplied to the test area at various times throughout the test. Other devices measure infiltration by capturing infiltrated water.

THE JEEP-MOUNTED INFILTROMETER

The jeep-mounted infiltrometer is believed to minimize the differences between an artificial storm-runoff event and a natural storm-runoff event and to provide adequate portability. Mounting the infiltrometer on a four-wheel drive vehicle allows the unit to be nearly as portable as a hand-carried unit. The carrying capacity of the jeep allows the equipment to be sufficiently large to test a more representative soil plot and to carry enough water to continue a test for periods longer than an hour.

THE INSTRUMENT

The simulator consists of three basic parts; the sprinkler head, the water supply system, and the water collection system. Figures I and II are scale drawings of the sprinkler head and the unit in use. The sprinkler head is a 1.25 meter diameter cylinder constructed of plexiglass. Water drops are produced from capillary tubes which are bonded at one inch centers to the lower plate of the sprinkler head. Water is supplied to the sprinkler from a constant head reservoir which embodies a Boyle-Mariotte tube to maintain a constant pressure in the reservoir. A flow meter in the supply line between the reservoir and the sprinkler head is used to measure the discharge of the sprinkler. Drops from the sprinkler fall on the test area and are absorbed into the ground untill the infiltration capacity of the test plot has been exceeded. When the infiltration capacity is exceeded, water begins to pond and is removed through a siphon system. The amount of water removed is recorded as a function of time, thus determining runoff.

WATER DROP CHARACTERISTICS

Capillary tubes of 0.1 inch inside diameter produce water drops which are 5 millimeters in diameter. The sprinkler head is placed at a one meter height above the test plot. Five millimeter drops falling from a height of one meter have a kinetic energy which is equal to that of three millimeter drops falling at terminal velocity. Studies by Laws and Parsons (1943) have shown that three millimeter drops are representative of the average raindrop size in storms having intensities which range from two to six inches per hour. The intensity of rainfall from the sprinkler is controlled by modifying the inflow rate from the reservoir.

TEST AREA

Water from the sprinkler ralls in and around a collection ring which is a steel cylinder having one-eighth inch wall thickness and a diameter of one meter. The area of the test plot is the horizontal projection of the area of the soil surface within the collection ring. The area of the test plot is maximum when the ring is placed in a horizontal soil surface. When the collection ring is placed on a slope, the area becomes less and is equal to the maximum area times the cosine of the slope. Inflow to the collection ring is computed by determining the ratio of the test plot area to the area of the sprinkler which is 1.25 meters in diameter and multiplying this ratio by the inflow to the sprinkler.

NON-DISTURBANCE OF SOIL

The soil in the test plot is disturbed by placement of the collection ring. The degree of disturbance is minimized by utilizing a relatively large test plot, one meter in diameter, and by careful placement of the ring. The soil is also disturbed through water drop impact, but because the kenetic energy of drops from the sprinkler were sized to equal the kenetic energy from natural raindrops no correction is believed to be required.

LATERAL FLOW

Marshall and Stirk (1950) indicate that lateral flow from infiltration is most effectively minimized by peripheral wetting. Water for peripheral wetting is supplied from the 0.125 meter radial overlap of the sprinkler beyond the catchment ring. It is believed that this overlap may be insufficient if wind occurs during an infiltration test. For this reason it is recommended that the test plot be surrounded by a wind screen.

METHOD OF MEASUREMENT

A constant measured rate of water is applied on the test plot and peripheral to the test plot from the sprinkler. Computation of inflow to the test plot was discussed on the preceding page. The rate of runoff from the test plot is variable from zero at the beginning of the test to a larger quantity later in the test. Runoff volumes are determined by siphoning all surface water from the test plot and recording the volume of siphon water as a function of time. Infiltration is computed by subtracting runoff water from inflow to the test plot.

CONCLUSIONS

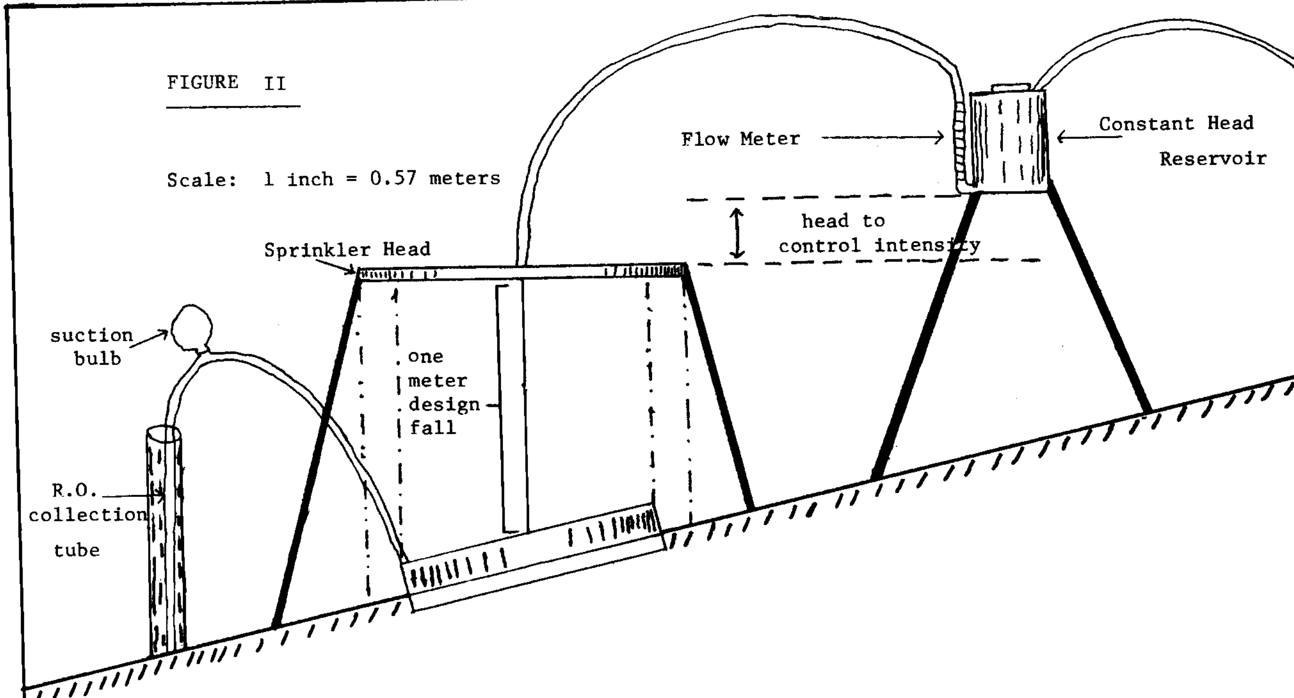
The principal conclusions from this study are:

1. The jeep-mounted rainfall simulating infiltrometer is designed to combine portability with a relatively large test plot and relatively long duration of testing. These advantages make possible more accurate predictions of infiltration rates in runoff potential studies.

2. The instrument has been designed to produce rainfall intensities of 2 to 6 inches per hour which are comparable to natural storm intensities in the Flagstaff area.

FIGURE II

Scale: 1 inch = 0.57 meters



When used on slopes, catch ring will appear as an elliptical projection on the horizontal sprinkler head.

$$Ae = \pi r^2 \cos \theta$$

θ = angle of slope

SCHEMATIC OF RAINFALL SIMULATOR

3. Capillary tubes of 0.1 inches inside diameter produce drops of five millimeter diameter. Five millimeter water drops from the simulator are equivalent in kinetic energy at impact to naturally occurring three millimeter drops travelling at terminal velocity.

4. Errors due to lateral flow are minimized through peripheral wetting.

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ABSTRACT

IMPROVEMENT OF WATER QUALITY BY AIR BUBBLING TO ELIMINATE THERMAL STRATIFACTION IN UPPER LAKE MARY, ARIZONA

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Thermal stratification has caused recurring quality-of-water problems in Upper Lake Mary--the principal water supply for Flagstaff, Arizona. Air bubbling conducted at one site in the 15,000-acre-foot reservoir eliminated the stratification for a distance of more than 2 miles. Natural oxygenation of the entire reservoir changed the hypolimnion from a reducing environment to an oxidizing environment and significantly improved the quality of the water withdrawn from the reservoir. The dissolved iron content in the water decreased from an average of 1.0 to 0.03 mg/l, the dissolved manganese decreased from an average of 0.5 to 0.01 mg/l, and the objectionable odor and color of the water were eliminated. The dissolved oxygen in the hyplimnion ranged from 0 to 20 percent saturation before air bubbling and from 40 to 70 percent saturation after the thermocline was destroyed.

The improvement in the quality of the water in Upper Lake Mary was accomplished at a net saving to the city. The cost of purchasing, installing, and operating the air-bubbling equipment was more than offset by the savings in electricity and chemicals previously used at the water-treatment plant.

THE NATIONAL WATER COMMISSION DRAFT REPORT[1/]

by

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BACKGROUND OF DRAFT REPORT

About 1200 copies of the Draft report were distributed to a critical audience in November, 1972. Another 400 copies of an 84-page Summary of the Recommendations were also circulated. The bulky report contains over 1100 pages (showing it), includes 16 chapters and 200 recommendations.

Although it was first believed that the Final report would be ready this Spring, it is my present understanding that it will not appear until mid-June of this year. The Commission record remained open until Feb. 15th. Some of the delay may be the result of gathering material at public hearings in January and February, in Spokane, Phoenix (which I attended), New Orleans and Washington, D.C. The hearings and other "input" may produce revisions of the Draft report, although I have not seen any of the new material.

The law establishing the Commission resulted from various efforts to examine water policies over a long period of time (notes here on earlier efforts) and recommendations made by previous commissions, including the Public Land Law Review Commission which submitted its Final report in June, 1970. Chapter 8 of that report relates to water resources and it recommended a course of action with respect to reserved rights that the National Water Commission had to struggle with also. But the main impetus for the statute creating the Commission came from proposals for development of the Colorado River Basin which raised fundamental questions regarding future national water policies. This background is discussed in the Preface to the Draft Report (iii).

A number of you may have worked on studies for the Commission and know about the Commission's composition and its staff work. The Commission has seven members. Charles Luce of New York is chairman. Roger Ernst, the former Arizona State Land Commissioner, is a member. All present members appear to me to be diligent and well qualified. They are served by a relatively small staff and a number of experts who prepared individual studies.

Most of you know that the Commission will go out of existence not later than September 26 of this year. Congress established the Commission in 1968 and gave it five years and up to 5 million to do the following (among other tasks):

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1. Presented at the Joint Annual Meeting of the Arizona Section of the American Water Resources Association and the Hydrology Section of the Arizona Academy of Science, Tucson, Arizona, May 4, 1973.

Duties of the Commission

Sec. 3. (a) The Commission shall (1) review present and anticipated national water resource problems, making such projections of water requirements as may be necessary and identifying alternative ways of meeting these requirements -- giving consideration, among other things, to conservation and more efficient use of existing supplies, increased usability by reduction of pollution, innovations to encourage the highest economic use of water, interbasin transfers, and technological advances including, but not limited to, desalting, weather modification, and waste water purification and reuse; (2) consider economic and social consequences of water resource development, including, for example, the impact of water resource development on regional economic growth, on institutional arrangements, and on esthetic values affecting the quality of life of the American people; and (3) advise on such specific water resource matters as may be referred to it by the President and the Water Resources Council.

COMMISSION PROPOSALS

This statutory charge is very comprehensive and required a great deal of work including about 25 or more separate studies which some of you have seen. Almost half of the studies deal directly with water law, and many others relate to "Institutional Arrangements" which are clearly dependent on law, e.g., interstate compacts, pollution controls, planning, project financing and environmental values, among others.

My effort here will be to call your attention to several sections of the Draft report and explain some reactions to them. General comments are saved for the end with the understanding that we are not speaking of the Final report which may include revisions in specific areas.

References to law and legal institutions are found throughout the Draft Report. In the time allotted I can outline only a few which are found principally in the following chapters:

Chapter 4 (83 pages) Water Pollution Control

All of you know about the 1972 legislation that was passed over the President's veto and that he subsequently impounded a large portion of the funds appropriated. I will not spend time on this matter except to point out that the report disagrees with the 1985 goal of "no dishcarge" in somewhat rhetorical fashion: "The Commission believes such an abstract and absolute approach ["No discharge" by 1985] to water quality management is as fundamentally unwise as to approach land use with a goal of placing no buildings on the land" (4-3). Instead, the Commission favors the 1965 Water Quality Act, statutory approach which simply indicated general Congressional intent that quality standards shall be such as "to protect the public health or welfare, enhance the quality of water" and provide for the uses stated in the act. (4-7). This chapter contains sixteen Recommendations found on pages 18-83. (I might add, parenthetically, that the law faculty voted to increase my course in water law from 2 to 3 hours, beginning this fall to allow time for water quality matters and enforcement procedures).

Chapter 5 (181 pages) Improving Water Related Programs

This long chapter covers the principal Federal water programs including inland water ways and the Reclamation Act. The chapter has 10 sections. There is time to refer briefly to only one, Section D. Acreage Limitations and Subsidies. The three recommendations following this section advocate discontinuing future irrigation subsidies and abolition of the 160 acre water right limitation in the future when full payment is made by project beneficiaries. (Remember, 160 acre limitation is on water not land.)

There are arguments on both sides of these questions and there isn't time to explore them. I do know, however, that something is wrong when subsidies for irrigation pay for cheap municipal supply, which is the situation in Phoenix as alleged at the hearing there in Janaury. Irrigation water is about $10.00 an acre foot; municipal users pay from $50 to $70.

Chapter 6 Procedures for Resolving Differences Over Environmental and Developmental Values

This chapter, with its detailed recommendations, would take a separate hour or more and I cannot take time to discuss it other than to say that the proposals deserve careful attention, particularly the conclusion that the NEPA requirements should be integrated into licensing procedures.

Chapter 7 (205 pages) Making Better Use of Existing Supplies

This chapter was of particular interest to me and should be to you because it emphasizes (Sec. B), Improved Ground Water Management; (Sec. D), Transfer of Water Rights, including the improving of record keeping, simplification of procedures and the need for changes in the law; (Sec. E), Improvements in State Water Laws to Recognize Social Values in Water; (Sec. F), A Permit System for Riparian States (which except for a few states in the East have no system whatever); (Sec. G), Reducing Water Losses by Improved Efficiency; and, last but equally important, (Sec. H), Re-Use of Municipal and Industrial Wastewater.

This chapter concludes with 68 separate recommendations that contain proposals that some of us have been advocating for over 20 years, e.g., the interrelationship in law of ground and surface waters, public management of ground water districts, the encouragement of re-uses and some clear answers to problems relating to rights in secondary and tertiary sources such as municipal effluent, and salvage and waste waters.

Your section Newsletter of the American Water Resources Association for December pointed out that the Commission has recommended local ground water management where conditions of overdraft exist as in Arizona. Three of the recommendations deserve to be read (if there is time):

Recommendation No. 1: State laws should recognize and take account of the substantial interrelation of surface water and ground water. Rights in both sources of supply should be integrated, and uses should be administered and managed conjunctively. There should not be separate codifications of surface water law and ground water law; the law of waters should be a single, integrated body of jurisprudence.

Recommendation No. 2: Where surface and ground water supplies are interrelated and where it is hydrologically indicated, maximum use of the combined resource should be accomplished by laws and regulations authorizing or requiring users to substitute one source of supply for the other.

Recommendation No. 3: The Commission recommends that states in which ground water is an important source of supply immediately commence conjunctive management of surface water (including imported water) and ground water, whether or not interrelated, through public management agencies.

Chapter 13 Federal-State Jurisdiction in the Law of Waters

This chapter, only 34 pages, is one all of you have heard about and will continue to hear about. It is probably the most controversial.

This chapter addresses several problems that have long been discussed in Congress and elsewhere, yet there continues to be much misunderstanding and folklore about water rights all the way from the oldest water right served by the *acequia madre* at Espanola, New Mexico, which was ordered dug by Juan Onate in 1598, on up to the highest levels of constitutional law as, for example, in the *Pelton Dam* case in 1955 which actually rests on the Supremacy Clause of the U.S. Constitution and not on the abrogation of private property rights or some *new* theory of Federal rights, as has been claimed.

The chapter relies heavily on Frank Trelease's study and is therefore weighted toward a private rights doctrine that has never been fully compatible with public development of land and water uses which became public policy at least with the Reclamation Act in 1902, if not earlier.

Chapter 13 recommends enactment of a "National Water Rights Procedures Act" parts of which are set forth in 11 numbered recommendations (Page 34). The central question faced relates to the nature and extent of "reserved rights." The recommendations adopt and carry forward previous recommendations on water resources made in the 1970 report of the Public Land Law Review Commission of which I was a member.

The proposed Procedures Act is directed to the determination and adjudication of water rights through quantification, the payment of compensation and the resolution of conflicts between Federal and state law. The matter of "reserved rights" has two separate dimensions, viz., non-Indian claims to waters on or from federal property, and Indian water rights. The report makes the point that the *Eagle County, Colorado* case in state court, in which the U.S. Supreme Court construed the McCarran amendment to mean that the U.S. could be joined in a state water adjudication (because the amendment waived sovereign immunity for that purpose), did not deal with Indian water rights. There is now pending in the U.S. District Court for Colorado, a suit by the U.S. which proposes to adjudicate water rights on a number of streams in southwestern Colorado which serve non-Indians as well as the Ute and Mountain Ute tribes. There is no time to cover this subject which I am sure the Commission is struggling with mightily.

There is much valuable material and thought in the three last chapters, 14, 15 and 16, which deal respectively with Cost Sharing, Captial Demands for Development and Basic Research and Data. They deserve your attention.

SOME REACTIONS TO THE DRAFT REPORT

As you might expect, the comments on the report divide themselves generally between the water user groups, on the one hand -- primarily agriculture and the developers, who are very critical of recommendations that will diminish or remove subsidies -- and environmentalists, on the other hand, who endorse the report except in particulars such as the recommendations on water quality controls. The latter group is joined by some economists.

Why this different reaction is seen in the following summary of controversial recommendations which do not all deal directly with water law:

The Committee Recommends

-- A general policy to charge water users the full cost of water services, relying heavily on free-market forces to work out the distribution.

-- An end to new irrigation projects, at least until the year 2000; the commission says they are not needed.

-- Limitation of new projects for flood-plain protection.

-- Imposition of time-limits for water project construction under new Congressional authorizations.

-- "De-authorization" and reconsideration of projects authorized more than 10 years ago, but not yet constructed.

-- Application of realistic interest rates in calculating both project costs and rates for repayment by beneficiaries, to eliminate hidden subsidies in federal water projects.

-- Watershed protection engineering should be taken away from the Agriculture Department; Interior's Bureau of Reclamation should be converted from construction to engineering and water management; and the Army Corps of Engineers should deemphasize construction and begin emphasizing information-gathering and water resource planning.

The creation of a board of review to look at projects and programs may threaten Federal agencies which are often prejudiced in project evaluations by self-interest.

SOME ADDITIONAL REACTIONS AND A CAVEAT

The projection of water demands and their discussion in Chapters 1 and 2 are not entirely satisfactory. But I am wary of projection techniques and the myths that numbers can create.

The market mechanism which is overplayed in the whole report seems to me a kind of naive return to Adam Smith's invisible hand and a secular theology that is not helpful in modern society. People are going to drink and use water no matter what it costs and public moneys have been invested in that proposition for so long I don't think it merits much discussion.

Some of the chapters were written or edited or influenced by lawyers including friends of mine with whom I have disagreed for years. Some of them are very good writers but poor or unsophisticated economists. And yet the report and its recommendations may be given an aura of plausibility or may create superficial attitudes with respect to environment problems and future policies because this group is now very influential in interpreting environmental policy and legislation. This is no reflection however on the Commission and its work. In fact, mine is small criticism for such a large and valuable effort.

LAKE POWELL RESEARCH PROJECT: HYDROLOGIC RESEARCH

by

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ABSTRACT

The Lake Powell Research Project is investigating the effects of man's activities on the Southeastern Utah-Northeastern Arizona region. A major portion of this project is devoted to the hydrology of Lake Powell, the largest recent modification in the region.

This hydrologic research is separated into the following subprojects and administrative institutions:

Subprojects	Institution
Streamflow Trends Evaporation Bank Storage	University of California at Los Angeles
Sedimentation Physical Limnology Lake Geochemistry	Dartmouth College

The project is now concluding its first year of full-scale research effort.

The UCLA subprojects are aimed at developing an overall water budget for the lake, both on an annual and long-term basis. The streamflow-trends study indicates that the Upper Colorado River Basin (UCRB) has shifted from a few extraordinarily wet decades in the early 1900's to several relatively dry decades up to the present.

Evaporation efforts so far are toward installing a data collection system capable of furnishing data for mass-transfer and energy-budget calculations.

The bank-storage study indicates that bank storage constitutes a large fraction of the impounded waters. Secondary as well as primary permeability may be of major importance in bank storage. The Evaporation and Bank Storage subprojects are working in close coordination with the Bureau of Reclamation.

The Sedimentation subproject has shown that the rate may be in general agreement with earlier estimates from river flow and suspended sediment data. However, the distribution is affected by sediment dams formed by slumping of canyon wall material.

Physical limnology studies indicate the presence of stratifications resulting from thermal and turbidity layers causing complex movements within the lake waters.

Field and laboratory efforts in lake geochemical analyses indicate that the precipitation of calcium carbonate may be the most important chemical process in changing the water quality of the lake.

INTRODUCTION

The Lake Powell Research Project is studying the effects of man's activities in the Lake Powell Region of Southeastern Utah and Northeastern Arizona. The project is funded by the RANN Division of the National Science Foundation. The project coordinator is Professor Orson Anderson from the University of California at Los Angeles.

The project is just completing its first full year of research. A major part of the Lake Powell Research Project is directed toward the hydrology of Lake Powell. The creation of Lake Powell is the largest recent man-made change in the region. The next changes will be large electrical generating stations and coal mining activities in the region.

Most of the subprojects related to hydrology are listed in the abstract. There are also three other subprojects relating to hydrology or limnology and they are administered by the University of New Mexico. These subprojects are 1) biological limnology, relating to biological water quality, productivity and eutrophication; 2) shoreline ecology, relating to shoreline vegetation and effects of recreation along the shoreline of the lake; and 3) heavy metals - studying the concentration of heavy metals, such as mercury, in the water and biota of the lake. Due to space and time limitations, these three projects, primarily concerned with biological limnology, will not be discussed here.

LAKE GEOCHEMISTRY

A description of this subproject is presented by Professor Robert Reynolds in another paper in this volume (q.v.). The major conclusion of this work is that calcite precipitation is the major chemical change affecting the water quality.

PHYSICAL LIMNOLOGY

The objective of the Physical Limnology subproject is to study the dynamics of water movement in the lake. The deeper regions of the lake are meromictic, or at best, only partially mixed. The shallower regions near the inflows and in the bays are mixed. In the summer there is an oxygen minimum layer at the top of the thermocline. In the winter, the lower, deeper regions have a conservative dissolved oxygen content due to dense, cold, oxygenated water flowing into the lake. In 1971 and 1972 there were advections of cold saline water, high in dissolved oxygen, in the late spring that reached the deeper portions of the lake. Dissolved oxygen content is a good tracer for water movements within the lake. The present residence time of water in the lake is about one year. If the lake is allowed to reach capacity, the minimum residence time will be approximately two and a half years. Thus, the lake retains many lotic, or river-like qualities, in spite of its size.

SEDIMENTATION

The objective of this subproject is to determine the rate of and distribution of sedimentation in Lake Powell. The total quantity of sediments is as anticipated from earlier studies of suspended sediment and flow from tributaries. The Colorado and San Juan Rivers supply the bulk of the sediments and these are deposited as deltas near the inflow sites.

However, additional smaller amounts are brought in by side tributaries due to local storms. Sometimes these inflows of sediment cause turbidity flows into the main channels. Some of these turbidity flows reach the turbines at the dam.

There are also slumps and rock slides from the canyon walls. These slumps and slides add sediment to the lake but are most important in forming underwater sediment dams that inhibit bottom transport of sediment. These sediment dams are mostly formed where the Chinle formation outcrops along the original canyon near or below the present lake level. Down the lake from the major slumps in the Colorado and San Juan arms the sediment thickness is considerably reduced.

The UCLA subprojects are primarily related to determining the water budget for Lake Powell.

STREAM FLOW TRENDS

The objective of stream flow trends subproject is to determine long range climatic trends and variations in streamflow in the Upper Colorado River Basin. It is also desirable to place recent measured flow in a better historical context.

Initially, the USGS records were reviewed. Three things were shown: first, that the earlier decades of the Twentieth Century were unusually wet, and the recent decades, beginning about 1930, have been relatively drier. Second, the Upper Colorado River Basin is not climatically homogeneous. There is a line, approximately following the trend of the Uinta Mountains, which separates two separate climatic regions. Third, the USGS records are too short to reveal long-term climatic trends.

The project is now using dendrochronology to estimate climatic records and establish a longer historical context for streamflow data. Twelve new tree-ring sites have been taken in areas where there are relatively undisturbed drainage basins with USGS stream gages at the outflow points. The tree ring series from these sites range from as far back as 1423 in the Uinta Mountains to 1793 in the Delores River Basin.

The tentative results indicate that the early decades of the Twentieth Century may possibly have been the wettest in the last four centuries. These early results also indicate that individual sub-basins may vary considerably in comparison to other sub-basins or the Upper Colorado River Basin as a whole.

EVAPORATION

The objective of this subproject is to determine evaporative loss from Lake Powell. The present estimates are an evaporation rate ranging from four to six feet per year. The Bureau of Reclamation maintains three mass-transfer stations in the Lake. In 1971, an estimated 400,000 acre feet of water were lost by evaporation. This may be a rather conservative estimate. The Lake Powell Research Project's estimate is that evaporation may have been as high as 550,000 acre feet. At full capacity, evaporation may range from 650,000 to 850,000 acre feet per year.

In cooperation with the Bureau of Reclamation, a fourth data station has been established at the northern end of the lake. Instrumentation on the data-station rafts has been substantially improved; also, instruments to permit estimation of evaporation by energy-budget methods have been installed. Each raft now has a Sierra Corporation SRC-700 recorder installed. Thermister probes are installed for surface-water and air temperature. Weather Measure model W103A-6L anemometers are being used. Two-meter winds are being measured at two rafts and two- and four-meter winds are being measured on two of the other rafts. Relative humidity is being measured at the two-meter level by Hydro-Dynamics model 15-7012WP Relative-Humidity Sensors. The rafts at the extreme south end and north end of the lake are equipped with Swissteco S-1 Radiation Balance Meters. Inflow and outflow temperature stations are being installed. The Bureau of Reclamation and Limnology subprojects are measuring temperature profiles to determine seasonal variations in heat content of the lake. An evaporation pan station is being installed at Hall's Crossing near the geographic center of the lake area. Most of the data is recorded on magnetic cassette tapes and will be reduced and analyzed at Lowell Observatory in Flagstaff.

BANK STORAGE

The present estimates of bank storage by the Bureau of Reclamation are about seven million acre feet through the end of the first decade of the lake's formation. Bank storage will be strongly influenced by elevation of the lake's surface. Regulation of the lake's surface is now pending court decision.

In the original estimates of bank storage, the primary porosity of the Navajo sandstone was the major consideration. It may be that other formations play a significant role in the bank storage situation. Also, fracture permeability may, in some local areas, be important with regard to bank storage.

A tentative model shows that the bank storage rate may decrease significantly after maintaining the lake at a more or less constant level for two years. It is possible that by lowering the lake level substantial return flow of bank storage may occur. The Lake Powell Research Project, in cooperation with the Bureau of Reclamation, is drilling two exploratory wells near the lake. One well is to test the Permian section at the north end of the lake, the other well will test a fractured area in the Navajo sandstone near the central area of the lake.

SUMMARY

At the end of the first year of research, only tentative conclusions have been reached about the hydrology of the Lake Powell region. At the end of the second year, the subprojects described above will furnish more specific results in regard to the water budget and limnology of Lake Powell.

August, 1973

COMPETITIVE GROUNDWATER USAGE FROM THE NAVAJO SANDSTONE

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ABSTRACT

Groundwater modeling is used to theoretically relate mining pumpage of the Navajo Sandstone to declines in the potentiometric surface at Navajo and Hopi Indian community, domestic, and stock usage locations. The shallow wells on top of Black Mesa are shown to be part of a perched water table condition which is dependent upon the hydraulic conductivity of an aquatard known as the Mancos Shale. The isolation of the aquatard allows the shallow wells to be treated as a problem separate from that of the artesian and recharge areas. Computer modeling of the groundwater system is concerned only with those Indian wells which directly tap the Navajo Sandstone in either artesian or free water table areas. The computer simulation developed is a modified version of the basic artesian aquifer routine used by the Illinois State Water Survey. Computer results correspond with the low percentage of storage withdrawal calculated for the artesian area under Black Mesa.

INTRODUCTION

Projections of future energy demands have resulted in the planned construction of coal-fired power generation plants in the "Four Corners" region of the Southwest (Arizona, New Mexico, Colorado, and Utah). A simulation of the effects of these power developments including physical, economic, and social variables is in progress at the University of Arizona under a grant from the Ford Foundation. The simulation effort has a two year duration with computer modeling consuming most of the first year and a workshop involvement with selected interest groups some time during the second year. The project emphasizes simulation as a decision and management tool and is presently nearing completion of the first year.

The objective of the Four Corners Program is to provide conflicting interest groups with an understanding of the long term consequences of various alternative plans proposed as solutions to coal-fired power developments in the Four Corners region. The enumeration of alternative plans is called a decision space, and the range of corresponding results is called an effects space. A number of computer simulation models link the decisions and effects. Figure 1 shows a conceptual block diagram of the simulation model. By virtue of the intended application, the simulation includes human participants; however, only the major section classifications (i.e., managerial, social, economic, and physical) have been diagrammed.

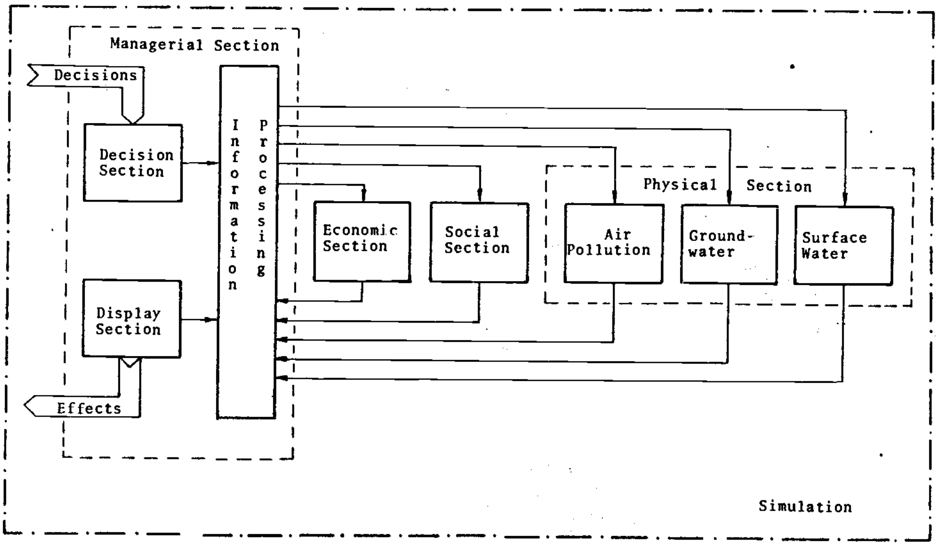

Figure 1. Conceptual Block Diagram of the Simulation.

A portion of the physical section of the simulation is concerned with the development of groundwater resources for strip mining on Black Mesa and the associated slurry transportation of coal to the Mohave power plant and, potentially, the Southern Nevada plant. The impacts of slurry pumping upon the local environment has been discussed in subjective terms by Budnik (1972, p. 102) and Clemmer (1970, pp. 5-11). The effects of slurry pumpage upon water interests of the Navajo and Hopi Indians are examined in this paper from a theoretical viewpoint with the aid of simulation model results.

SITUATION

Black Mesa, Arizona, is the location of low sulfur, bituminous coal deposits contracted for use at the Navajo and Mohave power generation facilities. At full operation, both facilities will consume a daily total of 39,000 tons of coal -- 23,000 tons at the Navajo plant and 16,000 tons at the Mohave plant (Environmental Statement, Navajo Project, p. 79). The 35 year contractual supply agreements are held by Peabody Coal Company. Navajo and Hopi Indian Tribes have leased 64,858 acres of Black Mesa to Peabody Coal although the company anticipates mining only 14,000 acres to meet the 35 year supply agreements. For the duration of Indian leases, the Environmental Statement, Navajo Project (p. 79) states:

> "The terms of the mining leases is for 10 years and so long thereafter as coal is produced in paying quantities, with land and specified improvements to be returned to Tribal ownership."

Transportation of the strip mined coal from Black Mesa to the distant facilities has prompted the use of a slurry pipeline. The pipeline is approximately 275 miles in extent linking the coal deposits with the Mohave facility near Bullhead City, Arizona. The slurry is a 50 percent mixture by weight of coal particles and water and travels in the pipeline at a velocity of about 5 miles per hour (Environmental Statement, Navajo Project, p. 87). A 78 mile rail line is planned for the other link from Black Mesa to the Navajo facility near Page, Arizona. Figure 2 shows the location of Black Mesa with respect to the Indian reservations, state lines, and the community of Page. Bullhead City is located off the map to the southwest near the intersection of Arizona, California, and Nevada.

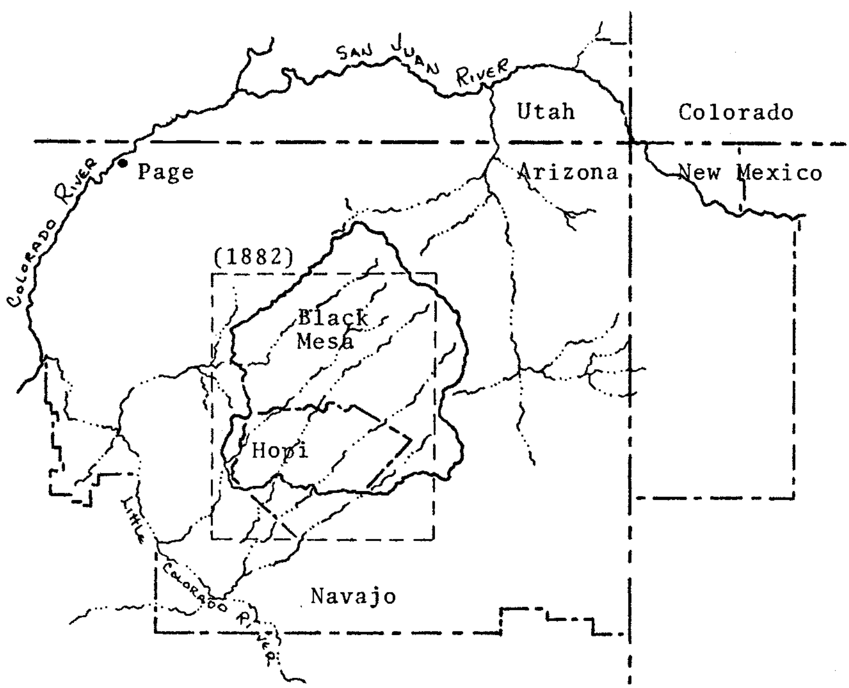

Figure 2. Location of Black Mesa on Navajo and Hopi Indian Reservation lands. (Dotted lines show Navajo-Hopi joint use area from 1882)

Water for the slurry pipeline is obtained from groundwater pumpage of the Navajo Sandstone aquifer. Six production wells owned by Peabody Coal presently extend from the surface of Black Mesa to depths ranging from 3535 feet to 3737 feet.[1] The Navajo Sandstone is the fourth member, numbered in ascending order, of the N-multiple-aquifer system (Southwest Energy Study, Appendix D, p. 51). The N-multiple-aquifer system demonstrates artesian conditions under Black Mesa and water table (unconfined) conditions in associated recharge or outcrop areas. As of 1961, a field inventory of reservation groundwater supplies resulted in the tabulation of 1248 drilled wells, 537 dug wells, and 955 springs (Cooley, et al., p. A4). A large portion of the Indian wells do not directly tap the Navajo Sandstone aquifer. For example, Indian wells in the Mesaverde formation (upper 600 to 800 feet) of Black Mesa are more than 1700 feet above the Navajo Sandstone in the vicinity of the Peabody Coal mining site.

MODEL OBJECTIVE

The objective of the groundwater model is to relate aquifer withdrawals by Peabody Coal to possible declines in the potentiometric surface of the Indian domestic and stock usage areas. Three main areas are of interest, they are:

1. Shallow Indian wells on top of Black Mesa.
2. Indian artesian wells tapping the Navajo Sandstone.
3. Indian wells in the recharge area of the Navajo Sandstone.

DECISION SPACE

Pumping rates and pumping durations for the Peabody Coal wells define the decision space of the groundwater model. Pumping rates of 2,000 gallons per minute and 4,000 gallons per minute are selected. The lower rate reflects current activities. The higher rate represents a speculation that the Southern Nevada plant might be built and might be supplied by a slurry pipeline in whole or in part. The observation time horizon is 55 years, and the pumping duration is 35 years. The effects space is the variation in potentiometric surface as a result of the decision space for the three areas defined by the model objective. Other parameters are changed for a sensitivity analysis of aquifer characteristics.

AQUIFER PROPERTIES

The Navajo Sandstone, Coconino Sandstone, and the De Chelly Sandstone are the principal water bearing aquifers located on reservation lands. Productivity of the water bearing units is low, and few wells are used for irrigation purposes. Typical pumping rates corresponding to various

[1]E. H. McGavock, well schedules, U.S.G.S., Flagstaff, Arizona.

water uses include 1-5 gallons per minute for stock and domestic windmills, 5-100 gallons per minute for municipal and institutional wells, and less than 200 gallons per minute for the few industrial wells (Cooley, et al., 1969, p. A48). The aquifers are generally separated by thick layers of sandy siltstone; siltstone, and mudstone which form confining layers or aquatards.

NAVAJO SANDSTONE

The Navajo Sandstone occupies the west and northwest portion of reservation lands. It is dissected by the San Juan River and roughly bounded by the Colorado and Little Colorado Rivers (Harshbarger, et al., 1957, p. 21). The Navajo Sandstone is wedge shaped varying in thickness from 1,400 feet in the northwest corner of the reservation to 15 feet near Chinle (eastward), 192 feet at Second Mesa (southeastward), and 525 feet at Tuba City (southward). The sandstone is characterized by large-scale crossbeds believed to be the result of wind deposition. About 98 percent of the sandstone is quartz grains. Hydraulic properties for the Navajo Sandstone obtained from pump tests and drill-core samples are summarized in Table 1 (Cooley, et al., 1969, p. A46).

Table 1 Hydraulic Properties of the Navajo Sandstone Aquifer

Parameter	Range
Porosity (%)[1]	25-35
Specific Retention (%)[1]	3-8
Specific Yield (%)[1]	18-29
Permeability (gal/day/sq ft)[1]	3-494
Transmissivity (gal/day/ft)[2]	450-3800
Coefficient of Storage[3]	.005

[1]Hydrologic laboratory tests on 24 samples.

[2]Computed from three pump tests.

[3]Computed from one pump test.

The Navajo Sandstone in confined areas exists between the Kayenta (lower) and the Carmel (upper) Formations. The Carmel Formation ranges in thickness from 0-300 feet and is principally siltstone and some sandstone. The Kayenta Formation ranges in thickness from 0-700 feet. In the northern part of the Navajo Indian Reservation, the Kayenta is a sandstone becoming a siltstone in the southern part of the reservation (Cooley, et al., 1969, Table 3, p. A7). Information on hydrologic properties of the Carmel and Kayenta Formations is scarce; however, representative ranges of permeability (hydraulic conductivity) are listed by Walton (1970, p. 36) according to rock type. Typical permeabilities for the rock types in units of gallons per day per square foot include 0.1 - 50 for sandstone, 0.001 - 2 for clay and silt, and 0.00001 - 0.1 for shale.

MANCOS SHALE

The aquifers of the Mesaverde Formation on top of Black Mesa are separated from other underlying aquifers by the Mancos Shale. Direct precipitation and infiltration from small ephemeral streams is the only source of recharge for the Mesaverde aquifers. The Mancos Shale is a thick aquiclude ranging in thickness from 500 to 700 feet in Black Mesa basin (Cooley, et al., 1969, p. A43). A perched water table condition exists in the Mesaverde Formation. Indian wells tapping the Mesaverde aquifers range in depth from 300 to 400 feet. In order for the slurry pumpage to affect the shallow Indian wells, the perched water table must be drained. Assuming the Mancos Shale to be free from major fractures or punctures and the cement grout around the Peabody Coal wells to be solid, the remaining means of Mesaverde drainage is by leakage through the Mancos Shale.

Leakage per unit of surface area is a function of the permeability and head differential across the Mancos Shale. Other modeling efforts for a pumping rate of 2000 gallons per minute for 30 years have theoretically obtained a 600 foot drawdown calculation for the center of slurry pumping (Southwest Energy Study, Appendix D, 1971, p. 57). Assuming the 600 foot head differential to be distributed only across the Mancos Shale, leakage velocity and travel time through the shale can be calculated with the aid of Darcy's Law. The leakage velocity ranges from 0.013 to 0.0000013 feet per day, and the associated travel times through the Mancos Shale become 126 to 1,260,000 years. Under the conditions stated above, leakage through the Mancos Shale as a result of slurry pumpage will not appreciably affect the shallow Indian wells on Black Mesa. The low permeability of the Mancos Shale, not considering the other intervening layers, effectively isolates the Mesaverde Formation from the Navajo Sandstone.

COMPUTER MODEL

The effective use of computer models cannot be divorced from a basic understanding of the phenomena being modeled. In the case of the groundwater model, an understanding of the hydrologic system can simplify and make applicable computer programming efforts. For example, recognition of the hydraulic isolation between the Navajo Sandstone and the perched water table on top of Black Mesa reduced what was perceived as a three-dimensional modeling problem into a two-dimensional one. Thus, computer modeling of the groundwater system is concerned only with those Indian wells which directly tap the Navajo Sandstone in either artesian or free water table areas.

Initially, three computer models were considered adaptable for use; however, no models were known to exist which could simulate the Navajo Sandstone in an "off-the-shelf" capacity. Modeling considerations included (1) free water table conditions, (2) artesian head conditions, (3) wedge shaped geometry, (4) possible leakage in the artesian areas, (5) possible recharge in the unconfined areas, and (6) hydrologic stress applied through well pumping rates. Given the considerations above, modifications to each computer program available was expected. The three models were:

1. A finite difference representation of aquifer cross sections in steady-state (Freeze, 1969).
2. A finite difference representation of the free water table aquifer in the Tucson Basin (Gates, 1972).
3. A finite difference representation of a basic artesian configuration with various hydrologic options (Prickett and Lonnquist, 1971).

The Freeze model required major modifications to convert the computer program from one based upon a steady-state solution (static) to a nonsteady-state solution (dynamic).

SELECTION CRITERIA

The groundwater model selected for use is a modified version of the basic two-dimensional, simulation program developed by the Illinois State Water Survey (Prickett and Lonnquist, 1971). The simulation mathematics utilize finite difference equations developed from the physical considerations of Darcy's Law and the conservation of mass. This simulation program was selected as the basic building block of the groundwater model for the following reasons:

1. Characterization potential as related to the physical problem.
2. Time scheduling requirements of the modeling effort.
3. Excellent documentation of the computer program.
4. Modification potential of the computer program.

FINAL MODEL

Figure 3 shows the final model grid for Black Mesa using 2 mile, nodal spacings. The grid is composed of 34 columns and 39 rows. Aquifer pumpage for most computer calculations utilize five Peabody Coal wells with location coordinates as shown. The pumpage effects at four points surrounding the Peabody wells is monitored. Three of the monitor points exhibit artesian conditions while Shonto is an unconfined aquifer location. In general, the nodal points within the perimeter of Black Mesa are assigned artesian properties, and the nodal points outside the mesa perimeter are designated as free water table. The hatched area in the upper left corner is the major recharge surface for the 66 by 76 mile section of Navajo Sandstone.

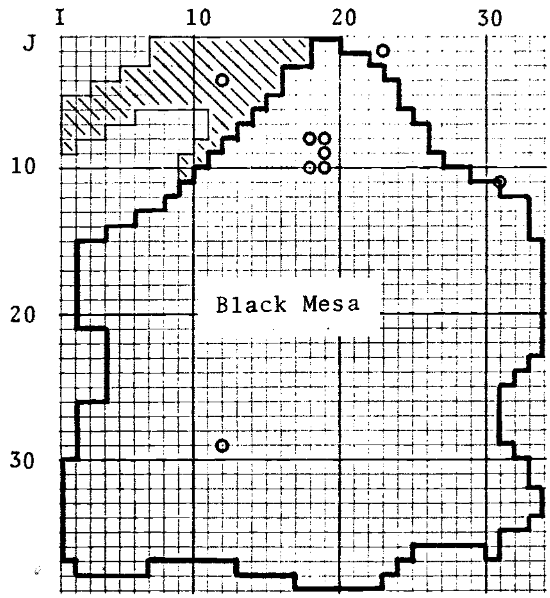

Figure 3 Final Model Grid

Interest Point	(I,J) Coordinates
Peabody Coal Well #2	(18,8)
Peabody Coal Well #6	(19,8)
Peabody Coal Well #5	(19,9)
Peabody Coal Well #3	(19,10)
Peabody Coal Well #4	(18,10)
Shonto	(12,4)
Chilchinbito	(23,2)
Rough Rock	(31,11)
Oraibi	(12,29)

The calibration of a simulation model for groundwater is generally based upon point source information in some historic sequence. A sufficiency of data in an historic sense is immediately lacking, and calibration of the present groundwater model in the traditional manner is not possible. However, the model will be used to access future effects based upon differential head changes in the potentiometric surface rather than accurate location of head elevations.

MODEL RESULTS

The theoretical effects of slurry pumping on the potentiometric surface at Chilchinbito, Rough Rock, Oraibi, and Shonto are tabulated in Table 2. The largest drawdown effects are obtained at Chilchinbito, the closest artesian location to the Peabody Coal wells. The following is a list of assumptions used in the model computations:

1. Recharge is zero.
2. Leakage is zero.
3. Total observation time is 55 years.
4. Slurry pumpage time is 35 years.
5. Slurry pumpage rates are either 2000 or 4000 gallons per minute (5 wells @ 400 gpm or 8 wells @ 500 gpm).

Table 2 Theoretical Effects of Slurry Pumping on the Potentiometric Surfaces at Various Locations (drawdown in feet)

Locations Pump Rate	Years										
	5	10	15	20	25	30	35	40	45	50	55
Chilchinbito											
Q = 2000 gpm	17	31	39	43	46	49	52	36	24	18	16
Q = 4000 gpm	30	56	70	79	85	91	96	68	45	33	27
Rough Rock											
Q = 2000 gpm	4	10	16	20	23	25	27	24	18	14	10
Q = 4000 gpm	9	21	32	41	47	52	56	48	37	27	21
Oraibi											
Q = 2000 gpm	1	4	8	12	16	18	20	20	18	14	10
Q = 4000 gpm	2	9	18	28	35	41	45	43	38	29	21
Shonto											
Q = 2000 gpm	0	0	0	0	0	0	0	0	0	0	0
Q = 4000 gpm	0	0	0	0	0	0	0	0	0	0	0
Peabody Well No. 5											
Q = 400 gpm	406	491	521	535	543	548	553	148	65	36	24
Q = 500 gpm	708	875	937	965	981	992	1001	296	132	74	49

Note: Pump 35 years, stop, T = 2000 gpdpf, SC(WT) = 0.15, and SC(A) = 0.0002.

SENSITIVITY ANALYSIS

Aquifer parameters were varied to determine ranges and characteristics of influence in potentiometric contours. Storage coefficient for the artesian conditions and aquifer transmissivity were of central interest. While most calculations used an artesian storage coefficient of 0.0002, the effects of changing the value one order of magnitude above and below this nominal choice was examined. A larger storage coefficient of 0.002 generally produced a reduction in drawdown and a time delay in the observation of slurry pumping effects at Chilchinbito, Rough Rock, and Oraibi. A smaller storage coefficient of 0.00002 produced a small increase in peak drawdown for the 35 year pumping period; however, the theoretical drawdowns had a sharper initial response reaching a stable reduction rate over most of the pumping period.

Variations in transmissivity,through aquifer geometry were accomplished by varying the aquifer thickness for a single value of permeability. The general wedge shape of the Navajo Sandstone in an east-west direction was used. Correspondingly, the greatest effect in potentiometric contours is observed at Rough Rock where the aquifer becomes thin. For the wedge geometry, two values of permeability were used -- 3 gallons per day per square foot and 30 gallons per day per square foot. The lower value of permeability is the low end of the hydraulic property range shown in Table 1. An increase in permeability by one order of magnitude decreases the peak drawdown and generally smoothes the observed pumping effects. The range of theoretical effects from low to high drawdowns as a result of the slurry pumping and sensitivity analysis are shown in Table 3.

Table 3 Range of Theoretical Effects for Slurry Pumping on the Potentiometric Surfaces at Various Locations (drawdown in feet)

	Years										
Locations	5	10	15	20	25	30	35	40	45	50	55
Chilchinbito											
Low Drawdown	0	2	6	8	9	11	12	8	7	6	6
High Drawdown	37	56	70	79	85	91	96	68	45	33	29
Rough Rock											
Low Drawdown	0	0	0	1	2	4	7	5	4	3	3
High Drawdown	17	22	32	41	47	52	56	48	37	27	21
Oraibi											
Low Drawdown	0	0	0	0	0	0	1	1	1	2	2
High Drawdown	13	19	20	28	35	41	45	43	38	29	21
Shonto											
Low Drawdown	0	0	0	0	0	0	0	0	0	0	0
High Drawdown	0	0	0	0	0	0	1	1	1	1	1
Peabody Well No. 5											
Low Drawdown	52	56	57	58	59	60	61	10	7	6	5
High Drawdown	708	875	937	965	981	992	1001	296	204	148	111

VOLUME USAGE

Although a measure of potentiometric surface effects was the desired goal, some simple static calculations add to an appreciation of the problem scope. Assume recharge and leakage to be zero and consider only the artesian area under Black Mesa. For an average aquifer thickness of 400 feet, a specific yield of 0.2, and a surface area of 2.1 million acres, the volume of water in the Navajo Sandstone (Black Mesa only) is calculated to be 168 million acre-feet. The Southwest Energy Study estimates present water pumpage from the Navajo Sandstone to be 3,264 acre-feet per year (Appendix D, p. 56). Over a 35 year period and at a constant pumping rate, the accumulated pumpage amounts to 114,240 acre-feet. The percentage of volume of water pumped over a 35 year period to the volume of water within the aquifer is calculated to be 0.07 percent. If variable ranges and calculation errors either increased the volume of pumpage by one order of magnitude or decreased the volume of aquifer storage by one order of magnitude, the percentage of volume usage would remain less than 1 percent.

CONCLUSIONS

Slurry pumping in the Navajo Sandstone is effectively isolated from the shallow wells on Black Mesa by the Mancos Shale and other intervening rock layers. Assuming no abnormal leakage paths, the mining withdrawals will not appreciably affect Indian wells located in the Mesaverde formation. Computer modeling has provided theoretical drawdown estimates for locations in the artesian and unconfined areas of the Navajo Sandstone. Since a variety of aquifer conditions are possible, ranges of drawdown are listed for the Indian communities of Chilchinbito, Rough Rock, Oraibi, and Shonto. The drawdowns vary from zero to less than 20 percent of the artesian head elevation above the Navajo Sandstone aquifer in confined areas. Similar modeling results were obtained for the Southwest Energy Study (Appendix D, 1971, p. 57). The volume of storage removal at a projected usage rate of 3,264 acre-feet per year for 35 years is significantly less than 1 percent of the water resources under Black Mesa.

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SALINITY PROBLEMS OF THE SAFFORD VALLEY:

AN INTERDISCIPLINARY ANALYSIS

by

Anthony B. Muller*

ABSTRACT

A change in groundwater quality, averaging approximately +0.13 millimhos electrical conductivity and +35 ppm chloride per year, has been documented between 1940 and 1972 with data from ten long-term sample wells.

The decrement in the water quality of the surficial aquifer seems to be attributable to four major mechanisms. An increase in salinity may be expected from leakage of saline water from the artesian aquifer. Such leakage would be stimulated by pumping-caused reduction of confining pressure, and by the puncture of the cap beds by deep wells. Water reaching the aquifer from natural recharge may contribute salts to the system. Such recharging water, if passed through soluble beds, could contribute to the salt. Lateral movement of water through similar deposits may be a contribution, and the concentration and infiltration of agricultural water could also add to aquifer salinity.

The economic analysis of the Safford Valley, based on the modeling of a "Representative Farm" analog, indicates that cotton will remain economical to produce on the basis of the projected salinity trends, for a significant time beyond limits of prediction. The analysis indicates that the optimum salt-resistant crops for the area are being cultivated, and, of these, alfalfa will cease to be productive in large areas of the valley by 1990. The entire valley will not produce alfalfa for profit by 2040.

The methodologies shown in the paper indicate how pumping influences salinity change and outline salinity control recommendations for the area.

SETTING

The Safford Valley, Graham County, Arizona, is a structural intermontane trough averaging 15 miles in width and lying between the Gila and Pinaleno Mountains in southeastern Arizona. The Gila River flows from east to west, for 32 miles through the valley.

The basement complex is at the surface on the upstream limb of the valley and extends at its deepest point to over 5,000 feet below the surface on the downstream limb. The material filling this trough may be divided into three major subdivisions, or facies groups. The lower basin fill is the primary contributor to this material. In this subdivision, a basal conglomerate is overlain by an extensive evaporite facies, which is subsequently overlain by a green clay facies and some delta deposits. The Gila River intercepts

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the lower basin fill in the bottom of the valley. Transitional deposits composed of calcareous and red facies cover the lower basin fill away from the river. The upper basin fill, of which only remnants seem to be apparent, terminates the series with tuffaceous lacustrine, piedmont and conglomerate facies. In plane view, the lower portions of the valley along the Gila River are seen to be Pleistocene and Holocene alluvial deposits, such as sand, gravel, silt and clay. The upper portions of the valley, which comprise the terraces, consist of Upper Pliocene Gila conglomerate, lake beds, lacustrine clays, silts, sand, tuff and limestone (Harbour, 1966).

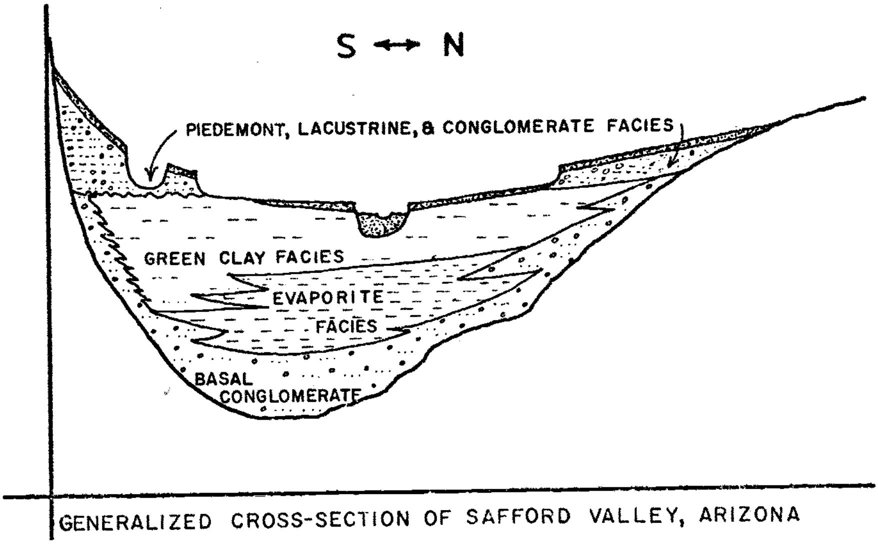


Figure 1.

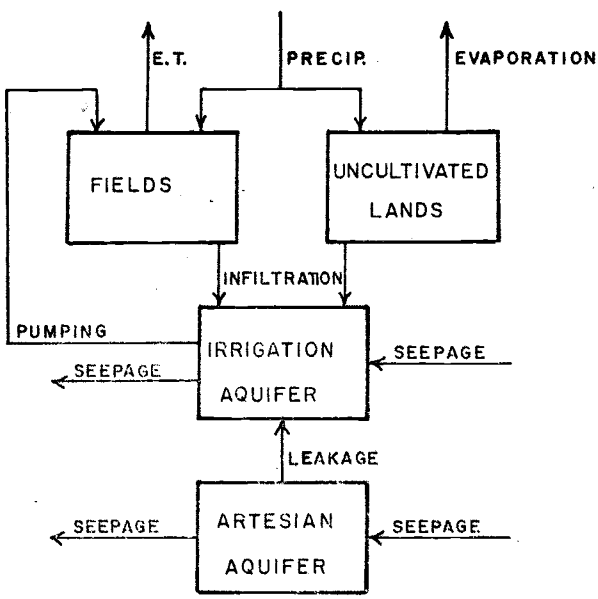
The Safford Valley is hydrogeologically divided into two aquifer systems. An artesian aquifer system occupies the lower extent of the basin fill, and is believed to extend to the bedrock-fill interface. The upper boundary of the system is, or is in, the green clay facies of the lower basin fill. Since the basement complex deepens in the downstream direction, and since the green clay facies (and red facies into which it grades in the western valley limb) is relatively level, the vertical extent of the aquifer similarly increases in that direction.

A surficial water table aquifer occupies the quaternary alluvial material in the stratigraphy. The upper surface of this aquifer, the water table, seems to conform with the configuration of the land surface when under static conditions.

SALINITY OF THE WATER TABLE AQUIFER

The water of the artesian aquifer is, in general, extremely saline, with electrical conductivity in the neighborhood of 8 millimhos. Extreme variability in salinity among samples from this aquifer has been observed. The water also exhibits thermal properties, and may be above 100°F in many locations (Muller, 1973). The material which makes up this aquifer was deposited in the lightly saline waters of a Pleistocene-Pliocene lake. The water of this lake is believed to still occupy the basal conglomerate facies. The artesian conditions are imposed by the confining forces of the green clay and evaporite facies which also contribute to the salinity of the aquifer. The intertonguing of the basal and evaporite facies brings water into contact with the remnant evaporitic salts of the lake (Knechtel, 1938).

The salinity of the water table aquifer averages approximately 4 millimohs (Muller, 1973), and thus is currently in the acceptable range for its agricultural uses. The identification of sources of salt to this water is more difficult than of those to the artesian aquifer, with a four-input model proposed (Muller, 1973). One input of both water and salt into the water table aquifer is natural recharge. Such recharge, primarily from wash-bottom sources during precipitation periods, is shown to have a lower salinity than groundwater. This source would then improve aquifer salinity, as shown by areas of high quality water at such wash locations on iso-chemical maps.



FLOW OF WATER AND SALT IN PHYSICAL SYSTEM

Figure 2.

A second input is water which infiltrates from irrigated areas. This would mean pumped irrigation water (groundwater), plus surface water irrigation, plus precipitation, less evaporation. A thermodynamic, chemical equilibrium model was applied to this mechanism (Dutt, 1972), using well cuttings as aquifer samples. It was found that if the water applied to the field had 735 ppm chloride, for example (which was found to be the average in irrigation water), the chloride content would be 2100 ppm by the time it reached the water table at 100 feet. This constitutes a three-fold increase in the salinity as indicated by chloride.

The third input is a source of salts within the aquifer itself, that is, lateral movement of water in the aquifer through soluble beds. The relative scarcity of evaporitic beds in this zone of fill, coupled with the low hydraulic gradients as a force moving water through such beds, limits the significance of this contribution.

The final, and deemed most important, contribution of salts to the system is leakage from the artesian aquifer system. Such leakage occurs through natural imperfections in the cap beds, as well as punctures of these confining layers by deep wells. The pressure of the artesian aquifer encourages such leakage of salts and water into the overlying aquifer. The contact between the water table aquifer and the artesian aquifer is a principal consideration in the development of groundwater resources in the valley. Such development should take into consideration the extent of these contacts under natural conditions as well as alternation of the extent of contacts by foreign conditions caused by pumping. Pumping, by imposing a hydraulic gradient artificial to the location, may set up flow lines in the two aquifers which encourage the occurrence of such contacts, or leaks. The volume of water entering the system in this manner is felt to be significant, although estimates are difficult (Muller, 1973).

At any point in the water table aquifer system these four contributions are believed to make up the salinity increase, although the extent of each mechanism varies from point to point. This increase in salinity has been found to be approximately +0.13 millimhos and +35 ppm chloride per year on the average (Muller, 1973), between 1940 and 1972. The rates do not seem to be uniform throughout the valley. Iso-chemical maps indicate the change may be divided into three major sections. From the Gila River entry point into the valley until Safford there seems to be a uniform, relatively stable, salinity condition with the highest quality water in the valley. From Safford to Pima there is a uniform, low-magnitude increase in salinity. From Pima to the downstream limit of the valley, the area exhibits extremely irregular salinity change conditions. Thus, progressing in the downstream direction, the water becomes more saline, more rapidly deteriorating and deteriorating in a less uniform fashion. Although downgradient movement of salts may be a contributing factor, the correspondence to the increasing artesian aquifer is not considered coincidental. The salinity of the artesian aquifer generally increases in this direction, leakage points are more common, and there may be more head differences between the aquifers. It is therefore recommended that the first area mentioned be that primarily pumped, and the last not pumped at all. Although this recommendation is obviously impossible to implement, canal companies can use wells in the stable region in preference to elsewhere, and further development may be planned with this consideration in mind.

ECONOMIC ANALYSIS

Production functions show productivity of the principal commercial crops in the valley (alfalfa, hay, sorghum, cotton and barley) to be all above 90 percent relative yield (except alfalfa), with the current root soil salinity in the neighborhood of 4 millimhos for the growing season. Yields of alfalfa are shown to have suffered considerably from salinity, with 84 percent relative yield. It shall be assumed that the 85 percent associated with the base year (1972), root zone electrical conductivity (RZEC) is the yield on which per acre gross returns of $146 are based (Wildermuth, 1969). Net returns at this yield are $64 per acre (Wildermuth, 1969), or 44 percent of the gross returns. Thus, alfalfa will yield no return whatever when relative yield falls below 56 percent, if ceteris paribus conditions are maintained. Since base yield (on which gross returns information is established) is only 85 percent relative yield, this value becomes 56 percent of 85 percent yield, or abour 48 percent relative yield. The production function shows alfalfa yield to approach this level as RZEC approaches 8 millimhos.

This level, though, is of no economic significance, since the acreage used for alfalfa will be used to produce another crop when net returns per acre from alfalfa fall below net returns from that other crop. In the case of the Safford Valley, the replacement crop will be barley since it is both more profitable and more salt-tolerant than sorghum. (Cotton is, of course, by far the most profitable crop grown in the valley as well as the most salt-tolerant. Acreage used for cotton production is restricted, and it is assumed that the farmer has already exhausted his acreage allotment. It is thus unlikely that alfalfa would be replaced by cotton.)

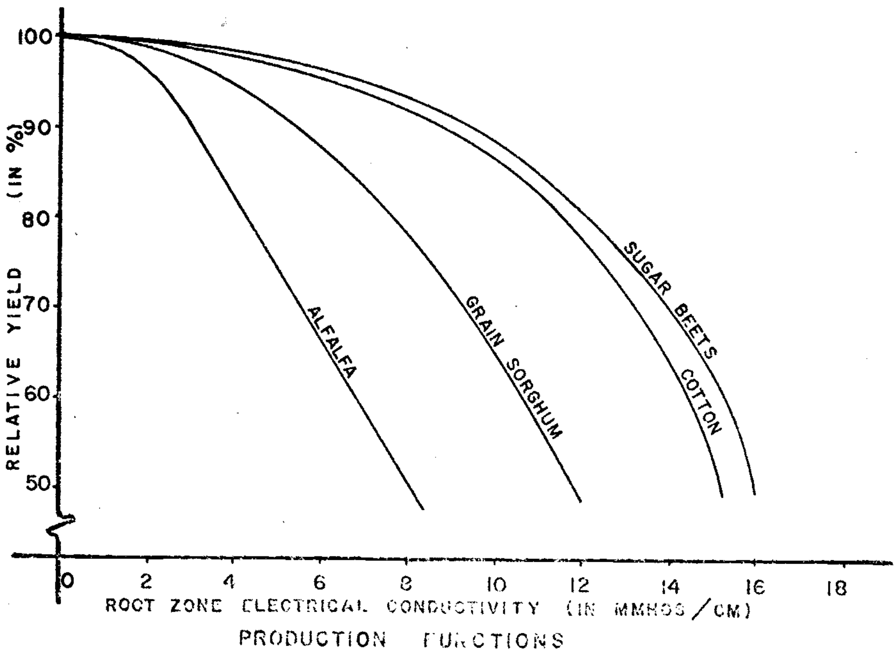


Figure 3.

Barley returns about $36 per acre (Wildermuth, 1969), so it may be assumed barley will replace alfalfa when returns from alfalfa production fall below this value. This, in fact, is an over-simplification, since farmers may grow alfalfa for other than simple costs-and-returns considerations alone. Alfalfa is used, for example, to return depleted nutrients to the soil in crop rotation practices.

Variable costs in the production of alfalfa are $64 to total $146 per acre. Holding variable costs constant, net return reaches $39 per acre, or net return equal to that provided by barley, when gross return is $121. This occurs at 82 percent of base year relative yield of 85 percent, or at 70 percent relative yield. Alfalfa is shown to reach this yield, in the production function, at a RZEC of 5.5 millimhos. At this level, cotton and barley remain virtually unaffected, while the relative yield of sorghum has fallen to 96 percent.

Clearly, as RZEC continues to increase, net returns continue to drop. At 8 millimhos RZEC, only barley is unaffected; sorghum yields have fallen to 83 percent, and cotton to 95 percent. It becomes apparent that within any extreme decrement in RZEC, cotton will remain economically profitable because of its high salt tolerance and because of the $84.40 per acre of cotton allotment currently supplied by the government.

SALINITY MODEL

An analysis of water quality data from irrigation wells that have been sampled repeatedly since 1940, and which thus yield 32 years of record, was performed for chloride and electrical conductivity measurements. Electrical conductivity yields an increase of +0.13 millimhos per year for a weighed mean slope. (Note: This is not a line of best fit.) Chloride measurements cluster into three definite groups, which correspond to the previously mentioned three areas. The area to Safford has an annual increase of +5.35 ppm chloride on the average. The Safford to Pima stretch shows a tight cluster of +34.90 ppm chloride per year, and the area beyond Pima increases by +61.52 ppm chloride on the average, per year. These results both support the pumping area recommendations and yield a basis by which to project future changes in the groundwater quality. The electrical conductivity increase rate, when applied to the 1972 electrical conductivity map (Muller, 1973), will yield projected values of electrical conductivity for any given year. If, on the other hand, a level of electrical conductivity is fixed, by using the rate change projection and the 1972 base year data for that point, the year in which the specified level is reached may be computed. The Department of Agriculture's equation to relate root zone soil salinity to salinity of irrigation water (Allison et al., 1954) was applied to the 5.5 millimhos RZEC. The irrigation water value obtained, 9.8 millimhos irrigation water electrical conductivity, was used as the limit for analysis in determining the year such a RZEC level would be reached, by the method just described. It was found that the three regions of salinity appeared here also. The area first to reach this level was that beyond Pima. A contour map of the replacement data of alfalfa shows the first changes from alfalfa production by 1990. The entire region will be unable to support alfalfa by 2010, on the basis of the economic criteria described. The Safford to Pima region will reach this RZEC level circa 2020, while the remaining region will achieve the level in 2040.

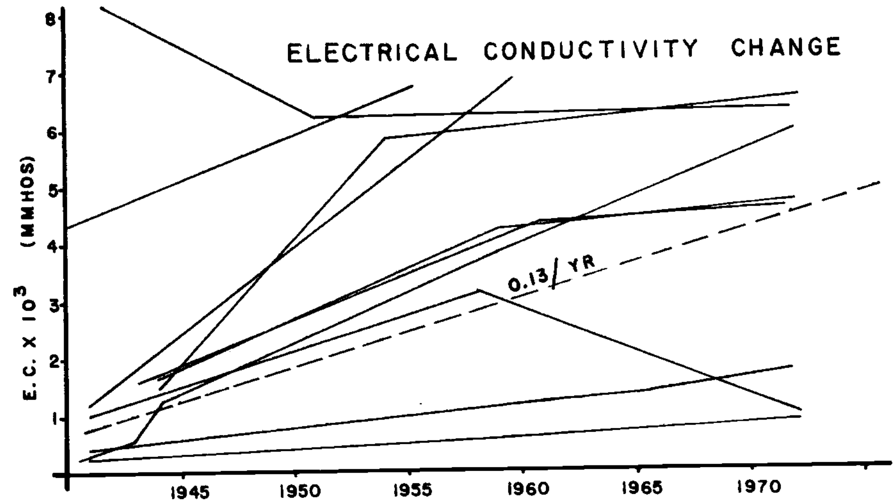


Figure 4.

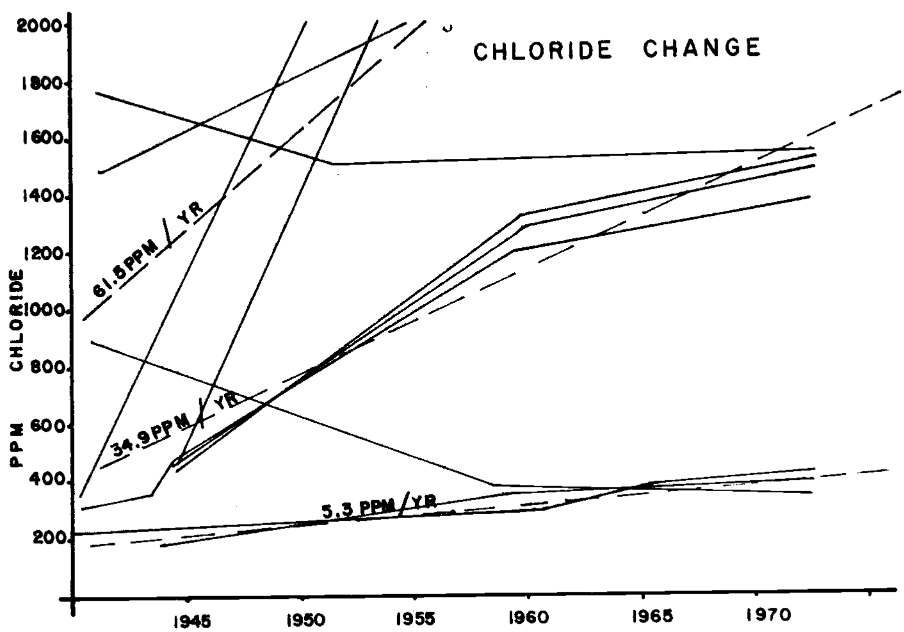


Figure 5.

Cotton is the primary agricultural product of the region, with an estimated 15,200 acres in production, or nearly half of the 32,350 acres of agricultural land (Muller, 1973). Cotton is at the same time the most salt-tolerant crop (with the exception of barley). Of the $173 per acre profit on cotton, $84.40 per acre is a government subsidy payment. Because of this payment, the replacement and no profit levels of the crop are pushed well out of the time range within which this study will make projections. It is safe to say that under ceteris paribus conditions the economic cultivation of cotton is under no danger for a significant time into the predictable future. Further, since cotton is the principal economic element in agricultural profits, the agriculturally-based economy, under these same conditions, is sound.

INTERDISCIPLINARY IMPLICATIONS

Sociologic analysis showed that, in general, the local farmers, based on 41 field interviews (which constitute 25 percent of the farms having 10 or more acres under cultivation), were cognizant of, yet showed little concern for, the water quality situation influencing their land (Muller, 1972). Many did not use salt-control techniques such as furrow shaping or efficient irrigation methods.

An inter-system flow model was constructed in an attempt to synthesize the sociologic, economic and physical systems which operate in the valley. System elements were broken down into state variables, internal characteristics of the system; and functional relationships; working system components. The interconnectors of the three sub-systems mentioned were isolated to be crop yield (physical to economic), net income and operational costs (economic to sociologic), farming practices (sociologic to physical), and observable field characteristics (physical to sociologic). The only factor which is effected by farmer behavior that can affect salinity is pumping practices, an element of farming practices (Muller, 1973). This functional relationship includes the time, location and volume characteristics of pumping which are under the control of the farmer. It is obvious that when irrigation of a crop is required, both time and volume pumped cannot be controlled. Perhaps, if it is a large farm, the selection of which well to be pumped is left to the farmer. The principal point at which pumping practices can extend some leverage on salinity control is through the canal companies which supply water to many of the farmers, since they have wells pumping into the canals throughout the valley. They can exercise the choice of pumping in the areas proposed in this paper. In further development of the water resources, the canal companies may concentrate on the development of wells in the upper valley, and downstream farmers can then depend further on "individual characteristics," which is a rather ambiguous variable describing all learned and experienced behavior of the farmer. Thus, education on salt-control farming methods and on the actual local salinity situation is the method by which salinity control measures may be realized.

Both the ambiguity of "individual characteristics" and the difficulties in introducing modern salinity control methods to farmers are principal sociologic problems which will govern the resolution of the salinity problems in the area. Since there is no terminal danger for productive agriculture, such a resolution would be a slowing of the water quality decrement and the maintenance of high relative yields.

RECOMMENDATIONS

In general, the obvious primary recommendation is to employ all the high quality river water allowable by the Gila Compact (Greiner, 1971). This is currently being done. Second, the pumping regime proposed should be both currently phased in and considered in the further development of groundwater resources in the valley. Third, salt-control farming methods should be encouraged by farmer information programs; and further investigation in methods such as night sprinkler irrigation is suggested. Fourth, although currently sugar beets have disease problems in the region, continued research at the Safford Farm branch of the Agriculture Experiment Station (Turner, 1971) should be encouraged for the possibility of future re-introduction of sugar beets as an economic and extremely salt-tolerant crop. Fifth, well drilling practices should be encouraged which would avoid puncture of the artesian aquifer confining layer and wells that do intercept this layer should be plugged in a manner such that the flow of saline water is both not allowed to contaminate the irrigation aquifer and is not allowed to flow into the local water sources. It should be noted that these recommendations emphasize the ceteris paribus nature of the investigation and conclusions. Factors such as the introduction of extensive copper mining or fluctuation of market status of the crops concerned has not been considered. Under these conditions, the valley has a long and productive agricultural life for the future, and with the recommendations employed may have even higher yields and an even longer period of productivity. Planning should be the fundamental consideration of the region's inhabitants.

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ABSTRACT

THE SIGNIFICANCE OF LOGISTICS TO HYDROLOGY

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Theories and methods of logistics are useful for solution of some problems in hydrology. While depression storage is often mentioned as part of the rainfall runoff sequence, it is generally ignored in working models because past formulations proved too inflexible in practice. However, if the filling of depressions over a watershed by rainfall is viewed as a problem in logistics, then utilization of the depression storage concept becomes feasible. Another application falls in the realm of aquifer depletion: logistics can furnish a model of the depletion profile. Turning from theory to methods developed in logistic applications for estimating parameters of nonlinear functions, one finds that such methods deserve inclusion among those recognized in hydrology. Estimation of parameters for infiltration curves from field data, for example, can be accomplished advantageously with logistic methods. A bibliography is appended because here it is intended to introduce merely the potentialities of logistic studies.

ON THE STATISTICS OF HYDROLOGIC TIME SERIES

by

Sidney Yakowitz

and

Jack Denny

INTRODUCTION AND PRELIMINARIES

A good many naturally-occurring phenomena of interest to engineers and statisticians are described by a sequence of $\mathcal{L}$-valued measurements $\{x_i\}$ which, of course, we imagine to be a sample sequence of some stochastic process associated with a probability P. In general, experience, physical reasoning, and intuition all exhort one to conclude that future behavior of the process is statistically independent of the distant past, or, equivalently, that the process $\{X_i\}$ is rth order Markov, i.e., for all events A

$$P[X_n \in A | X_j,\ n-r \leq j < n] = P[X_n \in A | X_j,\ j < n].$$

The desire to understand the process (e.g., for purposes of prediction or decision making) is tantamount to inferring P (or perhaps just some property possessed by P). It is clear that toward this end, one can attempt to follow

Plan I: "Infer P directly from $\{x_i\}$."

On the other hand, we will forward reasons to suspect that the alternative route

Plan II: A) "Infer the Markov order r of $\{x_i\}$"

and then

B) "Infer P"

has many merits as an alternative to Plan I.

The chief drawback to Plan I in our opinion is that statistical schemes are unavailable and that further, the sample space is so unwieldy as to be forbidding to attempts at providing a comprehensive theory for direct inference, even in situations of relatively simple structure. A strength of Plan I is that (trivially) all admissible or optimal statistics for the inference problem must be within the category of Plan I, but not necessarily within Plan II, inasmuch as "optimal inference of parameter a, and then, assuming a to be true, optimal inference of parameter b" may not even be an admissible scheme for joint estimation of a and b. But on the other hand, Plan II has intriguing strengths--if only somehow we can satisfactorily solve problem A. For example, if the x_i's are real, then if problem A has been completed, we are left with a collection of vector-valued variables to analyze, which is a more conventional sort of problem. We can then hope that extant theories or relatively easy mathematical analysis might then prevail. In particular, if x_i takes on values in a finite set, then problem B reduces to finding multinomial parameters from independent trials and for this task, maximum likelihood estimators are easily calculated [26, p. 392] and the nicely behaved Dirichlet distributions provide a natural conjugate family if Bayes methods are preferred [10, p. 174]. We are familiar with the literature of river flow modeling and note that without exception authors have followed Plan II by either assuming in advance that they know the order of the chain analyzing the autocorrelation function, or proceeding without comment to Plan II. In our own studies in this area [27], Plan II has seemed the natural choice for analytical and computational methods as well as our desire to maintain a heuristic understanding about the behavior of our models.

Even having chosen to follow the simplifying Plan II, we are faced with delicate issues--most conspicously the lack of a satisfactory small sample theory for problem A. This paper is devoted to proposing and exploring a solution to this problem, with particular attention to the specialization of finite state chains.

With respect to problem A it should be understood that not only do we seek the Markov order, but we seek the minimum Markov order. To illustrate the need for this insistance we note that the length of physical record required for a certain tolerance of estimation is very sensitive to the Markov order. In the case that interests us most-- x_i taken from a finite set $\mathcal{A}$--roughly speaking, the length of record required for estimation of the Markov transition matrix to a given confidence interval is proportional to the number of non-zero transition probabilities in the matrix. Let M denote the number of elements in S. Recognizing that there are only M possible successors to a r-tuple $(X_1,\ldots,X_r)$, M^{r+1} parameters determine the transition matrix of an rth order chain. If it can somehow be shown that the order is r_2 rather than $r_1 > r_2$, then the required record length is reduced (roughly) by a factor of

$$M^{r_2-r_1}$$

For M=5, a reduction in order by 2 results in a reduction of required record by a factor of 1/25. In hydrology, one is often forced to work with a record of fixed length determined by the time when information gathering was first initiated and in such cases the magnitude of the order can be critical in determining the possibility of quantitative decision making. In fact, in light of our desire to handle hydrology problems which have confronted us and which involve relatively small (about 1000) chain lengths, we have decided to eschew large sample methods and not analyze asymptotic properties of various hypothesis testing schemes which come to mind.

Let us then formally define our

MAIN GOAL: For any given level α, find a hypothesis test for finding the minimum order r of a finite Markov chain, within the hypothesis that the Markov order is t. The test should further have attractive statistical and computational properties.

It is necessary to stipulate that we are operating within the hypothesis of some known order for the technical reason that without some such assumption, inference is hopeless. One could be faced with the situation that

$$X_{n+1}|X_n,\ldots,X_1 \;=\; X_{n+1}|(X_n,\ldots,X_{n-r-1}),\; X_{n-q}$$

where q>>r and q is greater than the length of record and any "reasonable" scheme would falsely accept that the hypothesis is of order r.

Approaches to the order reduction problem taken in the literature have followed two very different avenues--large sample theory and Bayes methods.

The former attack is described in the survey paper by Billingsley [3], wherein the principal tool of analysis is the chi-square method. This paper specifically studies the order reduction problem and has a rich assortment of references. More recently, Boza [5] has described another large-sample method for hypothesis testing in which the study of large deviations replace the chi-square technique. We have had experience with large sample methods [11]. It is to be admitted in their favor that they are easy to implement on the computer. But on the other hand, when we are forced to analyze a relatively short sequence, as has been our wont in our hydrology projects, our confidence in these techniques lag. The size of the sample may be too small for the chi-square approximation to be good.

Let us carefully develop an interesting comparison of the popular chi-square technique discussed in Billingsley [3, p.29] and references therein, which is applicable to the Main Goal and the usual chi-square method for testing for homogeneity of multinomial probabilities (discussed, for example, in Blum and Rosenblatt [4, p. 425]) i.e., the test for the hypothesis that

$$p_{nq} = p_{vq}, \text{ all } q, n, \text{ and } v.$$

where p_{vq} denotes the vth multinomial probability associated with the qth multinomial vector. Let N_{vq} denote the number of outcomes in category v of the qth variate, $1 \leq v \leq V$, $1 \leq q \leq Q$. $N_{v.} \equiv \Sigma_q N_{vq}$, $N_{.q} \equiv \Sigma_v N_{vq}$, $N = \Sigma_{v,q} N_{vq}$. The chi-square test for homogeneity is based on the fact tha' under the hypothesis, asymptotically, the variable

$$\Sigma_{v,q} (N_{v,q} - N{\cdot}q(Nv{\cdot}/N))^2 / N{\cdot}q(Nv{\cdot}/N) \tag{1.1}$$

is distributed as the chi-square law with (V-1)(Q-1) degrees of freedom.

It will transpire that the standard large sample method for testing the hypothesis H: "order r" within K "order t" is given by summing up terms of the form (1.1) over all possible r-tuples and thereby getting a chi-square variable with a larger degree of freedom. Specifically, let us adopt the convenient frequency count notation used by Dickey and Lientz [13] and agree that,henceforeth j denotes an $\mathcal{L}$-valued r-tuple, i denotes an $\mathcal{L}$-valued (t-r) tuple, and k is a variable over $\mathcal{L}$. With respect to some $\mathcal{L}$-valued observed chain, $n_{ij}(k)$ is the number of occurrences of (ijk) in the chain, n_{ij} the number of occurrences of (ij), etc. Then in this notation the variable recommended by Billingsly [3, (equation 6.2 with substitution as in his displayed equation below)] for testing hypothesis H is

$$\Sigma_k \Sigma_{ij} (n_{ij}(k) - n_{ij}(n_{jk}/n_j)^2 / n_{ij}n_j(k)/n_j \tag{1.2}$$

which, as proven in Goodman [17], has asymptotically a chi-square distribution with (I-1)(K-1)J degrees of freedom, I, J, and K being the size of the range of values of k, j, and k respectively. Let us hold r-tuple j fixed. Then with the identification that

$$n_{ij}(k) = N_{ki}, \; n_{ij} = N{\cdot}i, \; n_j(k) = N_k, \; n_j = N$$

we see that the jth term of (1.2) is asymptotically a chi-square variable with (I-1)(K-1) degrees of freedom and thus in light of (1.1), the test (1.2) can be regarded as the chi-square test for the simultaneous homogeneity of multinomial probabilities over each of J different sets of multinomial experiments, each variable in the jth set being indexed by an i-value and taking on values in $\mathcal{L}$. Billingsly [3] and Goodman [17] have noted that tests for homogeneity of contingency tables and Markov processes hypotheses are similar, and statistics on these processes become asymptotically identical.

Our approach is both conditionally admissible and Bayes for testing H. Furthermore, our method has the especially nice property that the probability of Type I error is independent or parameter, and consequently the level of the test is not pessimistic or unnaturally large.

The Bayes approach to testing H: "Markov order = r" within $H \cup K$: "order = t" is nicely developed in the penetrating study by Dickey and Lientz [13] who set forth specific and computationally feasible rules for testing H when the prior is in a Dirichlet family. It turns out that their criterion too bears stong similarities to the methods to be expounded here. Let P_0 denote the hypergeometric probability with parameters displayed in the conditioning. One may then write their criterion as "accept if and only if"

$$\Pi_j P_0[\{n_{ij}(k)\} | \{(n_j(k),\ n_{ij})\}] > \Pi_j\ Q(n_{ij}, n_j(k)) \tag{1.3}$$

for Q a given function which depends on the prior. Our methods will lead to the criterion "accept if and only if"

$$\Pi_j\ P_0[\{n_{ij}(k)\} | \{(n_j(k),\ n_{ij})\}] > T_\alpha \tag{1.4}$$

for a constant T_α determined by the desired level of the test. We will avail ourselves of (1.3) to verify the conditional admissibility of (1.4). The obvious strength of criterion (1.3) is that it provides an admissible test, but the disadvantage, in view of our main goal, is that the distribution of the random argumment of Q is strongly dependent on the "true" parameter of H and thus the level of the test (which is determined by $\inf_{\theta \varepsilon H} E[Q|\theta]$, since the l.h.s. of (1.3) will be seen to be independent of $\theta \varepsilon H$) may be 0, or at least, one would anticipate that it is generally small. The books by Good [16] and Martin [20], while not containing material specifically addressed to our main goal, nevertheless provide discussions on Bayes methods which are related and may be useful.

The applied probability literature is rife with applications of Markov processes. A great assortment of practical examples are to be found in the texts by Bhat [2], Breiman [6], Feller [14], and Parzen [23], for instance. Let us pay homage to the origins of our statistical interests in the subject by documenting the popularity of Markov models in hydrology and its sister science, meteorology. Meteorologists use Markov chains taking values in {0,1} to indicate "rain" or "no rain" in precipitation models. We reference the papers by Weiss [25] and Lowry and Guthrie [19] on this point.

In hydrology there are a great many papers on mixed autoregressive moving average processes for describing the flow of a river past a given measuring station; i.e., if $\{x_n\}$ is the time series describing flow at epoch n, then $\{x_n\}$ is determined by the relation

$$x_n = \sum_{i=1}^{n} a_i\ x_{n-i} + \sum_{j=1}^{m} b_j\ e_{n-j}$$

where $\{e_n\}$ is some sequence of independent, identically distributed random variables. We mention the papers by Carlson et al. [7], Matelas [21] and Yevjevich [28] as well as the book by Fiering [15] on this model. Clainos et al. [9] have used Markov processes in an entirely different fashion for streamflow modeling. They are interested in describing a monthly series and discretize the range of flows to get finitely many states. Then they use a non-stationary Markov process to describe the discretized streamflow series, a different Markov matrix being associated with each month.

Pegram [24] uses a finite Markov chain to describe the effect of placement and outflow policies of dams on flood routing. Markov chains have long been used to describe water storage in dams, as related in the monograph by P. Moran [22]. Chow and Kaveiotis [8] use an autoregressive scheme to model annual surface runoff as a function of annual precipitation. Bagley [1] has described a more elaborate (but less specific) model for describing runoff on a daily basis.

Our attention to statistical aspects of Markov chains arises from our wish to find an accurate model for a river (specifically, the Rillito River) in an arid region whose behavior was clearly unexplainable by mixed autoregressive moving average Markov chains. We opined that a model of the more inclusive form [27]

$$x_{n+1} = f(x_n, \ldots, x_{n-r}) + e_n$$

where $\{e_n\}$ is an rth order Markov chain associated with precipitation (and motivated by the Markov models for meteorology mentioned above) would be a sufficiently rich class to describe our river. Being faced with finding the Markov orders, max $\{m,r\}$, we attacked this problem by using the standard large sample schemes [11], but with some lack of confidence in our results. Our attempts to sharpen the above study and the numerical results obtained by applying our analysis to our Rillito River data comprise the body of this article.

A PROPOSED CLASS OF TESTS

We continue to let i, j and k denote $\mathcal{L}$-valued vectors of length t-r, r, and 1 respectively and, with respect to some $\mathcal{L}$-valued sequence, $n_{ij}(k)$ denotes the number of occurrences of ijk, $n_j(k)$, n_{ij}, etc. being similarly defined. $D \equiv \{n_{ij}(k)\}$.

Our intention is to show that for a certain prior probability on the space of transition matrices and a certain function Q, the critical function

$$\phi(\{n_{ij}(k)\}) = \begin{cases} 1, & \text{if } (\Pi_{ijk}\, n_{ij}(k)!) > Q(\{n_{ij}\}, \{n_j(k)\}) \\ 0, & \text{otherwise} \end{cases}$$

is Bayes, and by virtue of a lemma to follow, it is consequently admissible. This will lead us to a scheme which is admissible conditionally on $D \equiv (\{n_{ij}\}, \{n_j(k)\})$ and have the advantage over other small sample schemes available in that its level is readily established. Certain other analytic and computational merits will be listed and in closing, we will point out how our test has certain approximations which are very easy to compute.

In the elementary lemma which follows, Θ denotes a parameter space for a hypothesis testing problem. ξ is a prior probability defined on the Borel subsets of Θ, and H and K are respectively the null and alternative hypotheses. The underlying space $(\mathcal{X},\mathcal{A})$ on which P_θ is defined is assumed to be finite, with $\mathcal{A}$ being the power set of $\mathcal{X}$.

Lemma: Assume for every Euclidean ball B, $\xi(B \cap H)$ and $\xi(B \cap K)$ have positive probability whenever the arguments are not empty. If $P_\theta(A)$ is continuous with respect to θ for each event A, then ϕ_ξ, the Bayes critical function with respect to ξ, is admissible.

Remark: The condition on ξ obtains whenever ξ dominates Lebesgue measure on the simplex $H \cap K$.

Proof: If, to the contrary, there were some rule ϕ better than ϕ_ξ, we would have

$$E_\theta[\phi(X)] \leq E_\theta[\phi(X)], \quad \theta \varepsilon H$$

$$E_\theta[\phi(X)] \geq E_\theta[\phi(X)], \quad \theta \varepsilon K,$$

with strict inequality at some θ_0. From the finiteness of $\mathcal{X}$, we have that for any critical function ϕ, $E_\theta[\phi(X)]$ is a continuous function of θ. Assume for now that θ_0 is in H. Then the strict inequality must also hold on some set $A = H \cap B$, where B is a sphere containing θ_0.

$$\text{Bayes risk under } \phi = \int_A E_\theta[\phi(X)]\, \xi(d\theta) + \int_{H-A} E_\theta[\phi(X)]\, \xi(d\theta) + \int_K (1-E_\theta[\phi(X)])\, \xi(d\theta),$$

and also

$$(\text{Bayes risk under } \phi) - (\text{Bayes risk under } \phi_\xi) \leq \int_A (E_\theta[\phi(X)] - E_\theta[\phi_\xi(X)])\, \xi(d\theta) < 0.$$

This contradicts our assumption that ϕ_ξ is the Bayes critical function. The argument for $\theta_0 \varepsilon K$ is identical.

Returning now to the problem posed in the Main Goal, our parameter space will be the set Θ of sequences $\theta = \{\Pi_{ij}\}_{ij}$ where for each ij, $\{\Pi_{ij}(k)\}_{k\varepsilon\mathcal{L}} = \Pi_{ij}$ is a multinomial parameter. With respect to the Markov chain, $\Pi_{ij}(k)$ tells us the probability that the string ij is followed by the element $k \varepsilon \mathcal{L}$. With this terminology,

$$\text{Hypothesis } H = \{\theta: \Pi_{ij} = \Pi_{i'j}, \text{ all } i, i', j\}$$

and

$$K = \Theta - H.$$

We will be assuming that the hypothesis testing is done on the basis of an observation of a chain $\{z_i\}$, of $z_i \in \mathcal{L}$, of fixed length n and a fixed starting state. Then $(\mathcal{L}, \mathcal{A})$ is finite as described above the lemma and

$$P_\theta[\{z_i\}_{i=1}^n] = \Pi_{ijk} \, \pi_{ij}(k)^{n_{ij}(k)}$$

is a continuous function of θ. The prior ξ of particular interest to us assigns probability a, $0 < a < 1$, to H and is arbitrarily defined on H with the restriction that it satisfy the condition of the lemma. We will take particular interest in a ξ such that under $\xi_\theta|K$, the Π_{ij}'s are independently distributed as the Dirichlet distribution with parameters (1, 1, ..., 1),

$$\text{i.e., } f(\Pi_{ij}) = s! \text{ on the simplex } \{\Pi_{ij}(k) \geq 0, \Sigma_k \Pi_{ij}(k) = 1\}$$

where s is the cardinality of $\mathcal{L}$.

Let us derive an explicit expression for the Bayes solution of the hypothesis testing problem. Let $D \equiv \{n_{ij}(k)\}$. Then the Bayes rule (for the 0 - 1 loss criterion) is "accept if and only if"

$$P[H|D] > P[K|D],$$

i.e.

$$P[H] \, P[D|H]/P[D] > P[K] \, P[D|K]/P[D]$$

or accept if and only if

$$P[D|H]/P[D|K] > \xi[K]/\xi[D]. \tag{2.1}$$

Let us recall a modification of Whittle's formula for the frequency count D of a Markov chain with parameter θ (e.g., Goodman [17, p. 483])

$$P_\theta[D] = T(D)(\Pi_{ij}[n_{ij}! \, / \Pi_k n_{ij}(k)!])\Pi_{ijk}(\pi_{ij}(k))^{n_{ij}(k)},$$

where T(D) is a function whose exact form will not concern us.

Under the prior we have been discussing,

$$P[D|K] = \int_k P_\theta[d] \, d\xi(\theta),$$

and so

$$P[D|K] = \Pi_{i,j}\left(\frac{n_{ij}!}{\Pi_k n_{ij}(k)!}\right) \left(\int_K \pi_{ij}(k)^{n_{ij}(k)} \, d\xi\right) T(D)$$

which, from the complete integral of the Dirichlet density, gives

$$P[D|K] = [\Pi_{ij}[(n_{ij}!/\Pi_k n_{ij}(k)!)] \frac{\Pi_k n_{ij}(k)!}{(n_{ij}+s-1)!} \; T(D)$$

$$= (\Pi_{ij} \, n_{ij}!/(n_{ij}+s-1)!) \; T(D) \tag{2.2}$$

It is important to what follows to notice from above that

$$P[D|K, \{n_{ij}\}, \{n_j(k)\}] = C\ T(D) \tag{2.3}$$

C being a constant depending on $\{n_{ij}\}$, $\{n_j(k)\}$.

Continuing with the derivation of the Bayes criterion for testing for H, note that under H, $\pi_{ij} = \pi_{i'j}$ and so letting i' be some fixed member of $\mathcal{A}^{t-r}$ we have

$$P[D|H] = (\Pi_{ij}(\frac{n_{ij}!}{\Pi_k n_{ij}(k)!}))\ (\Pi_{jk}\int_H (\pi_{i'j}(k))^{n_j(k)} d\xi)T(D)$$

and the Bayes test is (from (2.1)) "accept if and only if

$$\Pi_{ij}[\frac{(n_{ij}+s-1)!}{\Pi_k n_{ij}(k)!}]\ \ \Pi_{jk}\int_H \pi_{i'j}(k)^{n_j(k)} d\xi \geq \xi(K)/\xi(H)\text{"} \tag{2.4}$$

which because of the integral is not a pleasant test, computationally. Worse yet, while (2.4) is an admissible test, the level may be pessimistic and in any event it is excruciatingly difficult to find.

We can make progress as follows: If we condition the test on the statistic $D' \equiv (\{n_{ij}\}, \{n_j(k)\})$ then

$$P[D'|H] = Q(D')\ \Pi_{jk}\int \pi_{i'j}(k)^{n_j(k)} d\xi(\theta)$$

where $Q(D')$ is the number of sequences $\{z_i\}$ satisfying the condition $D' = \{n_{ij}, n_j(k)\}$ as well as the initial state and as before, i' is any fixed t-r tuple. Now

$$P[D|D',H] = P[D|H]/P[D'|H] = T(D)\ Q(D')^{-1}\Pi_{ij}(n_{ij}!/\Pi_k n_{ij}(k)!). \tag{2.5}$$

Recalling the criterion (2.1), the Bayes test for H on the basis of the statistic D D' is: accept if and only if

$$P[D|D', H]/P[D|D',K] > \xi(K)/\xi(H)$$

Substituting the expression (2.5) we have just developed and repeating (2.3), the above test becomes: accept if and only if

$$(\Pi n_{ij}(k)!)^{-1} > V(D')\ (\xi(K)/\xi(H))$$

where $V(D') = (\Pi(n_{ij}!)^{-1})C\ Q(D')$. It is important to note at this point because $\xi(H)$ may vary continuously on the unit interval by proper choice of ξ, any test of the form: accept if and only if

$$(\Pi\ n_{ij}(k)!)^{-1} \geq t \tag{2.6}$$

for any fixed constant t is admissible (with respect to the statistic $D|D'$).

It is instructive and reassuring to verify that test (2.6) agrees with our unconditionally admissible test (2.4) in the sense that (2.4) can be rewritten in the format of (2.6). Specifically, we may write the unconditionally admissible test as: accept if and only if

$$(\Pi\, n_{ij}(k)!)^{-1} > M(D')$$

where

$$M(D') = \Pi_{ij}[(\int \Pi_{i'j}(k)^{n_j(k)} d\xi(\theta))\ (n_{ij}+s-1!)^{-1}]\ \xi(K)/\xi(H).$$

At this point we have provided a Bayes test for H (and hence an admissible test) which has allowed us to get rid of the bothersome integrals which appeared in the earlier criterion (2.4). But the above test (2.6) is still short of the Main Goal because we have not resolved the question of level. But actually, since D' is a sufficient statistic for $\theta \varepsilon H$ (as is readily confirmed by recalling Whittle's formula) the probability of rejection does not depend on $\theta \varepsilon H$. Thus

$$P_\theta[D|D'] < t$$

$$= P_\theta[\text{type I error}]$$

$$= \text{constant, in } \theta \varepsilon H.$$

This gives us the important conclusion that the <u>level of the test is constant for $\theta \varepsilon H$</u>.

More may be derived from this line of reasoning. As is shown in Denny and Yakowitz [12], the test is similar (in the terminology of Lehmann [18]), and for testing for order n-1 given order n, $P_\theta[D]$ is boundedly complete. As our test is admissible among similar tests and the boundary is H itself, we are able to derive that the test is <u>unconditionally</u> admissible.

While we have shown above that the level is constant on the hypothesis set, it would be very difficult to compute this common value. But following ideas of Goodman [17], one may sidestep this trouble by making a very useful approximation.

Define

$$P_0(D|D') = \Pi_j[\frac{(\Pi_i n_{ij}!)}{n_j!}\ (\Pi_k n_j(k)!)]\ (\Pi_{ijk} n_{ij}(k)!)^{-1}$$

which, the reader will recognize, is the probability of frequency count $\{n_{ij}(k)\}$ in some independent contingency table, homogeneous in j and with each given dimensions $\{n_{ij}\}$, $\{n_j(k)\}$. Obviously, our test (2.6) is for an appropriate t', the same as the test: accept if and only if

$$P_0(D|D') > t'.$$

But Goodman [17, sections 3 and 4] has proven that for every $\theta \varepsilon H$,

$$P_\theta(D|D') = P_0(D|D')\ Q'(D)$$

where Q' is a function which converges to 1 in probability as the chain length increases. Consequently, for each $\theta \varepsilon H$, and chain of sufficient length,

$$P_\theta[\text{reject}] \;=\; P_\theta[P_0(D|D') \le t'] \simeq P_T[T \le t'] \;=\; F_T(t') \qquad (2.7)$$

where T here (in contrast to prior usage) is the induced random variable $T = T(V)$ for V a random frequency count having probability function $P_0(\cdot|D')$.

Let us summarize these findings in a theorem.

Theorem: The test with the critical region

$$D: \; P_0(D|D') \le t \; . \qquad (2.8)$$

is, for each t , both Bayes and admissible for testing H on the basis of the statistic $D|D'$. The level is constant for $\theta \varepsilon H$ and for long chains, this common level is close to $F_T(t)$, T being the variable defined in connection with (2.7).

The above test (2.8) is the one we advocate for the Main Goal. (2.8) will be our fundamental mechanism toward arriving at the Main Goal. Let us list a few positive properties of this algorithm.

1) It achieves a level α test for the approximating contingency table problem. (Randomization may be necessary.)

2) Conditionally (on $D' = \{n_{ij}, n_j(k)\}$) the test is admissible for testing H.

3) The test is Bayes against a "non-informative" prior for testing H.

4) Conditionally (on D') the acceptance region has the smallest cardinality among all tests of level α.

5) The conditional test satisfies the Neyman Pearson criterion for testing H against the uniform distribution on the points in the range of $D|D'$.

6) The test is not difficult to implement, as we will see. Approximations to the fundamental method can be implemented by desk calculator.

7) Asymptotically, the test coincides with the likelihood ratio test for simultaneous homogeneity of several contingency tables (and consequently is the usual chi-square test for Markov order for reasons discussed in section 1).

And some negative properties are:

1) The test is not unbiased (but we suspect that generally, unbiased tests are of the trivial type--no real use is made of D).

2) The test is not minimax (but the minimax action is independent of D).

NUMERICAL APPLICATIONS TO ARTIFICIAL AND ACTUAL DATA

The purpose of this section is to share with the reader some experience concerning how well the test in (2.8) works, and in particular how it compares with the usual chi-square test for Markov order which we have discussed and referenced in section 1. It transpires that the test is relatively sensitive. For example, if the transition probabilities $\Pi_j(k)$ are chosen uniformly, the value set $\mathcal{A}$ is {0,1}, the order n is 5, and the length of the chain is at least 200, then in the overwhelming proportion of times, the test rejects at the 5% level the hypothesis that the order is 4. Again with data generated so that the order is 5, our test seems clearly superior to the chi-square test in sensitivity.

In use on actual daily streamflow data from a river in the Tucson basin and a monthly record of the height of a lake in Hungary, application of our test provided useful information.

First, some details of our algorithm and some numerical difficulties are considered. For purposes of computational ease we approximate $Q'(D)$, in (2.7), by 1 and thereby allow ourselves the liberty of using the more easily computed contingency table P_0 in place of the exact conditional probability of the Markov frequency count.

Let us give an algorithm realizing our basic test for H: "order is r" given "order is t." Assume we have an observation $\{z_i\}_{i=1}^{m}$ of the chain, and α is the desired level.

1) Compute the sufficient statistic $D = \{n_{ij}(k)\}$ and the statistic $D' = (\{n_{ij}\}, \{n_j(k)\})$.

2) Compute $P_0(D|D') \;=\; \Pi_j[\Pi_i n_{ij}!)(\Pi_k n_j(k)!)/n_j!(\Pi_{ik} n_{ij}(k)!)]$ (3.1)

3) Compute t_α, the α percentile of the variable $T = P_0(D_1|D')$ with D_1 a random sequence with probability mass function $P_0(\cdot|D')$.

4) Accept if and only if $P_0(D|D') > t$.

All steps but (3) are easily and quickly implemented by computer. In principle--and in problems with small s and t-r, in practice--step 3a can be achieved by the brute force method of computing $P_0(D_1|D')$ for every sequency $D_1 = \{m_{ij}(k)\}$ satisfying D' and then ordering these points in the domain of $P_0(\cdot|D')$ in increasing P_0 value, $D_1(1), D_1(2), \ldots$. The critical region is then the set $C_\alpha = \{D_1(v), v \geq j\}$ where (assuming no randomization is needed)

$$\sum_{v \geq j} P_0(v) \;=\; \alpha.$$

But in our applications we have found this procedure computationally prohibitive because the range of the variable D_1 is so large, even for moderately long chains.

A technique we have found adequate, easily programmed, and sparing in memory is simply to use Monte Carlo methods to estimate t_α. Specifically, let D and D' be the statistics computed from the sample and D_1 a variable over the range of statistics $\{m_{ij}(k)\}$ satisfying D', and let $P_0(\cdot|D')$ be the discrete density function for D_1. Then by using simulated observations of the variable $T = P_0(D_1|D')$ one can approximate t_α as closely as desired. That is, let $\{T_j: \ j \leq m\}$ be a sequence of observations of the variable T. Then the α percentile of the empiric distribution function constructed from this sequence converges to t_α as $m \to \infty$.

In this simulation we have suggested for finding t_α, some simple precautions can reduce the computational expense. Let us consider a few of these.

For each state j, define $D^j = \{n_{ij}(k)\}_{i,k}$ and define $D_1{}^j$, D'^j similarly. If

$$P_j(D^j|D'^j) \;=\; \Pi_{i,k}(n_{ij}!n_j(k)!/n_j!n_{ij}(k)!)$$

then clearly

$$P_0(D|D') \;=\; \Pi_j P_j(D^j|D'^j).$$

We show for each string j how to compute a simulated variable T^j such that $T^j = P_j(D_1{}^j|D'^j)$. Observe that $T = \Pi_j T^j$. Therefore, once the problem of simulating T^j is resolved, the problem of simulating T itself falls out.

Let $D_j = \{d_1,d_2,\ldots,d_m\}$ be the domain of $P_j(\cdot|D'^j)$. We calculate, order, and store the m values $P_j(d_v|D'^j)$ so that $P_j(d_{(v)}|D'^j) \leq P_j(d_{(w)}|D'^j)$ if $v \leq w$. To simulate T_j, call for a random number u, and then find the maximum w such that

$$\sum_{v \geq w} P_j(d_{(v)}|D'^j) > u.$$

Then $T^j = P_j(d_{(w)}|D'^j)$. We found that 500 synthetic observations of T^j, each j, gave us a sufficiently accurate estimate of the distribution of T.

In order to gain experience and confidence in our techniques for testing the Markov order, we applied the algorithm just described to simulated chains. The routine for generating the chains operated as follows: the input card set the order n and the length N of the chain. For each binary n-tuple j and each simulated chain, the transition probability $\Pi_j(1)$ was chosen uniformly (and of course $\pi_j(0) = 1-\pi_j(1)$). The initial state was held fixed for all simulations. Successive binary digits were added to an already developed chain C by concatenating 1 or 0 to C according to whether the next random number u is < or > $\Pi_j(1)$, j being the rightmost n-tuple of C.

In the tabulation below, we list the outcome of our test as follows: Having calculated the empiric distribution function F_T of T, we calculate the percentile q associated with $P_0(D|D')$, i.e.,

$$q = F_T(P_0(D|D')).$$

Actually we tabulate below 1 - q. With this transformation, we reject at the α level if and only if the tabulated number $1-q \geq 1-\alpha$. We have tabulated 1-q rather than q itself for the purpose of comparing our test with the usual chi-square test discussed earlier. Thus for each simulated frequency count D, we have calculated (as per equation (1.2)) its chi-square statistic and then the associated percentile of the chi-square statistic. Of course, rejection occurs at the α level whenever the chi-square statistic percentile exceeds $1-\alpha$.

Table I

A Sequence of Tests on Simulated Chains
(All chains have order 5 and length 200)

Simulation	Percentile Order 5 ours	Percentile Order 5 χ^2	Percentile Order 4 ours	Percentile Order 4 χ^2
1	.13	.00	.99	1.0
2	.12	.00	.99	1.0
3	.98	.01	.99	1.0
4	.66	.01	.99	1.0
5	.09	.00	.99	.99
6	.18	.00	.99	1.0
7	.40	.01	.99	1.0
8	.13	.00	1.0	.99
9	.17	.01	.99	1.0
10	.69	.10	.99	.89
11	.88	.00	.98	1.0
12	.94	.47	.99	.94
13	.26	.00	.98	.97
14	.05	.00	.98	.95
15	.76	.18	.99	1.0
16	.63	.08	.99	1.0
17	.61	.02	.97	1.0
18	.79	.01	.99	1.0
19	.06	.00	.95	.94
20	.25	.00	.97	.93
21	.28	.00	.99	1.0
22	.79	.05	.88	.76
Sample Average			.98	.97

We see in Table I that both tests are highly effective. At the 5% level, out of 23 simulations, our method makes only two errors, one of each type. The chi-square method accepts falsely five times and no errors of the other type. In cutting down the length of the chain, we see in Table II that our test performs with noticeably fewer errors than the chi-square test. The results we are reporting are in the order that they were performed, unedited, and, we think, they are typical.

Table II

A Sequence of Tests on Simulated Chains
(All chains have order 5 and length 120)

Simulation	Percentile Order 5 ours	Percentile Order 5 χ^2	Percentile Order 4 ours	Percentile Order 4 χ^2
1	.13	.00	1.0	.98
2	.23	.00	.99	.98
3	.52	.00	1.0	.99
4	.17	.00	.93	.42
5	.11	.00	.95	.95
6	.44	.00	.95	.38
7	.73	.02	.97	.98
8	.53	.01	.64	.34
9	.12	.02	.58	.65
10	.00	.00	.00	.00
11	.24	.00	.93	.65
12	.49	.00	.92	.60
13	.25	.00	.94	.15
14	.38	.00	.99	.95
15	.95	.09	1.0	.99
16	.10	.77	.98	.99
17	.13	.00	.99	1.0
18	.59	.00	.98	.90
19	.38	.00	.83	.89
20	.03	.00	1.0	.83
21	.71	.04	.89	.91
Sample Average			.88	.75

At the 5% level, the only type I error committed in Table II is by our algorithm (once), but our method made three fewer type II errors than the chi-square rule (9 to 12). At the 10% level, the number of type II errors (10) made by the chi-square method is twice as many as those commited by ours, which still only causes one type I error. Note the disparity in the sample average value of T vs χ^2 statistic percentiles under the alternative.

Let us now turn our attention to some actual data. We studied a sixty year (1900 to 1959) record of monthly recordings of heights of Lake Balaton in Hungary. Visual inspection of this data made it plain that the behavior during January through April differed markedly from the rest of the year. Therefore, only data from May through December was used and thereby sixty different chains with eight entries each were compiled. The chain was converted to binary entries by assigning 0 or 1 according to whether height was greater or less than average. To obtain D we made a frequency count of the pooled data, ignoring the transition from the end of one year to the first state of May in the next. The analysis of the data is summarized below.

Table III

Analysis of Markov Order of Lake Balaton

	Order 5 within 6	Order 4 within 5	Order 3 within 4	Order 2 within 3
Percentile of T	.85	.96	.98	.98
χ^2 Percentile	.085	.80	.98	.98

In Table III, we have copied the outcomes of tests of order k within the hypothesis of order k + 1, for k = 5,4,3,2. The test for order 5 uses a chain of effective length 3 x 60 = 180, so in view of our simulations, it should be fairly discriminatory. In any event, it would seem very likely that the order is not less than 5.

Next we consider a record of daily flow measured at a single station of the Rillito River, a river in the Tucson basin in Arizona. Here the behavior of the river suggests a definite seasonal pattern, with winter-type flow in January through March and summer flow in July through September. We study these two annual periods separately, again deriving frequency counts from the pooled data over the thirty years of record. We take 0 to correspond to no flow, 1 to correspond to positive flow.

Table IV

Tests for Markov Order of Rillito River

(December - March)

	Order 5 within 6	Order 4 within 5	Order 3 within 4	Order 2 within 3
T Percentile	.90	.97	.229	.96
χ^2 Percentile	.033	.634	.15	.98

(July - September)

	Order 5 within 6	Order 4 within 5	Order 3 within 4	Order 2 within 3
T Percentile	.43	.97	.99	1.0
χ^2 Percentile	.44	.98	.99	1.0

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THE COGNITIVE STRAWMAN PLANNING METHODOLOGY: PUBLIC INPUT

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INTRODUCTION

Consideration of human welfare in broader terms rather than in solely national economic efficiency has been an evolving trend in water resource planning and project evaluation. Quality of life, aesthetic enjoyment, leisure time opportunities and environmental quality are included as important goals for project planners. While this trend to broaden the planning perspective is being discussed and reviewed, attempts to develop methodologies to quantify non-economic criteria have been ongoing. The historical development of multi-objective planning usually begins with the report of May, 1950 known as the "Green Book" which sets forth benefit-cost analysis and consideration of objectives other than the "general economic welfare." This proposed practice called for identification of all beneficial and adverse effects of a project in both tangible (i.e., monetary) and intangible terms. The Green Book was never adopted by any federal body, but the basic philosophy of benefit-cost analysis was embodied in Budget Circular A-47 issues by the U.S. Bureau of Budget in 1952.Congress then began calling for a liberalization of the benefit-cost system. Senate Document 97 was a response to this need. It called for the three prime objectives of water and related land resource projects to be development, preservation and the well being of people. However, the unwritten rule of the Office of Management Budget was to maintain as their standard for approval the tangible benefit-cost ratio. Various positions proposed by the Water Resources Council in 1968 culminated in two publications called the "Orange Books." These gave the purposes of water and land planning as the enhancement of: national economic development, regional development, the quality of the environment, and social well being. Developments have continued in this general area of multi-objective planning, but none so closely linked to the governmental decision making process. Similar efforts to the Orange Books were Goals for Americans (The President's Commission on National Goals, 1960) and Toward Balanced Growth: Quantity with Quality (National Goals Research Staff, 1970). A third approach by the National Planning Foundation defined six broad goal areas: health and safety, education, skills and income, human habitat, finer things and freedom, justice and harmony, and gross national product.

Another type of related effort to this style of planning methodology is the identification of social indicators. The HEW circular Toward a Social Report (1969) dealt with identification of social indicators to measure the performance of society in meeting social needs. Social Indicators (Bauer, 1966) focused on the potential of using statistical information in decision-making related to social goals. The Technical Committee of the Water Resources Centers of the Thirteen Western States, on the basis of these developments, formulated an applied systems oriented approach to present a comprehensive multi-goal methodology. The result was their report entitled Water Resources Planning and Social Goals: Conceptualization Toward a New Methodology, hereafter referred to as "Strawman."

The Strawman Planning System is a modeling concept relating water resource use to "social goals." The core of this planning system is a hierarchical array of societal goals with nine prime goals at the top, descendingly disaggregated into further categories called sub-goals and concluding with measurable physical indices called social indicators.

Figure I presents a schematic of this general plan. This report is the foundation of our work in its philosophy and design. The nine prime goals of the Strawman are:

1. Collective Security
2. Environmental Security
3. Individual Security
4. Economic Opportunity
5. Cultural and Community Opportunity
6. Aesthetic Opportunity
7. Recreational Opportunity
8. Individual Freedom and Variety
9. Educational Opportunity

It is the purpose of this paper to describe a further development of multi-objective planning methodology, the Cognitive Strawman.

COGNITIVE STRAWMAN

It was determined during a year long study by a tutorial team at the University of Arizona that in order for the Strawman to be effectively used and accepted as a valid planning tool, it needed revision in several areas. Two of these areas needing improvement were 1) the Strawman should reflect the way people, planners, and specialists think and 2) a system of weights or trade-offs must be determined between the various goals and sub-goals to reduce confusion resulting from the use of the Strawman.

The first area of concern resulted from the observation that the original Strawman attempted to link social, or rather technical, indicators directly with sub-goals of society. In order to obtain input directly from lay persons as well as specialists it would be necessary to create a new connection between social indicators and sub-goals. As a result an intermediate concept was developed which is called a mental indicator. Thus, the public should be asked about these mental or perceived indicators to ascertain their perceptual dimensions of selected sub-goals. An example may make this clearer. In the original Strawman, the achievement, or lack of achievement, of the sub-goal water aesthetics was measured by the social indicator turbidity. Such a term is physically descriptive but conveys little knowledge of perceived conditions to most people. We intend to introduce mental indicators as the lowest order sub-goals to circumvent this problem. In this example, one of the mental indicators for water aesthetics could be clarity, a term familiar to the general public.

The desire to achieve a weighting system is even more basic. To make the Strawman operational, a system is proposed to determine for specific groups of people their desired weights and trade-offs between the various sub-goals. Determining weights is necessary to reduce the amount of information from a taxonomy of 300+ social indicators to a more useful number of quantified sub-goals. This effort to define a system for producing weights on social goals has been called for by many experts in the field. Terrence P. Curran in "Water Resources Management in the Public Interest" calls upon the social scientist "to make greater efforts to describe community goals and social needs in measurable terms." (Curran, 1971, p. 38) A brief explanation of such a system has been proposed in an article entitled "What Will You Pay for Amenity?" The author states that figures used in planning analysis by civil servants should "mirror the community's underlying pattern of preference and value." (The Economist, May 23, 1970, p. 76) The public too is apparently aware of such a desire. David Brinkley, a prominent news correspondent, recently stated the need for a national poll to set budgetary priorities (Brinkley, 1973) Locally, the City of Tucson's Planning Department developed a Comprehensive Planning System whose objective was to identify problems, choose alternatives, and finally, select the best solution through extensive public input (City of Tucson Planning Division, 1972, p. 7).

The specific concern is to develop evaluative criteria for human goals. The steps needed to accomplish such an objective include the following: 1) human objectives must be enumerated and described, (note: the philosophy and design of this comes from the Strawman, although we will alter the specific terms and their description to better fit public perception) and 2) the various objectives must be compared in terms of their relative importance to each other and to human welfare in general. To structure these steps, we propose a planning paradigm which is based on the following statements.

1. Societal goals can be described by a dendritic structure consisting of nine main goals and numerous branches of sub-goals and sub-sub-goals.

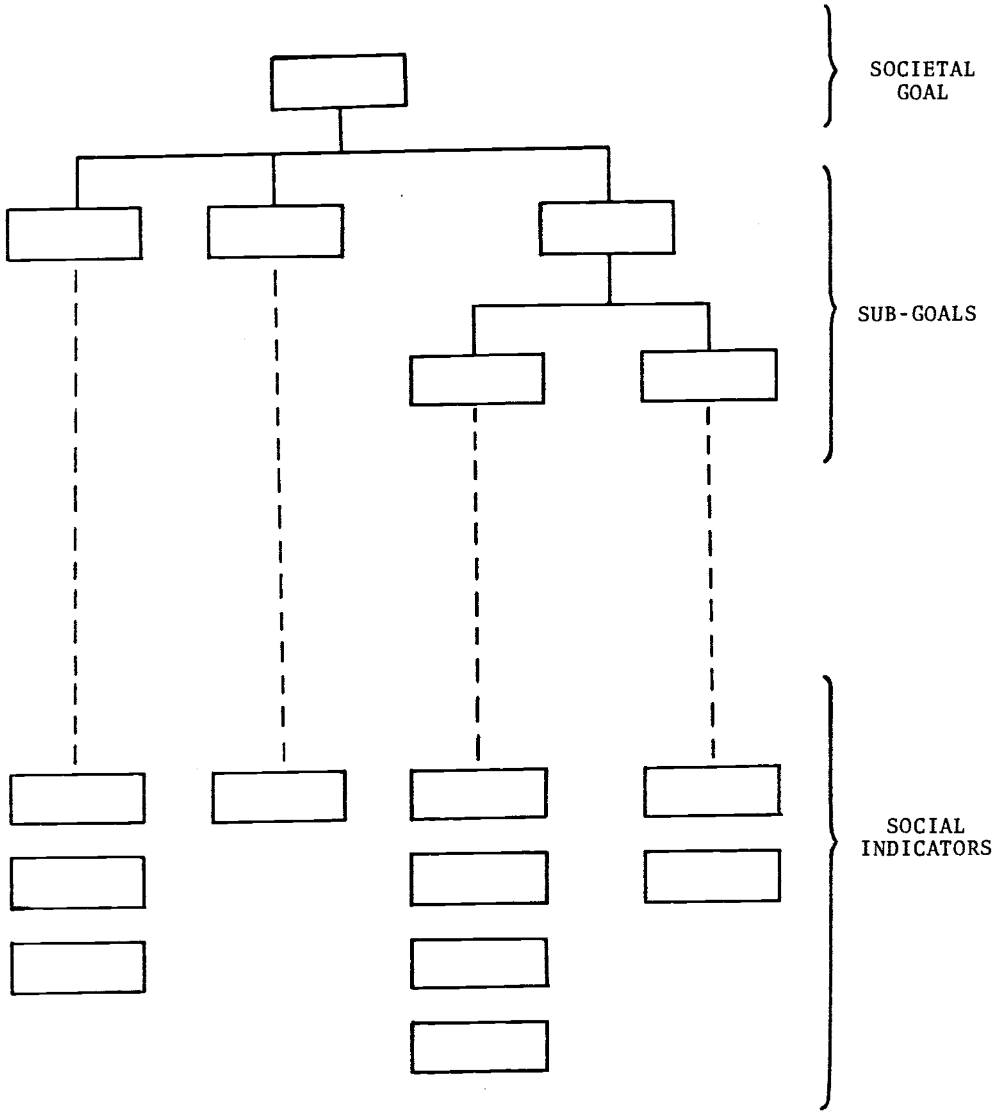


Figure I. General Design of the Strawman Planning System

2. People react to physical, economic, and social facts through their <u>perceptions</u> of these facts which are processed through their belief system.

The basis for acceptance of statement #1 is found in the Strawman. Statement #2 bases public reaction upon perception or cognition and requires further analysis.

PERCEPTION, ATTITUDES, BELIEFS, AND VALUES

Literature on the organization of human value systems is voluminous and confusingly overlapping. However, a recognized expert, Milton Rokeach,(1970) has presented a complete text on the subject in his book <u>Beliefs, Attitudes and Values, A Theory of Organization and Change.</u> Figure II, based on an analysis by Rokeach and Schiff, depicts the psychological process for preference attainment, or weight formation.

The mental process of weight formation (preference) begins with the subject or criteria to be evaluated. The first operation is physical perception (Schiff, 1971, p. 7) which is the mental-physical response to the stimulus. The next operation, social perception, is all inclusive and unique to the evaluator (Schiff, 1971, p. 8). This process makes use of three cognitive systems, attitudes, beliefs, and values, to interpret recognition or physical perception. Rokeach's definitions of these terms are:

> Attitude - an organization of beliefs around a subject. Virtually all theorists agree that an attitude is not a basic irreducible element within the personality, but represents a cluster or syndrome of two or more interrelated elements (p. 112).
>
> Belief - an inference made by an observer about underlying states of expectancy or any simple proposition, conscious or unconscious, inferred by what a person says or does. Rokeach conceives these beliefs to be arranged along a central-peripheral dimension containing five classes. They are:
>
> A) Primitive Beliefs - 100 percent Consensus. Non-controversial psychologically incontrovertible beliefs with unanimous social consensus, i.e.,"I believe this is a table."
>
> B) Primitive Beliefs - Zero Consensus. Psychologically incontrovertible potentially controversial beliefs without social approval, i.e., "No matter what others believe, I believe in God."
>
> C) Authority Beliefs. Potentially controvertible beliefs based on acceptance of personally known reference persons, i.e., family, class, peer group, religious group, etc.

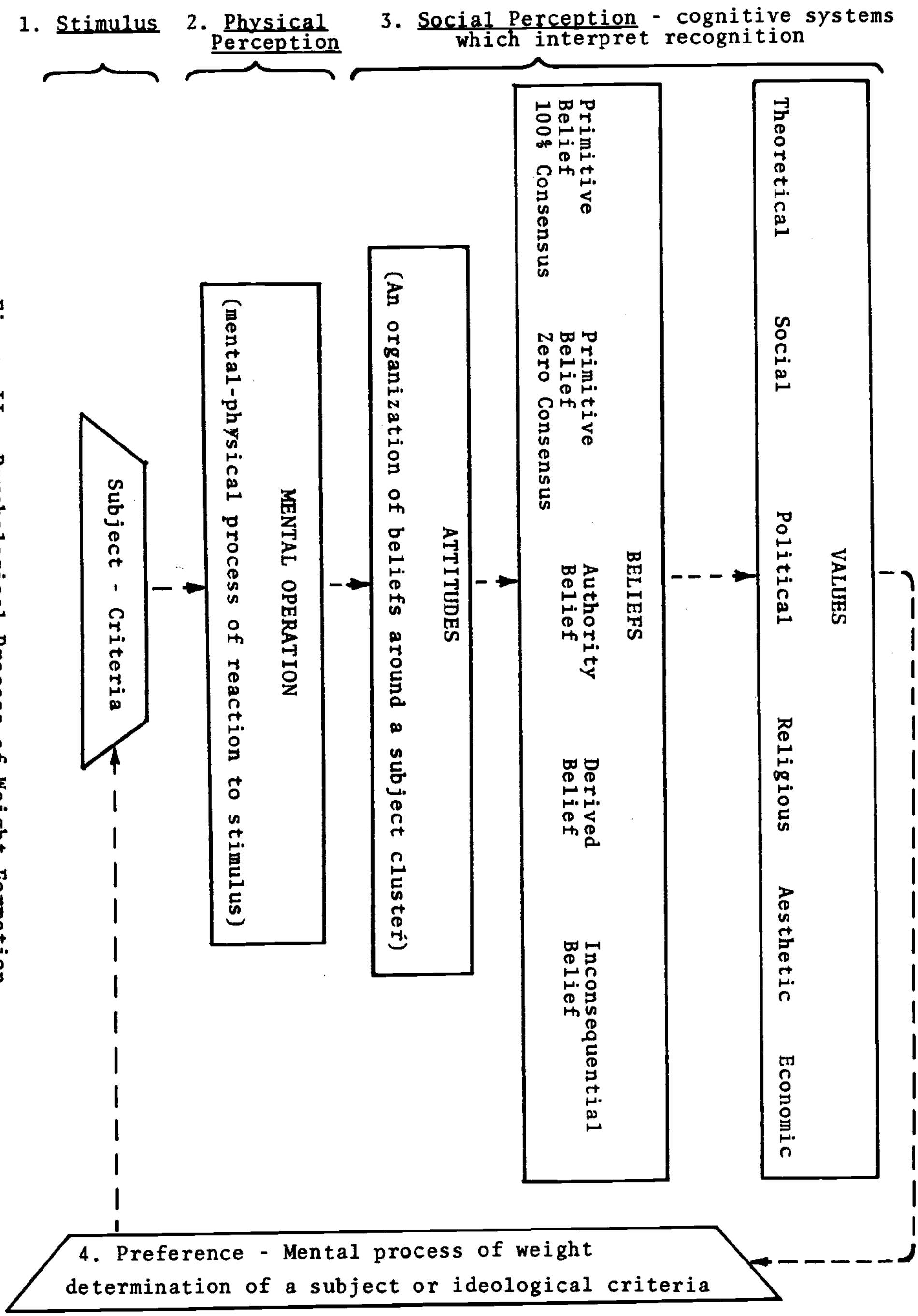


Figure II. Psychological Process of Weight Formation

D) Derived Beliefs.
   Potentially controvertible beliefs derived second-hand through processes of identification with authority.

E) Inconsequential Belief.
   Beliefs which represent arbitrary matters of taste.

(Note: In context of this planning paradigm only the last four categories of beliefs are relative to evaluation of social goals.)

Values - A type of belief, centrally located within one's total belief system (or cognitive system), about how one ought or ought not to behave, or about some end state of existence worth or not worth attaining. Values are thus abstract ideals, positive or negative, not tied to any specific attitude object or situation, representing a person's beliefs about ideal modes of conduct and ideal terminal goals. The Allport-Vernon-Lindzey Scale of Values distinguishes six classes of values: theoretical, social, political, religious, aesthetic, and economic (Rokeach, p. 125).

Proceeding with the process of preference formation, social perception begins with attitude evaluation of the recognized subject area. Processing this attitude, so as to be consistent with one's fundamental belief system, results in the potential for evaluation based on one's value system. The result can be considered to be a weighted preference on the subject.

ELITIST VERSUS POPULIST INPUT

Given the above structure for formulation of weighted preferences, two alternatives exist for enumerating and describing human goals and sub-goals. One is the elitist approach as taken by the Technical Committee in the original definition of the Strawman. The philosophy here is that specialists (i.e., the planning elite) are more capable of evaluating realities based on fact rather than cultural interpretation. Upon re-examination of Figure II, we note that the cognitive systems which are utilized in preference formation must be processed through one's fundamental beliefs and value systems. Thus, the planning specialist would have no inherent abilities over anyone else when it comes to evaluating abstract social goals. As Lowell E. Gallaway (1971) has noted, beneath the assumption of the elitist approach lies a belief in the minds of those who suggest it that they are possessed of a superior capacity for determining what "ought to be done" by others. In short, he says, it is a manifestation of a basically autocratic, if not authoritarian, mentality.

The second alternative for evaluating community goals is the direct use of public opinion for criteria assessment and weight assignment. According to Terrence P. Curran (1971) probably the most visible and immediate force pervading the water resources profession today is public concern for the environment. This concern, he feels, is leading to important changes in basic ideology. Curran notes that United States citizens in the last several years have become increasingly disenchanted and alienated (steming from disruptive events such as the Viet Nam War, student unrest, etc.) from the traditional political process, and he concludes that, if social needs are to be met, the public must be involved. Curran makes the observation: "The greatest challenge to those in the water resources field is to fully and forthrightly inform the public." (Curran, 1971, p. 34) Actually the idea of citizen participation is a little like eating spinach, no one is against it in principle because it is good for you. That is, in theory, participatory democracy rings true. However, implementation strategies cover a wide range of philosophies.

Sherry R. Arnstein (1969) describes a typology of eight levels of participation ranging from complete citizen power to citizen manipulation (see Table I). The typology itself is a descriptive continuum and the Cognitive Strawman, utilizing public input, would seemingly be located on Rung No. 4, Consultation.

Degrees of Citizen Power

8. Citizen Control
7. Delegated Power
6. Partnership
5. Placation
4. Consultation
3. Informing
2. Therapy
1. Manipulation

Table I. Eight Rungs of a Ladder of Citizen Participation

Public Opinion Defined. The term, public opinion, is in everyday usage, thus it contains features that weaken it as a tool of analysis. The usage of public opinion includes at least the implicit assumption that public opinion emerges as a dominant force qualitatively superior to the view of any single individual (Truman, 1951, p. 211). With the demand for more verifiable methods of studying social phenomena, this view, which was dominant during the 19th century poetic reverence of public opinion, has largely disappeared from academic literature, though not from the press. Thus we can say the following statement belongs to another period of academic exuberance:

"Towering over Presidents and state governors, over Congress and State legislatures, over conventions and the vast machinery of party, public opinion stands out, in the United States, as the great source of power, the master of servants who tremble before it." (Bryce, 1891, p. 255)

We can best approach the analytical view of public opinion by dividing the concepts into its component segments. For John Dewey (1927) the public are non-participants, or in his words, "Some human actions have consequences for others who are not immediate participants in these actions, the public are those 'others' who perceive or can be made to perceive these consequences." (Dewey, 1927, p. 17)

Opinion may be thought of in the simplest terms as the expression of an attitude on an issue or proposition (Truman, 1951, p. 219). (Note: opinion requires less mental processing than preference, see Figure II.) An individual may be said to have an opinion not only if he has expressed an attitude, but as well if he can express one when called upon to do so. Public opinion, therefore, consists of the opinion of the aggregate of individuals making up the public under discussion. It does not include all the opinions held by such a group of people, but only those relevant to the issue or situation that defines them as a public. Public opinion, it follows, is specific to a particular set of conditions.

Public Opinion Input. The history of public input to government policies in the United States has been primarily by interest group. The two, public input and interest group input, are so related that Katherine Warner (1971), in her report "Public Participation in Water Resources Planning" equates the two terms. Public input is not theoretically so restricted. Both direct communication with an elected official or bureaucrat (by letter, telegram, phone, etc.) and formal testimony by private citizens during a public hearing can represent public input unconnected to an interest group. But in general, the best organized and certainly the largest amount of public input originates with the well coordinated articulate advisory arguments of interest groups. While this type of input and the aforementioned methods of individual citizen input do cover the range of public opinion, they convey little of its quantitative nature. Only some type of public opinion survey or vote is quantitatively indicative of public opinion. Both of these quantitative systems could be used to ascertain opinion on substantive issues. Voting is primarily used for electing representatives though it is also used for such substantive issues as tax changes andbond issues. The public opinion survey method could also be used by a planning agency for quantitative analysis of substantive issues. The Cognitive Strawman system is a proposal to use the survey method to determine public opinion weights associated with societal goals. For a discussion of this method see "A Public Weighting of Four Societal Goals in Arizona and Oregon," (Kimball, et al., 1973).

The attempt is to retrieve public weights concerning the general goals of society and not opinions on specific technical matters. It is based on the theory that all people are capable of expressing an opinion concerning the general directions of society. Thus, the public and the planning elite are on an equal basis when they express their fundamental beliefs.

THE COGNITIVE STRAWMAN CONCEPT

In order to evaluate this proposal of publicly obtained weights concerning societal goals, a sub-set of the original Strawman's nine goals was selected. Based on the dual criteria of (1) relevance to water resource projects and (2) potential of retrieving public perception, three prime goals and one sub-goal were chosen. They are Recreation Opportunity, Aesthetic Opportunity, Economic Opportunity, and the Health Security sub-goal of Collective Security.

The prime goals, Collective Security and Environmental Security, were rejected on the basis that they are primarily technical and thus not conducive to public reaction. The remaining goals, Individual Security, Cultural and Community Opportunity, Individual Freedom and Variety, and Educational Opportunity are within the scope of public perception but have proportionately less relation to water resource proposals. The proposed public opinion survey, a mail questionnaire, had definite length limitations and therefore represented an additional constraint on the selection of the above mentioned goals.

In order to insure that these selected goals were logically enumerated and described, the following set of rules was adopted:

1. The structure of goals, sub-goals, and social indicators will be dendritic and non-overlapping in nature and will proceed from general, perceived goals to specific, technical indicators.
   (Public preferences can thus be obtained by asking trade-offs across the perceived, upper level of the structure while such goals and sub-goals can be physically measured by quantifiable indices at the bottom of the structure.)

2. Sub-goals and sub-sets of sub-goals may appear in more than one place in the overall structure.
   (While the "non-overlapping" constraint of Rule 1 applies within the disaggregation of any one goal, the appearance of similar sub-goals in different goal disaggregations is allowed.)

3. All sub-goals in any one category should be independent within the category.
   (Independence between sub-goals in any one category is seemingly inherent in the assumption that social goals can be disaggregated into specific components or sub-goals, and also the presence of such an

independence constraint facilitates public preference attainment in any given category. For example, if a category in Recreation Opportunity was defined to contain two components, crowding and quality of recreation sites, respondents might well experience difficulty in making viable preference judgments between the two if they considered crowding to be an integral part of the quality of recreation sites.)

4. The attempt is made to make the sub-goals in any category exhaustive.

(The disaggregation of a goal or sub-goal to a higher level of resolution (i.e., down the structure) should contain all components which are thought to comprise the goal or sub-goal. Admittedly, any sub-goal disaggregation presented herein is by no means exhaustive for all foreseeable Strawman applications, but the attempt is and should be made to provide the best possible weighted preference system.)

5. The maximum number of sub-goals in any one category must be 6 or 7 to facilitate the attainment of public preference.

(Psychologists estimate that 6 or 7 items is a functional limit for individuals to simultaneously judge independent variables. As the number is increased objects are generally grouped and then compared. (Schimpeler, 1967, p. 146))

Given the above rules for disaggregation, the research procedure to redefine the Strawman in terms of people's perceptions was based upon a lexicographic analysis, or content analysis, of the components of Recreation Opportunity, Aesthetic Opportunity, Economic Opportunity, and Health Security and analysis of the interviews by the researchers.

The specific methodology was to ask open-ended questions of the type, "Visualize air pollution and describe the components of your mental image" and "List five areas of economic concern which affect you personally and explain why." From such lists of goal descriptors and the relative frequencies which which our respondents mentioned each, our team selected a representative list and aggregated these publicly-obtained terms in such a way that they would be acceptable to planners and meaningful to the public. In accordance with the preceding rules for disaggregation, the goal descriptors were fitted into the structure. As a result, people's perceptions, balanced by planners' needs, are reflected in the resulting Cognitive Strawman.

A revised taxonomy of the Health Security sub-goal of Collective Security, the goals Economic Opportunity, Aesthetic Opportunity, and Recreation Opportunity follow as Figures III through VI.

Figure III Aesthetic Opportunity Goal Disaggregation

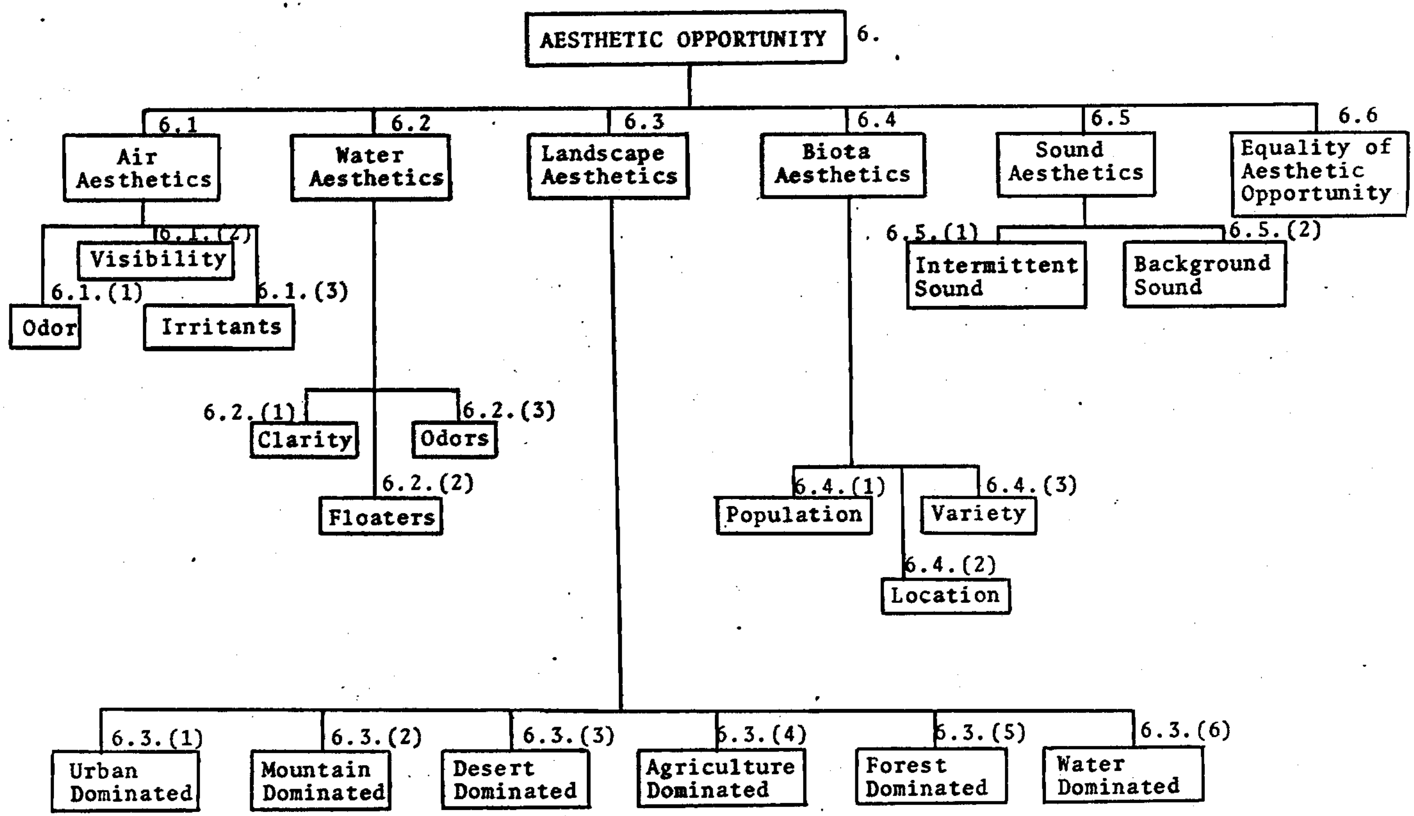


Figure IV Economic Opportunity Goal Disaggregation

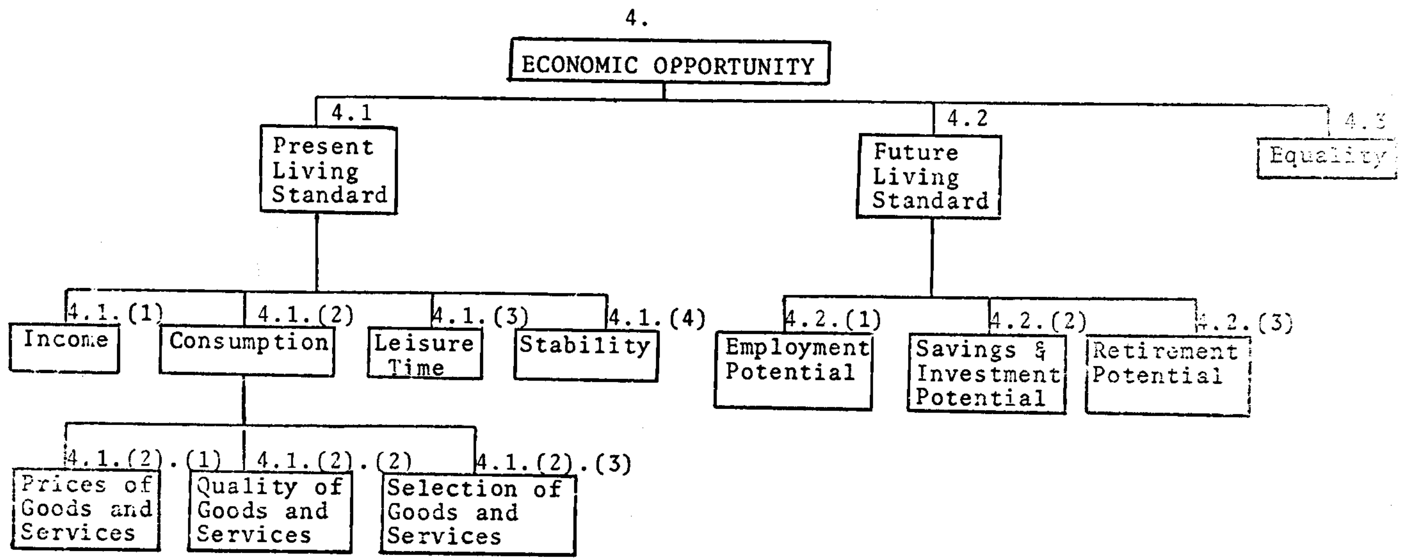
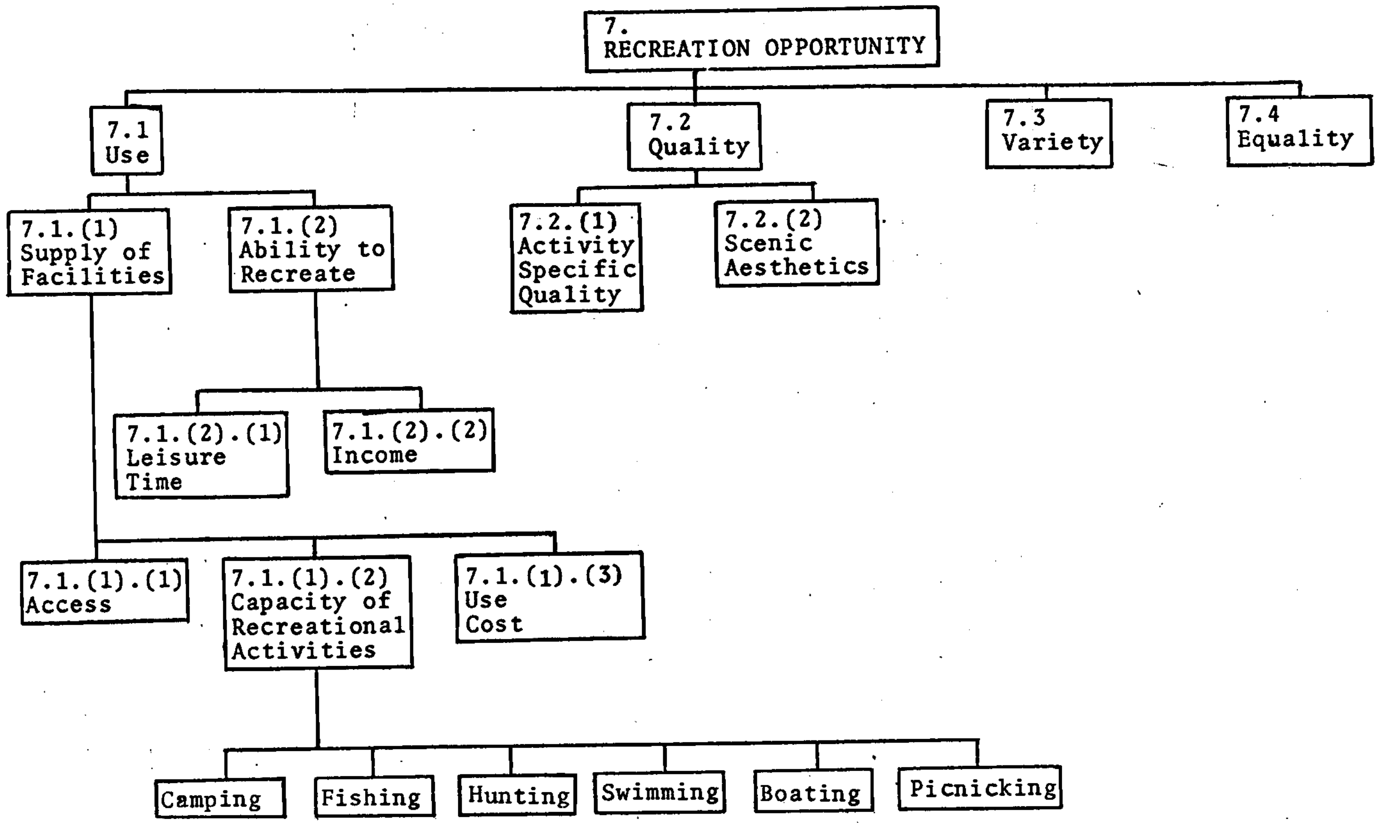


Figure V: Recreation Opportunity Goal Disaggregation



CONCLUSION

A simple model of the planning process should include the following steps:

1. Definition of the Problem
2. Information Collection
3. Development of Evaluative Criteria
4. Development of Alternative Plans
5. Selection of "Best" Plans, or less presumptuously a Set of "Good" Plans

The procedure discussed in this paper provides a fundamental framework for developing evaluative criteria, Step #3, emphasizing public input. It is hoped it will be a useful tool in evaluating societal goals by utilizing non-economic criteria. In addition, its use should aid in fulfilling the increasing demands of the populace for responsive open-channels to their planning specialists. In essence, it is an effort to supply an unfulfilled demand by the consumer of government services for planners to respond to his needs. In addition, however, it is believed the system is also compatible with the planning elite and will, if utilized, provide him with needed information to make informed decisions.

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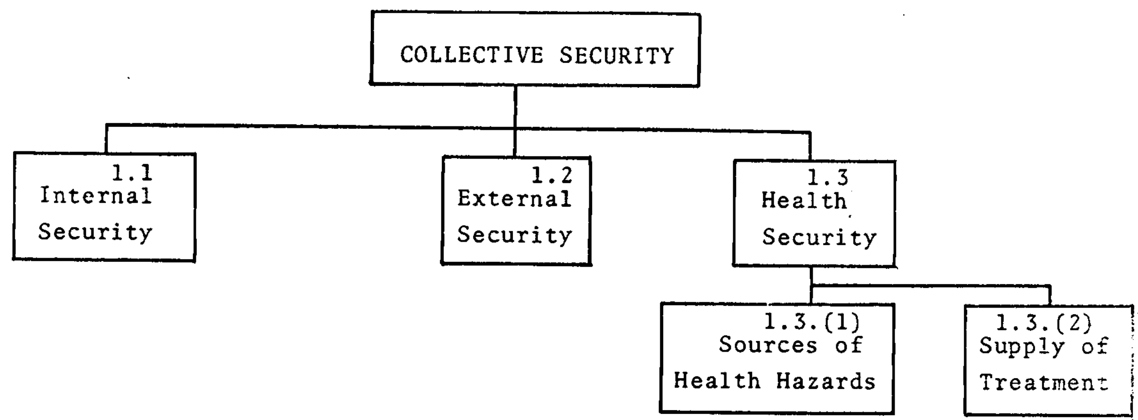
Figure VI Health Security Goal Disaggregation



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A PUBLIC WEIGHTING OF FOUR SOCIETAL GOALS IN ARIZONA AND OREGON

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In response to the increasing demand for multi-objective planning in the development of water resources, the Technical Committee of the Water Resources Centers of the Thirteen Western States has proposed a new planning methodology which relates water resource use to "social goals" (Peterson, et al., 1971). The planning system, referred to henceforth as the "Strawman," is based on a comprehensive set of nine societal goals which are delineated by an hierarchical array of sub-goals which are connected to water resource policy through social indicators, measures of socially significant phenomena. Provided that the form of connectives between social indicators, sub-goals, and goals have been defined (Gum, et al., 1973) and that the coefficients, or <u>weights</u>, of the connective algorithms are known, the Strawman is capable of providing aggregated information concerning the impacts of alternative plans on an array of goals and/or sub-goals to the policy maker. To incorporate weights into the Strawman planning methodology, two conceptual problems must be resolved. First, "who," the decision-maker, the general public, or interest groups, should be involved in weighting the improvement or attainment of specified goals? Second, what is the appropriate weighting methodology given the requirements of the Strawman planning system and the expertise of the designated individual(s) involved in the weighting process?

Mr. Weston Wilson, a Graduate Associate in Research at the University of Arizona, has addressed himself to the first of these two problems in his paper, "The Cognitive Strawman Planning Methodology: Public Input" (Wilson, et al., 1973). His conclusion is that general public input into the weighting of goals and sub-goals within the Strawman hierarchy is well justified and possible, provided that sub-goals are defined in such a way that they are easily perceived by the public. Mr. Wilson also states that three prime goals, Aesthetic Opportunity, Recreation Opportunity, and Economic Opportunity, and also a sub-goal of Collective Security, Health Security, are most amenable to public perception and also have special relevance to water resource development. Given these recommendations concerning the "who" and the specified goal areas as defined by the "Cognitive Strawman," the following discourse shall concern itself with the development of an appropriate weighting methodology and a test application in two states, Arizona and Oregon.

INTRODUCTION

Undoubtedly, the initial response to the development of a weighted preference methodology must be the basic question, "Can preferences be measured?" Measurement, as defined by Stevens (1946, p. 677), is the assignment of numerals to objects or events according to rules. The ease of application of such a definition to physical objects or processes is readily apparent, but measurement becomes more problematical when subjective entities are involved. Many psychologists have adopted Thorndike's dictum, "Whatever exists at all, exists in some amount," (Thorndike, 1918, p. 16) and have also adopted the corollary that whatever exists in amount, for example the quality of handwriting or the appreciation of a sunset, is measurable. Thus, the idea of measuring preferences in terms of the desirability of sub-goal improvements *per se* is not overly problematical, but the measurement of such non-physical attributes does imply the necessity of a rigorous consideration of two measurement enigmas. First, in the operation of scaling an attribute on a psychological continuum, what is the functional relationship between the method of scaling and the concept being measured? Second, given that a psychological measurement technique has evaluated a number of stimuli on a linear scale, what type of measurement scale has actually been achieved? Both of these questions have special relevance to the development of an acceptable weighted preference methodology and therefore must receive special attention.

A survey of the psychological literature yielded a plethora of methods, including ranking, rating, paired comparison, fractionation, magnitude estimation, and ratio estimation, all of which are capable of evaluating a stimulus on a linear preference scale. However, the nature of the Strawman planning methodology itself and also the requirement of obtaining weighted preferences from the general public produced constraints on the type of weighting methodology appropriate for inclusion into the Strawman's planning framework.

THEORETICAL CONSTRAINTS

The dominant theoretical constraint on the acceptability of a weighting methdoology is the level of measurement which must characterize the achieved preference weights and continua The proposed Strawman planning methodology is a quantified planning structure utilizing numerical weights, mapped-in social indicator values, and a function to aggregate information. The dependence of such a system on quantitative relationships and manipulations requires that the achieved preference weights be analogous in nature to cardinal numbers. That is, the weights must exhibit the properties of cardinal numbers so that all foreseeable mathematical manipulations and comparisons within the hierarchical structure of the Strawman are permissible. The imposition of such a restriction requires that any acceptable weighting methodology must attain an order of measurement equal to that of the cardinal number system.

REVIEW OF MEASUREMENT SCALES

Based on the fact that numerals can be assigned to objects or events under different rules and operations, S. S. Stevens

(1946) recognized that different kinds of measurement and different kinds of scales could be produced. As a result, he advanced a theory concerning scales of measurement which has as its basis the concept of invariance. That is, after a set of numbers has been assigned to reflect the outcome of a series of empirical operations, such as orderings, comparisons, or balancings, the type of measurement achieved can be ascertained by determining how the scale values can be transformed without the loss of empirical information. Using this criterion and the idea that the type of scale achieved depends upon the character of the basic empirical operations performed, Stevens developed a "Classification of Scales of Measurement" (Stevens, 1959, p. 25) which includes four scales of measurement, nominal, ordinal, interval, and ratio.

The lowest scale of measurement, nominal, requires only the determination of equality and thus represents the most unrestricted assignment of numerals. Although nominal scaling is not always thought of as a form of measurement because of its use of names or letters to designate categories and classes, it does perform the important function of classifying and identifying.

Steven's next higher order of measurement, ordinal, not only requires a determination of equality but also a determination of greater or less. This second operation usually takes the form of rank-ordering, or ranking objects in terms of the magnitude of some characteristic without measuring the amount of the characteristic possessed by each object. For example, on Mohs' Hardness Scale, diamond is harder than corundum, but from this scale nothing can be said about the relative difference in hardness or how hard each is in an absolute sense.

An interval scale of measurement, such as the Fahrenheit or Celsius temperature scales, requires the determinations of equality, greater or less, and the equality of intervals or of differences. From this scale of measurement it cannot only be said that, for example, 70°F is warmer than 60°F, but also that 70°F is 10°F warmer than 60°F. The equal intervals of temperature, or degrees, are scaled by noting equal volumes of expansion of mercury in a column. However, the zero point on such a scale is purely a matter of convention or convenience. It cannot be said that 70°F is twice as warm as 35°F. The zero point is arbitrary as can be shown by the fact that the scale remains invariant under multiplication by a constant and addition of a constant (i.e., a linear monotonic transformation). Almost all common statistical measures are applicable with interval scales, provided that such statistics do not require the knowledge of a true zero point. Problems arise, however, in the use of such a scale in higher-order mathematical operations such as logarithmic transformations.

The highest level of quantitative description, the ratio scale of measurement, is possible only when operations have been made for the determination of equality, greater or less, the equality of intervals, and the equality of ratios. Examples of this scale of measurement include the Kelvin or absolute temperature scale, length, time intervals, and the cardinal number system. In all ratio scales, an absolute zero is always implied though the zero point on some scales (e.g., the Kelvin scale) may never be produced. Once a ratio scale is erected, it can be said, for example, that 6 inches is twice as long as 3 inches or $100^{\circ}K$ is twice as warm as $50^{\circ}K$. Length or the amount of heat can be thought of in an absolute sense or an amount more than a rational or true zero point. The ratio scale, whose form is invariant only under multiplication by a constant (e.g., converting meters to centimeters) is the most restrictive class of measurement and exhibits properties which allow the application of all types of statistical measures and mathematical equations and manipulations.

IMPLICATIONS TO A WEIGHTING METHODOLOGY

Given the previously cited need for weights which are analogous in nature to the cardinal number system and the previous review of measurement scales, it becomes apparent that a weighting methodology appropriate for inclusion into the Strawman's planning framework must exhibit measurement of the highest order, a ratio scale. However, it must be noted that although ratio scales have been utilized in psychophysical studies, the use of ratio scales in general psychological studies of attitudes and preferences has been the exception rather than the rule. Most psychological data are legitimately expressed only as interval scales. It makes little sense to speak, for example, of zero intelligence or to be able to say that one person is 1½ times as anxious as another. However, the idea of a zero desire for an improvement in a sub-goal does make sense, and the ratio comparisons of preferences are essential to the Strawman in terms of the mathematical requirements of the system. Thus, an acceptable weighting methodology, in terms of the theoretical constraints, must be capable of achieving a ratio measurement of sub-goals on a preference dimension.

PRACTICAL CONSTRAINTS

The inclusion of a weighting methodology into the Strawman's planning framework also requires that certain practical constraints be met. Given the rationale for general public involvement in the weighting process, it was decided that any acceptable weighting methodology must be conducive to a mail questionnaire survey. If the target groups had been decision-makers or interest groups, an interview type survey would have perhaps been more suitable, but the desire to query a random sample of the general public over a large area required, due to limited resources, the acceptance of a mail survey. Thus, an acceptable methodology, in terms of the practical constraints, must be short, easily understood, and answerable in an uncontrolled atmosphere.

PROBLEMATICAL METHODOLOGIES

Based on the theoretical constraint of the achievement of a ratio scale of preference, it was realized that the most common psychological measuring techniques were unacceptable. Ranking, the most efficient method of psychological measurement (Eckenrode, 1965, p. 183), requires only that judges arrange stimuli in order of increasing excellence, quantity, or preference with respect to the psychological variable under consideration. Such an operation, however, yields strictly ordinal information and says nothing about the distance or ratio between the ranks. Rating, the most common procedure for measuring human judgments (Schimpeler, 1967, p. 99), involves the location of stimuli on a presumably unique psychological continuum which has been supplied with descriptive phrases to indicate position. However, because judgments are not made in terms of an absolute numerical scale, the resulting evaluations of the stumuli exhibit only interval characteristics. An interesting off-shoot of the rating technique is the Theory of Signal Detection (TSD) rating method (Daniel, et al., 1971), a technique which purports to separate an individual's perception of a stimulus from his "judgmental criteria." However, like the results of simple rating methods, TSD also yields strictly interval information. Another common psychological measuring technique is the successive paired comparison test (Thurstone, 1927). In its typical application, every possible combination of two stimuli is presented to the subject. The number of times each stimulus is preferred to every other one is calculated, and through an information processing algorithm, an interval scale may be derived from the paired comparison judgments. From this brief review of the scales of measurement attained by the most common psychological techniques, it can be discerned that adherence to such a theoretical constraint, the achievement of a ratio scale, is rare in general psychological testing, and thus the investigation of more obscure psychological measurement techniques was required.

A review of the psychophysical measurement literature did yield three ratio-scaling techniques, fractionation (Stevens, 1959), ratio estimation (Stevens, 1959), and magnitude estimation (Stevens, 1966), which were seemingly appropriate for our use on theoretical grounds. However, definite difficulties were quickly discovered when the practical nature (e.g., the complexity of instructions and the difficulty of usage in a mail questionnaire) of each method were considered.

INVESTIGATED WEIGHTING METHODOLOGIES

Although none of the more common attitude and preference measuring techniques nor the better known ratio-attaining psychophysical techniques could be adopted to produce the desired weighted preference system, the literature did yield three techniques, the Thurstone Paired Comparison Test with

Rational Origin Assumption (Thurstone and Jones, 1959), the Comrey Paired Allocation Test (Comrey, 1950), and the Metfessel General Allocation Test (Metfessel, 1947), which were seemingly appropriate and therefore required further investigation.

It must be noted that these tests represent the two basic lines of development in scaling, "indirect" and "direct" (Ekman and Sjöberg, 1965). In general terms, indirect scaling, as represented in the Thurstone Paired Comparison Test, requires only a minimal amount of information from the subject and then determines the scale from the experimental data using a set of assumptions concerning variability. On the other hand, direct scaling methods such as the Comrey Paired Allocation Test and Metfessel's General Allocation Test require subjects to directly report quantitative judgments. The use of such direct methods requires the acceptance of the assumptions that subjects are capable of estimating quantitative relations between subjective experiences, and, in the case of ratio-attaining direct methods, observers can directly estimate the ratio of two or more psychological entities. A brief discussion of the three weighting methods and the test results of each follows.[1]

<u>Thurstone Paired Comparison Test with Rational Origin Assumption</u>. The Thurstone Paired Comparison Test with Rational Origin Assumption is a technique which utilizes the Law of Comparative Judgment (Thurstone, 1959, p. 34-49) to produce an additive measurement scale of subjective values and which also experimentally determines a subjective, or rational, origin. In such a test, subjects are asked to express their preferences in paired comparison-type questions between each of all single stimuli (e.g., A or B), all combinations of paired stimuli and all single stimuli (e.g., AB or C), and each of all combinations of paired stimuli (e.g., AB or CD). Then, using the Law of Comparative Judgment, all single stimuli and also all combinations of paired stimuli are assigned scale values. The rational origin, or zero point, is then simply determined by calculating the mean difference between the sum of the scale values of all pairs of stimuli (e.g., A and B) and the respective scale values of the combinations of the pairs of stimuli (e.g., AB). Thus, Thurstone's basic assumption is that the utility derived from both A and B (i.e., AB) is equal to the sum of the utility derived from A and from B separately. Providing one accepts this assumption, a zero point exists, a ratio scale of preference is achieved, and thus all mathematical manipulations are permissible.

However, the Thurstone Paired Comparison Test with Rational Origin Assumption has definite practical and theoretical shortcomings. First and foremost is the required length and redundancy of the test. The need to consider single-single, double-single, and double-double stimuli comparisons requires 21 paired

[1]For a more complete discussion, see Gum, et al., 1973.

comparisons for 4 stimuli and 55 paired comparisons for 5 stimuli and thus commonly evokes irreverent comments from subjects such as "redundant, boring, and idiotic." Another shortcoming of the Thurstone technique lies in its basic premise, the additive assumption. Although Thurstone does restrict such linearity to small composites of stimuli (i.e., AB or ABC), the assumption in certain cases must be regarded as questionable, and therefore the basing of a weighted preference system on such an assumption must be considered extremely problematical. On these grounds, the requirement of considering an unwieldy number of paired comparisons and the adherence to the questionable additive assumption, the Thurstone Paired Comparison Test with Rational Origin Assumption was rejected from further consideration.

Comrey Paired Allocation Test. The Comrey Paired Allocation Test utilizes the traditional paired-comparison technique, but requires more than a mere preference judgment between a pair of stimuli. Subjects are also asked to allocate 100 points, votes, dollars, etc., between stimuli pairs, and therefore, not only is simple preference information conveyed, but also the amount and ratio that one stimuli is preferred to another. The Comrey technique requires that all possible pairs of stimuli (e.g., 6 stimuli yields 15 pairs) be considered in the previous manner. Through an information processing algorithm developed by Comrey, such paired allocation data is transformed to a ratio scale of measurement.

The validity of this transformation was tested in a ratio estimation experiment of physical line length. In this experiment, 15 students and faculty at the University of Arizona were presented with pairs of lines of different length and asked to allocate 100 points between the lines in each pair to convey the ratio of the perceived line lengths. For example, if a 3 to 1 ratio was perceived, the appropriate allocation would be 75-25. The responses were processed using Comrey's scaling algorithm, and the resulting measurement scale of perceived ratios of line length correlated highly (correlation coefficient (r) = .975) with the scale of the ratios of true line lengths.

The implicit nature of ratio judgments in the Comrey test itself and the validation of the technique by achieving a very high correlation between judged physical ratios and actual physical ratios confirmed the capability of the Comrey Paired Allocation Test to achieve ratio scales of perceived, physical entities. However, it was realized that the adoption of such a method to obtain ratio measurement of a more subjective entity, preference, required the acceptance of the assumptions previously mentioned in regard to direct scaling methods.

Further testing of the Comrey Paired Allocation Test included an application of the method to groups of sub-goals from two of the Strawman's prime goals and inclusion in a Pre-Test questionnaire which was sent to 200 Arizonans. Although the general response to the questionnaire was relatively good (31% returned), definite practical and theoretical problems were readily apparent. The primary impediment of Comrey's technique is the requirement that every stimulus be compared with every other stimulus. The necessity of considering 15 paired comparisons for 6 stimuli is burdensome even for the most enthusiastic test-taker. Another problem inherent to all tests with a paired comparison format is the possibility of the appearance of intransitivities. The viability of paired comparison scaling methods rests on the premise that the subject's preferences are transitive. That is, if A is preferred to B, and B is preferred to C, then A is preferred to C. However, upon analysis of the scales of preference, as obtained by the Comrey test in the Pre-Test questionnaires, it was realized that highly anomalous results were occurring in many of the individual scores for the six stimuli (15 pair) questions. Further examination of the actual point allocations revealed that these anomalous results were due to the presence of intransitivities. That is, A was preferred to B, B was preferred to C, but C was preferred to A. A review of previous studies on intransitivity (Davis, 1958) revealed that there is no conclusive evidence for the existence of stable intransitivities, intransitivities that occur repeatedly with the same stimuli and observer. Therefore, based on these seemingly uncorrectable enigmas, the required length of the Comrey Paired Allocation Test and its apparent tendency to produce intransitivities, this test was rejected from further consideration as an acceptable weighting methodology.

Metfessel General Allocation Test. Like the preceding method, the Metfessel General Allocation Test utilizes as its basis the assignment of 100 points, and therefore, as Metfessel states, the subject "either actually or symbolically manipulates units of the ratio scale of cardinal numbers, so that his manipulation of the cardinal numbers expresses his judgments of quantitative relations among the items on a given dimension." (Metfessel, 1947, p. 230). More specifically, subjects are asked to distribute or allocate 100 points not just between two stimuli, as in the Comrey test, but simultaneously between all stimuli in question. The resulting assigned numbers serve as immediate sources of the ratio values between the psychological magnitudes corresponding to all stimuli.

Like our experiment with the Comrey test, a similar experiment using line length was conducted with Metfessel's test. The subjects were asked to allocate 100 points among five lines to express the apparent ratios between them. The results were of better quality than the ratios achieved by Comrey's test and yielded a correlation coefficient of .997 to the true line ratios.

Given the apparent ability of Metfessel's test to accurately portray perceptual ratios of physical phenomena and the acceptance of the assumptions of direct scaling methods, Metfessel's General Allocation Test was further tested by applying it to the dendritic structure of a Strawman prime goal in a section of the previously discussed Pre-Test questionnaire. The results of this portion of the questionnaire were encouraging. Respondents could apparently allocate 100 points simultaneously between a number of stimuli to express their desire for an improvement in each stimulus. It appeared that this method, while immediately yielding ratio relationships, was also very economical in terms of a subject's time and effort and thus well suited to the concise style required of mailed questionnaires. The scaling algorithm required with Metfessel's test is very direct for achieving individual ratio-scaled preferences and simply requires calculating the mean for group preference scales. However, this technique is not without minor shortcomings. First, as Metfessel himself notes, the method does require a fair degree of arithmetical sophistication of the subjects. The problem of respondent's allocations not summing to 100 was readily apparent in Pre-Test questionnaires. Other problems, which could be discerned in the Pre-Test and other experiments with Metfessel's technique, were the apparent duplication of an individual's pattern of dividing points among the same number of stimuli and also the apparent tendency of respondents to be more concerned about having their point allocations add to 100 than allocating points to accurately portray their preferences. However, it appears that if some of Metfessel's instructional aids, such as ranking stimuli before allocating points, are added, the occurrence of these problems can be minimized. Therefore, although there are shortcomings and assumptions that must be considered in the use of this technique, the Metfessel General Allocation Test appears to be, given the theoretical and practical constraints previously mentioned, the most acceptable weighting methodology for inclusion into the Strawman's planning framework.

APPLICATION OF THE WEIGHTING METHODOLOGY: ARIZONA AND OREGON

Based on the acceptance of the Metfessel General Allocation Test as an appropriate weighting methodology and the desire to query the general public via mail questionnaire in regard to improvements in sub-goals and goals as specified in the "Cognitive Strawman," a demonstration of the proposed weighting methodology was undertaken in two states, Arizona and Oregon. These states were deemed suitable sites for such a "test" because, in addition to providing different physical settings, it was thought that an analysis of the resulting weights could lead to some valuable insights into the apparent environmental policy divergence between Arizona and Oregon and could perhaps answer the following hypothetical question: "Are the desires of the people in two states different or are the governmental officials' desires different in regard to state policy concerning protection of the environment?"

Therefore, a questionnaire was developed which included the Metfessel General Allocation Test applied to groups of sub-goals in the dendritic structures of three prime goals, Recreation Opportunity, Aesthetic Opportunity, and Economic Opportunity, and a sub-goal of Collective Security, Health Security (see example below). The test was also applied to all three prime goals and Health Security simultaneously.

Example: Distribute 100 points to indicate your desire for improvements in the following aspects of air aesthetics: Visibility (the distance you can see), Odor, and Eye Irritants (eye discomfort caused by airborne substances).

Visibility	_____
Odor	_____
Eye Irritants	_____
Sum	100

A biographical section was included in the questionnaire to obtain socio-demographic data which was thought to bear relevance in regard to later analysis of the weights. More specifically, the questionnaire contained questions relating to age, sex, residence, race, years of education, occupation, employment sector, family income, political affiliation, and self-rating questions regarding environmental knowledge and environmental activity. A rating of ten state problems, which were thought to be appropriate to the areas under study, was also included.

To achieve a representative public weighting of the elements in the specified goal hierarchies, a random sample of individuals in the two test states was desired. Therefore, a random smaple (N=2500) of Arizonans was obtained from a private marketing firm, while the Oregon Department of Motor Vehicles provided co-workers at Oregon State University with a random sample (N=2000) of individuals in Oregon. Questionnaires, preceded by a "warm-up" letter and followed by a "reminder" letter, were then sent to individuals in these random samples.

PRELIMINARY RESULTS

After a 3½ week questionnaire-return period, the return rate of usable Arizona and Oregon questionnaires was 18% and 13% respectively. Analysis of the socio-demographic data revealed that individuals responding to the questionnaire were on the average much alike in both states and could be characterized as being more educated and having higher incomes than individuals drawn randomly from the two states. In regard to the interest concerning similarities or differences in the desires of Arizonans and Oregonians with respect to the protection of their environment, the following three indices were considered: the rating of state problems, the results of the environmental activity and knowledge questions in the biographical section, and the mean sub-goal and goal weights.

As can be discerned in Table 1, the individuals in the responding samples of the two states rated the seriousness of these ten problems in a very similar manner. More precisely, a comparison of the average rating scores for the two states yielded a correlation coefficient of .907 . Table 1 also includes the resulting rank of each state problem by state. An analysis of the ranking differences between the two states produced a Spearman Rank Correlation Coefficient of .952 (significant at .001).

TABLE 1

Average Rating and Ranking of State Problems by State

	Normalized Rating Scale (Σ=100)		Rank Order	
	Arizona	Oregon	Arizona	Oregon
Uncontrolled Growth	10.52	10.62	5	4
Water and Air Pollution	12.63	13.53	1	1
Taxes	12.23	13.47	2	2
Flood Control	8.06	5.83	8	10
Crime	11.99	10.88	3	3
Employment and Wages	9.50	9.68	7	7
Water Conservation	9.77	9.97	6	6
Drugs	10.54	10.13	4	5
Transportation	7.34	7.65	10	9
Welfare System	7.42	8.25	9	8

To determine if individuals in the Oregon responding sample rated themselves significantly higher on the environmental activity and environmental knowledge scales than responding Arizonans, a Chi-Squared Test at the 5% level of significance was applied to the results of these two questions in the biographical section of the questionnaire. The results of the test revealed that the state in which an individual resides and the environmental activity or knowledge levels are independent. That is, the responding individuals in Arizona and Oregon are not significantly different in the way they rate themselves concerning their environmental activity or knowledge.

Table 2, a comparison of the mean weights by state, reveals that only 5 of the 18 sub-goal groups have a different rank order for the two states and that the mean weights of Arizona and Oregon on the whole are not significantly different. More precisely, the results of a Student's T-Test at the 5% level of significance demonstrated that of the 61 sub-goals and goals weighted, only 18 of the mean weights are significantly different between the two states. In regard to some of the specific differences, it appears that responding Oregonians were more concerned with improvements in their leisure time,

TABLE 2

MEAN SUB-GOAL AND GOAL WEIGHTS BY STATE

	Arizona	Oregon
Recreation		
*Leisure Time	45.06	49.74†
Income	54.94	49.50†
Access	31.69	26.67†
Admission Cost	24.17	24.08
Capacity of Recreation Activities	41.14	47.72
Facilities	47.50	44.05
Ability to Recreate	52.50	54.80
*Camping	21.02	26.19†
Fishing	19.55	18.04
Hunting	11.50	12.95
Swimming	15.09	14.77
Boating	12.84	11.32
Picnicking	20.02	15.60†
Quality of Recreation Activity	38.82	35.48
Scenic Aesthetics	61.18	64.14
Supply and Ability to Use Recreation Facilities	29.50	29.98
Quality	25.88	24.29
Variety	18.68	19.25
Equality of Opportunity	25.94	25.72
Aesthetics		
Intermittent Sound	43.68	41.89
Background Sound	56.32	56.59
Visibility	43.31	37.78†
Odor	25.78	30.48†
Eye Irritants	30.91	30.59
*Clarity	38.68	35.09†
Odor	25.63	27.22
Floating Objects	35.69	37.31
*Urban	37.00	39.97
Mountain	10.70	8.13†
Desert	11.43	6.57†
Agricultural	9.51	10.54
Forest	10.88	12.25
Water	20.48	21.77

*Different sub-goal group rank order for the two states.

†Mean Weights significantly different between the two states at 5 percent level.

Table 2. Continued

	Arizona	Oregon
Aesthetics (Continued)		
Population	42.46	42.53
Variety	28.13	28.83
Location	29.40	25.60†
*Air	26.27	22.82†
Water	19.37	23.50†
Landscape	15.04	14.32
Biota	11.80	11.97
Sound	11.78	13.24
Equality of Opportunity	15.75	13.78
Health		
Decrease in the presence of Health Hazards	53.82	53.10
Number of Medical Facilities and Personnel	46.18	46.14
Economics		
Prices	50.35	45.78†
Quality	30.80	32.06
Selection	18.85	21.77†
Income Level	29.96	26.73†
Consumption of Goods and Services	16.51	16.62
Leisure Time	17.11	20.45†
Stability of the Economy	36.41	36.20
Potential for Future Employment	27.20	28.55
Potential for Savings and Investments	36.97	36.28
Potential of Retirement Plans	35.83	34.79
Present Standard of Living	30.58	30.40
Future Standard of Living	37.21	37.50
Equality of Opportunity	32.21	31.72
Recreation	18.13	19.08
Health	28.84	25.68†
Aesthetics	20.52	20.81
Economics	32.51	34.43

*Different sub-goal group rank order for the two states.

†Mean Weights significantly different between the two states at 5 percent level.

while responding Arizonans desired greater improvements in their income level. Weighting differences which are seemingly explained by physical divergences between the two states include the following: the greater Arizona concern for improvements in the air, mountains, and desert, the greater concern of responding Oregonians for improving the condition of their waters, and the significant lack of Oregon concern for improvements in mountain and desert areas.

CONCLUSIONS

In a general sense, the Arizona and Oregon mean weights are identical. This evidence for the congruity of desires for improvements in these goal areas is further substantiated by the similarities revealed in the rating of state problems and the environmental activity and knowledge levels. Thus, given the limited size of the responding sample and the realization that perhaps the true sources of divergent public desire were not measured or for some reason could not be measured, the tentative conclusion, based on preliminary results, must be that Oregon's image of being more progressive than Arizona with respect to environmental concern and action is apparently the result of divergences in state leadership and/or interest group orientation.

In regard to the viability of the weighted preference methodology, the Metfessel General Allocation Test, the Arizona-Oregon demonstration has shown that this weighting technique can be done by the general public and is appropriate for a mail survey. Although the return rate of the questionnaires was adequate for a demonstration of the methodology, it must be noted that for the weighting methodology to be truly useful in an actual planning situation, more intensive follow-up techniques and other efforts, such as interviews, must be undertaken to insure greater and more balanced input from all segments of the population.

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PUBLIC PERCEPTION OF WATER QUALITY AS A PLANNING TOOL

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Recent trends in resource planning can be interpreted as a demand for greater public participation in policy formulation and project evaluation. The purpose of this paper is to discuss one method of increasing public input. The first section will examine the perceptual dimensions of the environment and propose a means of integrating public perceptions into the planning process. The next section proposes a way to quantize the perceived environmental dimensions. Finally, the system is validated for one dimension.

PUBLIC PERCEPTION OF THE ENVIRONMENT

An individual's perception of his environment is influenced by three things: 1) what is expected, 2) what past experiences can be related to the current experience, and 3) the amount of information received from the environment. The individual can then form a judgment on the worth of the experience. The problem, then, is to elicit the judgmental categories from the individual, and disaggregate them into component parts.

DEFINITION OF DESCRIPTORS

The procedure used to elicit the perceptual categories is discussed in a previous paper (Wilson, et al., 1973). In brief, it is based on a determination of what is important to people by the words they use (Slobin, 1971). To elicit the descriptors,open-ended questions were asked. These related to the state of the environment and required respondents to provide words or word groups describing a given scene. The words were then analyzed by a frequency analysis to select those used most often. Like terms were grouped by generic name, e.g., trash and scum are under floating objects. The generic groupings for water quality split into sight and smell (Judge, Everett and Gum, 1973). Further disaggregation provided descriptors of Clarity and Floating Objects under Sight and Odor under Smell. The above procedure provided a list of descriptors meaningful to both the general public and the professional for decision making. The inclusion of these descriptors of water quality in an information system allows their use in the planning process.

PROPOSED INFORMATION SYSTEM

One possible information system is a hierchical structure relating the perceived dimension of water quality to a set of measurable indicators. Such a system is shown in Figure 1.

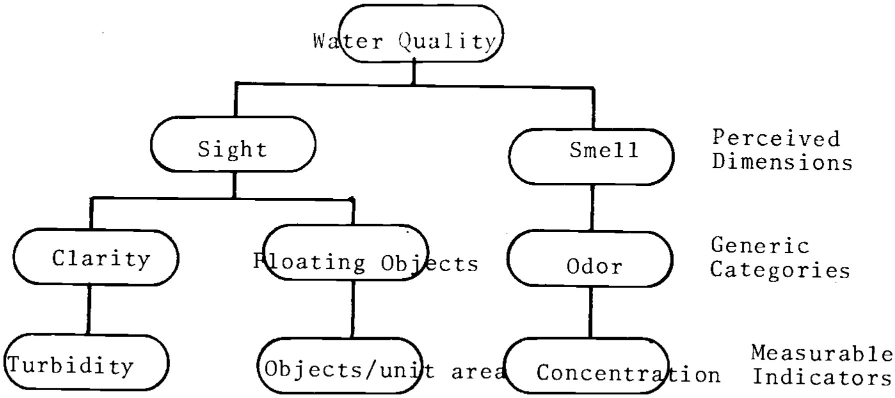

Figure 1. Proposed Information System

The information system allows a decision maker to estimate environmental effects of an alternative water resources management plan and evaluate the effect on the public. Information is aggregated at the highest level, Water Quality, but the system can be entered at lower levels for a more detailed evaluation. A more detailed description of the information system is given in Wilson, Gum and Roefs,(1973). One major step in the development is selection of measurable quantities to quantize the perceived dimensions of the environment. The quantification allows the aggregation of information and facilitates the comparison of alternative management plans.

QUANTIFYING PUBLIC PERCEPTUAL DIMENSIONS

The purpose of quantifying information is to allow aggregation and comparison. Qualitative information is difficult to use. The grading of meat into prime, choice, good and utility is a good example. If the consumer goes from a piece rated good to one rated choice, the increase in quality is unknown. The increase in price becomes painfully obvious. A quantized system might list fat, protein, vitamin and water content. The quantized information allows comparison. However, the amount of information would be confusing. This would be alleviated by aggregating the information into a few scales. The information system proposed operates on this principle. The measurable quantities must define the generic categories and be amenable to aggregation.

MEASURABLE INDICATORS

Figure 1 shows the indicators selected. The set is neither exclusive nor exhaustive. Each indicator was picked to meet two criteria: 1) it should be a standard measurement, and 2) it should relate to the generic heading. Single indicators were selected for each generic category as a first iteration and to simplify the developmental process. The measurable indicators relate to the fat and protein content of beef. They provide a set of numbers that can be readily compared within a generic category. It is almost impossible to compare between generic categories. How many units of turbidity equal one unit of concentration of odor? This limits the ability to aggregate the raw information and requires a transformation of the new data to a more general scale.

PERCEIVED QUANTIFICATION

The system proposed has been based on the public perception of the environment: Consistent with this theme is the use of a public value system, to transform the raw data, measurable indicators, to a form that can be aggregated. The transformation is accomplished by a satisfaction function. Satisfaction can be defined as the feeling of well-being felt by an individual because of the state of his environment. The satisfaction function, then, provides a numerical rating of the perceived well-being or satisfaction.

DERIVATION OF THE SATISFACTION FUNCTION

Hypothesize the existence of a satisfaction function, such that, the perceived satisfaction with the environment is a function of the measurable indicators. Or,

$$S_{wq} = f(X_1, X_2, X_3) \tag{1}$$

where,

S_{wq} is satisfaction with water quality,

X_1 is turbidity

X_2 is objects per unit area

X_3 is concentration of odor.

Differentiating (1),

$$dS_{wq} = \sum_i \frac{\partial f(X_i)}{\partial X_i} dX_i \tag{2}$$

Dividing (2) by S_{wq} and multiplying by $\frac{X_i}{X_i}$ j

$$\frac{dS}{S} = \sum_i \frac{\partial f(X_i)}{\partial X_i} \frac{X_i}{S} \frac{dX_i}{X_i} \tag{3}$$

Elasticity of S with respect to X_i is defined as:

$$e_i \equiv \frac{\partial f(X_i)}{\partial X_i} \frac{X_i}{S} \tag{4}$$

Combining (3) and (4)

$$\frac{dS}{S} = \sum_i e_i \frac{dX_i}{X_i} \tag{5}$$

Integrating (5)

$$S = \prod_i X_i^{e_i} \qquad (6)$$

where,

S is satisfaction

X_i is the measurable indicator

e_i is the elasticity of S with respect to X_i.

Equation (6) then is the transformation of the measurable indicator to a scale capable of being aggregated. The integration of equation (5) requires an assumption of constant elasticity over the range of the variable X_i. This assumption is based on the work of S. S. Stevens (1946, 1968) and others (Bruno, Hefferline and Suslowitz, 1971 and Moskowitz, 1972) that perception of stimulus is modeled by a power law. In light of later experiments, this assumption is not unreasonable.

VALIDATION OF POWER LAW

Validation of the power law as a model of the satisfaction derived from the environment was conducted with undergraduate students of the University of Arizona. The subjects (S's) were exposed to samples of water of varying turbidity and asked to rate their satisfaction with each sample as compared to a standard. Each sample was shown twice not in any sequence. The S's were then shown the samples and asked to estimate the clarity of each. The purpose was to test the hypothesis that value and magnitude were differing judgmental processes. Figure 2 shows the results of a least square regression of the averaged satisfaction and magnitude ratings for the S's. The results of the experiment tend to confirm the hypothesis that the power law models the judgment of value, or satisfaction, derived from the state of the environment. Further, it strengthens the hypothesis of a different judgmental process for value and magnitude.

	Satisfaction With Clarity	Magnitude of Clarity
Exponent	-1.86	-.94
r^2	.91	.95

(all values significant at .01)

Figure 2. Satisfaction and Magnitude Comparison

CONCLUSIONS

The conclusions of this research are: that the perceived dimensions of the environment can be determined; that an information system to quantize these dimensions is possible; and, that a power function may be used to model the satisfaction felt because of the state of the environment. The perceived dimensions of the environmnet are defined by the words, or word groups, most often used to describe various states of the environment. These descriptors form a basis for an information system derived from public values. Measurable indicators are then selected to describe the perceptual dimensions. A power function models the satisfaction derived from each perceptual state and quantizes the information. The quantified information can then be aggregated and compared to evaluate the effects of alternative management plants

Extensions of this research should be directed to verification of the measurable indicator list, development of water quality descriptors for other geographic areas, and development of a set of decision rules for use in a management system. In addition, it is essential that this perceptual approach be integrated with economic and technical methods for a complete evaluation of water resource management alternatives.

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