Peter F. Ffolliott

VOLUME 2

# HYDROLOGY and WATER RESOURCES in ARIZONA and the SOUTHWEST

PROCEEDINGS OF THE 1972 MEETINGS
OF THE
ARIZONA SECTION—
AMERICAN WATER RESOURCES ASSN.
AND THE
HYDROLOGY SECTION—
ARIZONA ACADEMY OF SCIENCE

MAY 5-6. 1972. PRESCOTT, ARIZONA

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# AN INVESTIGATION OF COLORADO RIVER TRIPS: A USER STUDY[1]

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The number of people taking Colorado River trips through the Grand Canyon National Park and Monument has increased dramatically during the past few years, usually on the order of 60% to 70% over each preceeding year. This increased user-intensity of the River has led to observations that the environmental carrying capacity[3] for the area has been reached, causing a possible quality degradation in this unique recreational experience. In response, both the National Park Service and the Colorado River Outfitters Association have adopted new policies to improve the quality of the trip, and to protect this valuable amenity for future generations.

Specifically, the National Park Service limited the use of the Colorado River beginning with the 1971 season to help protect the resource. The limit was based on user-days,[4] and each outfitter was allocated a number of user-days based on his past volume of business. Essentially, the limit was set at approximately

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1 This research was supported by the Grand Canyon Natural History Association.

2 Mark A. Boster, National Science Foundation Graduate Trainee in Water Resources Administration, Russell L. Gum, Associate Professor of Hydrology and Water Resources, University of Arizona, Tucson 85721.

3 Kormondy (1969) defines the environmental carrying capacity as the limit at which the environment can support a population.

4 A user-day is defined as an individual participating in a recreational activity during one day. Thus, a person taking a 10 day river trip will count as 10 user days.

10,000 users for the 1971 season, the same number of users who participated in the 1970 trips.

However, information on which to base policy is severely limited at this time. Consequently, there is a need to gain a greater awareness and understanding of visitor expectations, perceptions, interactions, and satisfactions and dissatisfactions.

## OBJECTIVES OF STUDY

The primary objective of this study is to obtain information directly from users on the recreational use of the Colorado River for river trips, which can be used in the policy-making process. Specific objectives are to:

1. investigate user's perceptions of crowding;
2. determine the underlying structure of the variables obtained from the questionnaire;
3. determine the types of people who participate in river trips.

## METHOD OF STUDY

The researchers decided to investigate the long-term effects of the experience by sampling past "river-runners," using a mailed questionnaire to 1967 through 1970 participants. A random sample of 2,622 people were sent questionnaires, and 65% completed and returned them; indicating a strong interest and concern for river trips. A second attempt to contact the 35% of those sampled who did not return their questionnaires was not made. Although this might bias the results of the study, it was judged that the additional cost necessary to collect these data was not justified.

## METHOD OF ANALYSIS

The data analysis consisted of both individual question tabulation and multivariate data-cluster analysis. The tabulation presents the intensity of responses to the individual questions. Cluster analysis was used to determine if any underlying structure of the data existed, and consisted of both variable and object analysis (Tryon and Bailey, 1970).

## CROWDING?

Recreation areas of the United States are experiencing increased crowding of available facilities, and according to some observers, the Colorado River from Lee's Ferry to Lake Mead is no exception. Of critical importance is the user's perception of a crowding problem and the relationship of this perception to regulatory actions. Since user limits have been set for the River by the Park Service, this study attempted to determine if they were justifiable, adequate, and equitable from the viewpoint of past users. Ecological parameters were not considered.

One can distinguish and measure two distinct types of crowding conditions, or potential crowding conditions, associated with river trips: (1) the size of each individual group or trip, and (2) the interaction of different groups along the River at any one time. The size of an individual trip remains constant for the duration of the excursion. On the other hand, the second condition may only exist sporadically throughout the day, when one group overtakes another group. Two or more groups often stop at the same scenic or historic place for hiking, and on occasion, more than one group will share a camping site at night.[5]

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[5] This situation is avoided whenever possible by all the outfitters operating on the River.

Individuals were asked to indicate approximately how many people were on the river trip they took, and to evaluate this as: too few, too many, or about right. Table 1 reports the results of these questions. Virtually none responded "too few" regardless of the size of their trip. The percentage of those answering "too many" seems to increase as the size of the trip increases, whereas those answering "about right" decreases as the size of the trip increases. When considering these data together, one sees that the percentages answering "about right" are greater than those for answers of "too many" until trip sizes of 41 to 50 people. This indicates that in general, for trip sizes less than 40 people, the majority of users are satisfied with the trip size, but when trips get larger than 40 people, the majority felt the size was too large.

The second type of crowding that can occur on the River is also important to river managers, outfitters, and users, as it concerns the total number of groups on the River at a given time, and their spatial distribution. Respondents were asked several questions to determine if they had seen other parties, what the effect of seeing other parties was, and what their opinions were about future management policies. More than 90% of the sample responded that they had seen other parties on the River, and their reactions are shown in Table 2. Notably, more people thought that the other groups were "reassuring" rather than "very annoying." The largest number (35.85%) thought that the other trips were "not annoying" while nearly equal numbers thought that they were "tolerable" and "enjoyable." The significance of the large number who felt that the other trips were "enjoyable" and "not annoying" should not be overlooked by the decision-makers. Only about 12% of the sample were annoyed or very annoyed at seeing other trips on the River. All other users were not overly upset by the other groups,

Table 1. Individual Reactions to Different Size River Trips

| Size | % of Sample on Trip | Too Few | About Right | Too Many |
|---|---|---|---|---|
| 1-5 | 0.31% | 40% | 60% | -- |
| 6-10 | 3.86% | 3% | 93% | 3% |
| 11-20 | 21.03% | -- | 83% | 16% |
| 21-30 | 35.74% | -- | 79% | 21% |
| 31-40 | 21.79% | -- | 62% | 38% |
| 41-50 | 7.98% | 1% | 40% | 57% |
| over 50 | 9.06% | 1% | 36% | 62% |

Table 2. Reactions to Seeing Other Groups on the River

| | Total | 1967 | 1968 | 1969 | 1970 |
|---|---|---|---|---|---|
| Very annoying | 3% | 3% | 3% | 2% | 3% |
| Annoying | 9% | 11% | 8% | 7% | 10% |
| Tolerable | 25% | 21% | 21% | 28% | 26% |
| Not annoying | 36% | 34% | 43% | 36% | 34% |
| Reassuring | 4% | 4% | 3% | 2% | 4% |
| Enjoyable | 22% | 25% | 19% | 23% | 22% |

and in fact the majority found them at least tolerable.

One might expect some variation in response to the spatial distribution of trips when tabulated by years. The year 1967 had the fewest participants, and 1970 the most participants, so it would be useful in policy decisions to note any variations in the users' perceptions for the various years, and thus, in the different user-intensities.

There was a difference in the number of people who saw other groups when the yearly tabulations were compared to the aggregate results. In 1967, 75% of the sample saw other groups, compared to 85%, 89%, and 96% for 1968, 1969, and 1970, respectively, reflecting the increased user intensities per year. Thus, in 1970, it was virtually impossible to make the journey without encountering another party, whereas in 1967, three out of every four trips saw other groups. However, when the reactions to seeing other groups on the River are compared to both the aggregate response and to other years, there does not appear to be a difference (Table 2). Regardless of the user-intensity, and thus the incidence of encountering other groups, the participants' reactions to the other groups is consistent. However, the population of river trippers may have changed over time such that present populations have a greater tolerance of crowding than previous populations.

Cluster analysis was used to analyze information on the users' backgrounds and to help understand their perceptions of crowding.

## WHAT IS CLUSTER ANALYSIS?

Understanding the entities that compose man's world has always been the goal of science, and cluster analysis is a general logic procedure that groups entities on the basis of their similarities and their differences. A set of variables can logically be grouped according to one's own rational composition; however, this logical grouping procedure takes no account of the correlations among the variables. Cluster analysis of multivariate data affords the opportunity of studying the underlying structure of the data.

Cluster analysis of variables (V-analysis) was utilized to: 1) condense the variables into basic dimensions that captured a significant amount of the general covariation among the variables, and 2) select homogeneous subsets of variables that are observable representations of the basic dimensions. The goals of cluster analysis on the objects (O-analysis) was to: 1) compute the position of each of the objects within the dimensions described in variable analysis, and 2) find the points of concentration within the dimensions, i.e., the groups of people with similar characteristics in terms of the clusters defined in V-analysis.

## APPLICABILITY TO COLORADO RIVER TRIPS' DATA

Computer analysis on all of the relevant variables produced clusters that were not judged "good" by the researchers using the criteria established by Tryon and Bailey (1970). Therefore, it was decided to use subsets of the data. The "Yes-No" questions (policy), "True-False" questions (individual values), and outdoor activity data were each run as an independent group. Other variables, e.g., income, age, etc., were included with the sub-groupings, but none of these variables appeared in final clusters.

## CLUSTER ANALYSIS ON OUTDOOR ACTIVITIES

Cluster analysis was used to determine whether a relationship exists between the type of outdoor activities in which "river-runners" participate and their income level. V-analysis defined three clusters, but none of the income categories entered into the resulting clusters. This indicates that there is no tendency for any group of river trip participants in the same income category to participate in certain outdoor activities.

The three clusters defined by the algorithm can be described as follows: (1) wilderness activities, (2) individual activities expending relatively little energy, and (3) individual activities expending high energy values (see Table 3). These three clusters explain 94% of the total generality and 91% of the total variability. The reliabilities (repeatability of the clusters) for the three clusters are 64%, 54%, and 46%, respectively--somewhat lower than desired, suggesting only a moderate chance of reproducing the same clusters if another sample is drawn.

These clusters are dimensions on which one can measure the individual respondents, which is the next step--developing profiles for groups of respondents using O-analysis.

Factor scores were calculated for each individual. Using the concentration points in the score space, O-analysis was able to distinguish eleven different O-types, describing all but 24 of the total sample (less than 1%). The groups are relatively "tight" with homogeneities all between 79% and 92%. Figure 1 is a graphical representation of the factor means of the eleven profiles and also, the number of participants in each O-type. In order to become a useful input, each of the groups should be studied, characterized, and stereotyped. For example, the largest group is represented by O-type 5. These are the people who ranked about average on the wilderness and strenuous activities clusters, but low relative to others on the less strenuous individual activities of cluster 2. In contrast, the smallest O-type is number 11, people who ranked very high on clusters 1 and 3, and at about the same place as O-type 5 on the less strenuous

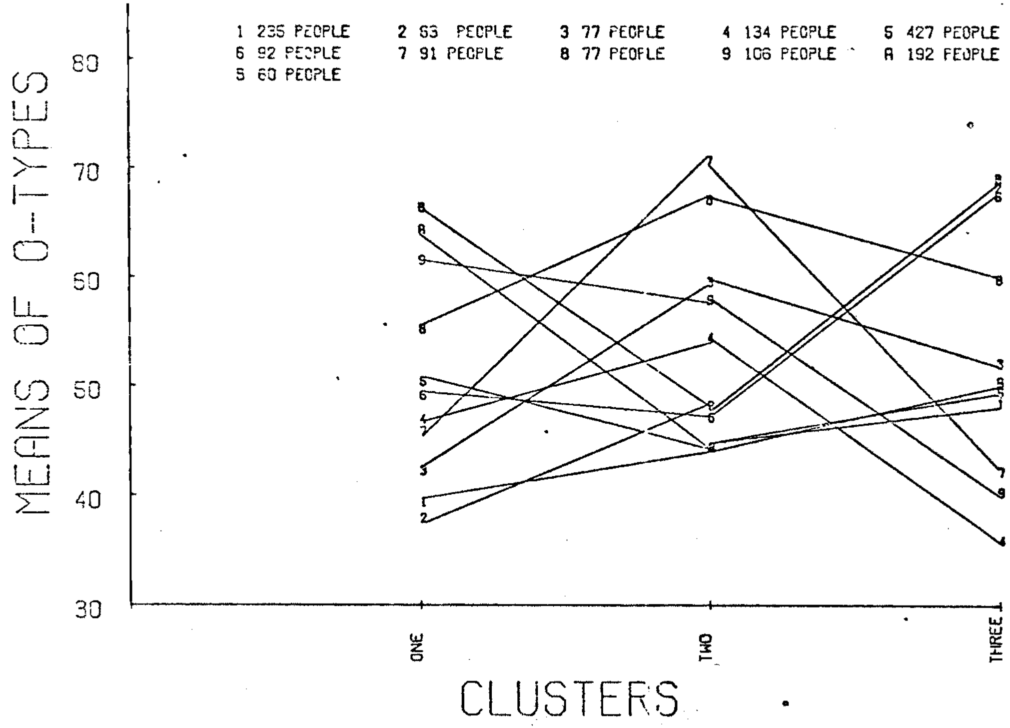


Figure 1. O-type Profiles on Outdoor Activities

Table 3. Clusters Defined on Outdoor Activities

| Cluster | Definer |
|---|---|
| 1. Wilderness Activities | Back-packing<br>Hiking<br>Camping<br>Mountain climbing with equipment |
| 2. Individual Activities Expending Relatively Little Energy | Fishing<br>Hunting<br>Golf<br>Boating |
| 3. Individual Activities Expending High Energy Values | Tennis<br>Snow skiing<br>Water skiing<br>Not fishing |

activities, cluster 2. The difference in intensities shown in Figure 2 is interesting. O-type 11 might be stereotyped as the "thrill seekers." They are interested in the thrill of the strenuous individual sports and also mountain climbing, hiking, back-packing, and camping; whereas, they are not interested in the more popular outdoor activities found in cluster 2.

Similar stereotypes can be assigned to each of the groups, giving the decision-makers a means of describing the people on the trip. For example, the second largest O-type is number 1, whose members can be described as having relatively little experience in the wilderness, and are generally non-participants in other outdoor activities. On the other hand, the second smallest group is O-type 2, whose members also have had little wilderness experience, but are very active in outdoor activities, especially in the strenuous ones.

O-types 9 and 10 are fairly large, and represent the other extreme--people with considerable wilderness experience. However, they differ in the type of outdoor activities in which they engage. Members of O-type 9 participate in the less strenuous outdoor activities rather than the strenuous ones, while the group represented by O-type 10 prefers the strenuous activities.

O-type 8 only represents 77 people, but they are the "super active" ones. They have had moderate wilderness experience and appear to engage in both types of outdoor activities regularly. The 91 members of O-type 7 are an interesting group. They have had some wilderness experience, do not generally engage in strenuous activities, but are the most active in the non-strenuous ones.

CLUSTER ANALYSIS ON "YES-NO" QUESTIONS

V-analysis on these data defined three distinct clusters that explain 78% of the total generality and 71% of the total variability for the data. The cluster reliabilities are lower than the desired 100%; 60%, 45%, and 42%, respectively, however they still provide valuable information. The clusters can be described as: (1) more trips, (2) management appreciation, and (3) wilderness regulation (see Table 4).

Once again, a factor score for each participant was calculated on each cluster, and the resulting factor means are shown in Figure 2. Ten O-types were distinguished in the O-analysis, classifying all but 69 of those sampled (less than 1%). The overall homogeneities of the O-types are generally high (81% to 95%), suggesting "tight" groupings.

There appear to be only two response areas for cluster 2: those who did not appreciate the trip management, and those who did; while clusters 1 and 3 seem to attract strong, medium, and low responses. For example, O-types 2, 6, and 10 can

Table 4. Clusters Defined on the "Yes-No" Questions

| Cluster | Definer |
|---|---|
| 1. More Trips | A river trip is something I would do more often if I had more time. |
| | I would go on more river trips if I had more money. |
| | Do you expect to take more Colorado River trips through the Grand Canyon? |
| | Do you plan to run other wilderness and/or white water rivers? |
| 2. Management Appreciation | Did the boatmen make your trip more enjoyable by pointing out natural phenomena? |
| | Were you encouraged to leave your campsites in better condition than when you had found them? |
| | Did you find what you were looking for on the trip? |
| | Did you feel that you had enough time for exploring on your trip? |
| 3. Wilderness Regulations | Should people be encouraged to take river trips within the Grand Canyon? |
| | Do you consider an outboard motor a necessary part of a safe, pleasurable trip? |
| | Should more developments like the Phantom Ranch be built along the Colorado River? |
| | Not: Do you believe that the Colorado River should be maintained as a wilderness river within the Grand Canyon? |
| | Not: Should the number of people taking river trips through the Grand Canyon be limited? |

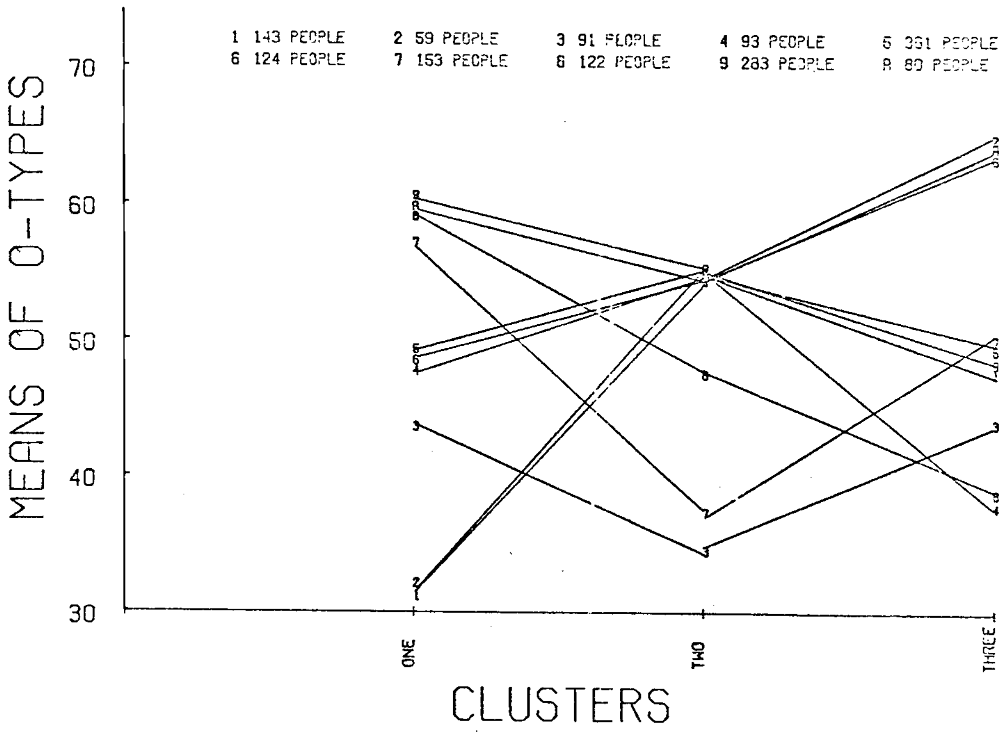


Figure 2. O-type Profiles on "Yes-No" Questions

be characterized as really appreciating the trip management, wanting to see regulatory policies initiated, and vary from desiring no additional river trips to planning to make more Colorado River trips. These groups all contain nearly the same number of participants. Those wanting to make additional river trips and also wanting regulatory policies appear somewhat hypocritical.

The largest group, O-type 5, are those who are moderate about taking more trips and desiring more regulations, but really enjoyed the trip. O-types 7 and 3 did not enjoy the management, were moderate about wilderness regulations, but O-type 7 was slightly more interested in taking more river trips. These are the people who perhaps wanted a type of experience that they did not completely receive.

CLUSTER ANALYSIS ON "TRUE-FALSE" QUESTIONS

V-analysis distinguished four distinct clusters that were able to account for 98% of the total generality and 95% of the total variability. The clusters can be described as: (1) learning, (2) trip experience, (3) "get away," and (4) wilderness adventure (see Table 5). The reliabilities for the first three clusters are reasonable (69%, 59%, and 60%, respectively), but the fourth cluster's reliability is low, 28%. The cluster was retained, however, for O-analysis.

The O-analysis found seven O-types using the "True-False" data (see Figure 3). The overall homogeneity of O-types ranges from 75% to 97%, and only 99 participants (less than 1%) are not represented in the O-types.

O-types 1, 6, and 7 represent the three largest groups of people, and are very similar in their characteristics. All three rate high on the trip experience, thought that it provided the opportunity to "get away," and perceived the trip as a wilderness adventure. Those members of O-type 1 felt that the trip was less

Table 5. Clusters Defined on the "True-False" Questions

| Cluster | Definer |
| --- | --- |
| 1. Learning | allows one to learn about himself<br>allows one to gain communion with God<br>allows one to learn about other people<br>allows one to test his physical abilities |
| 2. Trip Experience | is a unique experience<br>allows one to observe the beauty of Nature<br>allows one to enjoy the companionship of camp life<br>allows one to get away from the impersonal urban environment |
| 3. "Get away" | allows one to get away from the technology of civilization<br>allows one to "get away from it all"<br>allows one to get away from pollution<br>allows one to get away from people |
| 4. Wilderness Adventure | is full of excitement and adventure<br>is a wilderness experience |

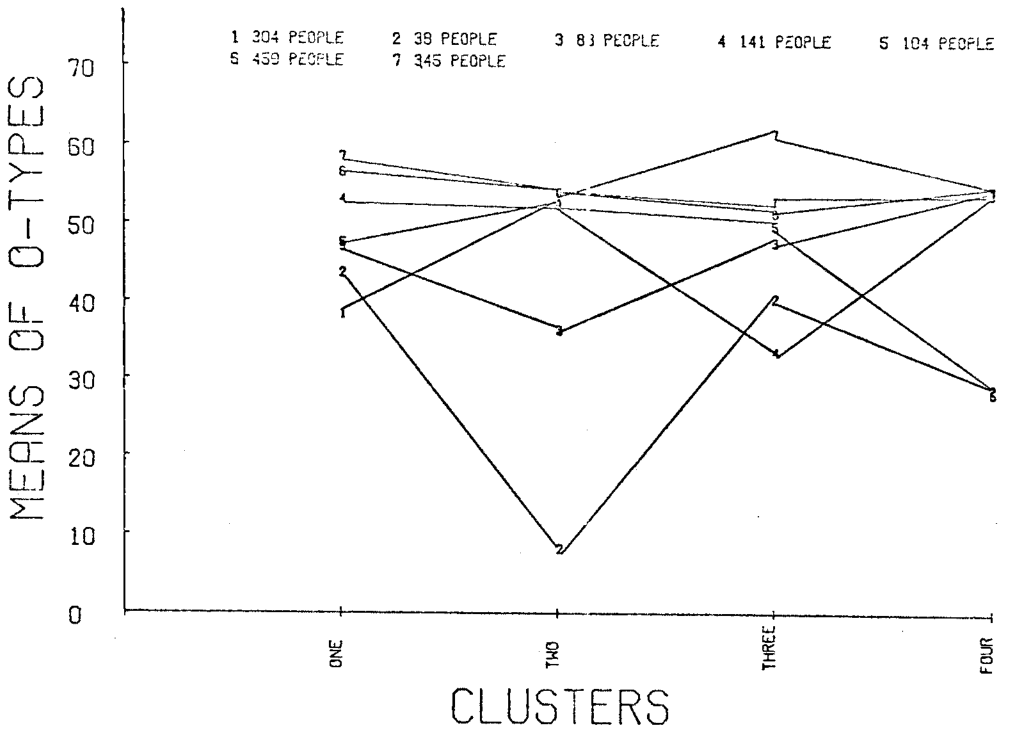


Figure 3. O-type Profiles on "True-False" Questions

of a learning experience than those in the other two O-types. Generally, these are the people who probably enjoyed their entire river trip. In contrast to these three O-types, the smallest group is represented by O-type 2. These are the people who tended not to appreciate the trip; they did not believe the trip was a learning experience; they ranked considerably lower than any of the other O-types on the trip experience cluster, and they did not feel that the trip allowed them to "get away" or that it was a wilderness adventure. These people are probably the chronic complainers who regularly give negative feedback to the National Park Service.

## MANAGEMENT IMPLICATIONS

The cluster analysis results can become a useful tool of policy-making institutions. Both the Park Service and the outfitters should gain some understanding of the variables that comprise each of the clusters, and look at the profiles of the people, noting the size of each of the O-types. This will enable them to know more about the people who make rivers trips, and thus, to better determine the type of trips desired.

Cluster analysis will be used in future work to predict which groups of people defined in O-analysis perceive a crowding problem.

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# HYDROLOGY AS A SCIENCE?

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Mankind has been curious about the earth on which it lives since ancient times. Multitudes of questions have arisen because of this curiosity. Some of these may have been -- What makes the sea salty? What is the source of water coming from springs? What makes rain?

As mankind sought answers for these, other equally perplexing questions arose. How does water occur in the ground? How do you get water out of hard rock formations? How do some streams erode hard rocks and yet fail to move sand?

How did mankind attempt to solve these questions? In the past, scientific knowledge was minimal and methodology nil. Today guidelines, established in the past or present, are used to build up a method of investigation for the compilation of scientific knowledge required for their solutions. Today it is called Hydrology - the science of water.

We now have called Hydrology a science! A question still remains with many as to the truth within the statement. In order to establish the truth or destroy the disbelief present, let's look at what is science.

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## WHAT IS SCIENCE?

What is the definition of science? According to the Random House Dictionary of the English Language, science is a branch of knowledge or study dealing with a body of facts or truths systematically arranged and showing the operation of general laws; knowledge gained by systematic study. Systematic study implies experimentation, observation, classification, and practice.

Also of interest is the legal dissertation of science from Corpus Juris Secundum [79 C.J.S. 455, Science (1952)]: Quote "SCIENCE. In a sense 'science' means knowledge obtained individually by study of facts, principles, causes, etc.; knowledge of many methodically digested and arranged so as to be attainable by one; accumulated and accepted knowledge which has been systematized."

Many other definitions of science have been developed and recorded but in all instances the implication of attained knowledge is included. Now we may ask again - Is hydrology a science?

## DEVELOPMENT OF SCIENCE OF HYDROLOGY

Two broad types of knowledge may be attained -- experimental and historical. Experimental knowledge is derived from experiments designed and developed to obtain specific data. These provide the bases for practically all our knowledge in the realms of physics, chemistry, and other hard sciences. The experiments could be repeated by any competent observer and the results could be verified if the same data were utilized. From these experiments, universal laws of nature were established.

The second type of knowledge is historical which consists of facts not attained through experimentation. Data cannot be obtained at will. The knowledge is certain and authentic when observed by competent observers and does not negate the laws of causality. Many of the phenomena of hydrology are isolated and natural events which cannot be duplicated by experimentation. Thus these data are historical as mankind has been unable to "back-up" in time.

Within the purview of the previously stated definitions, hydrology can be considered a science. All data must be defined and verified, organized and correlated, and must be universal in application. Systematic study is required; the study of water.

When did the science of hydrology begin? The exact origin is hard to trace and is probably buried in antiquity. Man has forever realized that water is essential for his survival and has worked toward maintaining its supply. The science of hydrology was involved throughout history. The progressive extensive exploitation of waters of the world in recent times has necessitated an intensive study of water in its natural form and occurrence. The body of knowledge developed by the scientific investigations and studies is recognized as hydrology throughout the world.

It is an interdisciplinary science, involving the integration of other earth sciences in order to explain the life history of water and its physical, chemical, and biological character. Other sciences that overlap into the science of hydrology are agricultural sciences, meteorology, climatology, geology, geomorphology, chemistry, physics, hydraulics, mechanics, and mathematics.

## DEVELOPMENT OF A HYDROLOGY CURRICULUM

In order to effectively carry out the scientific investigations relating to water, properly trained personnel with the ability to recognize, organize, correlate, etc. the derived data became a prerequisite. To satisfy this prerequisite educational programs with this type of training as its prime objective were developed.

Knowledge required for understanding and solving complex water problems may be considered as a continuum extending from the basic physical sciences through the applied sciences and into the behavioral sciences. The breadth of knowledge encompassed is as great or greater than in any other field of study. The educational program in hydrology strives for a general education across the continuum especially at the undergraduate level with specialization available at the graduate level.

Historically, training in hydrology at the university level has been implemented within several established disciplines with little or no integration among these disciplines. Also training in the behavioral sciences with emphasis on water has been very limited especially in the social, political and economic fields. Recently there has been concerted effort on several campuses to integrate and broaden hydrology education by the development of interdisciplinary programs.

The importance and need of having properly trained scientists in hydrology are receiving greater recognition and emphasis as our society attempts to cope with the complex problems surrounding water. With greater demands being placed on our natural resources, virtually insurmountable problems develop in providing an adequate and suitable water supply at a desired location or for a specific requirement. Many agencies make decisions relating to water; these decisions involve the populace introducing the complex interrelationship between socio-politico-economic problems. These problems require special attention and skills; their solutions are more complex than the empirical solutions generally developed in engineering and science technology. Methodologies rather than specific models are essential as the life of a model may be short; new knowledge is an exponential function implying that knowledge of today will probably be superseded within a students program. A continual update of hydrology training or any other training program, must exist in order to maintain this pace.

## HYDROLOGY FOR THE POPULACE

As stated previously, decisions relating to water are made and the decisions involve the populace. Too frequently the populace is ignorant or uninformed as to the ultimate effect of these decisions. Hence, an informed populace would permit or require decisions which were sound and in the best interest of the populace as a whole.

In order to try to inform the populace, an introductory course in hydrology was introduced within the Department of Hydrology and Water Resources at the University of Arizona. This course is in addition to the professional hydrology courses in the department and is intended for the non-scientific oriented students. It was felt that water could be the one topic upon which attention could be focussed and demonstrate the interrelations between water and many other disciplines. The course was thus aptly entitled "Water and the Environment."

This program consists of two parts with each part a four unit course thus requiring two semesters for the completion of the total eight units assigned to the program. It is considered a laboratory science course and includes a 3-hour laboratory period in addition to the 3-hours of lecture per week. Because it is a laboratory science course it has been positioned beside the classical basic science courses such as chemistry, physics, biology, and geology. Students enrolled in some of the university's curricula can satisfy their science requirements by satisfactory completion of the course.

The direction of the course was to introduce the occurrence of water through the use of the hydrologic cycle. After this introduction, each phase could be covered in detail to provide a physical basis for water. The course then would build upon the basic physical concepts presented and would permit discussion of the present and future problems for each area. Figures 1 and 2 represent an organizational chart for each of the two parts of the course. This arrangement or sequence was considered to be most desirable as the physical aspects of the system must be known prior to delving into specific problem areas. For example, the statement "the Colorado River is polluted" may be a true statement but it by itself is not adequate information from which the problem can be solved. The physical system - nature, type, extent of pollutant, etc. - must be known before any solution can be derived. Thus the direction of the course attempted to provide the physical aspects followed by the specifics.

Several problems were encountered during the initial offering of the course; some of which were predicted while others were unexpected. One of the major problems was, and still is, the lack of a suitable text. Because of the broad scope of the course, several books would be required but this requirement is somewhat unrealistic for economic reasons if for no other. There are other reasons; one of which being the student. Since freshmen are the intended student, expecting them to utilize 3-5 books for a course is futile. They are not attuned to this type of activity. To circumvent the problem a select list of references was compiled and these books were made available at the University's library. The list as included in this paper is by no means complete but does cover a very broad spectrum of water and its related areas. This does not solve the problem of an inadequate text but does permit a temporary solution to the problem.

Another major problem was scientifically designed laboratory experiments. These experiments are a prerequisite for the course if it is to satisfy the requirement as a laboratory science. Scientific method must be stressed but yet

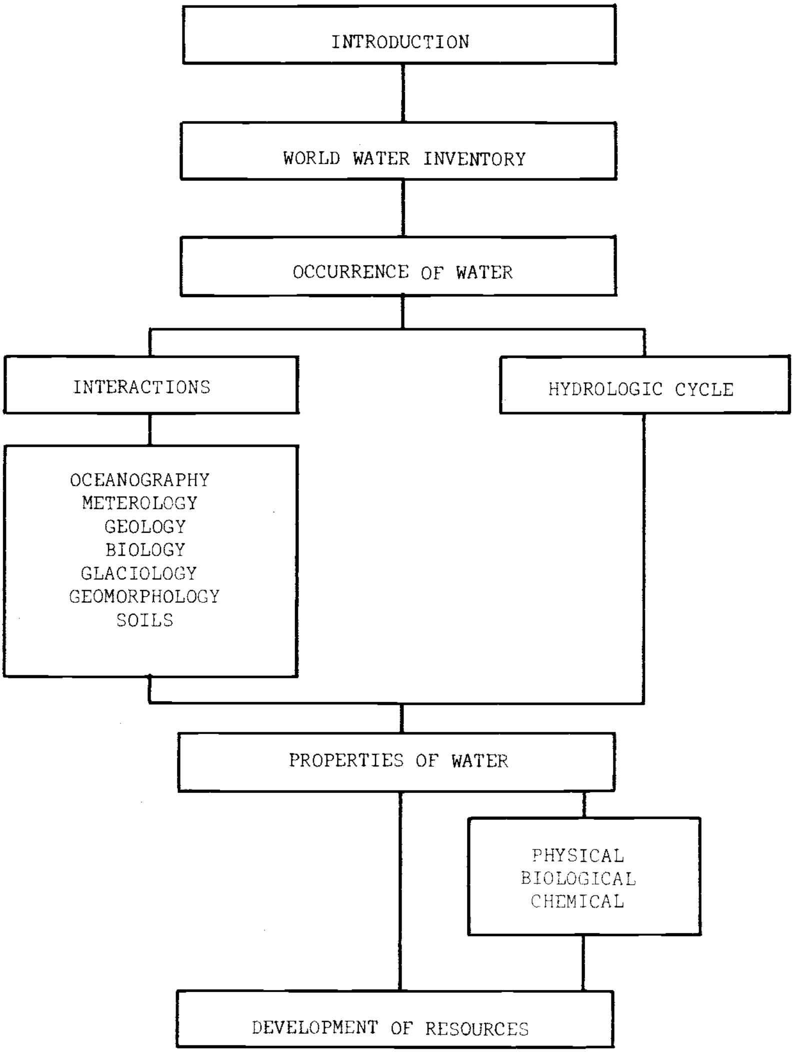


FIGURE 1. Subject matter organizational chart for the first semester of the course - "Water and the Environment"

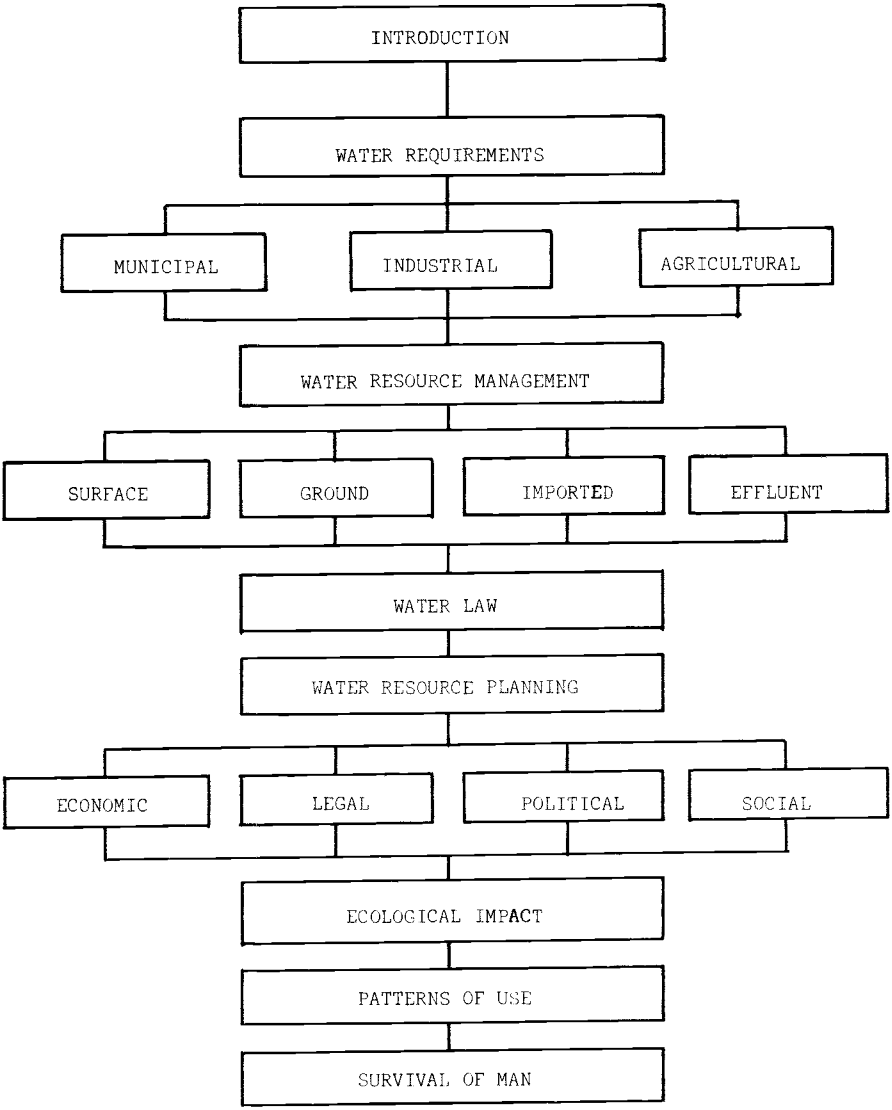


FIGURE 2. Subject matter organizational chart for the second semester of the course - "Water and the Environment"

many of the subject areas do not lend themselves to this type of effort. The physical, chemical, and biological aspects of water were easily demonstrated through laboratory endeavors but other topics such as ecological impacts, water planning, legal considerations, etc. were extremely difficult, sometimes impossible, to demonstrate within a prescribed or recognized "scientific method" framework. Allied to this problem are the logistics required to implement a truly desirable laboratory program. During the initial stages of any new program, space and equipment are not always at the immediate disposal of the program directors. This thus requires a rather complex and diplomatic management scheme for utilization of existing facilities. Only time and/or money will permit the optimal development of this segment of the program.

Another problem, probably the most partentuous, lies within the students. It is extremely hard to evaluate much less rectify because of the nature of the source of the problem. One aspect of this problem is the extreme ecological orientation of the student. He/she is not interested in the why or how of the problem but rather want an immediate solution. As stated previously they are unaware of why the Colorado River is polluted and it really did not matter so long as they could say "this" is the solution. It became extremely difficult to convey to them the importance of knowledge of the system before an effective solution could be rendered.

A second aspect of the student problem was the student's preparatory training toward handling a laboratory science course. Most had never encountered a lab science course and were not attuned toward the quantitative thinking necessary in this segment. Part of this stems from a lack of training and/or desire for mathematics. Although mathematics was not a stated prerequisite, an implied

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knowledge was expected. This implication was ill-founded as most of the students were unable to handle even the simplest mathematical manipulations necessary to attain the quantitative solutions. This thus forced some modification upon the objective function of the course.

## CONCLUSION

The concept of hydrology as a science course is sound but some problems make it rather difficult to administer within its stated objective function -- to inform the populace about water. Experiences accrued during the first offering of such a course indicate that some of the problems are very real and difficult to evaluate. These same experiences, however, did permit satisfaction, in part, of the stated objective function. The manner of satisfaction may not have been as initially designed but this is rather academic if the function was in fact satisfied. Time will tell as to the true impact that a course of this type may have upon water decisions.

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# AN ANALYSIS OF YEARLY DIFFERENCES IN SNOWPACK INVENTORY-PREDICTION RELATIONSHIPS

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## INTRODUCTION

Inventory-prediction relationships between snowpack conditions and forest attributes may be useful in estimating water yields derived from snow. Conceivably, these relationships may also provide insight relative to potential changes in water yields resulting from land management re-directions (e.g., thinning and clearing of forest overstories). Thus, snowpack inventory-prediction relationships may be considered an integral part of water yield forecasting and improvement programs.

Unfortunately, it is often necessary to develop snowpack inventory-prediction relationships from source data collected over a limited time period (e.g., one, two, three years of record). This constraint has been common to previous studies designed to develop snowpack inventory-prediction relationships in Arizona (Ffolliott and Hansen, 1968; Garn, 1969; Ffolliott and Thorud, 1972), possibly restricting the usefulness of these investigations.

Inventory-prediction relationships developed from source data representing a limited time period may be strictly applicable to the conditions (e.g., precipitation inputs, temperature patterns, etc.) encountered in the time period. Analyses of long-term data available from the USDA Soil Conservation Service (SCS) Federal - State - Private Cooperative Snow Survey (Enz, 1970) suggest inventory-prediction relationships developed from limited data may have more general application, however.

Snowpack inventory-prediction relationships satisfy the following objectives: (1) to estimate the mean snowpack water equivalent on a watershed or basin, and (2) to describe the trade-off (e.g., the rate of exchange) between snowpack water equivalent and forest attributes. In many instances, inventory-prediction relationships developed from limited data are of restricted value in satisfying the first objective. But, if defining the trade-off between snowpack water equivalent and forest attributes is the objective, the results of the study suggest more general application of relationships developed from limited data.

## DESCRIPTION OF STUDY

This paper describes a study designed to empirically analyze inventory-prediction relationships developed from long-term Snow Survey records. Specifically, regression equations defining relationships between snowpack water equivalent at peak seasonal accumulation and different expressions of forest density were evaluated. Snowpack water equivalent at peak seasonal accumulation was selected as the dependent variable in the regression equations because, conceptually, this quantity represents one of the best estimates of potential water yields derived from snow (Ffolliott and Thorud, 1972). Expressions of forest density selected as independent variables included measures easily obtained for direct application in snowpack inventory-prediction relationships.

Available records from 18 snow courses (Table 1) located throughout the ponderosa pine (*Pinus ponderosa* Laws.) type in Arizona (Figure 1) provided source data in the study. Courses selected for study were located within forest stands vis-a-vis in natural openings adjacent to forest stands or in cienegas and parks.

TABLE 1. Mean of Variables on Snow Courses.

| Name | Basal Area | Aspect | Slope | Elevation |
|---|---|---|---|---|
| | -- sq. ft./ac. -- | | -- percent -- | -- feet -- |
| Bear Wallow | 134 | NE | 35 | 8100 |
| Beaver Head | 34 | NW | 5 | 8000 |
| Canyon Creek | 88 | N | 5 | 7500 |
| Chalender | 43 | N | 15 | 7100 |
| Copper Basin Divide | 42 | NE | 5 | 6720 |
| Forest Dale | 23 | NE | 5 | 6430 |
| Fort Valley | 81 | S | 5 | 7350 |
| Gentry | 81 | -- | 0 | 7650 |
| Happy Jack | 53 | -- | 0 | 7630 |
| Heber | 53 | -- | 0 | 7600 |
| Mingus Mountain | 68 | SW | 5 | 7100 |
| Mormon Lake | 38 | NW | 5 | 7350 |
| Mormon Mountain | 46 | -- | 0 | 7500 |
| Munds Park | 37 | -- | 0 | 6500 |
| Newman Park | 28 | E | 5 | 6750 |
| Nutrioso | 78 | SW | 5 | 8500 |
| Rose Canyon | 46 | NW | 30 | 7300 |
| White Spar | 70 | NE | 25 | 6000 |

Each snow course consisted of a series of individual sample points at regular intervals within relatively homogeneous forest cover and site conditions. The number of sample points and the interval between the points were variable, however; eight to 12 points at 50-foot intervals was the most common sample design, although as few as five points at 25-foot intervals was encountered.

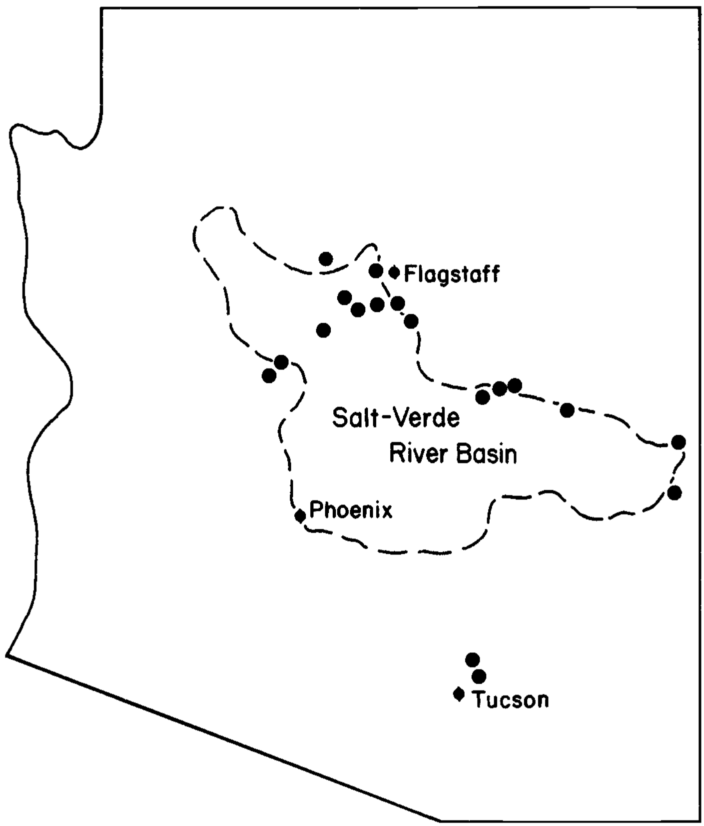


Fig. 1. Location of the 18 snow courses within the ponderosa pine type in Arizona.

Since the snow courses were established in locations where the snowpack is expected to persist throughout the winter-spring water yield forecasting period, the courses are not necessarily an unbiased sample of forest cover and site conditions occurring within the ponderosa pine type. Therefore, interpretations of the results of the study should be restricted to the range of conditions sampled.

Measurements of snowpack water equivalent are made on a regular bi-monthly schedule, starting January 15 and extending through April 15. Measurements taken on this schedule, and representing maximum snowpack accumulation on each course for each year of record, were assumed indicative of peak seasonal accumulation.

Snowpack measurements have been obtained on the snow courses evaluated for a variable number of years, depending on the installation date. Over 30 years of record were available from the older courses, with only five years of record from the course most recently installed. All available data were utilized, though, because of the need to analyze as many among year variations in snowpack inventory-prediction relationships as possible.

Expressions of forest density were developed from point sampling procedures using standard mensurational techniques (Beers and Miller, 1964; Dilworth and Bell, 1968). Employing a basal area factor (BAF) of 25, the tally of trees from all sample points comprising a snow course provided the basis for developing the expressions of forest density.

Initially, basal area, number of trees, and cubic-foot volume were considered for evaluation. Only basal area will be discussed in this paper, however, as this expression of forest density is more widely accepted and applied than the others. Furthermore, basal area is easily and objectively determined in the field, is readily converted to other expressions of forest density, and is an important term for many multiple use land management relationships.

For each snow course, regression equations describing snowpack water equivalent as a function of basal area was developed for each year of record. The family of regression equations representing an individual snow course was then analyzed in terms of similar statistical characteristics.

Snow courses were analyzed individually (e.g., discrete "case histories") because of discontinuities in forest cover and site conditions among the courses.

The primary hypothesis tested in the study, and evaluated by statistically analyzing the family of regression equations representing a snow course, was that, given a precipitation input, the distribution of snowpack water equivalent at peak seasonal accumulation is determined by the spatial arrangement of the forest cover (e.g., basal area).

The hypothesis was tested and evaluated within the framework of the following assumptions: (a) the linear mathematical model selected to empirically describe snowpack water equivalent at peak seasonal accumulation and basal area was appropriate; (b) a regression intercept, which indicates regression elevation, reflects the precipitation input for the year of record; therefore, considering a family of regression equations representing a snow course, regression intercepts can be expected to vary with different precipitation inputs; and (c) a regression coefficient, which indicates regression slope, reflects the trade-off between snowpack water equivalent at peak seasonal accumulation and basal area; therefore, considering a family of regression equations representing a snow course, it was assumed that regression coefficients will remain statistically constant with varying magnitudes of precipitation input.

Covariance analyses (a = 0.10) were used in testing the hypothesis (Snedecor, 1959). In these analyses, it was assumed that all source data were drawn from normal populations with homogeneous variances.

## RESULTS AND DISCUSSION

Essentially, the purpose of the above-mentioned analyses was to test for statistical similarities among regression coefficients developed for each snow course in the study. Discontinuities in conditions among the courses resulted in different regression coefficients among the courses. However, the purpose of the analyses was to ascertain within snow course changes in regression coefficients as attributed to time, and not to demonstrate similarities in regression coefficients among the courses.

Of the 18 snow courses in the study, nine possessed a family of regression equations with at least one statistically significant regression coefficient (Table 2). Families of regression equations on three of these courses possessed both statistically significant and non-significant regression coefficients. These courses were dropped from further analyses, and they were considered not to support the primary hypothesis of the study.

TABLE 2. Summary of Regression Coefficient Analyses on Snow Survey Courses

| Name | Years of Record | Remarks[1/] |
|---|---|---|
| Bear Wallow | 21 | 1 * |
| Beaver Head | 31 | 2 ** |
| Canyon Creek | 11 | 3 * |

TABLE 2. (continued)

| Name | Years of Record | Remarks[1/] |
|---|---|---|
| Chalender | 22 | 4 *** |
| Copper Basin Divide | 6 | 1 * |
| Forest Dale | 30 | 2 ** |
| Fort Valley | 22 | 1 * |
| Gentry | 18 | 1 * |
| Happy Jack | 18 | 1 * |
| Heber | 19 | 3 * |
| Mingus Mountain | 22 | 2 ** |
| Mormon Lake | 23 | 1 * |
| Mormon Mountain | 6 | 1 * |
| Munds Park | 19 | 3 * |
| Newman Park | 6 | 4 *** |
| Nutrioso | 32 | 1 * |
| Rose Canyon | 21 | 4 *** |
| White Spar | 5 | 1 * |

1/ Explanation:

1 No single regression coefficient statistically significant; therefore, regression coefficients considered similar.
2 Statistically significant and non-significant regression coefficients.
3 Statistically significant and similar regression coefficients, with exception of years of low precipitation input.
4 Statistically significant, but not similar, regression coefficients.
* Supports primary hypothesis in the study.
** Does not support primary hypothesis in the study.
*** Inconclusive.

Families of regression equations characterizing the remaining six snow courses were analyzed for statistical similarities among regression coefficients. With the exception of years with low precipitation input (e.g., total seasonal precipitation input less than one-fourth of the long-term mean), families of regression equations on three courses possessed statistically similar regression coefficients (Table 2). The regression intercept values differed (Figure 2), but, again, this was attributed to varying magnitudes of precipitation input. These three

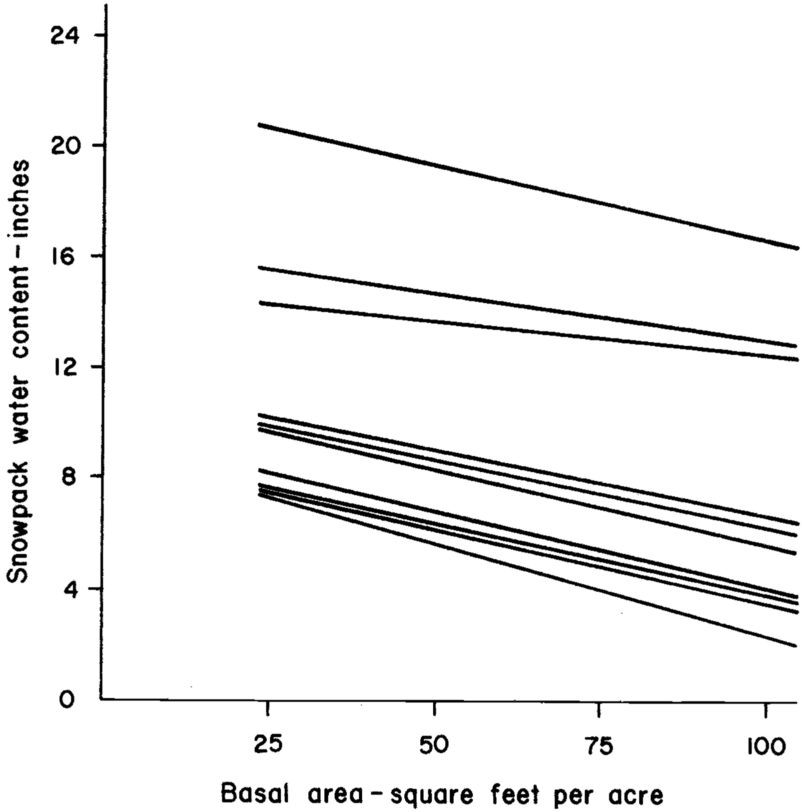


Fig. 2. Family of regressions illustrating relationships between snowpack water equivalent at peak seasonal accumulation and basal area for different years of record (1958-67) on the Canyon Creek snow course.

courses appeared to support the assumption of constant regression coefficients with different precipitation inputs.

On two snow courses, regression coefficients associated with two-thirds of the regression equations in the respective families were statistically similar, while regression coefficients associated with one-half of the regression equations in the family representing the other course were statistically similar (Table 2). These three courses were considered inconclusive regarding substantiation of the primary hypothesis of the study.

Of the 18 snow courses analyzed, nine possessed a family of regression equations in which no single regression coefficient was statistically significant (Table 2). The families of regression equations did differ in regression intercept values (Figure 3), however, which was attributed to different magnitudes of precipitation input. With all regression coefficients similar, the assumption of regression coefficients remaining constant with varying magnitudes of precipitation input was seemingly upheld.

## CONCLUSIONS

Generally, 12 of 18 snow courses evaluated in the study appeared to support the hypothesis that, given a precipitation input, the distribution of snowpack water equivalent at peak seasonal accumulation on a site is determined by the spatial arrangement of the forest cover (e.g., basal area). Three courses did not support the hypothesis, while three courses were considered inconclusive. More testing may be desirable before accepting the hypothesis as fact. However, until additional long-term source data are available, the results of the study may provide information re-

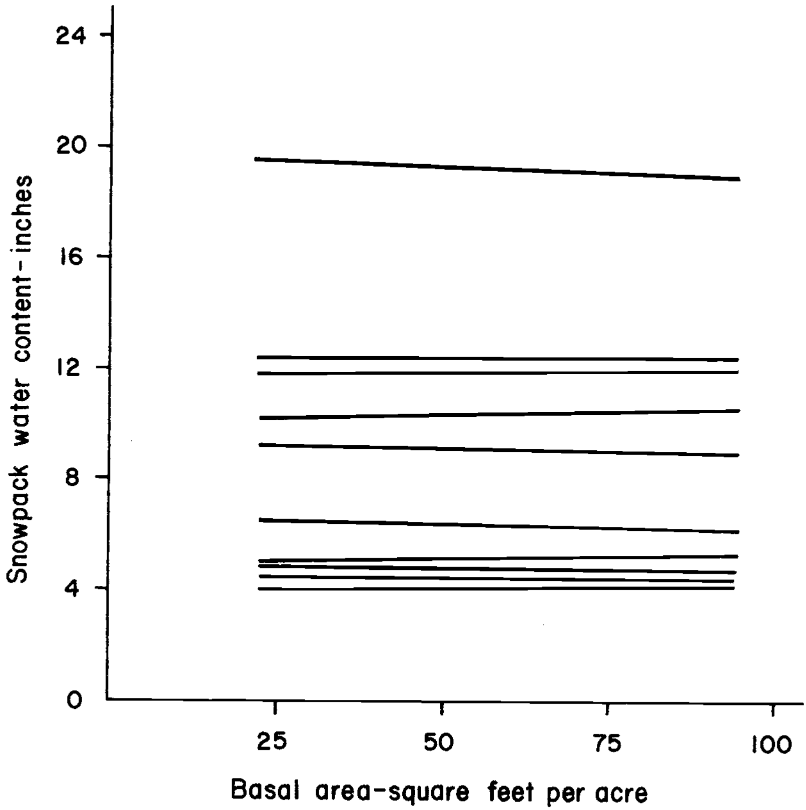


Fig. 3. Family of regressions illustrating relationships between snowpack water equivalent at peak seasonal accumulation and basal area for different years of record (1958-67) on the Mormon Lake snow course.

lative to applying inventory-prediction relationships developed from limited data to define the trade-off between snowpack water equivalent and forest attributes characterizing a given site.

While snowpack water equivalent may change as a function of precipitation input, the trade-off between snowpack water equivalent and forest attributes frequently remain statistically unchanged. Therefore, given a precipitation input, the distribution of snowpack water equivalent may be described by applying an inventory-prediction relationship developed from limited data, with knowledge of pertinent forest attributes.

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# DESIGN AND PILOT STUDY OF AN ARIZONA WATER INFORMATION SYSTEM

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A water information system may have highly different demands placed upon it depending upon the interest of the individual querying the system. Typical queries may be rainfall-runoff data for a research hydrologist, water level data for a perspective homeowner about to drill a well for domestic use, water quality data for a state agency or information from a water activity file pertaining to present and past work in water resources in a specific scientific or geographic area of interest.

Demands such as those listed above create a broad spectrum of response to be provided by an information system. Information in this discussion represents a broader term than data, and information is usually gleaned from analysis or some type of display of data; i.e., a hydrograph graphically displayed as streamflow rate versus time is information, whereas data is the specific streamflow rate recorded or measured at a specific time.

Prior to collection of data and recording on magnetic tape, the literature was searched for experiences others have encountered. A visit was also made to the State of Texas for a first hand review of their data system (Weiss, 1971). Several systems (Botz, 1970;

Cooper, 1967; Crouse and Maxey, 1967; Lang, J.M. & Leonard, 1967; Robinson, 1967, & Turner, 1967) are in some state of operation, but everyone developing such a system seems to have an approach peculiar to his needs.

The first step necessary in the design of an Arizona Water Information System (AWIS) was personal contact with those who would be data suppliers and/or information users of such a system. Emphasis was placed on benefits in time saved by researchers, planners, consultants and others seeking data to apply to an area of interest or convenience of an automated system to handle water data or information dissemination to the public by State agencies, and the convenience associated with the computer's capability to condense data via programmed computation and presentation of information in a desired format.

Several State of Arizona agencies were contacted to determine which would be interested in such a system. Currently, six Arizona State agencies* have specific statutory responsibilities or administrative needs for inventorying, conserving, developing, managing, operating, protecting, or monitoring the water resources and related facilities of the State. A potential use for the AWIS by state agencies lies with in-house manipulation of data. Each agency possesses different information needs to fulfill its objectives, but much of this information is based upon common data. For example,

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* Arizona Highway Department, Arizona Game and Fish Department, Arizona Water Commission, Arizona State Health Department, Arizona State Land Department, and the Arizona Department of Economic Planning and Development.

the Arizona Highway Department conducts hydrologic studies pertaining to highway design; the State Health Department collects and analyzes water samples for chemical and bacteriological constituents; the State Land Department collects and files information dealing with ownership and legal description of groundwater wells; and the Water Commission generates additional data on groundwater. A means of disseminating this type of information in-house and between agencies could reduce redundant data collection efforts.

A survey of needs at the University of Arizona has resulted in an expressed interest for a water activity file and various data needs for research. A water activity file is descriptive information pertinent to each water resource activity.

The varied interests of Arizona water information users calls for a flexible operating system. A pilot program was developed with the premise that a knowledgeable base of information concerning on-going and past water resource activities must be initially developed. Once this water activity file has been initiated, then data generation cumulating from the water activities may be sought. The location of data may not always result in its inclusion into AWIS, however, as many sources are capable of disseminating data themselves. In this case AWIS could provide users the location of data. In some instances the water activity will not generate data at all, however, the water resource information available on file will be valuable for reference.

The water activity file consists of title of project (this may be a data collection program or a research project), person in charge of project, address of person in charge, period of operation, key words to describe activity, brief summary of the work, professional papers or articles resulting from activity, data availability (where data, if any, may be obtained), and province, county, river, basin, township, range and section where activity was conducted. Thus far many of the projects in the activity file lack information pertaining to data availability; however investigators are to be contacted to determine if data exists and how a potential user can best access it. If data resides with the AWIS for the project described, it may then be accessed. Water resource information and/or data, if it exists, may be retrieved by key words, or a broad range of geographic locations from Arizona's three provinces down to a quarter, quarter section or latitude-longitude designation.

Figure 1 is a Water Resources Questionnaire developed to collect the activity information for entry into the AWIS. The information collected to enter into the questionnaire was secured in several ways. These include: (1) personal contacts; (2) coordination with the Water Resources Research Center; (3) review of files of the coordinator of Research, University of Arizona; (4) utilization of the Current Research Information System (CRIS); (5) information provided by the Office of Water Data Coordination (OWDC), Washington D.C.; and (6) annual reports.

Although the pilot system is not truly statewide at this time, the continuing efforts include formal ties with Northern Arizona

| ARIZONA WATER INFORMATION SYSTEM<br>WATER RESOURCES QUESTIONNAIRE | | | | |
|---|---|---|---|---|
| 1. Project Code: | | 2. Title of Project: | | |
| 3. Principal Investigator or Personnel in charge: | | 4. Name and Address Where Activity conducted: | | |
| 5. Professional School | 6. Period of Operation | | 7. Level of Funding | |
| 8. Supporting Agency | 9. Key Words: | | | |
| 10. Brief Description of Project (include what data is being collected form, ie. charts, tables, punched card, tape; frequency of collection; will this data be available for introduction into a centralized water information system): | | | | |
| 11. Professional Papers of Articles Resulting from Project (Copies of Reprints would be helpful): | | | | |
| 12. Data Availability: | | 13. Province: | | 14. County: |
| 15. Basin: | 16. Township: | 17. Range: | 18. Section: | 19. Longitude-Latitude: |

Figure 1, Water Resources Questionnaire

University and Arizona State University, as well as the Statewide Arizona Resource Information System (ARIS). Additional types of information, including legal decisions, will eventually be incorporated. Close ties with the U.S. Geological Survey, Office of Water Data Coordination, and the Department of Economic Planning and Development are essential to avoid duplication of federally or state derived data; however, work must still be done to maximize use of the federal STORET system.

The pilot study has involved a substantial amount of public relations and sales work. It is not uncommon that a potential user can be turned into an actual user by simply demonstrating the product As more data is stored, such capability demonstrations will most assuredly create new users. The reporting system has not been finalized, but progress is encouraging. Currently, a systems software package written for the University of Arizona CDC 6400 is in use and all information is stored on magnetic tapes for subsequent retrieval.

Once operational the long range structure of AWIS may appear as shown in Figure 2. The movement of data between agencies, universities, and planners working in the field of Arizona's water resources actually represents an exchange of data and information inasmuch as these groups may maintain active files that could benefit from previous work and data collection activities of others. The linkages as shown in Figure 2 are avenues for the movement of data requests and for the exchange of data between the various components. The major functions of AWIS will be: (1) to serve as a central con-

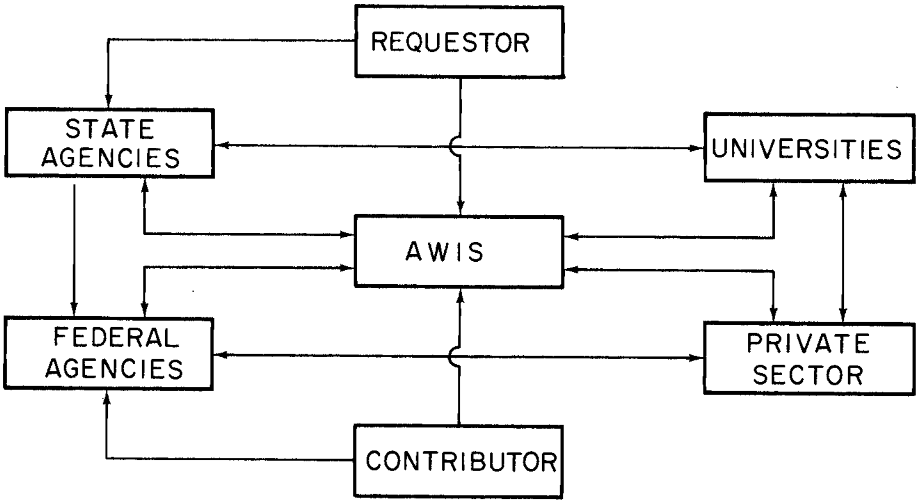


Figure 2. Linkage of Units Within the AWIS

tact point to which users of water resource information may direct requests; (2) to maintain a central bank of data and information for those wishing to delegate dissemination of their data to AWIS; (3) to refer requests to the proper agency disseminating their own data; and (4) to assemble knowledge as to the location of water data in the State. The above functions could have several advantages: (1) decrease data collection efforts of some agencies; (2) allow researchers to know of activities in the State, thus reducing duplication of effort, and (3) location of areas where additional emphasis should be placed on research.

Work will be conducted during FY 72/73 with the State of Arizona to determine how AWIS can best participate in the State source Information System (ARIS). AWIS may eventually become an operational subsystem of ARIS; but for the present time, the Office of Arid Lands Studies, representing AWIS, will serve as active member of the ARIS advisory work group.

Currently, the data base is small, limited to water quality data supplied AWIS by the State Department of Health for approximatel 450 wells in the Tucson Basin, and to water level, storage, storage coefficient, and transmissibility data supplied by the Arizona Water Commission for the Tucson Basin and Avra Valley.

This year's study has brought to light two important considerations in the design of an information system. These are standardization techniques for:
(1) data-station identification and (2) data quality.

The techniques used by many of the collector agencies for identifying data stations vary. For data-station identification a definition is necessary for three descriptive terms: identifier, locator, and descriptor. An "identifier" is a code that is unique to a particular station and once assigned is never changed. A "locator" is a code or item of information that helps pinpoint the location of the station in the field. The AWIS is currently using two locators: the USGS range Township Section code and latitude-longitude correct to the nearest minute. A "descriptor" is a code or item of information that describes the station, its installation, or record.

Data quality is a subject of great importance to the overall value of a data-handling system. Users should be aware of any potential problems that might arise as a result of the quality of the data. The quality of a measurement is affected by not only the method used but also by the field conditions at the time of measurement. Hence, two codes should accompany the measurement--one to indicate methodology and the second to provide information on the field conditions that may have affected the application of the method or technique, and hence the quality of the parameter measurement. The two codes at this time have not been formulated; however, a method for successfully delineating and reporting data quality will receive greater attention this coming fiscal year.

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# MAN-NATURE ATTITUDES OF ARIZONA WATER RESOURCE LEADERS

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## INTRODUCTION

Human attitudes are an important element in natural resource management. Most studies of attitudes in a natural resource context have considered only users of natural resources (Folkman et.al.,1968; Folkman, 1965; Hendee et. al., 1968; West et. al., 1968; West and Merriam, 1969). This paper reports on a pilot study conducted in Arizona to develop a man-nature attitude scale for natural resource professionals and leaders (Kanerva, 1969).

## METHODS

The study attempted to measure the frame of reference used by professionals and interested laymen in the field of water resources in a decision-making context. The study model consisted of a man-nature interaction system which was viewed as an open system representing the general interaction between man and nature (Figure 1). The area within

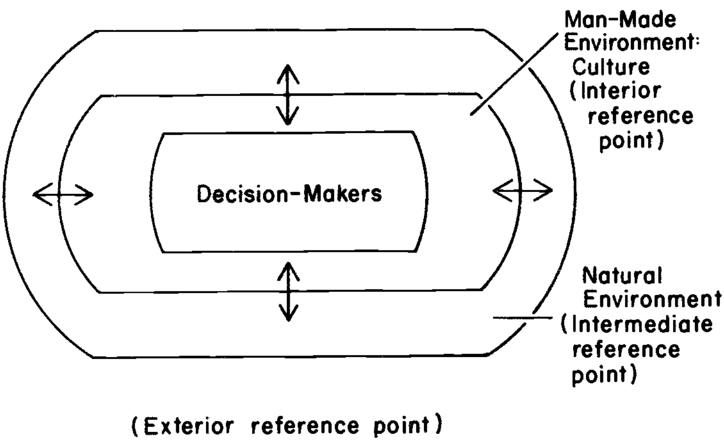


Figure 1. General system for the man-nature interaction.

The area within the largest oval represents the natural environment. The area within the middle oval represents the man-made environment, culture, the medium through which man views the natural environment. The smallest oval represents man as a part of the man-made and natural environments. The interactions, shown by the arrows, occur between man and the natural environment with cultural processes acting as a mediary (Smith, 1966).

The decision-making of man within the general man-nature system was represented by another open system shown in Figure 2 (Campbell et.al., 1960). The actual decision process is made up of three parts: the frame of reference of the decision-maker, the integration or thought process, and the input stimuli. In the study the integration process was taken as given. The frame of reference was considered the critical factor and was analyzed by comparing it to a variable set of input stimuli (questions).

In the general man-nature interaction system there are three basic frame of reference positions: the cultural or interior, the natural or intermediate, and the balanced or exterior (Figure 1). The exterior reference position is the most general; not wholly dependent on either the cultural or natural environments. When the decision-maker takes this mental step outside the whole system, he is able to view it in perspective and effect a more objective decision than from the other two reference positions.

The three reference positions are shown in Figure 3 on an interaction continuum from culture to nature. The exterior reference position is shown at the midpoint. In order to obtain a measure of an individual's position on this interaction continuum it is necessary to choose a qualitative variable and develop an attitude scale to measure it. The qualitative variable chosen was attitude towards human management.

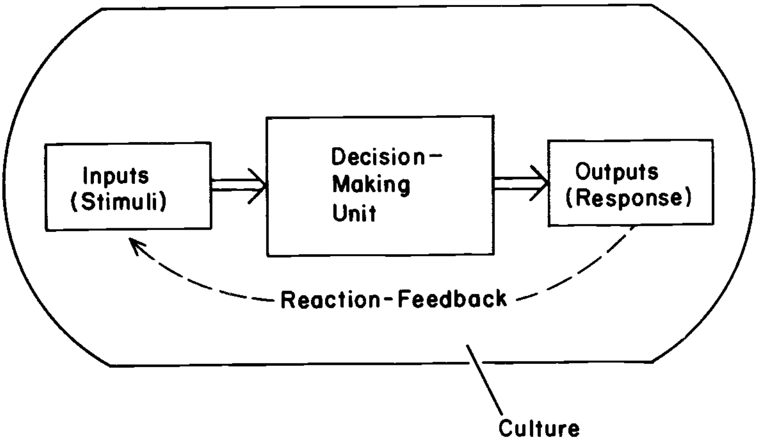


Figure 2. A decision-making model.

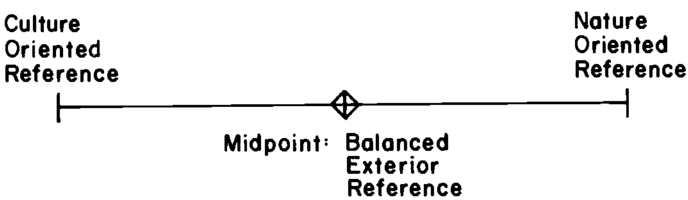


Figure 3. Frame of references interaction continuum.

The human management attitude continuum is shown in Figure 4. The continuum represents, from left to right, an increasing degree of positive attitude towards human management. The left part represents a low degree of human management attitude and a cultural reference position. The middle part of the scale represents a medium degree of the attitude and a balanced reference position. The right portion of the scale represents a high degree of the attitude and a nature oriented reference position. These portions may be described in more familiar, and perhaps controversial terms, left to right, as exploitation, conservation, and preservation attitudes.

Guttman scalogram analysis was used to develop the human management attitude scale (Stouffer et. al., 1950). The Guttman procedure was chosen because it makes possible the development of a unidimensional attitude scale along which respondents can be rank ordered. The procedure is too long to be completely detailed in this paper. A thorough explanation is provided by Anderson, Watts, and Wilcox (1966) in Legislative Roll Call Analysis. Briefly, the process includes developing a set of questions relevant to the qualitative variable. In this case, a set of 12 dichotomous questions were prepared and pretested (Oppenheim, 1966). Next, "favorable" responses to each questions must be determined based on question content. A favorable response is one that indicates a positive attitude towards the qualitative variable. The questions are then administered to a group of individuals. The remainder of the process involves the analysis of the responses. This analysis proceeds, through several steps, to determine whether questions are scalable or not through examination of question response

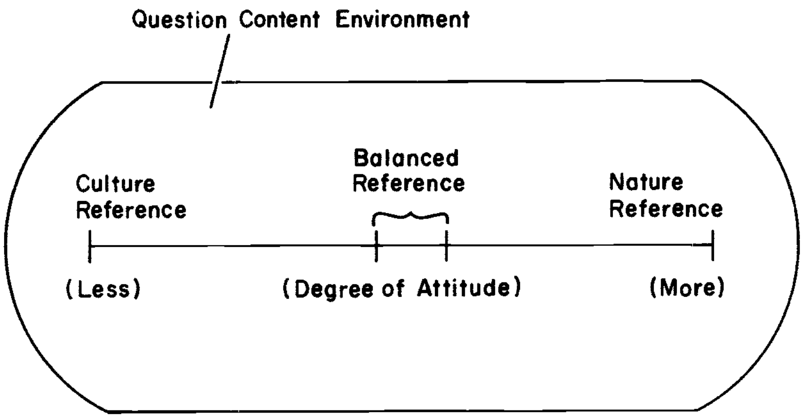


Figure 4. The human management attitude continuum.

patterns. The end result is a set of questions constituting the scale. The total number of favorable (positive) responses given by a respondent is his scale score.

Because this was a pilot study, a purposive sample was taken (Kish, 1965). The main study group of 86 persons consisted of people who had attended the 1967 Arizona Watershed Symposium and faculty members at the University of Arizona. In addition, one group of Watershed Management graduate and undergraduate students was used to pretest the questions. A second group of Watershed Management students was used to check the scale after it had been developed from the responses of the main study group.

## RESULTS

The original set of 12 questions was reduced to eight by the scalogram analysis, four questions were found to be nonscalable. The eight questions forming the human management attitude scale with their positive responses indicated were:

A) As a manager planning natural resource uses, which factor would be more important in your decisions: man's physical or man's emotional needs.

___+___ 1. Man's physical needs

_______ 2. Man's emotional needs

B) In general, which do you think is more important: deciding who gets what amounts of our useable natural resources or increasing the supply of our useable natural resources?

___+___ 1. Deciding who get what

_______ 2. Increasing the supply

C) In making decisions about using natural resources, would you take into account how people behave or would you not include this factor?

___+___1. Take into account

______2. Not include this factor

D) To protect certain areas, would you choose not developing these areas for public use or managing the public that uses these areas?

___+___1. Not developing these areas

______2. Managing the public

E) In general, do you think the manipulation of natural resources is a harmful practice or a beneficial practice?

___+___1. A harmful practice

______2. A beneficial practice

F) In making decisions about using natural resources, would you consider municipal and industrial demands important or consider these demands not so important?

______1. Consider these demands important

___+___2. Consider these demands not so important

G) Which would you choose: managing to take into account the opinions of groups that use natural resources, or managing according to the opinions of the managers?

______1. Take into account the opinions of groups

___+___2. According to the opinions of the managers

H) Do you think that the amounts of natural resources people use everyday are more than is needed or that these amounts are needed for modern living?

___+___1. These amounts are more than is needed

______2. These amounts are needed

The scale was found to meet all of 5 criteria:

1. The coefficient of reproducibility was .91, greater than the standard of .90 (Stouffer et. al., 1950).
2. There should not be a large number of respondents who all have the same non-scale pattern of responses. Three respondents were the most that fell into any one of 28 different non-scale patterns (Torgersen, 1958).
3. The initial set of dichotomous questions should be greater than ten (Stouffer et.al., 1950). Twelve were used.
4. The error ratio was .26, less than the suggested upper limit (Riley, 1963).
5. The responses of a group of students were checked against criteria 1, 2, and 4 above with favorable results. Although the students' scale scores were more evenly distributed than the main study group, they did not differ greatly.

As mentioned previously, the scale represents an increasing degree of positive attitude towards human management. A low score indicates a culture based reference position. A high score indicates a nature based reference. A middle score indicates a balanced reference.

The distribution of scale scores for the main study group were:

| Scale Score | Percent of Respondents |
|---|---|
| 0 | 0 |
| 1 | 9.3 |
| 2 | 30.2 |
| 3 | 32.6 |
| 4 | 9.3 |
| 5 | 12.8 |
| 6 | 3.5 |
| 7 | 1.2 |
| 8 | 0 |

The median score was 3. This means the study group was approaching a balanced attitude towards human management and, thus, approaching a balanced and objective reference position.

Respondents were asked whether they felt man and nature are in conflict. Sixty-nine percent said yes and 16 percent said no. This is in general agreement with the groups' overall attitude.

Sixty-nine percent of the natural resource professionals in private business or self-employed had scale scores in the 0 to 2 range (Table 1). This indicates a development philosphy and a culturally oriented reference position. Government professionals approached a more balanced attitude and reference position, only 35 percent of them had scale scores in the 0 to 2 range. While 30 percent of the natural resource educators had scale scores in the 0 to 2 range, 44 percent had scores in the 4 to 8 range. Thus, educators tended to have a balanced reference position and a philosphy of conservation. These groups, however, have different modes and degrees of influence on natural resource management decisions.

Respondents were asked to list the groups, clubs, and civic organizations to which they belonged. Respondents with membership in a greater diversity of groups tended to have higher scale scores than those belonging to a less diverse set of groups (Table 2). It would be expected that a person exposed to more points of view would have a more balanced attitude.

Respondents who had lived longest most of their lives in the West or East tended to have lower scale scores than those who had lived longest in the Central region (Table 3). It might be hypothesized that the Central region has an environment that is, in some unknown way, conducive to a more balanced reference position.

Table 1 Distribution of Respondents by Scale Score and Type of Employer

| Scale Score Group | Employer: Government | Private Business or Self-Employed | Educational Institution |
|---|---|---|---|
| 0-2 | 33% | 69% | 30% |
| 3 | 44% | 15% | 26% |
| 4-8 | 22% | 15% | 44% |

$X_4^2$ = 10.0 significant at .2 level

Table 2 Distribution of Respondents by Scale Score and Number of Group Types

| Scale Score Group | Number of Group Types: 1-2 | 3-5 |
|---|---|---|
| 0-2 | 45% | 22% |
| 3 | 28% | 48% |
| 4-8 | 28% | 30% |

$X_2^2$ = 4.1 significant at .2 level

Table 3 Distribution of Respondents by Scale Score and Region Lived in Longest

| Scale Score Group | Region | | |
|---|---|---|---|
| | West | Central | East |
| 0-2 | 47% | 20% | 50% |
| 3 | 33% | 36% | 33% |
| 4-8 | 20% | 44% | 17% |

$X_4^2 = 7.5$ significant at .2 level

## CONCLUSIONS

The pilot study was successful in constructing a scale measuring attitude towards human management. The scale, however, should receive further testing. One approach would be to test the scale on two groups believed to have greatly different attitudes, such as the Sierra Club and the Arizona Mining Association. A more interesting test would be to compare scale scores of natural resource managers with actual management decisions.

The analysis of associations between scale scores and respondent characteristics was not completely successful because the sample size was inadequate. Another similar study, but utilizing a larger sample, should be done to determine if such associations do exist. The existence of such associations would tend to validate the scale.

It is felt that techniques and studies such as these can provide natural resource managers with effective means of objectively evaluating and improving their management decisions.

Acknowledgements: The authors express their gratitude to the respondents whose cooperation made the study possible. Donald E. Freeman, former Assistant Professor of Government at the University of Arizona was especially helpful in the questionnaire design stage of the study. This study was published as Arizona Agricultural Experiment Station Paper 45.

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The Groundwater Supply of

Little Chino Valley

by

W. G. MATLOCK AND P. R. DAVIS*

## INTRODUCTION

Little Chino Valley is located about 15 miles north of Prescott in the west central part of Arizona (Figure 1). The area was originally settled as an agricultural community and in 1915 a surface water supply from Lake Watson Reservoir on Granite Creek near Prescott was obtained for irrigating about 2400 acres.

The first well in the area for irrigation was drilled in 1927. Although not recognized at the time, this well reached an artesian aquifer. The first flowing well was drilled in 1930 and had an initial flow of about 350 gallons per minute (gpm). Since that time more than 100 wells have been drilled into the artesian aquifer, not all of them successful, but some flowing at rates as high as 1500 gpm. Presently most of the artesian wells do not flow because the ground surface is above the piezometric surface of the aquifer, but with pumps may supply up to 3000 gpm.

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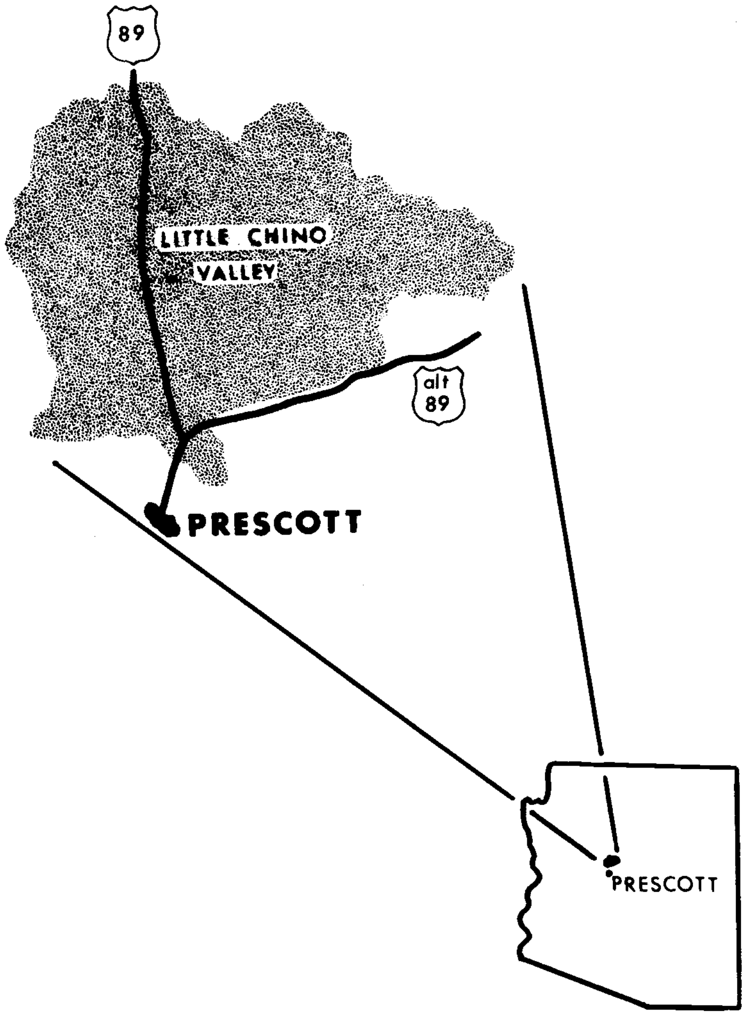


FIGURE 1. LOCATION MAP OF LITTLE CHINO VALLEY

Groundwater resource investigations were initiated in Little Chino Valley by the Arizona Agricultural Experiment Station at the request of local farmers in 1937. Use of water from the artesian aquifer for irrigation was small, but already annual and seasonal lowering in artesian pressure had become a cause for concern.

Lowering of water levels and artesian pressures has continued. To prevent any further development of wells for irrigation, Little Chino Valley was included in the Granite Creek Critical Groundwater Basin created by decree of the State Water Commissioner in 1962.

## DESCRIPTION OF THE BASIN

The surface drainage basin covers the 215 square miles shown in Figure 2. Rock formations define the basin boundary except in the southeast section where the divide is along the Prescott-Jerome Highway.

The valley is composed largely of gently rolling hills and flatlands covered by short grass. Little erosion is evident. Valley floor elevations range from about 4400 to 5300 feet with surrounding mountains extending to nearly 7900 feet.

The climate is dry and windy. Mid-afternoon relative humidity is less than 20 percent prior to the summer rains. Afternoon wind is from 10 to 15 miles per hour during the March-September period. Average annual rainfall of about 11 inches occurs principally in two seasons: about 40 percent during July, August and September, and 30 percent in December, January, February and March. (Schwalen, 1967).

Three sub-areas of surface drainage can be defined. Big Draw drains a strip about 3 miles wide along the western edge; Little Chino Creek drains a

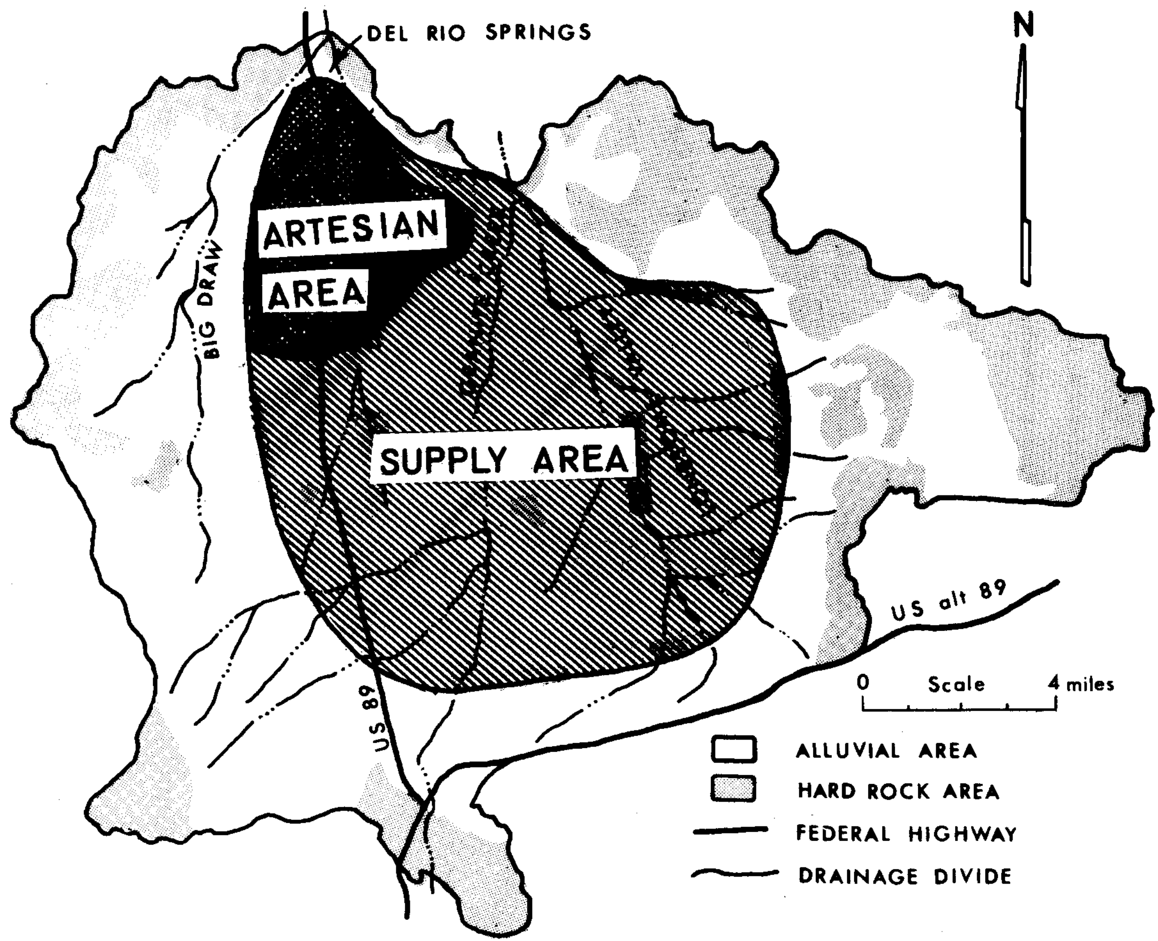


FIGURE 2. LITTLE CHINO VALLEY DRAINAGE BASIN

central portion; and Granite Creek drains Lonesome Valley, the eastern two-thirds of the area.

There is no perennial flow in the surface water channels of the drainage basin except in the extreme lower end. Del Rio Springs, located just north of the artesian area, was known to early pioneers as a source of excellent quality water. The flow here is supplied largely by upward leakage from the artesian formation with some seepage from overlying alluvial materials. From 1904 to 1927, the City of Prescott pumped water from Del Rio Springs through a 21 mile pipe line.

The geology was defined by Schwalen (1967) and will only be summarized here. The area is in the mountain section of the Basin and Range Province in Arizona bordering the southwestern edge of the Plateau Province. Dense pre-Cambrian granites, schists and related rocks surround most of the groundwater basin forming both a surface divide and a barrier to groundwater flow.

In the previously noted southeastern section, the surface divide is entirely on valley fill. Water level measurements in this area suggest that the ground-water divide is at approximately the same location.

Alluvial valley fill occupies about three-quarters of the surface area with an older alluvium near the edges of the valley and as erosional remnants in the central part. Buried volcanic flows (basalt) are interbedded in the older alluvium and as isolated outcrops on the valley floor.

A younger Quarternary alluvium occupies the lower central portion of the Granite and Little Chino Creek drainages. Recent alluvium is found in the Granite Creek floodplain and in the flat-bottomed swales of other major tributaries.

## THE AQUIFER SYSTEM

Groundwater studies in the Little Chino Valley area are interesting because of the multi-phase aquifer system which exists. The extent of the artesian area and its associated supply area are shown in Figure 2. Water table conditions prevail throughout the supply area.

Figure 3 is a simplified cross-section of the aquifer system. The artesian system is comprised of a series of interconnected, buried lava flows which are porous and frequently of cavernous character. The lava flows are interbedded with valley fill, largely clay, of varying thickness which provides the confining layer.

The lava flows have not been completely defined, nor even located in all parts of the groundwater basin. South of the artesian area lava was found in only a few wells some distance away. Lava was not found in the wells in the Big Draw drainage on the west side. Lava was found in two wells in Lonesome Valley, but there is insufficient evidence to make meaningful conclusions about its continuity.

The transition zone between the water table conditions in Lonesome Valley and the artesian area is poorly defined. Few wells are available from which to draw conclusions.

There is a definite hydraulic barrier on the north end of the basin which significantly reduces leakage from the artesian aquifer in that direction. The exact locstion of the barrier is unknown, but the sudden increase in hydraulic gradient which occurs there is taken as proof of its existence.

A water table aquifer in the overlying slluvial materials appears to be continuous throughout the artesian area. It is fed largely by excess irrigation water, but may also receive recharge from the local stream flow.

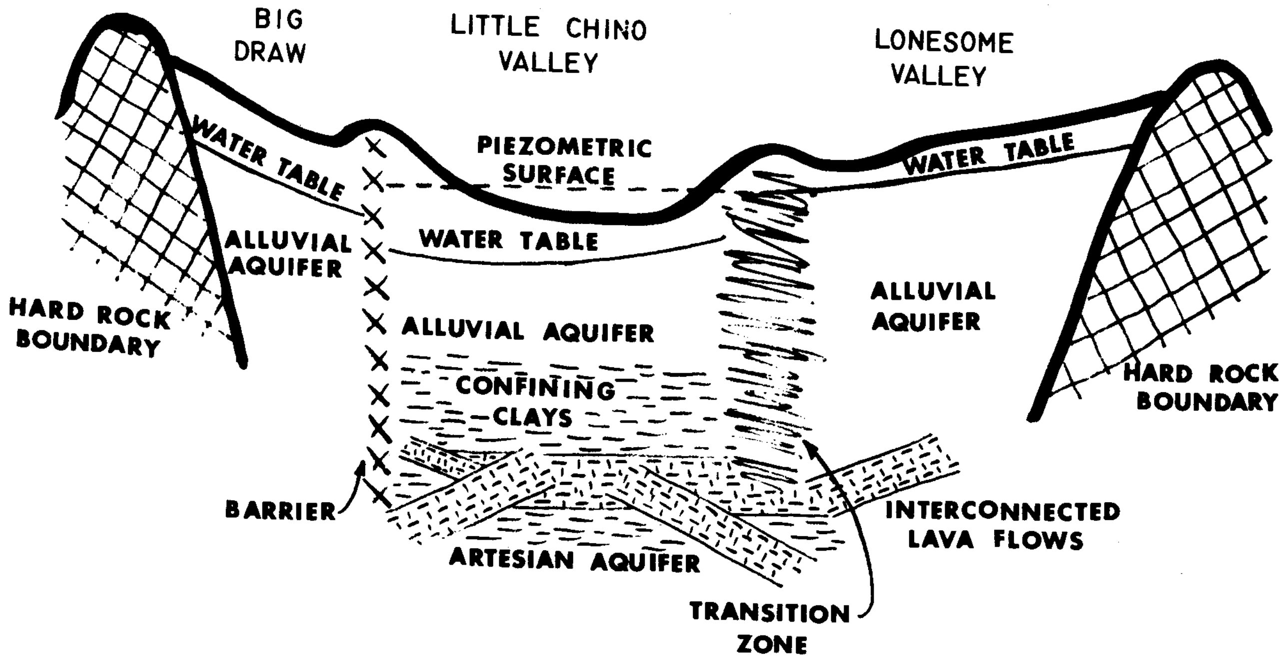


FIGURE 3. SIMPLIFIED CROSS-SECTION OF THE AQUIFER SYSTEM

Seepage from the water table aquifer occurs at the northern end of the area in Little Chino Creek and from the sloping lands south of Del Rio Springs. Analysis of water quality shows that at this point the water from the water table aquifer mixes with the upward leakage from the artesian aquifer.

Another water table aquifer is located in the Big Draw drainage area. An undefined barrier or discontinuity separates it from the artesian aquifer and the overlying water table aquifer.

## DATA COLLECTION

Since 1938 basic groundwater data have been collected each year. Measurements of water levels and artesian pressures are made each Spring in about 75 wells. Well logs and other drilling information are obtained from local drillers; water samples are taken for chemical analysis and well discharges are measured. A crop survey is made to determine irrigation water use; other water uses are measured or estimated.

### Water Levels and Artesian Pressures

Hydrographs of annual measurements in selected wells are shown in Figure 4. Wells in the artesian area all show the same lowering trend, about 2 feet per year, even though they are miles apart. Water table wells in Lonesome Valley have also declined at the same rate. Wells outside the artesian basin, however, show little or no change in water levels over the 35-year period.

As is typical of artesian aquifers a seasonal fluctuation in pressures takes place. In the summer months when water withdrawal for irrigation commences almost simultaneously throughout the basin a rapid decrease in pressures results; then when pumping stops, pressures quickly rise. The magnitude of the fluctuations varies with location. The maximum pressure variation occurs in the

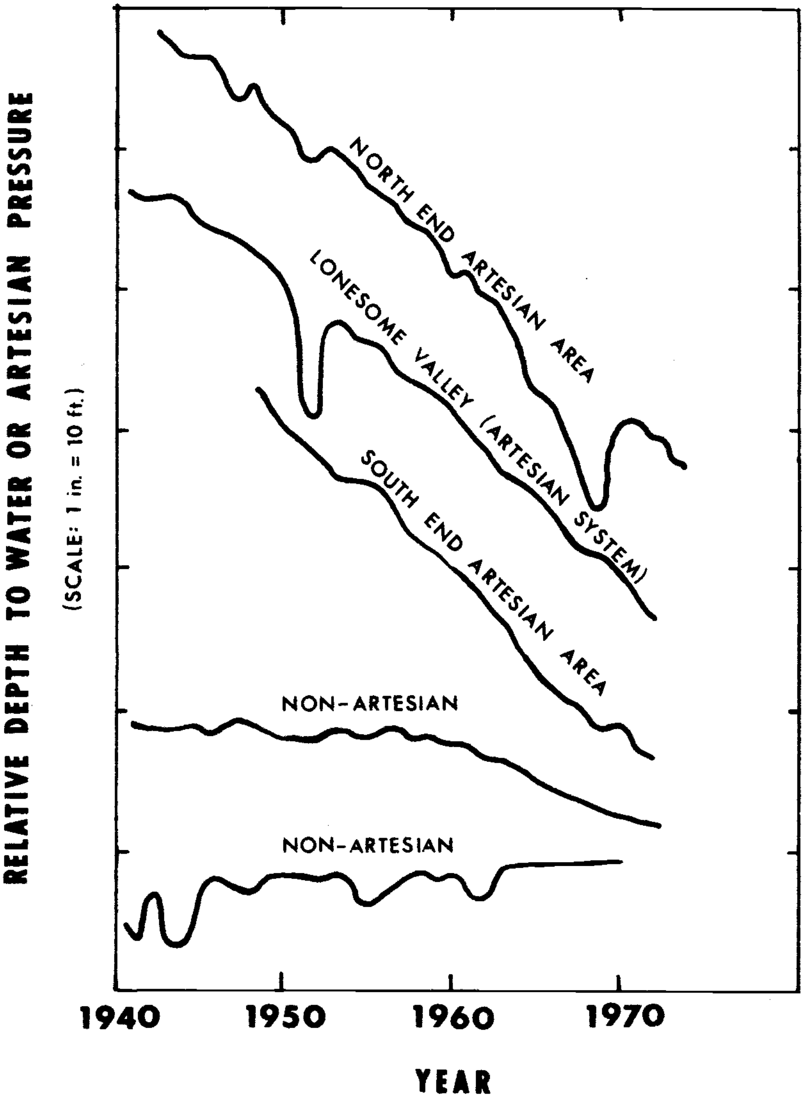


FIGURE 4. HYDROGRAPHS OF ANNUAL MEASUREMENTS IN SELECTED WELLS

north with a decreasing effect with distance to the south. A slight fluctuation can be seen in Lonesome Valley wells (Figure 5).

Water level fluctuations also occur in the surface water table wells in the irrigated area (Figure 5). Highest water levels occur at the end of the irrigation season and lowest levels are found just before irrigation starts in the Spring indicating the recharge of excess irrigation water.

Figure 6 is an artesian pressure surface and water level contour map of the artesian aquifer system based on measurements made in the Spring of 1972. Dashed lines are used in this and following figures in areas of uncertainty. Maps made from earlier measurements are similar in general character, contour shape, etc., indicating flow from Lonesome Valley and the south into the artesian area with all flow exiting in the north.

The previously discussed hydrographs show the lowering of water levels and artesian pressures at individual wells; Figure 7 is a map of the entire basin with lines of equal lowering for the time period 1940-72. It shows more clearly the areal extent of the lowering into the expanses of Lonesome Valley.

A North-South profile through Little Chino Valley with artesian pressure surfaces and water levels in the Spring of 1940, 1960, and 1972 is shown in Figure 8. The piezometric surface is seen to be almost level for a distance of 6 miles.

Quality of Water

The chemical quality of water from the artesian aquifer is excellent and remarkably uniform. Samples taken at wells some distance apart show almost the same concentration of constituent elements (See Table I.).

Water samples from surface water table wells, on the other hand, are more

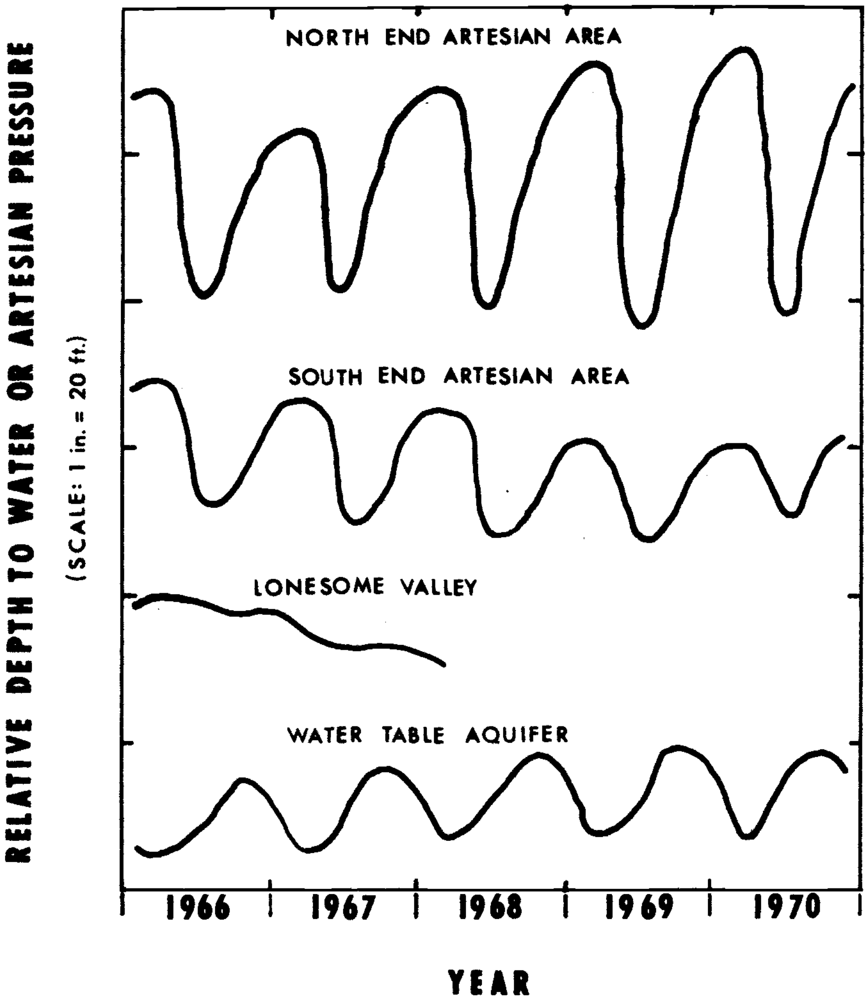


FIGURE 5. SEASONAL FLUCTUATIONS IN WATER LEVELS AND ARTESIAN PRESSURES

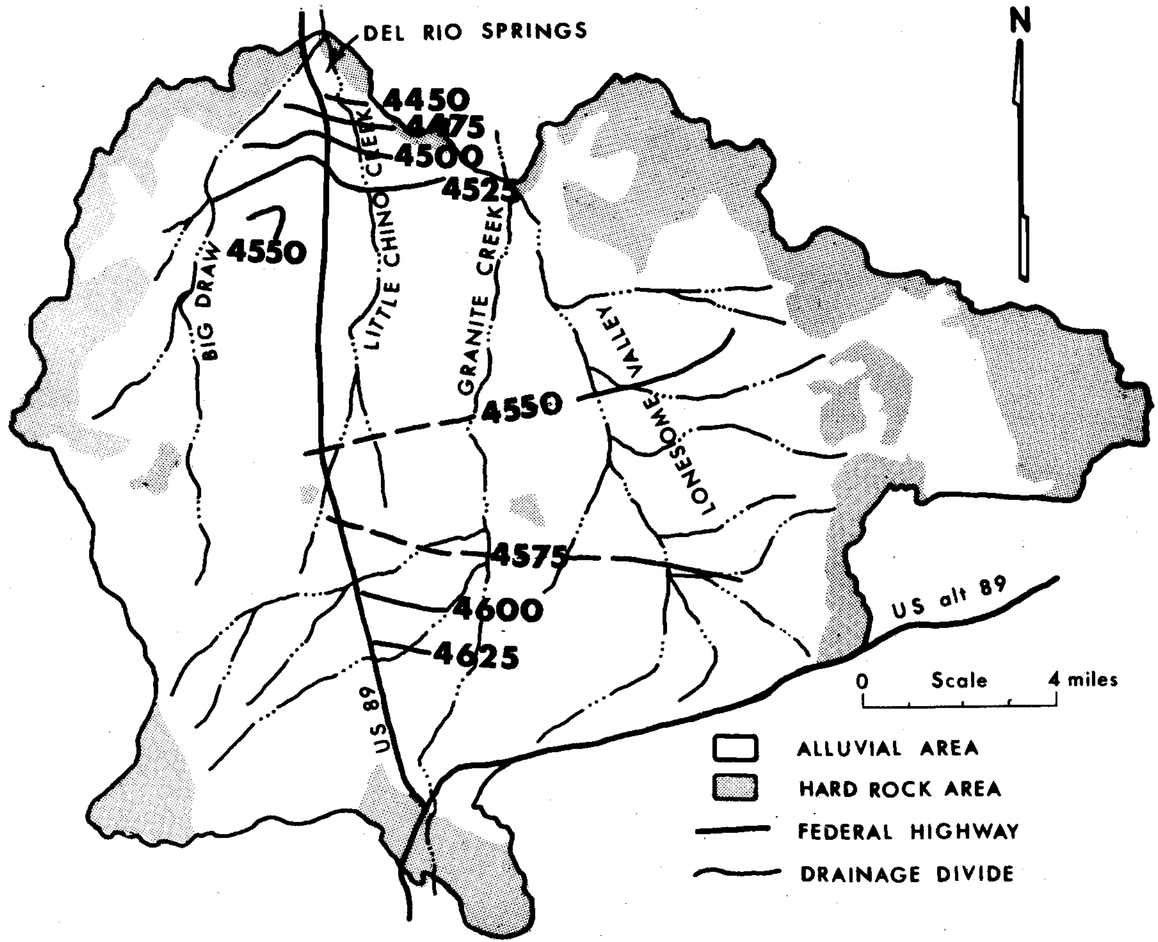


FIGURE 6. ARTESIAN PRESSURE SURFACE AND WATER LEVEL CONTOURS, 1972

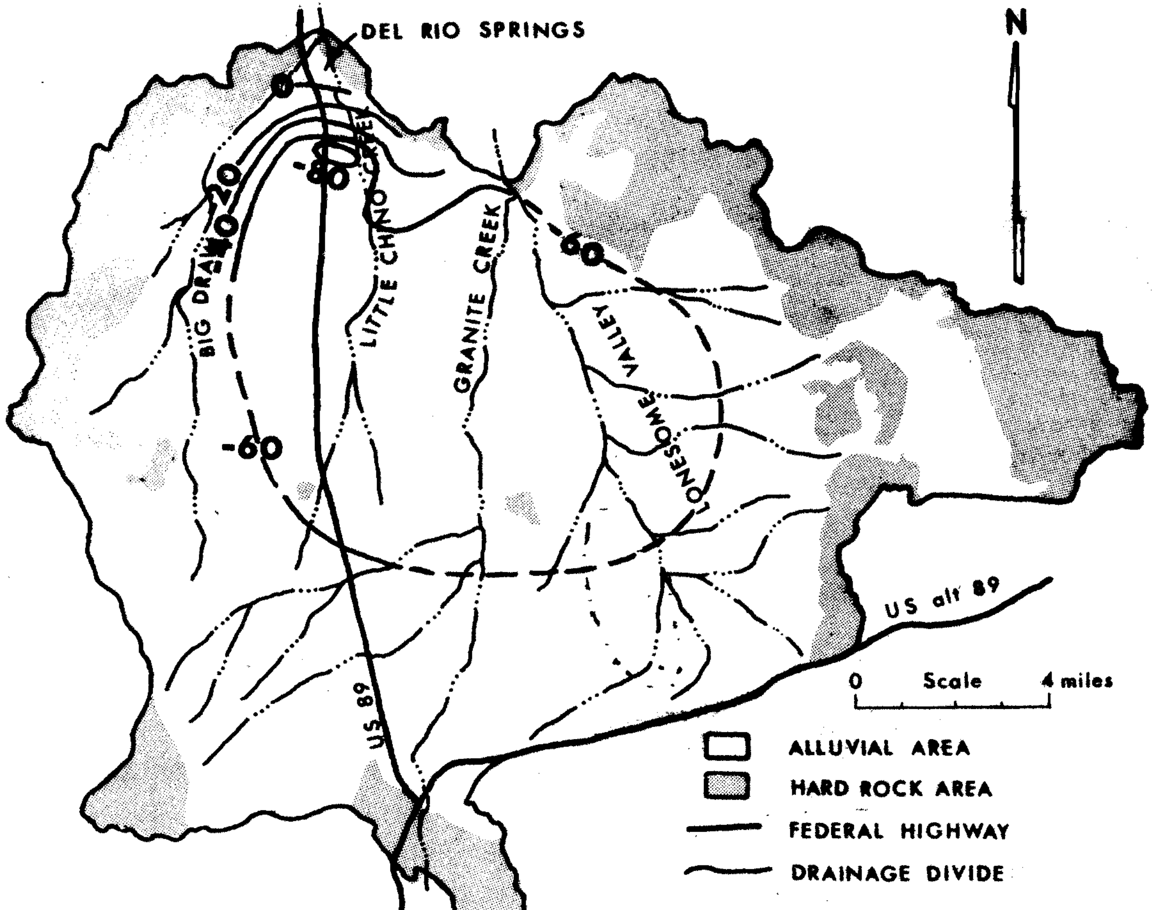


FIGURE 7. LOWERING OF ARTESIAN PRESSURES AND WATER LEVELS, 1940-1972

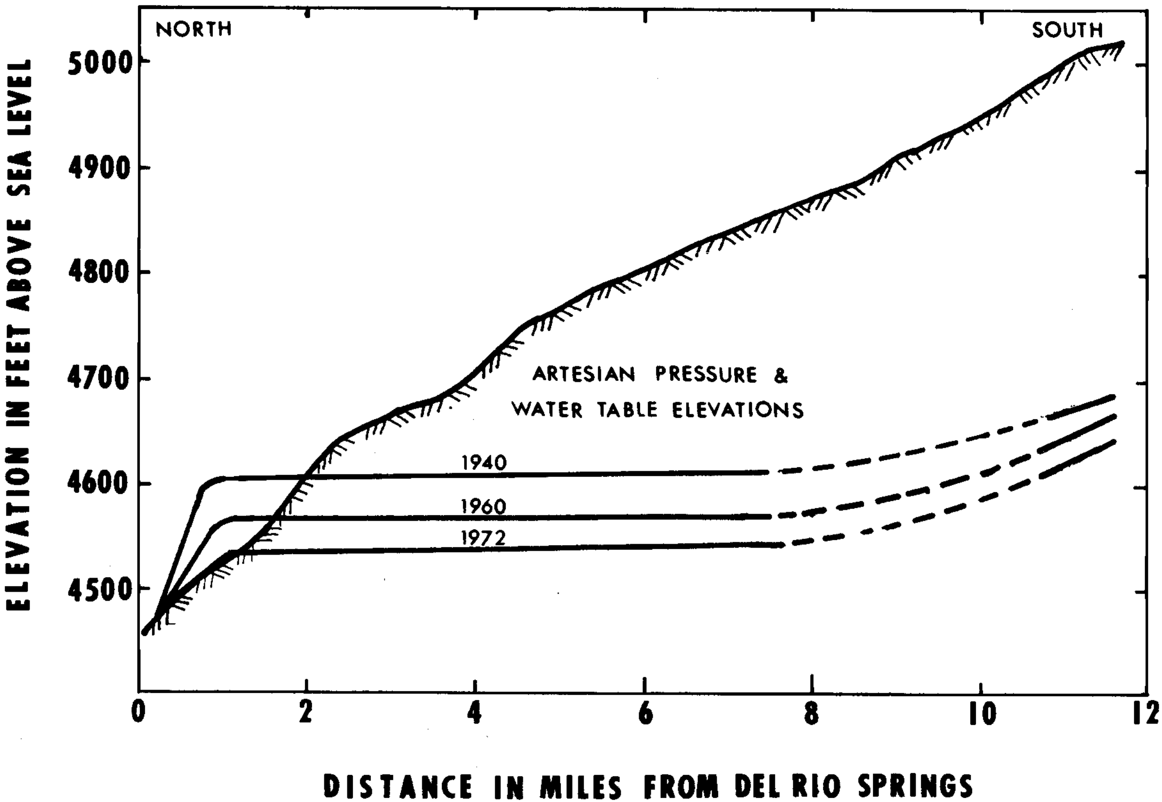


FIGURE 8. NORTH-SOUTH PROFILE OF ARTESIAN AQUIFER SYSTEM

variable in their individual characteristics and also are considerably harder and more saline. Considering that these waters are in part the result of deep percolation of excess irrigation water, the quality exhibited is not surprising.

TABLE I. Chemical Quality of Groundwater

| TSS | Ca | Mg | Na | Cl | $SO_4$ | $CO_3$ | $HCO_3$ | $NO_3$ | F | Hardness (gr/gal) |
|---|---|---|---|---|---|---|---|---|---|---|
| ppm | | | | | | | | | | |
| ARTESIAN WELLS | | | | | | | | | | |
| 230 | 30 | 12 | 14 | 16 | 16 | 0 | 137 | 5 | | 7.3 |
| 215 | 30 | 10 | 13 | 20 | 14 | 0 | 122 | 6 | | 6.8 |
| 220 | 30 | 12 | 11 | 16 | 0 | 0 | 151 | - | 0.3 | 7.3 |
| SURFACE WATER TABLE WELLS | | | | | | | | | | |
| 704 | 96 | 50 | 46 | 170 | 98 | 0 | 244 | - | 0.3 | 26.0 |
| 827 | 203 | 11 | 42 | 256 | 91 | 0 | 274 | - | | 32.2 |

## Water Use

Water is pumped in Little Chino Valley principally for irrigation and municipal (domestic) use. There is no local industrial use other than for agricultural purposes.

In 1948, the City of Prescott purchased and retired agricultural land from production in Little Chino Valley to obtain a municipal supply. Since that time more than 34,000 acre-feet of water have been pumped and transported by pipeline to the City of Prescott.

Most irrigation wells are not equipped with flow measuring devices, hence no records of pumpage are available. Water pumped for irrigation is computed from crop surveys on the basis of 3.6 acre-feet of water applied per irrigated acre. Water pumped and crop acreage for selected years in Little Chino Valley are given in Table II.

TABLE II. Crop Acreage and Water Pumped -- Little Chino Valley Artesian Area (selected years).

| Year | Irrigated Crop Acres | Irrigation Water Pumped (acre-feet) | Metered Pumpage City of Prescott (acre-feet) | Total Water Pumped (acre-feet) |
|---|---|---|---|---|
| 1939 | 1004 | 3714 | 00 | 3714 |
| 1943 | 1714 | 5658 | 00 | 5658 |
| 1947 | 2783 | 9183 | 00 | 9183 |
| 1951 | 3501 | 11559 | 1129 | 12688 |
| 1955 | 3362 | 12105 | 1054 | 13159 |
| 1959 | 3556 | 12802 | 1594 | 14396 |
| 1963 | 3590 | 11235 | 1974 | 13209 |
| 1967 | 2600* | 9360 | 1960 | 11320 |
| 1971 | 2758 | 9930 | 2000** | 11930 |

* Estimate -- incomplete survey showed 2093 acres

** Estimate

## WATER BUDGET FOR THE ARTESIAN AQUIFER SYSTEM

A preliminary estimate of the relationships among recharge, discharge and change in storage for the artesian aquifer system was made by Schwalen (1967). An evaluation for the 1967-72 period can now be made based on the newer data.

Before development of the Little Chino Valley artesian aquifer by man, the natural discharge from the aquifer was probably in equilibrium with the natural recharge to it. Surface flow of about 3000 acre-feet per year was measured at Del Rio Springs before significant withdrawals from the artesian aquifer were being made. Seepage from the water table aquifer was negligible. Additional natural discharge occurs as underflow in the Del Rio Springs area and westward to the hard rock boundary. Based on an estimated cross-section 4000 feet wide and 200 feet thick with an estimated permeability of 50 gallons per day per square foot, and a hydraulic gradient of about 150 feet per mile measured in 1938, an annual flow of about 1000 acre-feet was calculated.

Thus, average natural recharge was about 4000 acre-feet, a value that should still be valid today since nothing is known to have occurred to influence significantly the recharge. Major sources are: 1) direct recharge in fractures and joints in the rock boundaries of the basin and lava (basalt) outcrops throughout the valley, 2) seepage losses from canals and sandy stream channels south of the artesian area, and 3) deep percolation losses from irrigation in the same area.

Falling artesian pressures do not result in significant water release from storage, but in the supply area composed primarily of Lonesome Valley and the area south of the artesian area, a change in water level represents loss from storage. Total volumetric unwatering for the five year period was computed as 400,000 acre-feet or an annual average of 80,000 acre-feet.

Because of the decrease in hydraulic gradient at the barrier from 150 feet per mile to about 80 feet per mile the natural discharge as surface flow at Del Rio Springs and underflow from the aquifer also has decreased. For the period 1967-72, it was estimated to average only about 2300 acre-feet per year. Average annual pumpage for the same period was 11,500 acre-feet.

A hydrologic budget equation is:

$$W = R + SY_s$$

in which:

$W$ = withdrawal (pumpage and leakage)

$R$ = recharge

$S$ = change in storage (aquifer volume dewatered)

$Y_s$ = specific yield (amount of water released from a unit volume of aquifer when the water level falls 1 foot)

Using data for the 1967-72 period, the equation is:

$$13{,}800 = 4000 + 80{,}000\ Y_s$$

Specific yield for the period was calculated to be 12 percent which is comparable to the values computed for previous time periods.

## SUMMARY

The Little Chino Valley area is an interesting area for groundwater study. Like most areas in Arizona the groundwater is being depleted (withdrawal exceeds recharge). Water levels and artesian pressures are falling in the artesian system at a rate of about 2 feet per year over most of the area. A surface water table aquifer overlies the artesian aquifer and receives recharge from excess irrigation. Water quality in the artesian aquifer is excellent for most purposes; the water from the surface water table aquifer is somewhat higher in total salts, but still suitable for domestic and agricultural supplies. Specific yield for the supply area to the artesian aquifer was computed to be 12 percent on the basis of estimated annual recharge of 4000 acre-feet and leakage from the aquifer of 2300 acre-feet.

## ACKNOWLEDGEMENTS

Much of the descriptive information presented in this paper was taken from Technical Bulletin 178 of the Arizona Agricultural Experiment Station, "Little Chino Valley Artesian Area and Groundwater Basin," by Harold C. Schwalen, Professor Emeritus of Agricultural Engineering. Professor Schwalen originated the groundwater studies in Little Chino Valley and supervised activities there until his retirement in 1965.

Thanks are also given to the property owners of Little Chino Valley and nearby areas for allowing access to their wells and providing other information. Local well drillers have furnished well logs; their cooperation is gratefully acknowledged.

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Schwalen, H. C., 1967, "Little Chino Valley Artesian Area and Groundwater Basin," Technical Bulletin 178, Arizona Agricultural Experiment Station.

# SUBSIDENCE DAMAGE IN SOUTHERN ARIZONA

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University of Arizona

In western Pinal County, South Central Arizona, land is subsiding over a heavily mined aquifer. As a consequence, damages to structures on the surface occur and these damages have given rise to concern as to what remedies should be enacted to stop these subsidences.

The principal proposals to eliminate these damages are to stop the over-draught, either via water importation or, legislation to limit water pumping. The purpose of this paper is to inventory these damages to help provide a basis upon which cost studies can be performed to help determine what particular steps and actions can be taken to lessen the economic impact of these damages. To fully understand the type of damage occurring, it is necessary to examine the mechanism by which subsidence occurs. Essentially, this is due to water table drawdown and increased loading stress. The two principle ways by which this occurs are changes in the bouyant support of the grains in the zone of change, and changes in the water table or of the piezometric head, or both, which induce hydraulic gradients and seepage stress in the deposits (Lofgren, 1961). Generally, if mining conditions exist, these result in a one-directional compression of the deposits which result in subsidence of the land surface. This sinking, as indicated by U.S.G.S. maps of the area (Schuman, 1970), occurs in a fairly uniform

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fashion as the land mass sinks, with a gradually declining rate as one migrates away from the center of heaviest water withdrawal. Another visible manifestation of subsidence is the appearance of cracks or fissures around the subsiding basins. These cracks roughly parallel surface contours and transect natural drainage patterns. The trends of many of these fissures seem to conform with linear zones of steep gravity gradients. Most of these steep gravity gradients are adjacent to mountain masses and may reflect buried fault scarps along the periphery of the subsiding basin. If this is true, the buried fault scarps are sites of maximum tensile stress, and therefore the most likely sites for new fissures (Robinson and Peterson, 1962).

A second type of subsidence is very evident in other basins of similar geologic conformation. This type of activity is generally referred to as "near-surface" subsidence, and results chiefly from the compaction of deposits by an overburden load, as the clay bond supporting the voids is weakened by water percolating through the deposits for the first time (Bull, 1961). Clearly, this is the area between the surface, and the original water table. Extensive studies of this type phenomena have been carried out in the San Joaquin Valley of California. The principle clay to be found there is montmorillomite. In the study area, considerable amounts of clay (Hardt and Cattany, 1965 and Hardt, Cattany and Kister, 1964), including montmorillomite types have been found to be present, indicating that the area may be susceptible to this type of activity, though at present, case studies have not been extensively carried out.

Because of the lack of data necessary to differentiate between the two types of subsidence activity, all damages which may be attributable to subsidence were first assumed to be due solely to so-called "deep" subsidence. To alternatively suggest that both types of subsidence may be occurring in concert certainly seems reasonable. Presence of near-surface subsidence susceptible clays in the previously unwetted zone; plus large amounts of water being applied (far in excess of natural rainfall) for irrigation purposes, allowing percolation to these heretofore unwetted depths suggest near-surface subsidence to be a clear possibility. Indeed, there are numerous cases where wells now have water entering them at levels that are far above the original water table indicating the creation of artificial perched water tables. As an aside, several of these wells have required capping in this area to prevent this water's entering the wall because of its poor quality due to mineralization as it has percolated downward. If it were possible to relate specific amounts of the subsidence to the wetting of these layers, it would be most helpful, however these clay beds and lenses are extremely scattered and attributing uniformity to their role in compression would seem at best far-fetched. Probably their most likely role is that of individual slumpings which vary in magnitude as to the bed size, thickness, depth beneath the surface, and rate of wetting.

Analysis of the various damages were performed by submitting questionnaires, personally interviewing residents of the area, and on-site inspections. After preliminary efforts, it was found that these various damages fell into several categories, each of which will be examined individually in terms of both possible causes noted above.

## 1. DAMAGES TO NATURAL STRUCTURES

In strictest economic terms, a resource has no value unless it is being used. The sinking or cracking of raw desert land that is not being used incurs no cost, i.e., no damage. Arroyo generation and its subsequent contribution to streambed sedimentation is probably not of sufficient magnitude to consider here. Damages to the esthetic or scenic value of the area would also be extremely difficult to determine and in any event would probably go unnoticed as due to other than natural causes unless one were informed as to their origin.

## 2. DAMAGES TO MAN-MADE STRUCTURE

Damages to Agriculture: (A) Field Releveling: Excessive requirements for field releveling corrections to maintain proper irrigation were initially thought to be a major indication of subsidence. Reviews of Soil Conservation Service data failed to indicate this to be so. Total acreage leveled in Pinal County is near 4000 acres annually (Annual Reports, Soil Conservation Service, Eloy Soil Conservation District, 1971), (less than 2% total irrigated acreage) but less than 10% of this was necessary because of gross changes in slope. Most of the 4000 acres worked was performed at the request of the farmer to bring his fields into even better shape than when they were initially prepared. Because of increasing costs of water, brought about by greater lifting requirements, farmers are progressing toward level or near level fields to increase their water use efficiency. Other reasons for releveling are to correct disturbances created by either turning tractors or allowing water to enter the field too rapidly. In terms of near-surface subsidence activity, there are a few instances of slumpings or cracks that do not follow surface contours in fields that are not near mountain slopes. Indeed one of these actually runs at right angles to surface contours. Geologic records in the area show there are clay beds as thick as thirty-five feet in the zone above the original water table in this area. Again, the exact extent and importance of these is subject to question yet the possibility of their near-surface type reaction exists.

(B) Ditch Repair: Approximately 63,000 feet of ditches were installed or repaired in Pinal County in 1971. Again the S.C.S. states that only a very small proportion of this may be due to subsidence; less than 10%. New ditching and enlarging old ditches constitute the greatest proportion of this activity. A ditch that has definitely reversed in slope is near the area where the crack noted in (A) is, again indicating evidence of near-surface subsidence phenomena.

(C) Well Damage: Damage to wells has been caused by subsidence throughout the entire area. In the active zone of grain reassortment, the overburden "pulls" the casing downward, crushing it in this area. This is generally discovered as the well begins to produce large amounts of sand that are allowed to enter through the crushed casing area, or when the pump bowls are ruined by the sand. Records show that eight to ten wells fail in this fashion annually (Interviews with various pump and well installation and repair firms, 1972).

Near-surface subsidence also effects wells, but in a somewhat different fashion. The case of water entering wells at levels above the original water table from artificial perched water tables has already been mentioned. An effect of this is noted in another more costly type of damage which occurs occasionally where the well passed through a dry layer of clay. Subsequent to application of water on the surface, this layer became wetted and collapsed the well casing in a horizontal fashion, i.e., laterally from the sides. These are much less frequent than the vertical collapse due to deep subsidence.

Damages to Transportation Facilities: (A) Highways: Damage to highways has perhaps attracted the most attention. The principal area is near Picacho Pass, on Interstate Highway 10. A crack crosses beneath the highway that requires repair as frequently as four times a year. The other area is on State Highway 87, between the towns of Picacho and Coolidge.

In both of these areas, there is considerable indication that there may be some "near-surface" subsidence activity also occurring. In the former, the highway acts as a "mini-watershed" in time of rain. Large amounts of water pond near each side and may create further surface subsidence as a consequence. Near the area on State Highway 87 there is a large concentration of irrigated agriculture which may also be contributing to a problem of similar nature. This is in the same area where the atypical crack occurs. There are other areas where surface cracks and subsidence are "threatening" roadways. These have as yet provided no problem.

(B) Bridges: Damages to bridges have been suggested, however no evidence of same has been found. State and county highway department personnel have suggested that bridge damage is due solely to floods or inadequate construction.

(C) Interstate Pipelines: A major pipeline passes directly through this area. This is owned by El Paso Natural Gas, and provides some three million cubic feet of natural gas to Southern California daily. Several times, cracks have opened across the pipeline at right angles, however to date no damage has been experienced other than the necessity for refilling the hole. Surveillance of the area is maintained, but not at more frequent intervals than the rest of their pipelines. Interviews with their management personnel indicated that their construction techniques allow for earth movements (including earthquakes) and that the problem is minimal to them.

(D) Railroads: Interviews with personnel of the Southern Pacific Railroad provided information indicating their damage due to deep subsidence is minimal.

The same crack that crosses I-10 also crosses beneath their roadbed. They indicated that special maintenance beyond injecting concrete in holes at 45° angles under the track to stabilize it has not been required. The company "settles" the track approximately once per year, but this is not abnormal. Southern Pacific personnel made particular note of the fact that their problems in this area were much more severe following heavy rains which allowed water to collect and stand. This is very similar in nature and location to that cracked area previously noted in the discussion of highways.

Damages to Domestic and Urban Structures: Problems and damage in these areas do not seem to be much more severe than any other locations. No certain evidence exists to indicate that deep subsidence has caused extensive problems to these type structures. Near-surface subsidence caused problems may be more severe but there is little indication that they are excessive. Interviews with consulting engineers maintain that the cracking of walls in homes and street surfaces reflects little more than normal settling, and could probably have been avoided by initial construction techniques.

Based on the above catalog of damages, cost of repair will be applied to determine annual costs due to both deep subsidence and near-surface. This will help to shed light on the steps necessary to plan future land use and development.

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GROUNDWATER CONTAMINATION IN THE CORTARO AREA, PIMA COUNTY, ARIZONA

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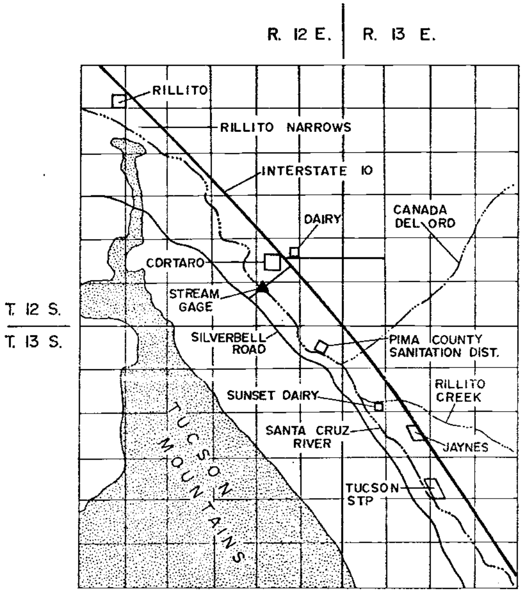
## INTRODUCTION

High concentrations of nitrate have been periodically found in water samples from wells north of the City of Tucson Sewage Treatment Plant. Cortaro, Arizona, is about 6 miles north of the plant and 5 miles south of the Rillito Narrows (Figure 1). The Cortaro Area is bounded on the south by the confluence of Canada del Oro with the Santa Cruz River, about 2 miles south of Cortaro. The Tucson Mountains lie to the west and are a complex assortment of volcanic rocks. The igneous and metamorphic sequences of the Tortolita and Santa Catalina Mountains bound the Cortaro Area to the east. The northerly boundary is where a shallow bedrock constriction occurs in the subsurface, a locality known locally as the Rillito Narrows.

Most of the Cortaro Area is within the floodplain of the Santa Cruz River, an ephemeral stream which passes northward through the Tucson Basin. The average rainfall is about 11 inches per year, much of which occurs during summer convective storms. Farming had commenced in the area by the turn of the century and was accentuated in 1920. Shallow water levels ranging from 30 to 50 feet in depth and excellent well yields resulted in a




LOCATION MAP OF CORTARO AREA

Figure 1.

substantial agricultural development. The Cortaro Water Users Association (CWUA) was formed in later years and has continued operation to the present. The CWUA owned and operated about 25 active irrigation wells in the Cortaro Area in 1971 and the annual pumpage was about 18,000 acre-feet. More than 15,000 acre-feet of this water was exported out of the Cortaro Area via canal for agricultural use in the Marana area north of Rillito Narrows. Under natural conditions, the Cortaro Area had perhaps the most favorable groundwater characteristic in Arizona.

In recent years, groundwater contamination has been observed in the Cortaro Area, from both chemical and bacteriological aspects. However, no published, detailed study of groundwater quality has been accomplished in this area. It is the purpose of the author to explore the possible sources of groundwater contamination, the extent of the contamination, and the relation to future water resources management in the Tucson Basin. In the light of more knowledge on the technical aspects of the effects of disposal of Tucson sewage effluent, more optimal decisions can be made by water resource managers in the future.

## TUCSON SEWAGE EFFLUENT

The Tucson Sewage Treatment Plant consisted of a primary treatment plant prior to 1951, when the first of three secondary plants was constructed. The present plant includes two activated sludge units and one trickling filter. The volume of sewage effluent from this plant was 2,800 acre-feet in 1940, 4,600 acre-feet in 1950, 16,300 acre-feet in 1960, and 33,000 acre-feet in 1970 (DeCook, 1970). Prior to 1951 most of the effluent was used for crop irrigation near the treatment plant, and excess volumes were percolated into the Santa Cruz River channel. A contract was finalized in 1955 with local farmers for the use of all the sewage effluent not used for irrigation at or near the plant. The excess effluent was used to irrigate about 1,500 of 2,100 irrigable acres near Cortaro from 1955 to 1970. The amount of effluent used on cropland near Cortaro was estimated to be

5,000 acre-feet in 1956, 8,000 acre-feet in 1960, 10,000 acre-feet in 1965, and 12,000 acre-feet in 1969.

As larger amounts of sewage effluent became available, increasing volumes were percolated in the Santa Cruz River channel. An estimated 2,300 acre-feet were percolated in 1956, 5,100 acre-feet in 1960, 9,200 acre-feet in 1965, and 16,800 acre-feet in 1969. Matlock (1966) found an infiltration rate of about 2 feet per day in the river channel between the treatment plant and Cortaro Road (6.3 miles). Use of the effluent for irrigation in the Cortaro Area was halted in October, 1970. Thus, in 1971 more than 90 per cent of the sewage effluent was percolated into the Santa Cruz River channel. From 1969 to 1971, the amount of effluent disposed of in the channel almost doubled, from about 17,000 to 32,000 acre-feet.

## OTHER SOURCES OF CONTAMINATION

Because of the excellent aquifer characteristics and low natural contents of dissolved solids in the groundwater, little chemical contamination has occurred except for nitrate. The city treatment plant serves all of Tucson south of Rillito Creek and west of Pantano Wash, whereas the Pima County Sanitation District, or Ina Road, treatment plant serves the Canada del Oro, Catalina foothills, and Tanque Verde areas. The Ina Road plant consists of sewage lagoons near the Santa Cruz River north of Canada del Oro. About 500 acre-feet of effluent were produced in 1970, however, upon completion of a large interceptor in 1972, the volume will increase to about 5,600 acre-feet per year.

A sanitary landfill is in the same area as the county treatment plant and is a potential source of nitrogen compounds. A meat-packing plant and several dairies are also in the Cortaro Area. Several small housing development and the KOA Campground have septic tank disposal systems. Anhydrous ammonia is widely used from March through June on the farmland of the area. Despite these and natural sources of nitrogen compounds in the Cortaro Aria, City of Tucson sewage effluent is the primary source of nitrogen forms.

## GEOLOGY AND HYDROLOGY

The thickness of alluvium ranges from less than 200 feet near the edge of the valley to about 600 feet in the central part beneath the present-day river. Five miles north of Cortaro, a bedrock constriction acts as a barrier to groundwater movement. This barrier produces relatively shallow groundwater levels, which range from 80 to 120 feet in depth. Water levels declined from 1920 to about 1965. However, water levels have stabilized or slightly risen since 1965 due to the increasing volume of sewage effluent recharge. Water level measurements in 1972 for the CWUA wells indicated an average rise of nearly three feet per year. The direction of groundwater movement is to the northwest. Wilson and DeCook (1968) studied infiltration of runoff in the Santa Cruz channel at a site one mile south of the Tucson sewage plant. Results indicated that about one-third of the recharged water reached the water table almost instantaneously, whereas slow drainage of the remainder from the unsaturated zone took up to 6 months.

A flownet, chemical analyses, and water temperatures at the well discharge suggest two distinct sources of groundwater. Groundwater in the eastern portion is from Canada del Oro underflow, mountain-front recharge from the Tortolita and Santa Catalina Mountains, and infiltration of Canada del Oro streamflow (Davidson, 1970). Groundwater in the western part of the Cortaro Area is from Santa Cruz River underflow, underflow of percolated sewage effluent, and infiltration of Santa Cruz River streamflow and sewage effluent. Sewage effluent formerly applied for irrigation in excess of crop requirements has also been a significant source of recharge. A rough water balance calculation shows that by 1965, an approximate balance of input and output to the groundwater was in effect, with about 35,000 acre-feet of input. Transmissibilities are high in the Cortaro Area, averaging about 350,000 gallons per day per foot for short-term pump tests run on CWUA wells in 1966. These excellent aquifer conditions minimize the effects of contamination from sewage effluent and other sources.

## REGIONAL NITRATE AND CHLORIDE DISTRIBUTION

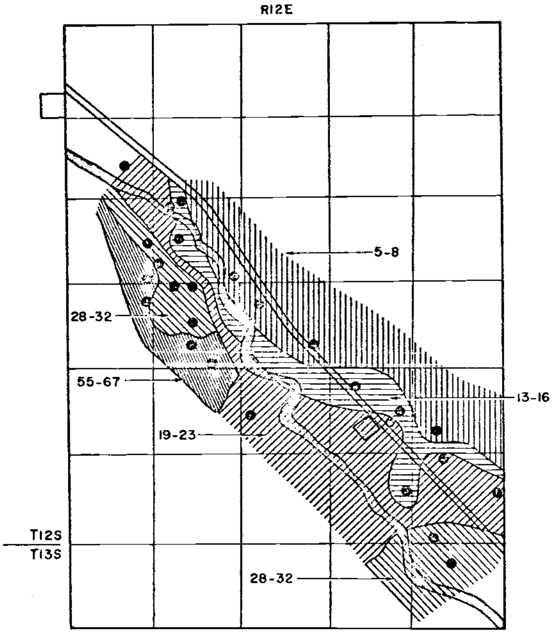
The Agricultural Engineering Department, University of Arizona, collected water samples for chemical analysis in June and October of 1970 and 1971 from many wells in the Cortaro Area. Cluff, DeCook, and Matlock (1971) discussed some of the analytical results for 1970 and suggested the complexity of assessing the nitrate situation. The areal distribution of nitrate shows several groups of wells with contents over 45 parts per million (ppm), or the U.S. Public Health Service limit recommended for potable water. One large area with nitrate exceeding 45 ppm in the groundwater was beneath farmlands surrounding the Tucson Sewage Treatment Plant, where sewage effluent has been used for irrigation for many decades. Laney (1970) had previously noted the relation of high nitrate to farmlands where sewage effluent was used for irrigation. A large holding pond (near Ruthrauff Road) which was used as part of the sewage effluent distribution system also occurred above the high nitrate zone. Two other localities with nitrate contents over 45 ppm were near Cortaro where sewage effluent was formerly used for crop irrigation. Nitrate contents greater than 25 ppm generally occur in groundwater beneath the Santa Cruz River floodplain, except near the confluences of Rillito Creek and Canada del Oro with the main channel.

Chloride is perhaps an ideal tracer in the environment due to its mobility in the groundwater, the low content in natural groundwater, and the abundance in sewage effluent. Chloride contents clearly demonstrate the influence of sewage affluent on groundwater of the Cortaro Area. Contents over 90 ppm occur for at least 3 or 4 miles downgradient from the Tucson Sewage Plant and reflect the movement of sewage effluent percolated from farmlands and the large holding pond near the plant and from the Santa

Cruz River channel. Chloride contents greater than 90 ppm also occur beneath farmlands formerly irrigated with sewage effluent near Cortaro. The average chloride content of sewage effluent from the City of Tucson treatment plant has ranged from 80 to 90 ppm in recent years. The distribution patterns for nitrate and chloride do not exactly follow the present-day channel of the Santa Cruz River, and suggest that the percolated effluent follows high transmissibility zones in the aquifer.

## SAMPLING OF LARGE-CAPACITY WELLS

Sampling of shallow wells of low yield often reflects local conditions, such as nearness to septic tank disposal systems, rather than the gross water quality of the aquifer. In order to understand the gross aquifer characteristics, one should sample large-capacity wells which have been pumping for several weeks or months. Twenty large-capacity wells operated by the CWUA were sampled monthly by the author during the 1971 and 1972 irrigation seasons. Hach field test kits were used for nitrate (cadmium reduction method) and chloride (Mohr titration) determination. Water temperature and electrical conductivity were also measured. Results for nitrate showed that these wells could be grouped according to the average contents measured (Figure 2). The lowest nitrate wells (5-8 ppm) were to the northeast and the highest nitrate wells (55-67 ppm) were to the west. This distribution is related to three primary factors: (1) the regional direction of movement of groundwater in the area; (2) the occurrence of




AVERAGE NITRATE(PPM) FOR CORTARO WELLS
1971-1972

Figure 2.

lands formerly irrigated with sewage effluent; and (3) well construction.

Wells with nitrate contents exceeding 55 ppm were in the western part of the Cortaro Area, where dilution by low nitrate water from the east does not occur. As Davidson (1970) noted, there is no mountain-front recharge from the Tucson Mountains to the west. These wells are thus primarily affected by underflow from the Santa Cruz River and infiltration of excess applied sewage effluent for irrigation. All wells with nitrate concentrations exceeding 55 ppm had the top of the perforated casing within 50 feet of the land surface. Percolating water can enter these wells above the water table, because the present-day level averages about 100 feet in depth. This cascading water acts as a short circuit and can substantially reduce the ordinary travel time of water from the land surface to the water table. The same group of wells do not penetrate more than 150 feet of the aquifer, and thus tend to sample relatively shallow water. The shallow groundwater tends to have higher nitrate contents because the primary nitrogen sources are at the land surface.

## CHEMICAL HYDROGRAPHS

Chemical hydrographs were prepared for all wells for both nitrate and chloride. These hydrographs can reflect many parameters. The total nitrogen content of sewage effluent is variable seasonally and from year to year. The total nitrogen content of sewage effluent from Plant No. 1, an activated sludge unit, averaged 20 ppm in 1971, with a maximum of about 40 ppm in the spring and a minimum of 10 ppm in the fall. The total nitrogen decreased from 30 ppm in 1969 effluent to 25 ppm in 1970 and 20 ppm in 1971, apparently due to decreases in the influent total nitrogen (E. J. Trueblood, Tucson Sewage Plant, personal communication, 1971). A plot of the average nitrate and chloride for the CWUA wells sampled shows a direct relation (Figure 3) between the two anions. This relation is highly suggestive that sewage effluent is the primary source of both anions.

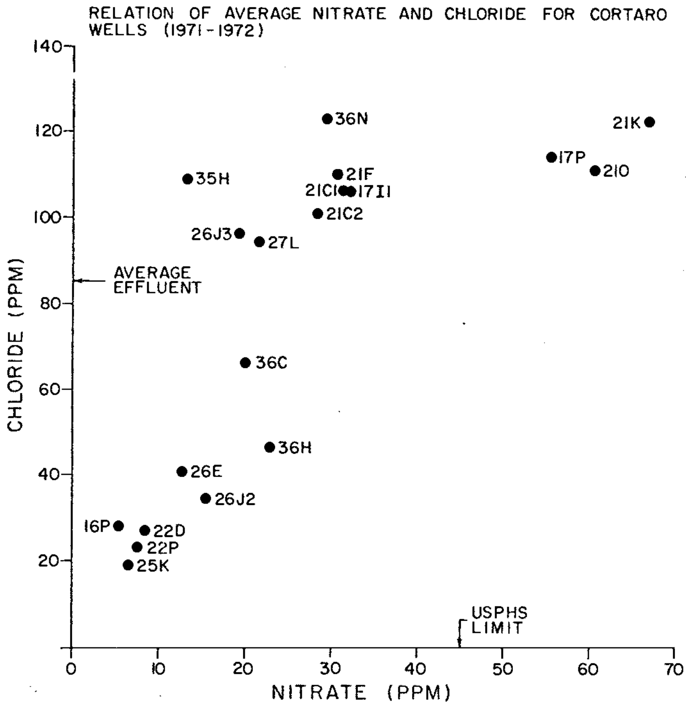
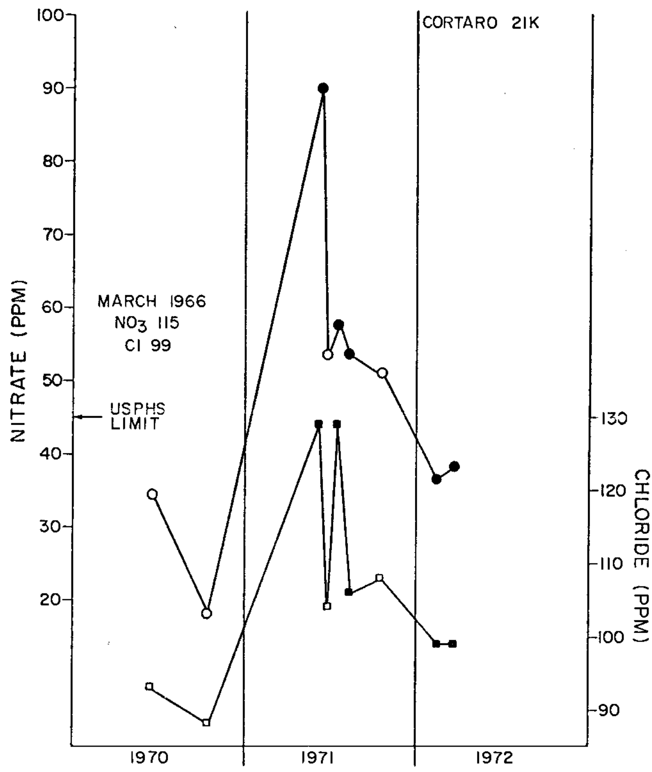



Figure 3.

Several well hydrographs demonstrated a seasonal nitrate pattern similar to that of total nitrogen in the effluent. However, several more years of record are necessary to confirm this relation. If this relation occurs, it indicates rather rapid infiltration of sewage effluent in the Santa Cruz River channel. Low nitrate and chloride contents in the fall for groundwater may be due to dilution by flood flows in the Santa Cruz River during July and August. Many wells show similar nitrate and chloride hydrographs, whereas others have opposite trends. As noted by



CORTARO 21K
NITRATE (PPM)
100
90
80
70
60
50
40
30
20
MARCH 1966
NO3 115
Cl 99
USPHS
LIMIT
CHLORIDE (PPM)
130
120
110
100
90
1970
1971
1972


Figure 4.

Schmidt (1972) in the Fresno, California area, these opposite trends may be related to denitrification.

The most dramatic changes that occurred in 1971 and 1972 are related to the recent alterations in the method of disposal of sewage effluent. Wells with the highest nitrate contents have shown a rather uniform decline from about 90 ppm in May, 1971 to 35 ppm in April 1972 (Figure 4). Sewage effluent was removed from farmlands in this area in the fall of 1970. The remaining nitrogen sources would include agricultural fertilizers and sewage effluent in the Santa Cruz River channel. CWUA wells to the east, which have low nitrate and chloride contents, have shown increasing contents. Nitrate concentration for one well increased about 8 ppm per year and chloride about 10 ppm per year (Figure 5). This situation is likely due to the increasing volumes of sewage effluent percolated from the river channel north of Cortaro since 1970. The significance of other nitrate sources besides sewage effluent seems to be confined to within several thousand feet of the source, such as the dairy east of Cortaro.

## SUMMARY

Based on available data, little contribution of nitrate from sources other than sewage effluent is apparent. Natural or background contents of nitrate are less than 5 to 8 ppm. Most of the high nitrate contents in water from large-capacity wells are related to farms where sewage effluent was formerly used for crop irrigation. After October, 1970, virtually all of the excess

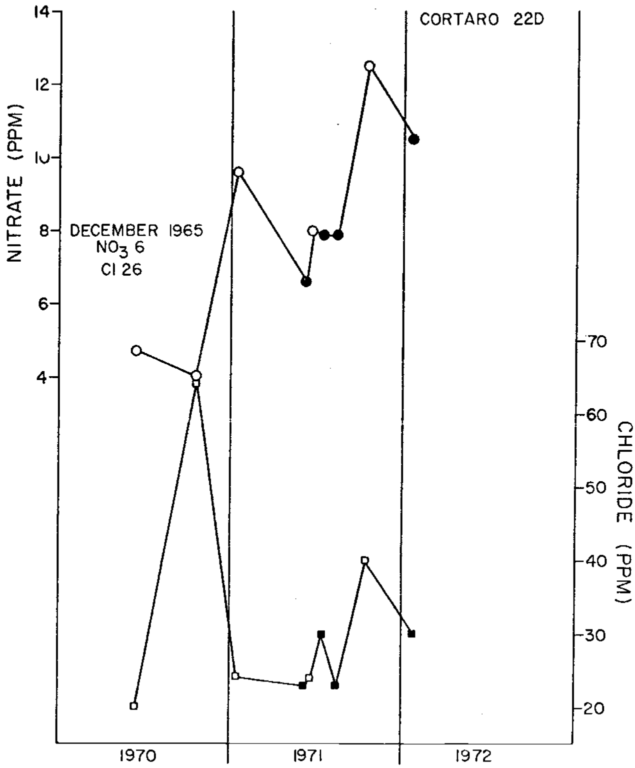

CORTARO 22D
NITRATE (PPM)
14
12
10
8
6
4
DECEMBER 1965
$NO_3$ 6
Cl 26
70
60
50
40
30
20
CHLORIDE (PPM)
1970
1971
1972

Figure 5.

sewage effluent not used at or near the plant was disposed of in the Santa Cruz River channel. Data show decreasing nitrate contents in groundwater beneath lands where effluent was formerly applied for irrigation, and increasing contents for wells near the Santa Cruz River. Evidence suggests that a large amount of denitrification occurs when the sewage effluent percolates from the Santa Cruz River channel. This occurs in the reach south of Ina Road where the channel is narrow and continuously wetted. Anaerobic conditions favor the retention of nitrogen in the soil and nitrification may not occur. In the channel north of Cortaro, a periodic wetting and drying should favor nitrification. Measurements of nitrogen species content in the surface effluent suggest that little nitrification occurs south of Cortaro, but significant nitrification occurs near Rillito Narrows.

It is not yet clear if or when an equilibrium will be established for nitrate contents in groundwater of the Cortaro Area. However, current data suggest that the disposal of effluent in the Santa Cruz River channel may be viewed as a possible form of tertiary wastewater treatment. If large amounts of phosphate and nitrate are removed during percolation, as has been suggested in the Phoenix area by Bouwer, Lance, and Rice (1971), then recharged effluent could possibly be used for municipal purposes by the City of Tucson. Cluff, DeCook, and Matlock (1971) have suggested a transfer of sewage effluent to Avra Valley for irrigation use. Another alternative is herein proposed. The City of Tucson could develop the source of recharge by means of a well field along the Santa Cruz River, assuming legal complications could be resolved. The beneficial use of this water could be for recreation and municipal purposes with careful chemical and biological monitoring.

This study of groundwater nitrate will hopefully be eventually combined with studies of the surface effluent flow in the river by Sebenik, Cluff, and DeCook (1972) and specific studies in the unsaturated zone by L. G. Wilson (personal communication, 1971). In this manner, perhaps a true understanding of the effects of sewage effluent on groundwater of the Cortaro Area can be obtained before large-scale water management schemes are enacted. Present data suggest little contamination of the gross aquifer system, a situation primarily due to the excellent aquifer conditions. As the volume of sewage effluent greatly increases in the future, this situation will likely change.

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# TRANSMISSIVITY DISTRIBUTION IN THE TUCSON BASIN AQUIFER

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## INTRODUCTION

This study is an outgrowth of an effort by the University of Arizona, Department of Hydrology and Water Resources to construct a digital model of the Tucson basin aquifer. Of particular importance in the construction of digital and electric analog models of non-homogeneous aquifers is the spacial distribution of transmissivity. A casual inspection of the areal distribution of transmissivity within the Tucson basin aquifer as determined by pumping tests (Universtiy of Arizona Agricultural Engineering files) and as reported by Anderson (1970) indicates that the values of transmissivity are not totally randomly distributed in space. Low values of transmissivity (less than 5,000 gallons per day per foot) occur around the periphery of the basin whereas larger values of transmissivity (greater than 100,000 gallons per day per foot) occur within the central and outlet parts of the basin which is manifested by a general trend of increasing transmissivity in the direction of groundwater motion.

Gates (1972) found the frequency distribution of transmissivity in the Tucson basin as determined by pumping tests to be normal or log-normal. Such a sample is biased in that people tend to drill wells in known areas of high transmissivity and avoid drilling wells in areas of known low transmissivity. The frequency distribution of transmissivity as derived from a calibrated digital model of the

Tucson basin aquifer is believed to be more nearly representative of the real world because the aquifer sample is without bias in the sense that the sample constitutes nearly the entire aquifer and not just a small part of it.

## GEOHYDROLOGIC SETTING

The Tucson groundwater basin is located in the Basin and Range physiographic province of the arid American southwest which is characterized by infrequent but sometimes violent local thunderstorms in the summer and widespread frontal storms in the winter. Recharge to the groundwater flow system is primarily by infiltration of runoff in the various stream channels throughout the area which generally flow only during periods of precipitation. Average yearly precipitation is in the range of 10 to 12 inches.

The aquifer is primarily phreatic and consists of Tertiary to Quaternary alluvium which occupies down-faulted blocks between the surrounding mountains. The direction of groundwater flow is generally from south to north. In some areas the alluvium may be as much as 7000 feet thick (Davis, 1965). The alluvial material ranges from fine-grained silts and clays through poorly or well sorted sands to poorly sorted gravels.

## ELECTRIC ANALOG MODEL

An electric analog model study of the Tucson basin aquifer was conducted by Anderson (1970). This model covers approximately 750 square miles and is constructed with a nodal spacing of one-half mile. The area covered by the analog model is shown in Figure 1.

## DIGITAL MODEL

In keeping with the present trend toward digital modeling of aquifers, a digital model of the Tucson basin was developed using the same nodal spacing as

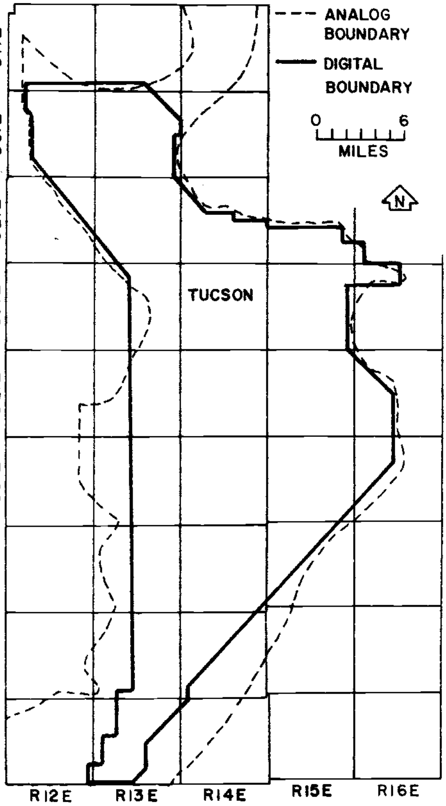


FIGURE 1. Map of Tucson basin, Arizona, showing boundaries of analog and digital computer models.

the electric analog model and covering approximately the same area. The digital model has a total of 2497 nodes each covering an area of 1/4 square mile. The area covered by the digital model is shown in Figure 1. A calibration process was used in which transmissivity values at individual nodes were adjusted by trial and error until calculated water level changes matched historical water level changes. The calibration process was halted when the average nodal error (the difference between calculated and historical water level changes over the 19 year period, 1947 to 1966) was reduced to 5.65 feet. A contour map of transmissivity within the Tucson basin as determined by the digital modeling process is shown in Figure 2. It should be mentioned that this represents the present state of the aquifer. In some parts of the aquifer the transmissivities are somewhat less than they were prior to the present period of heavy pumping which caused a decrease in saturated thickness.

## THEORY OF TRANSMISSIVITY DISTRIBUTION IN AN ALLUVIAL AQUIFER

The alluvial sediments within the Tucson aquifer were deposited upon some sort of pre-existing erosional surface. One can expect that this erosional surface, prior to being covered by alluvium, had some sort of drainage network. If one assumes that Horton's laws of stream orders were just as valid in the past as they are in the present, one can expect that they applied to the drainage network of the aforementioned erosional surface.

Horton's laws state that there is an exponential relationship between stream order and drainage network parameters such as area of drainage basin and length of stream. Stream order is a measure of the hierarchy of stream tributaries in which (for a given map scale) the smallest stream is assigned the lowest order of 1 and the larger streams are assigned succeeding higher orders. For example,

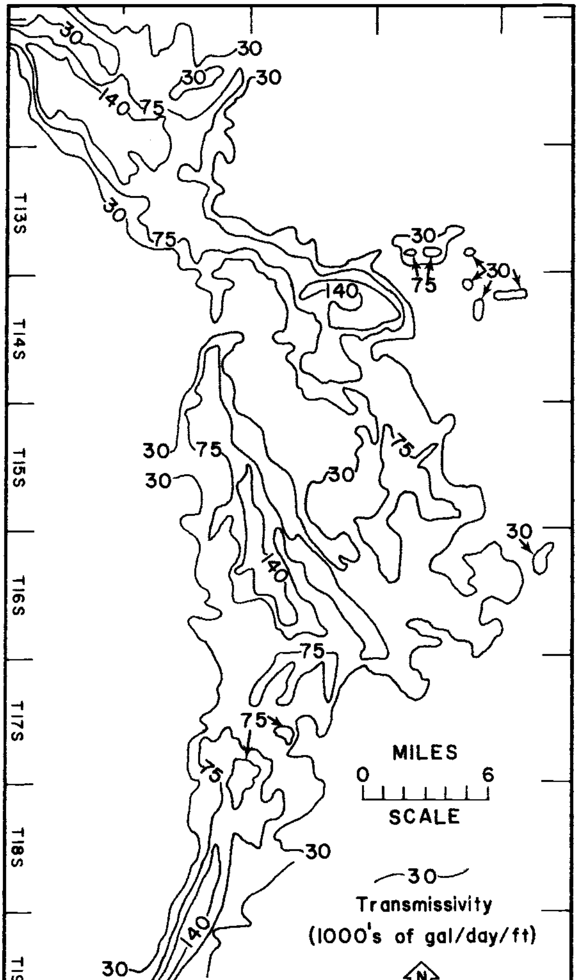


FIGURE 2. Transmissivity map of Tucson basin.

a first order stream has no tributaries whereas a second order stream has only first order streams as tributaries. A third order stream has only first and second order streams as tributaries and so on. If one plots on semi-log paper, for any surface drainage basin in the world, stream order versus length of stream or area drained, one obtains a straight line where the highest order stream has the longest length and drains the largest area whereas any lowest order stream has the shortest length and drains the smallest area. Likewise, a semi-log plot of stream order versus number of streams is also a straight line where the first order streams for any particular drainage net are most numerous.

In consideration of the above relationships, it is herein hypothesized that there should be an exponential relationship between transmissivity of an alluvial aquifer and the area of the aquifer having the specified transmissivity. One would thus expect the alluvial aquifer to have more low-valued transmissivities than high-valued transmissivities. In terms of a probability density function (PDF) this would be

(1) $$f(T) = \frac{1}{\lambda} e^{-T/\lambda} \qquad \text{for } T>0$$

$$f(T) = 0 \qquad \text{otherwise}$$

where

$T$ = transmissivity

$\lambda$ = mean value of transmissivity.

It should be noted that this PDF refers only to the frequency distribution of transmissivity and not to the locations of transmissivity in space.

## TEST OF HYPOTHESIS

In order to test the PDF hypothesis, a statistical study was made of the transmissivity values used in the digital model of the Tucson basin aquifer.

Values of transmissivity ranged from less than 1000 gallons per day per foot to as high as 450,000 gallons per day per foot. The arithmetic mean of all 2497 values was 54,848.

The probability P that T should lie between specified limits $T_1$ and $T_2$ is given by

$$(2) \qquad P[T] = \int_{T_1}^{T_2} f(T)dT$$

where f(T) is the PDF given by equation (1).

A plot of cumulative probability versus transmissivity for equation (2) with $\lambda$ = 54,848 is given by the solid curved line in Figure 3. For example, the probability of transmissivity values less than or equal to 10,000 is 0.1667, the probability of transmissivity values less than or equal to 50,000 is 0.5981, and the probability of transmissivity values less than or equal to 450,000 is 0.9997. This means that chances of finding areas with transmissivity greater than 450,000 are practically nil.

The plotted points in Figure 3 represent probabilities of transmissivity as determined by actual count of the number of nodes in the digital model having transmissivity values less than or equal to the plotted transmissivity value. It can be seen that there is a reasonably close fit between the calculated curve and the observed data points. A summary of the data points used to plot Figure 3 is given in Table 1.

## DISCUSSION OF RESULTS

It is common knowledge that alluvial basins consisting of coalesced alluvial fans have the coarsest detritus around the periphery of the basin. But because of the thinner saturated thickness around the basin periphery, the transmissivity

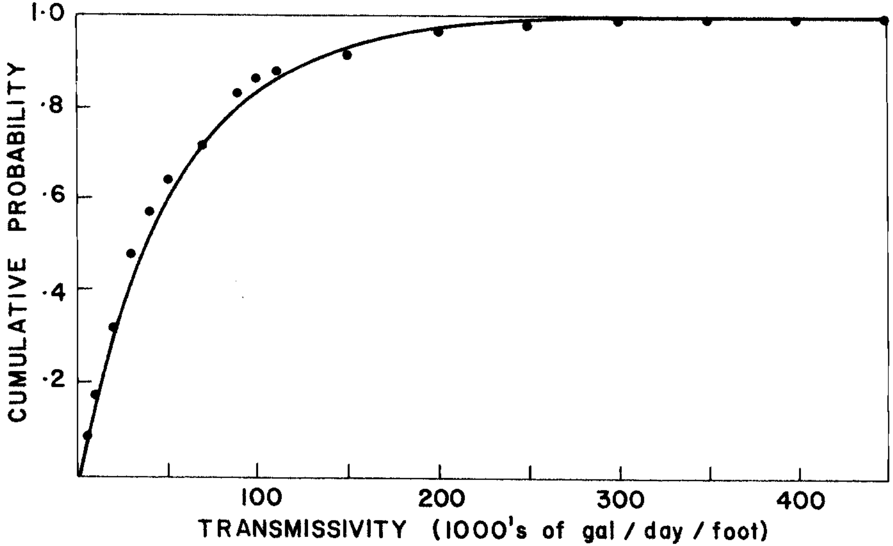


FIGURE 3. Cumulative probability of transmissivity in Tucson basin aquifer.

TABLE 1. Cumulative Probability of Transmissivity in Tucson Basin Aquifer

| TRANSMISSIVITY (1000's gpd/ft) | CUMULATIVE PROBABILITY | CUMULATIVE NO. OF NODES IN DIGITAL MODEL | |
|---|---|---|---|
| | | CALCULATED | ACTUAL |
| 5 | .0871 | 218 | 157 |
| 10 | .1667 | 416 | 431 |
| 20 | .3056 | 763 | 804 |
| 30 | .4213 | 1052 | 1222 |
| 40 | .5178 | 1293 | 1444 |
| 50 | .5981 | 1494 | 1621 |
| 70 | .7209 | 1800 | 1808 |
| 90 | .8062 | 2013 | 2088 |
| 100 | .8385 | 2094 | 2174 |
| 110 | .8654 | 2161 | 2211 |
| 150 | .9351 | 2335 | 2336 |
| 200 | .9739 | 2432 | 2379 |
| 250 | .9895 | 2471 | 2470 |
| 300 | .9958 | 2487 | 2486 |
| 350 | .9983 | 2493 | 2495 |
| 400 | .9993 | 2495 | 2496 |
| 450 | .9997 | 2496 | 2497 |

there is generally lower. Although transmissivity, being the product of permeability and saturated thickness, is a derived mathematical parameter, it appears to be a parameter which has real meaning in the real world of alluvial aquifers in terms of a quantity which is the resultant of the various inputs which combine to form a dynamic alluvial aquifer system. Given the aquifer in its present state, an increase in mean annual rainfall would cause an increase in the mean recharge rate. This in turn would result in more water flowing through the system which under similar hydraulic gradients but with greater saturated thicknesses would manifest itself by an increase in transmissivities within the aquifer. Conversely, a decrease in the quantity of water flowing through the system would result in decreased transmissivities in the aquifer.

It thus appears that the mean value of transmissivity for an alluvial basin aquifer given by $\lambda$ in equation (1) is a function of the climate and the drainage area of the basin. If this indeed is the case, it then might be possible to predict the mean value of transmissivity within an alluvial basin if one were given the proper climatic data and the area of the rainfall catchment for the basin, even though no information is available on the thickness or permeability of the sediments. In following this reasoning, if one would find another groundwater basin similar in size and climatic conditions to the Tucson basin, one would expect that basin to have a mean transmissivity similar to that of the Tucson basin.

Now that more and more aquifer systems are being modeled by means of digital computers, analyses such as the one described in this report could be carried out routinely. It would then be a simple matter to check on the validity of the relationships postulated herein.

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# AN EVALUATION OF CURRENT PRACTICES IN SEEPAGE CONTROL

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Abstract. The need for increased control of seepage from both natural and artificial small ponds and lakes has become more apparent with the increased frequency of their construction and use on the farm, ranch, and in recreational urban use. Seepage control methods are also becoming more numerous. Unfortunately, comparisons as to effectiveness, longevity and costs are not readily available. This paper investigates some control techniques being used in this region and evaluated them according to the above criteria. Emphasis was on the use of available physiochemical methods other than rubber membranes and concrete liners. Examples of the types of controls in use include plastic, soil compaction aids, hydrophobic chemicals and monovalent cation applications, such as sodium chloride. Some examples of the use of these methods in Arizona are shown and the results of some field comparison tests conducted using 8 x 8 square foot double-ringed infiltrometers presented. Recommendations are made of additional research that should be undertaken to improve the technology of the control of seepage losses.

## INTRODUCTION

In Arizona's water deficient climate, the importance of evaporation and seepage should be recognized in any water management plan that includes the surface storage of water for domestic, recreation, farm, ranch or industrial use. Evaporation loss has been studied widely and can be predicted to a greater degree of accuracy than seepage loss. Seepage loss is difficult to determine from an existing reservoir and extremely difficult to estimate in advance due to such variables as soil texture, structure, composition and the chemistry of the stored water. For lack of more convenient or accurate methods,

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measurement of seepage is often done through "back-door" techniques by measuring all gains and losses such as inflow, rainfall on the surface, and evaporation to a storage system. The difference or remainder after the calculations have been made is considered the loss due to seepage. Unlike evaporation however, methods are available for positive seepage control. In the design of a reservoir an economic study of evaluating cost versus effectiveness and longevity of various treatments should be made. Unfortunately, comparisons as to costs, effectiveness and longevity have not been readily available while options as to the type of method to be used have increased, especially in the area of chemical sealants. Because of this situation the optimum seepage control method is not always used and in some cases complete failure of the reservoir results. This paper will look at some of the control techniques available and attempt to present a comparison of the effectiveness of these treatments.

## THE SEEPAGE PROBLEM

Control of seepage problems is a factor in most areas of Arizona where surface water is stored. It is an important item in the urban recreational lake facilities now being constructed in many areas as well as in the more traditional stock tank and irrigation reservoir systems. Its control is also important with regard to disposal of liquid wastes. These seepage problems are discussed below.

The increase in Arizona's water oriented recreation is typified by the construction of artificial lakes and ponds in conjunction with home subdivisions, country clubs, mobile home parks and retirement communities. The scale of some of these developments can be seen in the Phoenix area where one organization, Diversified Properties, Inc., has constructed a 50 acre lake on 320 acres of farm land. Over 1,000 homes and 1,100 apartments are under construction at the site which is called "The Lakes." The major feature attracting the public is the lake which is not only used for boating and fishing but also greatly adds to the aesthetic quality of the development.

The control of seepage is an important part of the design of any such recreational system. As overdrafts from the groundwater supply continue and water levels in alluvial basins decline, water costs will rise. The rising water costs should provide the necessary economic incentive to keep losses due to seepage at minimum levels.

State officials and at least one State agency has stated that the allocation of water for use in artificial lakes is not wasteful and is considered beneficial in that economic and recreational benefits result. Governor Jack Williams was reported in a television interview regarding "The Lakes" on April 13, 1971 to have stated that the end use of such private-lake water produces tax income from homeowners that exceeds that from comparable acreages of farmland (Diversified Properties, 1971). In a statement to the State Land Department, The Arizona Water Commission took the position that transfer to this type of water use results in higher economic and recreational benefits and cannot be considered wasteful (Avery, 1971). At this writing, however, the legal aspects of such transfer of use are awaiting clarification from the State Attorney General (Gulatto, 1972).

In many rural Arizona areas, stock tanks and reservoirs are generally filled from surface runoff that is infrequent in nature. In arid and semiarid regions seepage control is essential to assure a stock water supply during dry months and/or years. An economical and dependable water supply is especially needed since extensive or costly treatments may not be in the economic range of some ranches. Generally these stock tanks must be periodically cleaned due to sedimentation. This fact must be taken into consideration in selecting the seepage control method.

Control of seepage from industrial or municipal wastes is important to prevent pollutants from being recharged to the ground water system. Some examples include liquid wastes present in mine tailings ponds or sewage lagoons or solid wastes that are subject to leaching. Here the effectiveness of seepage control is most important because of the rate of movement of groundwater. Once introduced into the groundwater system the waste chemicals may remain as a source of contamination for many years until dilution occurs. In this situation, more expensive membrane linings such as butyl rubber are often utilized to ensure against contamination.

## DATA COLLECTION

The Water Resources Research Center at the University of Arizona in addition to being involved in testing and experimentation in seepage control methods is also collecting data on seepage methods being used in Arizona. This information will then be available to consulting engineers, construction firms and others to reduce the number of cases where misconceptions and a lack of knowledge of the available alternatives cause reservoirs to fail or to be overpriced for the effectiveness provided.

Two examples should clarify the above statement. In a recent issue of an contracting magazine, mention was made in a feature article on a new recreational lake being constructed near Phoenix. Specifications were given for treating the sides to make them water resistant but the bottom was not treated because, according to the article, it was clay and therefore naturally impervious. Though the hydraulic conductivity of clay is much less than that of sands, movement still occurs and may be significant in some situations. Some clays have a relatively high seepage rate. Also, the effect the water quality may have on the clay minerals must be considered especially if the water is high in calcium and magnesium salts.

In another example a designer felt that seepage losses using a soil liner would be lower than with use of polyethylene plastic. Actually, positive seals using the plastic can be obtained provided that proper installation is provided and severe physical damage is avoided (Cluff, 1969). These points are discussed in more detail later.

To assist in the collection of data a questionnaire on seepage methods was prepared and sent to over 120 organizations and individuals who we felt might be involved in the design and construction of an artificial lake or pond. Also included in the mailing were land development companies that are using artificial lakes to attract potential buyers. Professional and consulting engineers were selected from descriptions of their specialties listed in the Phoenix and Tucson directories. Excavating contractors were also chosen. Additionally, known firms and dealers that are involved in seepage control products were contacted.

The questionnaire was designed to provide information in three basic areas. The first area was to determine the number and type of use of the artificial installations and to determine if any seepage control methods were used in seepage control. The third section requested evaluation of the method in use including estimates of seepage and total losses, maintenance needed and upkeep costs.

Response to the questionnaire has been far less than expected. Out of the 14 replies received, only eight indicated they had been involved with artificial lakes and the control of seepage. Specific information on methods and cost was often not included and evaluation if mentioned was limited to remarks such as "appears to be holding water" or performance was "satisfactory to excellent".

## CURRENT SEEPAGE METHODS

The remainder of the paper will be devoted to a discussion of the various methods currently used in Arizona. It is recognized that positive seals can be obtained from use of methods not included here such as sheet steel, butyl rubber, cast-in-place concrete and asphaltic pavement. The costs of these treatments are high and generally not competitive except in situations where there is a considerable economic consequence if seepage occurs such as in the storage of toxic wastes.

Generally, there are three different methods by which seepage can be controlled. First is the placement of a physical barrier such as a soil blanket with a low permeability, plastic or rubber membranes, and concrete of asphaltic sealants. A second method is the use of chemicals to create a hydrophobic condition in the soil. Treatment to improve sealing characteristics of the natural clays in the soil is the third method that may be used.

Table I provides a comparison of some of the plastic, petroleum and soil concrete treatments that are available. Material costs are included so

TABLE I

Plastic, Petroleum and Soil Cement Sealing Alternatives

| Method | Material Cost (/yd$^2$) | Estimated Labor Needed Skilled | Estimated Labor Needed Unskilled | Special Equipment | Expected Life | Expected Seepage Loss |
|---|---|---|---|---|---|---|
| Asphalt-fiberglass | \$0.70-.80 | 2 | 2-3 | Pump, sprayer | Indefinite unless dried, then seal coat, 8-10 years | Impermeable |
| Soil covered Asphalt-polypropylene | 0.75 | 1 | 3-4 | Asphalt pump, sprayer, soil ballast spreader | Indefinite | Impermeable-soil ballast used to minimize loss due to puncture |
| Soil covered polyethylene plastic [a,b] (10 mil) | 0.134 | 1 | 2-3 | Loader, dump truck w/gravel spreader (WRRC design), or self-propelled chip spreader | In excess of 20 years | Impermeable-soil ballast protects in cases of puncture |
| Soil covered PVC plastic (Unfabricated, 10 mil) | 0.357 [c] | 1 | 2-3 | Same as poly-ethylene | In excess of 25 years | Impermeable-soil ballast protects in cases of puncture |

TABLE I (cont.)

Plastic, Petroleum and Soil Cement Sealing Alternatives

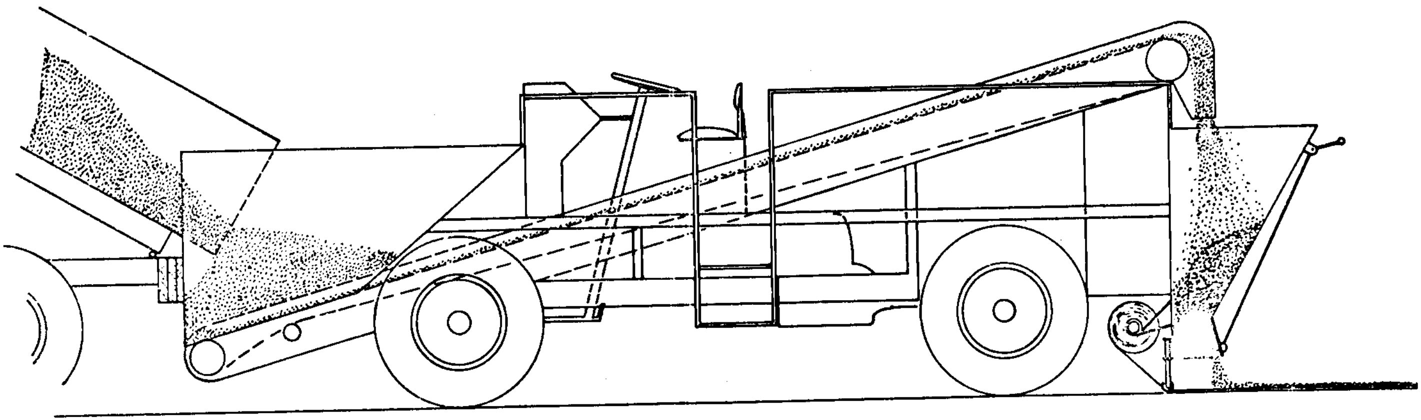
| Method | Material Cost (/yd$^2$) | Estimated Labor Needed Skilled/Unskilled | | Special Equipment | Expected Life | Expected Seepage Loss |
|---|---|---|---|---|---|---|
| Soil covered PVC plastic (10 mil fabricated) | \$0.675 [d] | 1 | 4-6 | Loader, dump truck, soil ballast spreader such as small dozer | In excess of 25 years | Impermeable-soil ballast protects in cases of puncture |
| Soil Cement [e] | 0.30-.60 | 2 | 4-6 | Harrow or spreader, rotary mixer, compaction rollers, wetting equipment | Indefinite | Impermeable-unless cracking occurs |

a - Available at this price up to 32 feet widths for minimum area of 39,000 ft$^2$.
b - 1/4" Beaded Mastic for lap sealing available at \$3.60/clf. ("Presstite" from Interchemical Corp., St. Louis, Mo.)
c - PVC cost for 54" widths. Solvent to weld laps excluded.
d - Available at price for minimum 72,000 ft$^2$ area (.90/yd$^2$ for less area). Fabricated up to 42 foot widths.
e - Material cost varies with soil type available at site, thickness desired.

that comparisons may be made. Since labor costs vary greatly, no attempt was made to quantify them except to indicate men and equipment needed. All of these treatments provide a physical barrier to water penetration. For this type of barrier polyethylene plastic is the least expensive with material costs at 0.13/$yd^2$ while the asphalt-polypropylene is the most expensive at 0.75/$yd^2$.

The use of polyvinyl chloride (PVC) and polyethylene in both seepage control and water harvesting has been investigated by the Research Center in the past few years. A pond at Tucson's Fort Lowell Park was lined and sealed using PVC. Basically the method developed for both types of membranes is to unroll the plastic over smoothed surfaces, seal the edges and backfill with minimum of two inches of silt of clay-gravel ballast to reduce the effect of unavoidable holes. With polyethylene, edges can be sealed using a beaded mastic and/or tape while with PVC a solvent can be used. On slopes a cover of up to six inches is recommended for protection against mechanical damage. A maximum slope of 1:2.5 is recommended in order to avoid slippage of the soil ballast. Figure 1 shows a method utilizing a chip spreader developed by the Center for laying the plastic and placing a layer of gravelly earth on top at the same time. The process was described in detail by Cluff (1969). Seepage was measured at less than 0.04 inches per day after installation.

The application of as little as two inches of a covering layer can prevent seepage from becoming significant if holes or tears develop in polyethylene. Results of a test conducted at the WRRC Field Laboratory in the spring of 1971 are shown in Figure 2. A one inch diameter hole was made in each of two plastic membranes that had been installed in 8 x 8 ft double-

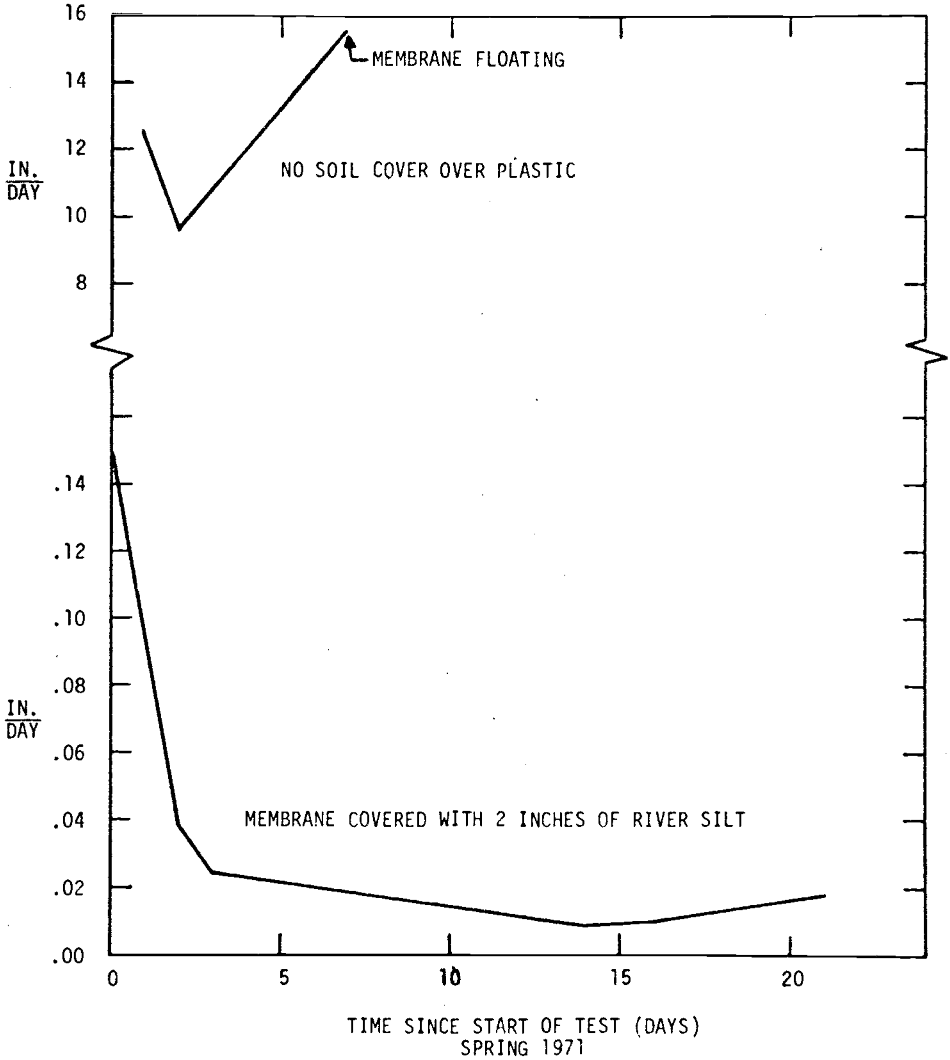


| WATER RESOURCES RESEARCH CENTER<br>UNIVERSITY OF ARIZONA | | |
|---|---|---|
| TITLE PROPOSED MODIFICATION OF CHIP SPREADER FOR INSTALLATION OF PLASTIC MEMBRANES | | |
| DESIGNED BY | DRAWN BY | DATE |
| A.R. HURLEY | [illegible] | 11/10/69 |

FIGURE 1

FIGURE 2

SEEPAGE RATE COMPARISON FOR TEN MIL POLYETHYLENE WITH ONE INCH DIAMETER HOLE IN 6 x 6 $FT^2$ DOUBLE-RING INFILTROMETER



ringed infiltrometers. One membrane was covered with two inches of a silt loam soil and experienced a loss of about 0.02 inches per day while after seven days the uncovered plastic floated to the surface. Apparently the rising of the membrane to the surface was due to air in the soil voids beneath the polytheylene being displaced by the infiltrating water. The experiment shows the necessity of placing ballast on the plastic. Additional reasons for a soil cover are for the protection from damage by animals, preventing of deterioration by ultra-violet radiation and possible wind damage if the water level is lowered or the pond drained.

Recent observation of the one-third acre Research Center pond lined with 8-mil clear polyethylene and described by Cluff (1969) showed a loss of about 11 inches over 35 days (about 0.31 inches per day). Using a conservative May-June daily pond evaporation rate of 0.25 inches, the rate of loss due to seepage is about 0.06 inches per day. Since construction in 1962, the pond banks have been damaged by rodents and removal of weed overgrowth with little apparent effect on the initial seepage rate of 0.01 inches per day.

Polyethylene plastic costs vary with the amount ordered. For 10-mil polyethylene the material is available in widths up to 35 feet. The material cost of $0.13/yd$^2$ is given for drop shipments of greater than 2000 pounds or 39,000 square feet when weight calculations are made. PVC prices differ depending on whether it is purchased in five foot widths or fabricated to wider widths. Costs do not include sealers which are purchased separately.

Soil cement has been used successfully in lining recreational and mining leach ponds in Arizona during the past ten years. Material costs of $0.30 to

$.60 per square yard are dependent on the soil type. Soils containing more clays require larger amounts of cement. Linings of 4 to 6 inches are usually desired. Slopes of banks are generally less than 1:4 to allow for the use of compactors. The area is prewetted and cement mixed at 20 to 40 pounds per square yard. The mixture is compacted when the water content is sufficient so that the mixture can be formed into a firm ball. Curing is done for a minimum of three days while keeping the cement moist for the entire time (Portland Cement Assoc., 1958).

Asphalt and fiberglass, and polypropylene and asphalt are two additional methods being used in Southwest reservoir construction. The first method was developed by the Water Conservation Laboratory of the U. S. Department of Agriculture, Agricultural Research Service, in Phoenix. This method has been used successfully in the construction of nine water catchments and reservoir linings. Briefly, installation proceeds by spraying asphalt emulsion over a relatively smoothed surface, laying unwoven fiberglass matting over the sprayed soil, and respraying again with emulsion. After a curing period, a sealcoat of asphalt-clay roofing emulsion is applied (Frasier, 1970; Myers and Reginato, 1969). Current material costs are estimated to range $0.70-0.80 per square yard (Myers, 1972).

Polypropylene plastic has the strenght characteristics of nylon-reinforced butyl but must be protected from sunlight. In order to retain flexibility the material is fabricated into a mesh. One brand is marketed under the trade name of "Petromat" by the Phillips Petroleum Company. The fabric in unrolled over a surface prepared so as to eliminate sharp projections. Overlaps are sewed or sealed with adhesive. The entire liner is then sealed using an asphalt emulsion. Positive sealing is obtained by use of 1-2 inches of ballast. On-site material costs (fabric plus sealants) are estimated to be about $0.75 per square yard.

Table II shows some natural materials and chemical substances that are currently in use in the Southwest. Soil and water analyses are recommended before the use of any of the treatments since effectiveness varies widely with different soil and water characteristics. For example, clay particles which have been dispersed by sodium in natural clay soils or in bentonite can be flocculated by water high in calcium and magnesium. Thus a seal caused by the dispersed clays would usually be only temporary under these conditions unless additional sodium was added.

With both locally available and bentonite clays the material is disked into the native soil and compacted. For bentonite the powdered material is spread over a dry surface at a minimum treatment rate of one pound per square foot for a soil containing small amounts of sand (Shen, 1959). Rollins and Dylla (1970) used as much as four pounds per square foot at an extremely sandy site in Nevada with the treatment being effective for four years of wetting and drying. Compaction may be done by drum or sheepsfoot rollers but the latter would require a higher bentonite treatment rate. For sealing silty to sandy materials, high-swell bentonite is commonly used. Low-swell bentonite mixes readily with water but treatment rates are higher and flocculation in hard waters occurs more easily. Granular material spread over the water surface of an existing pond may be used but seals are less effective than with dry spreading (Rollins and Dylla, 1970).

It has been known for many years that addition of sodium salts will

## TABLE II

## CLAY AND CHEMICAL SEALING ALTERNATIVES

| Method | Estimated Material Cost ($/yd^2$) | Estimated Labor Needed Skilled | Unskilled | Special Equipment | Estimated Expected Life [a] | Range of Expected Seepage Loss and % Reduction from Initial or control rates [a] |
|---|---|---|---|---|---|---|
| Wyoming Bentonite Clays (Powdered) | $0.304 (min treatment @ 1 lb/sq ft) | 1 | 2-3 | Disc for mixing, Drum or sheepsfoot roller to compact | Surface Blanket[b] 4-5 years<br>Buried Membrane[b] 8-12 years | Yr 1: 0.1-0.4 in/day, 94%+[e]<br>Yr 4: 5.5-9 in/day, 65%+ |
| Local Clays | Variable | 1 | 2-3 | Same as Bentonite | 2-3 years [b] | Variable depending on clay content and water quality |
| Enzymes | | | | | | |
| New Installation | 0.031 | 2 | 2-3 | Disc or harrow, compaction and spray equipment | >5 years | 0.2-0.4 in/day, 90%+ [f] |
| Existing | 0.043 | 1 | 1 | Mixing (by boat) | Unknown | Not as effective as above application |
| Hydrophobic | 0.25 | 1 | 2-3 | Spray Equipment | Unknown | Unknown |
| Resinous Polymers ("SS-13") | | | | | | |
| New | 0.142 | 2 | 2-3 | Disc or harrow, compaction equipment | >10 years [c,d] | 0.2-3. in/day, 80%+ [g] |
| Existing | 0.89 | 1 | 2 | Mixing (by boat) | >10 years [c] | 1.5-4.5 in/day, 60%+ [f,g] |

TABLE II (cont.)

CLAY AND CHEMICAL SEALING ALTERNATIVES

| Method | Estimated Material Cost (/yd$^2$) | Estimated Labor Needed Skilled | Estimated Labor Needed Unskilled | Special Equipment | Estimated Expected Life [a] | Range of Expected Seepage Loss and % Reduction from Initial or control rates [a] |
|---|---|---|---|---|---|---|
| Sodium Carbonate (soda ash) | \$0.026 (@2500 lb/acre) | 1 | 1-2 | Disc or harrow | >5 years [b,c] | 0.15-0.3 in/day, 95%+ [h] |
| Sodium Chloride (salt) | 0.049 (@10,000 lb/acre) | 1 | 2-3 | Disc or harrow, compaction equipment | >5 years [b,c] | <0.1 in/day 95%+ [f] |

a - Expected life and seepage loss estimates can vary widely depending on soil type and properties, water quality, plant life, and animal traffic. Estimates are made for comparison purposes only and do not necessarily indicate performance at a particular installation. Soil and water analyses should be made before treatments applied.
b - Efficiency dependent on water quality. High calcium-magnesium ("hard") waters will decrease efficiency with time. Expected life estimates footnoted here have been experienced using water with a high Ca-Mg content.
c - Wetting-drying cycles decrease expected life.
d - Compaction aid "Paczyme" used prior to polymer application. Price includes both materials.
e - Results reported by Rollins and Dylla (1970).
f - Results determined by WRRC measurements and/or experiments.
g - Results provided by manufacturer.
h - Results reported by Myers (1969).

reduce permeability by the mechanisms of wetting and clay dispersion. Greater swelling will occur in montmorillonite-type clays since its structure allows for expansion. However, dispersion occurs with any clay as the monovalent cation concentration and repulsion between particles increases. Dispersion blocks the water-conducting channels present in the soil and the seepage rate is therefore reduced.

Treatment with sodium has been successful on a one acre catchment at Page Experimental Ranch near Oracle, Arizona. Here runoff from a sodium-chloride treated one-acre catchment is stored in a 200,000 gallon sodium-chloride treated pond. The soil at Page is deep and contains about 29 percent clay. Recent measurements show a seepage loss of less than 0.1 inches per day. In addition to the salt treatment, it is believed that seepage was further reduced by sodium dispersed clays being transported from the salt treated runoff plot.

The method of application used is to broadcast the salt over the soil surface at a rate dependent on the amount of clay present in the soil. Decker (1965) recommends 0.20 to 0.33 pounds per square foot (8799-14500 lbs/acre). In one treatment a simple cyclone seeder (used for spreading grass seed) was utilized. The salt is disked or harrowed into the surface to the treatment depth desired. Compaction is not necessary but increases effectiveness since as the soil density is increased the available water paths are decreased.

A sample of the salt treated soil from the bottom of the reservoir at Page was removed, compacted in an infiltrometer at our Field Laboratory and subjected to four wetting and drying periods from Spring 1971 to Spring 1972.

The results, given in Figure 3, show an increase in water loss through each cycle. One explanation may be the use of water high in calcium and magnesium for the test. Total hardness of the water from our Field Laboratory well is about 340 ppm (expressed as calcium carbonate). Water of this hardness is considered "very hard" by the USGS. Also, large cracks and a massive, blocky structure were noticed after each drying period. These cracks, some extending as deep as two inches, may not completely close when rewetted and therefore act as infiltration channels to carry water into the untreated soil beneath. Longevity is decreased with each successive cycle making retreatment necessary.

Another sodium salt that can be used for seepage control is sodium carbonate. The Water Conservation Laboratory of the Department of Agriculture in Phoenix has conducted successful tests using this material in calcareous soils containing expandable clay materials. Soil compaction is not absolutely necessary to get a good seal but should increase the effectiveness. One advantage seen over NaCl is that the carbonate and the displaced calcium provide additional cementation and more resistance to flow. Cost of the soda ash at $125 per acre for a 2500 pound application is higher than for an equal application of sodium chloride. However, a higher rate of NaCl must be applied to obtian equivalent effectiveness. Retreatment may be necessary with this method also. Reginato, et al. (1968) presents a suggested guide for a combination of sodium carbonate and sodium chloride that can be applied to the existing water surface.

In the case where the seepage rate of the natural clays and bentonite treatments described above have increased with time, losses may be reduced by application of the sodium salts mentioned above.

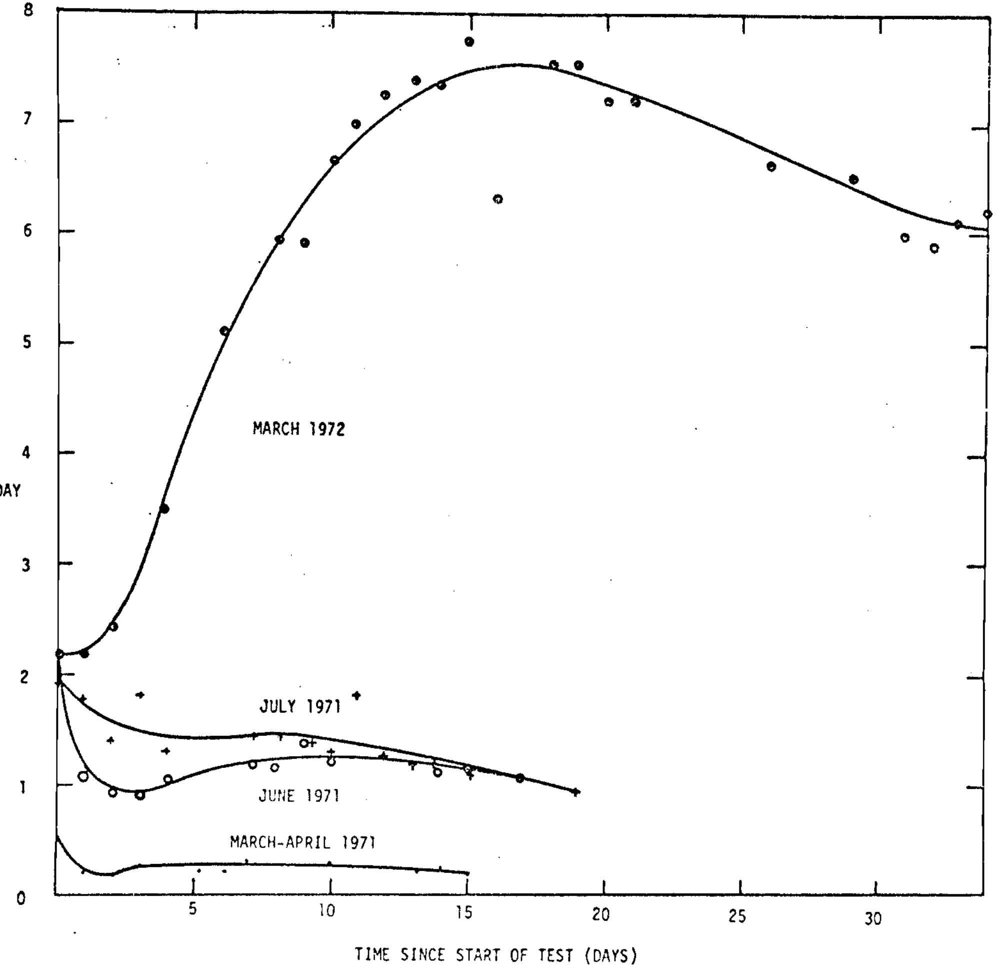



FIGURE 3 SEEPAGE RATE COMPARISON FOR COMPACTED SALT-TREATED SOIL IN 8 x 8 $FT^2$ DOUBLE-RING INFILTROMETER

Other types of chemicals are being marketed in Arizona. Most of these provide a barrier to seepage by decreasing permeability. A hydrophobic chemical has also recently been sold for seepage control. This type of chemical provides resistance to infiltration until the high liquid-solid contact angle is overcome. Chemical cost information is presented in Table II. Descriptive information about these chemicals, their use and effect, is presented in the paragraphs below.

An oil-soluble emulsion of resinous polymers called SS-13 (Larutan Corp., Anaheim, Calif.) has been used as a water inhibitor since 1957. It is designed for use in areas where high calcium and magnesium levels in water minimize the effect of clay present. The polymerized organic molecules are capable of being adsorbed on both clay and silaceous materials. The large molecules extend into the void spaces between particles decreasing the available paths for water movement.

The product works best on coarser grained soils containing silaceous particles. A series of infiltration pit tests conducted several years ago at our research facility showed its effectiveness to be less in a clay loam type soil than in sandy loams (Figure 4 and 5). Possibly the reason for this is that large size polymers may have difficulty interacting in the small clay void spaces. The seepage rate monitored for ten months after installation and use at "The Lakes" in Tempe averages slightly less than 0.3 inches per day (Blundell, 1971).

Application of the SS-13 material may be made by addition to the water of existing ponds and lakes at a rate of 1:1000 to the water. In newly constructed impoundments, the material is being marketed in conjunction with a product called Paczyme (Larutan Corp.). This product is designed to

FIGURE 4

RESULTS OF APPLICATION OF SS-13 TO PIT WITH SECOND PIT AS A CONTROL

CLAY LOAM SOIL

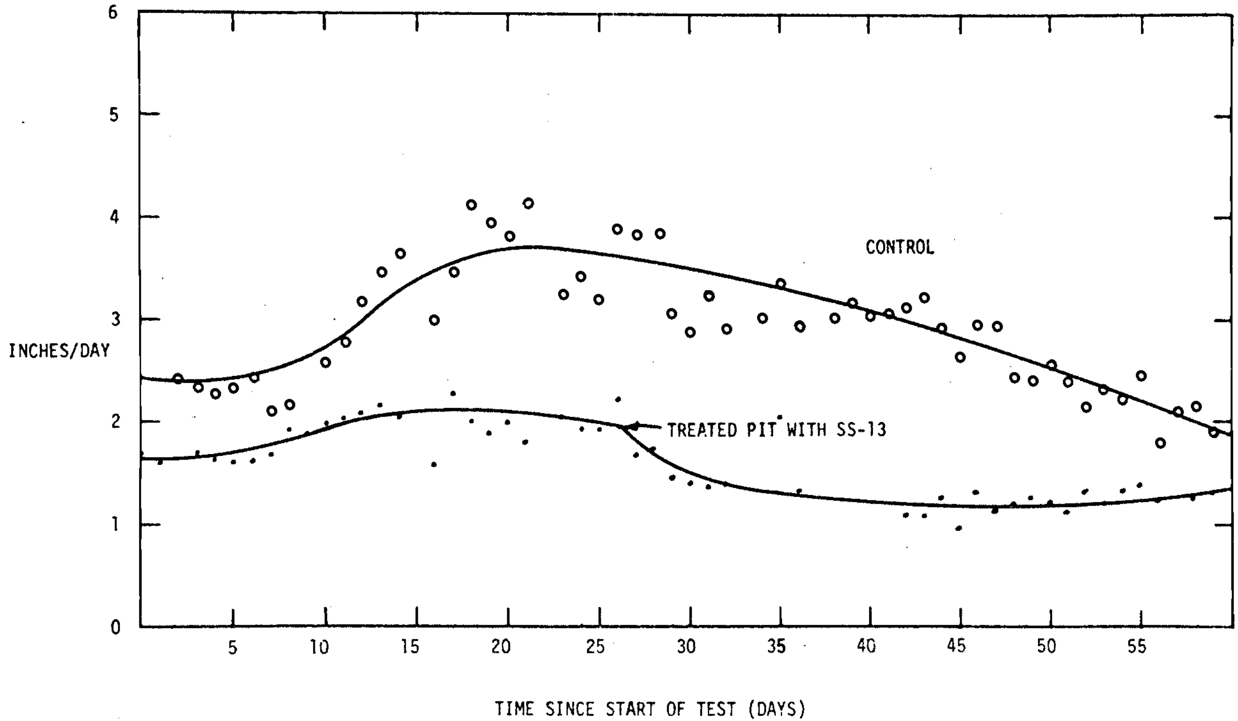
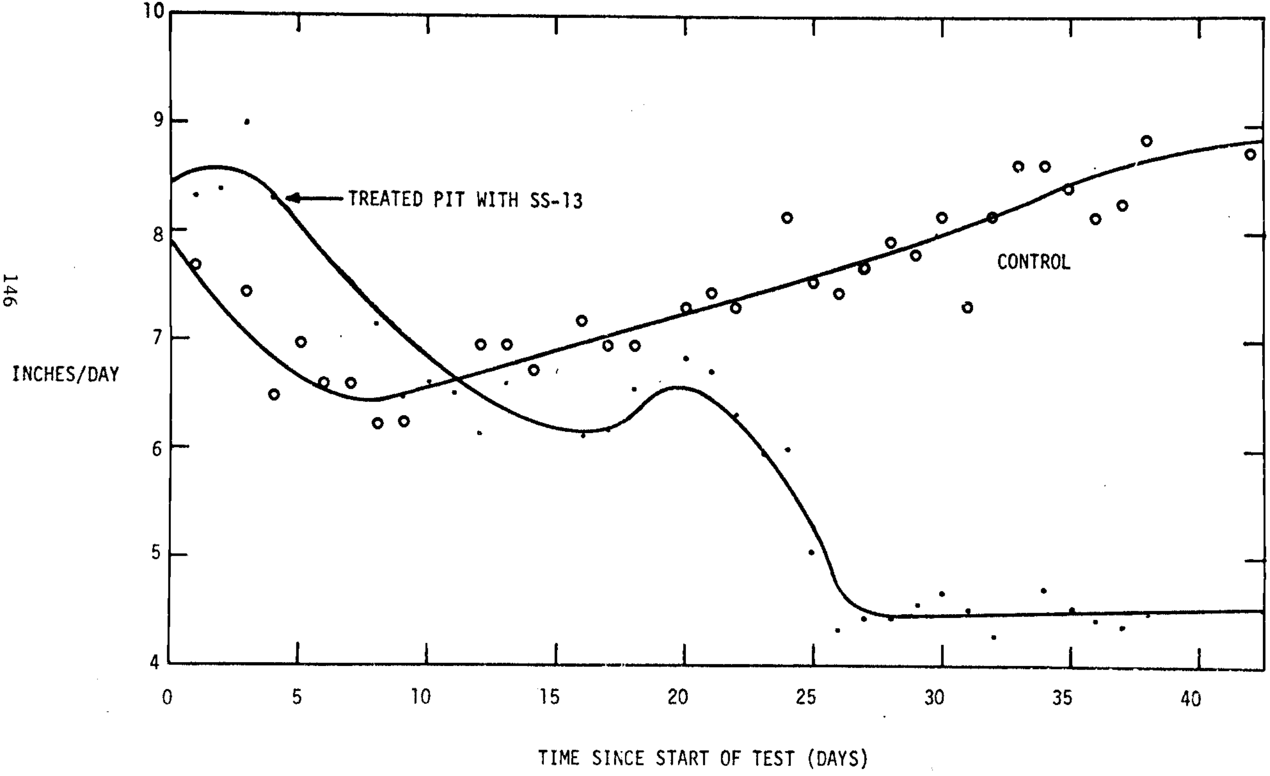


FIGURE 5

RESULTS OF APPLICATION OF SS-13 TO PIT WITH SECOND PIT AS A CONTROL

SANDY CLAY LOAM SOIL



assist in compaction by lubricating the particles to provide a better degree of packing and alignment. The compaction aid is mixed with water, disked or blended into the soil with a grader or rotatiller at a rate of one gallon per 25-35 cubic yards of soil. Compaction to a thickness of 6-9 inches at optimum density is performed. The reservoir is then partially filled and the SS-13 applied at the rate of one gallon to 100 gallons of water.

Wetting and drying cycles and resultant soil stress can cause a loss of effectiveness in the SS-13 treatment making further applications necessary. The material will kill fish when first applied due to the oil in the emulsion but is non-toxic to humans and animals.

Organic enzymes (Enzymatic Soil of Tucson) mixed with lubricants are used as aids in compaction and also provide sealing by reorientation of the flatter surfaces to one another and cementation of the fines to form larger particles (Wallace, 1972). WRRC experiments show initial results of enzyme treatments to be highly variable depending on degree of compaction, soil type and possibly temperature and/or micro-organism effect. Studies made by the Research Center at Twin Lakes north of Tucson in 1965 and at El Dorado Country Club, Tucson, in 1970 (Boyer and Cluff, 1971) show seepage rates to be about 0.10 and 0.33 inches per day respectively. Recent studies made using a 29 percent clay soil in one of the double-ringed infiltrometers and soil from Page Ranch at the WRRC Field Laboratory show a final rate of about 0.2 inches/day compared to untreated rate of 4.2 inches/day. However, large variations occurring over several days early in the test are as yet unexplained.

Application is made by adding the enzyme material to the water used for compaction at a rate of 1:1000. The soil surface is disked or harrowed then compacted at optimum moisture density to the desired thickness (usually 6-9 inches). One gallon will treat approximately 30 to 40 cubic yards of material. Compaction, density and moisture content should be monitored during application. For existing installations, one 55 gallon drum per surface acre is applied to the water surface.

Recently, hydrophobic chemicals have been introduced into the Arizona market. Testing of one (Formula 125, R & D Lab., Tucson) was done using an infiltrometer at the WRRC Field Laboratory but results were inconclusive generally because of problems in mixing the chemical and soil. Application is now being conducted by spray methods. The treatment rate varies with soil type and moisture content. One catchment and storage area has been recently constructed on the Papago reservation near Sells, Arizona.

## TREATMENT EFFECTIVENESS AND LONGEVITY

Included in the tables is an attempt to quantify the effectiveness and longevity of treatments. As mentioned earlier, wet-dry cycles can have a most important effect. This was investigated in our series of sodium chloride tests mentioned earlier and was listed as a deficiency in some chemical treatments. Another variable is the planned use of the facility. Installations that will be used for stock watering, for example, must have provisions for use without disturbing the bottom and side seals though Rollins and Dylla (1970) believe that stock traffic might be beneficial in providing additional earth compaction. In the case of the plastic the thickness of cover material must be increased for large amounts of traffic. In some areas freezing and thawing effects must also be considered. Rollins and Dylla (1970) also consider these in addition to wetting and drying cycles in their work with bentonite.

The questionnaire failed to get substantial response with regard to effectiveness and longevity. Answers such as the pond is "holding well" or "can now be filled to capacity" were received. With the exception of the project in Tempe, continuing measurements of seepage from the treated ponds investigated are not being made.

Investigations providing what information is available have been conducted by public agencies such as the University of Arizona, other state institutions, Bureau of Reclamation, the Agricultural Research Service and the U. S. Water Conservation Laboratory. Information provided by these agencies tend to be in the form of technical reports but some are available for use by the layman. Table III contains a listing of several organizations which can provide more specific information on seepage control. Commercial suppliers can furnish information on their specific treatment methods.

TABLE III

SOME PUBLIC AGENCIES SUPPLYING INFORMATION ON SEEPAGE CONTROL

Bureau of Reclamation, USDI, Denver Federal Center,
Denver, Colorado 80202

Colorado State University, Extension Service,
Fort Collins, Colorado 80521

U. S. Water Conservation Laboratory, ARS, USDA,
4331 E. Broadway
Phoenix, Arizona 85040

University of Arizona, Agriculture Experiment
Station, Tucson, Arizona 85721

University of Arizona, Water Resources Research
Center, Tucson, Arizona 85721

Seemingly, many purchasers of commercial treatments are satisfied with the initial results as provided for in the treatment contract. Warranties are provided by some installers if certain criteria such as pretreatment investigation and details of installation and operation are met. Complaints, however, are often not made unless the owner suspects an extraordinary amount of water loss. To many of those with an ample water supply who have had ponds or lakes installed, seepage losses are included under operating expenses and, unless excessive, are not considered important enough to warrant constant measurement. But to those with limited water supplies, such losses are important and information on seepage control should be available to aid in the planning of their facilities.

Measurements of seepage loss from existing installations should be continued beyond the time of initial treatments so that effectiveness and longevity of various treatments could be compared. Additionally, continuing efforts should be made to monitor research developments and effectiveness of commercial products now being marketed or that could be used for seepage control.

## CONCLUSION

The owner or designer of an artificial lake has available a wide variety of choices in seepage control with the cost of treatment versus the cost of non treatment being the major factor in his decision. The effectiveness and longevity of treatments were estimated from data presently available. Alternative choices have been presented in such a way that those involved in seepage control design should be able to make more meaningful decisions. However, further observation and research is needed to verify the continuing effectiveness of some of the procedures presented in this paper.

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The assistance of Sol Resnick, Associate Director of the Water Resources Research Center is gratefully acknowledged. It was under his direction that the seepage control program at the WRRC was initiated in 1963.

The continued assistance and support of Gordon Dutt, Professor with the Department of Soils, Water and Engineering is also appreciated.

The authors wish to thank those organizations and individuals who completed and returned the questionnaire. Also, we are thankful for the assistance provided by the commercial firms which provided us with the product information, current prices and, in some instances, material for testing. However, mention of companies or products is for the purpose of providing specific information and does not imply recommendation or endorsement by the Water Resources Research Center over those not mentioned. Additionally prices are subject to change depending on increased manufacturing costs, product availability, shipment size and distance to the reservoir site.

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COLOR IT EVAPORATION

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## INTRODUCTION

In the arid and semi-arid regions of the nation and the world, evaporation consumes vast quantities of the generally limited water supply within these areas. An estimated 17 million acre feet are lost annually within the 17 western states of the United States through this mechanism alone. Considering the demographic explosion occurring in this area, water and efficient water use is becoming more and more important. To attain an efficient utilization of current water resources, a thorough knowledge of evaporation must be known. More accurate and usable methods of determining evaporation are imperative in planning and operations of water resource systems.

Evaporation is determined by direct measurement or through the utilization of several empirical relationships which relate evaporation rate to some pertinent meteorological variables. This paper presents a rather unique parameter and its utilization in predicting evaporation or understanding evaporation phenomena from free water surface.

## BRIEF REVIEW OF LITERATURE

Interest in evaporation is not new as early writings on the subject data back to 1798 when Dalton demonstrated that the rate of evaporation was proportional to the difference in vapor pressure between the evaporating surface and the atmosphere.

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The Dalton principle that evaporation is a function of vapor pressure differences is the basis for all vapor transfer methods developed since that time.

Since Dalton's work, much effort has been concentrated on evaporation phenomena with almost all meteorological parameters considered at one time or another. Abbe in 1905 reported the effect of sunshine and heat and further indicated that these parameters were studied in England as early as 1691. Meyer in 1915 developed one of the earlier evaporation formulas using vapor pressure deficit and wind velocity.

Cummings and Richardson in 1927 considered evaporation primarily a function of solar radiation and introduced the term insolation for this parameter. Richardson in 1931 introduced the term back radiation from the water and used same in the proposed empirical relation. Rohwer in 1931 developed a formula for estimation of evaporation based on the climatic factors, barometric pressure, wind velocity and vapor pressure deficit. Blaney and Morin in 1942 published an empirical formula using the parameters of mean monthly values of temperature, relative humidity and annual daytime hours. Hargreaves in 1956 developed a similar relationship using only the temperature and humidity relations.

Since Dalton's first thesis, one can see that extensive activity in determination, either directly or indirectly of evaporation has taken place and it continues to date. Most of the efforts are devoted to modification of existing methods, combination of presently studied parameters, or continuation of studies for purpose of verification of existing models. Thus, the parameter color is being added to the long list of parameters considered.

## EXPERIMENTAL PROCEDURE

The need for evaporation data in the Lubbock, Texas region of the High Plains of Texas developed during a study relating to the use of playa lake water in artificial recharge operations. In order to determine subsidiary losses of water from the playa lake, evaporation was considered to be the primary loss. Much printed data exists for potential evaporation from free water surfaces exists but yet most do not specify the exact quality of water used in data determination. Since the waters used in the recharge study were a unique type - surface runoff with high sediment content - it was not clear that printed values for evaporation would be consistent with the actual values. The question of quality was the primary factor resulting in this study.

In order to determine evaporation rates, Class "A" U. S. Weather Bureau land pans were used. Since quality was the initial parameter of concern, three (3) waters, each with different quality characteristics were used during the initial phase. These waters were identified as clean (municipal water), sewage effluent (secondary treatment effluent), and surface runoff (collected in playa lakes). Major visible characteristic differences between the waters were in their color. The effluent possessed a dark-black-green color while the surface runoff was a reddish-brown color with evidence of suspended sediments. Two (2) additional waters were introduced after initial observations indicated their value. These waters were beef cattle feedlot runoff and artificially colored. The feedlot runoff possessed a higher density than municipal water as well as being dark-brown in color with evidence of suspended matter. The artificially colored water served primarily as a check as municipal water was used and was colored dark green using a dye. The chemical quality was not altered and the only difference was the color.

Measurements of ambient temperature, humidity, wind, barometric pressure, solar radiation, electrical conductivity and water temperature were made in addition to the direct measurement of evaporation. Evaporation data was obtained daily during select periods weather permitting and at 2-hour intervals periodically. The other measurements were made incidental to the evaporation data. Chemical analyses were performed periodically to ascertain the chemical quality of the waters.

## RESULTS AND DISCUSSION

Evaporation rate data dates back to 1966 and is available through the summer, 1970. After indications that differences in rates existed for the different colored waters, greater emphasis was placed on the color parameter. The results presented herein will be restricted to that which relates to this parameter.

Table 1 shows a typical chemical analysis of the three (3) waters - clean, effluent, and surface runoff. As can be seen from the table, chemical quality varied for the three waters but the difference was not significant to affect evaporation. The runoff water was extremely low in total dissolved solids yet had an observed evaporation rate greater than the clean and almost the same as the effluent.

TABLE 1

TYPICAL CHEMICAL ANALYSIS

| Ions | Sewage | Runoff | Clean |
|---|---|---|---|
| Calcium | 64 ppm | 20 ppm | 58 ppm |
| Magnesium | 44 | 12 | 28 |
| Sodium | 420 | 3 | 60 |
| Potassium | 20 | 9 | 6 |
| Bicarbonate | 195 | 122 | 323 |
| Chloride | 560 | 20 | 69 |
| Sulfate | 600 | 40 | 80 |
| Carbonates | 36 | 0 | 0 |
| TOTAL | 2353 | 206 | 588 |
| pH | 8.8 | 7.3 | 7.5 |

Figure 1 shows the evaporation rates for runoff, clean, and effluent waters during six months in 1967. As can be seen from the figure, evaporation rates were highest during May and June with maximum rates of approximately 0.5 inches/day from the darker colored waters - sewage effluent and surface runoff. As temperatures increase during July and August, the highest rates were observed for runoff and effluent whereas during the cooler months, the clear water had the higher rate. Salt content appeared to have no effect upon evaporation rate as was initially considered.

Similar evaporation rates were observed for four months in 1968. Figure 2 illustrates these rates and again the trend of higher rates for the darker waters and higher temperature months were noted. The maximum rate of evaporation was again approximately 0.5 inches per day.

Artificially colored green water was added to the waters observed in 1969. The purpose of its use was as a check to evaluate the effect of coloration on evaporation. The trend developed in 1967-1968 indicated that color may be a controlling parameter and the intent was to verify this trend. Figure 3 shows evaporation rates for clean, effluent, and green water. The highest rates for any of the years of record were observed during this time period. This was primarily due to the extremely hot dry weather during the study period. The figure again shows the evaporation rate increase during the hotter months for the colored waters. The artificially green water had a rate of evaporation equal or greater than the effluent with both having higher rates than the clean. Thus further substantiation of color as a parameter of influence.

A fifth category of water was used in the 1970 study. This water, beef-cattle feedlot runoff, possessed very dark color characteristics and was similar

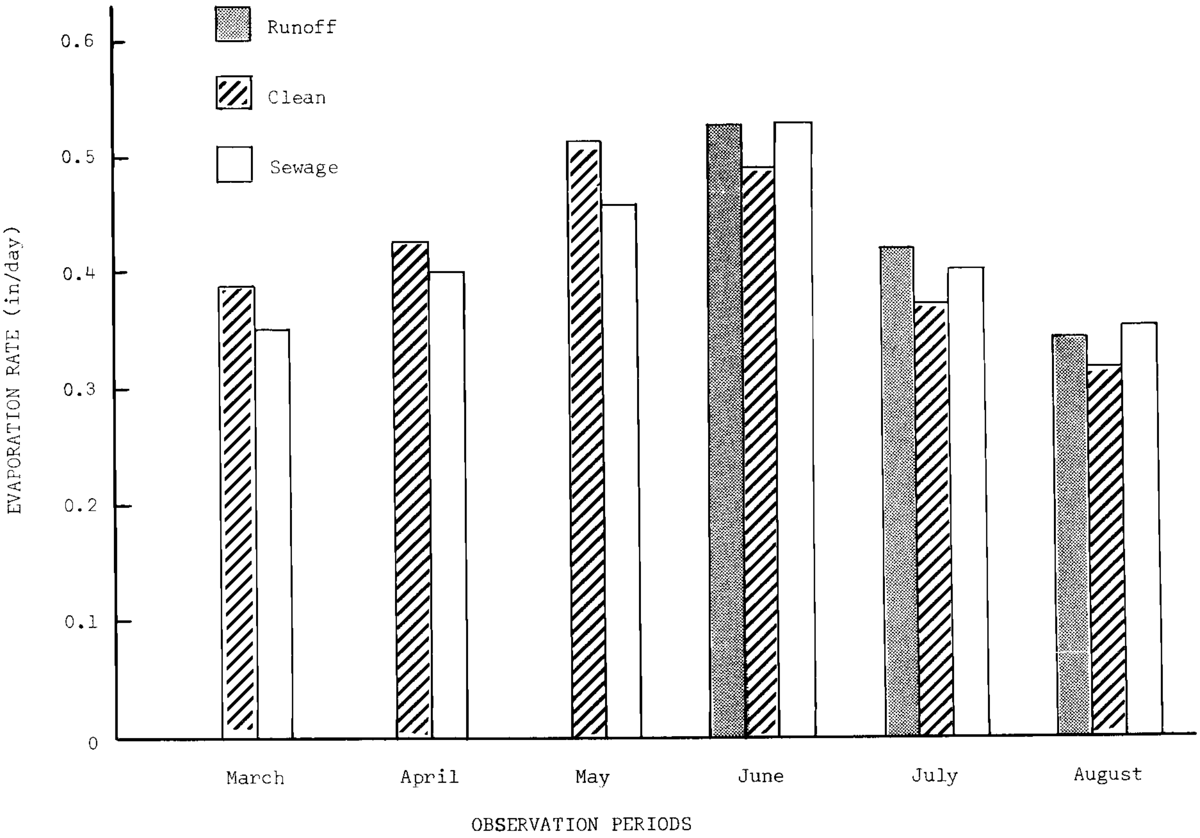


Figure 1. Average evaporation rates from free water surfaces of select waters in 1967.

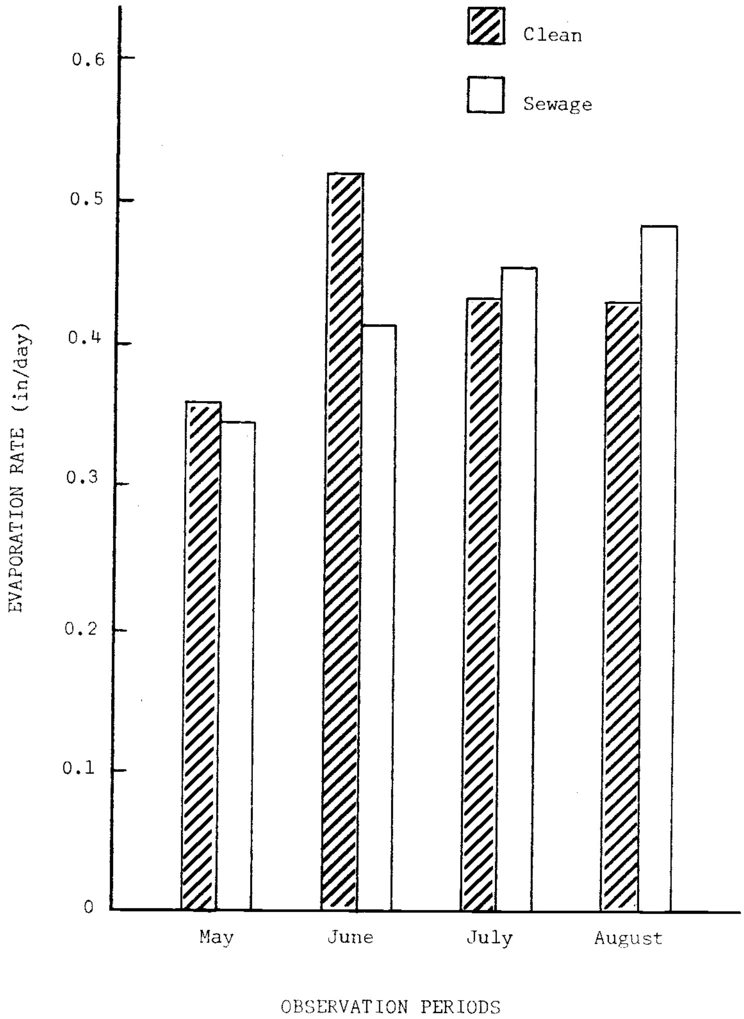


Figure 2. Average rate of evaporation from free water surfaces of sewage effluent and clean water in 1968.

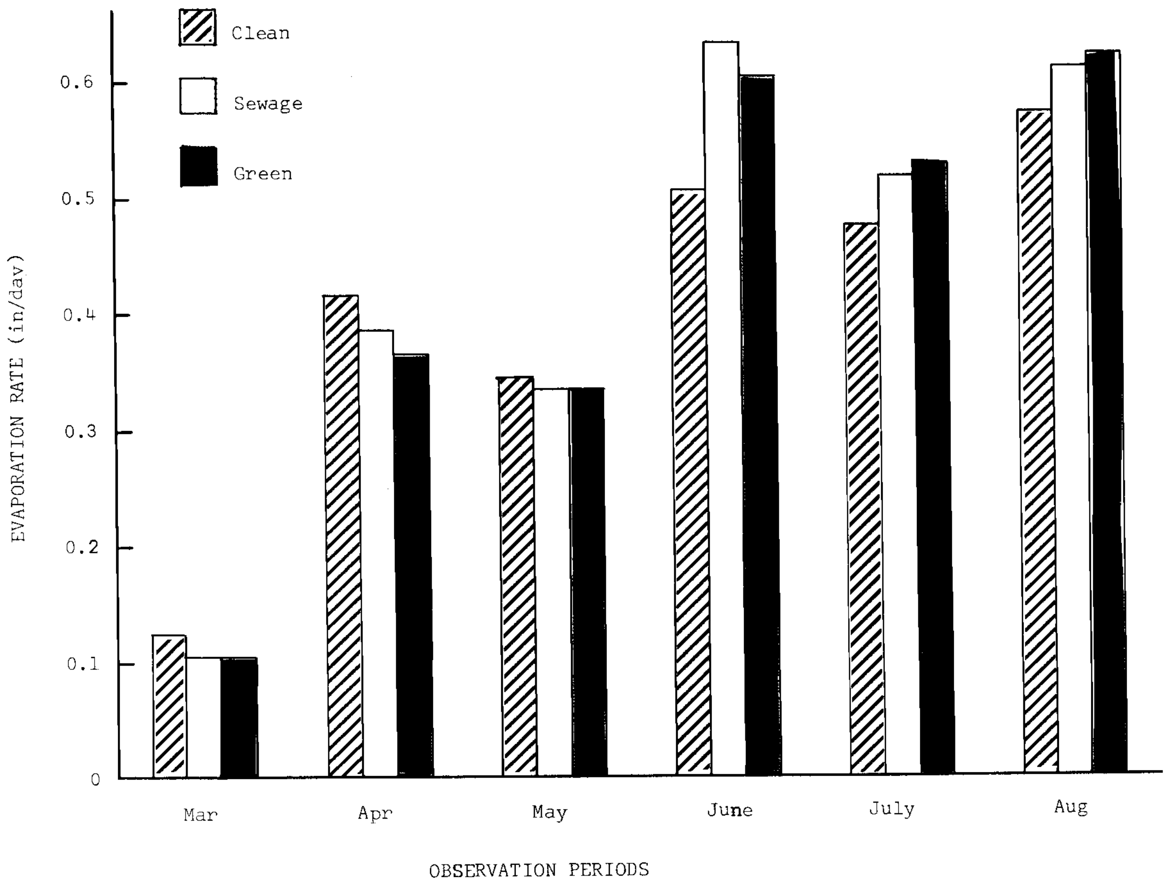


Figure 3. Evaporation rates for select waters and periods in 1969.

to the sewage effluent and green water in color. The rates observed are shown in Figure 4 and again indicate an increased rate for the darker colored waters especially during the hot summer months.

Since a trend appeared to correlate high evaporation rates with increasing ambient temperature, especially for the colored waters, rate measurements were made at 2-hour intervals. Figure 5 represents these observations during June, 1968 for clean and effluent water. Evaporation rates are higher for clean water during the evening and early morning hours but less during the daylight hours. Most significant differences occur during the mid-afternoon when air temperatures are the greatest. Water temperature measurements were made to determine if an increase in temperature could be detected. The effluent water temperature was approximately 2°F higher than for the clean. This indicates that back radiation probably was increased from this dark colored water and probably led to the increased rate of evaporation.

Figures 6 and 7 show the 2-hour relations for select waters but for different periods of the year - January, 1970 and July, 1969. The colored waters possessed higher rates during daylight hours in July, 1969 with the pattern very similar to that observed in June, 1968. However, in January, 1970, the colored waters possessed higher rates for all observations. This indicates that color is a parameter which influences both absorption of radiation and back radiation from the water and thus influences evaporation rates.

The measurements of humidity, temperature, wind, and radiation have not been analyzed to date to correlate their effect on the evaporation study as conducted. Indications are that no significant effects will be found within this analysis.

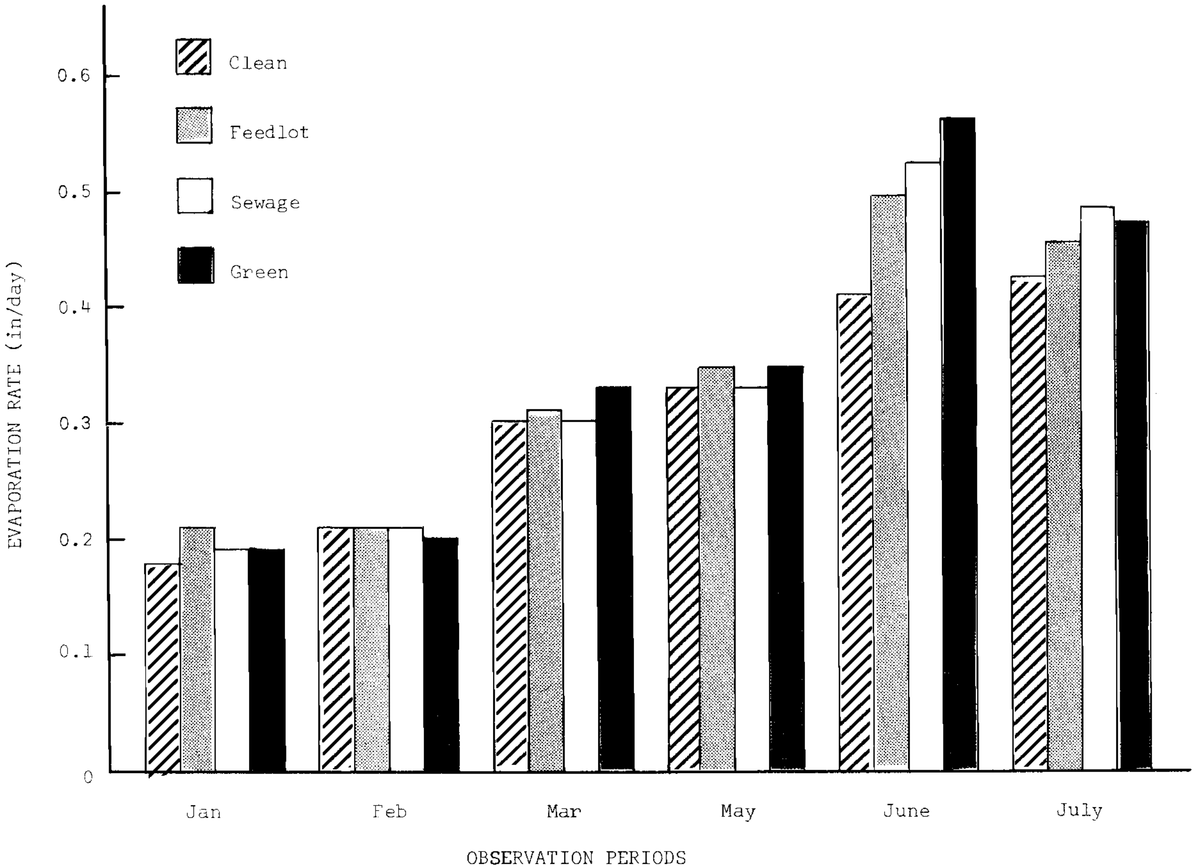


Figure 4. Evaporation rates for select waters and periods in 1970.

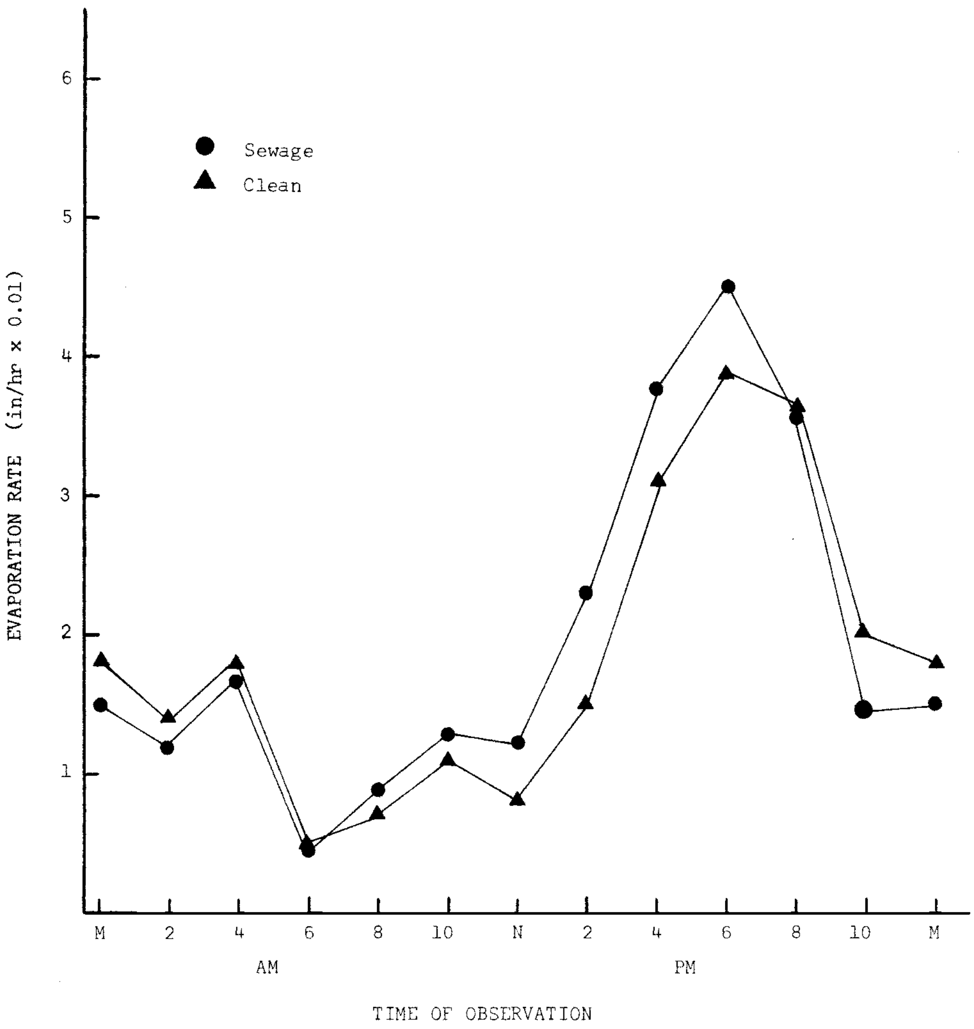


Figure 5. Evaporation rates for sewage effluent and clean water at 2-hour intervals in June, 1968.

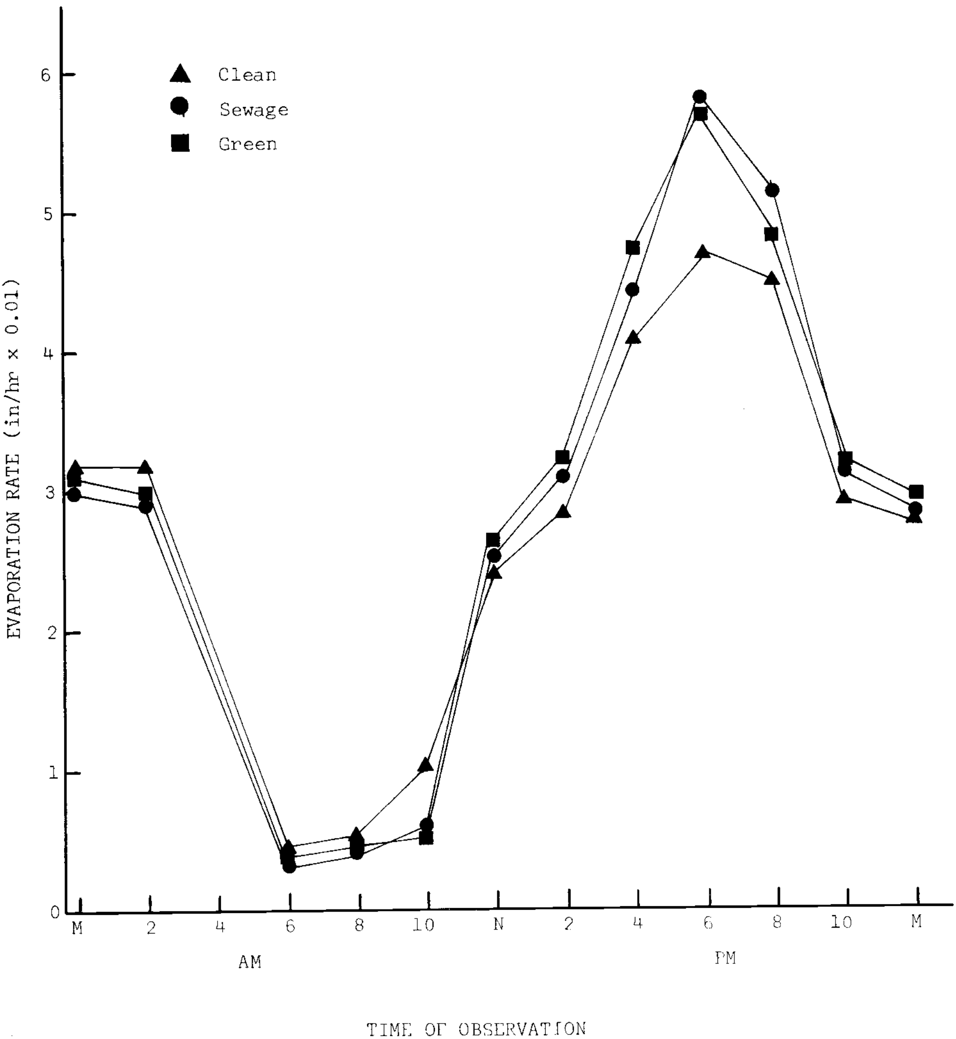


Figure 6. Evaporation rates for select waters at 2-hour intervals during July, 1969.

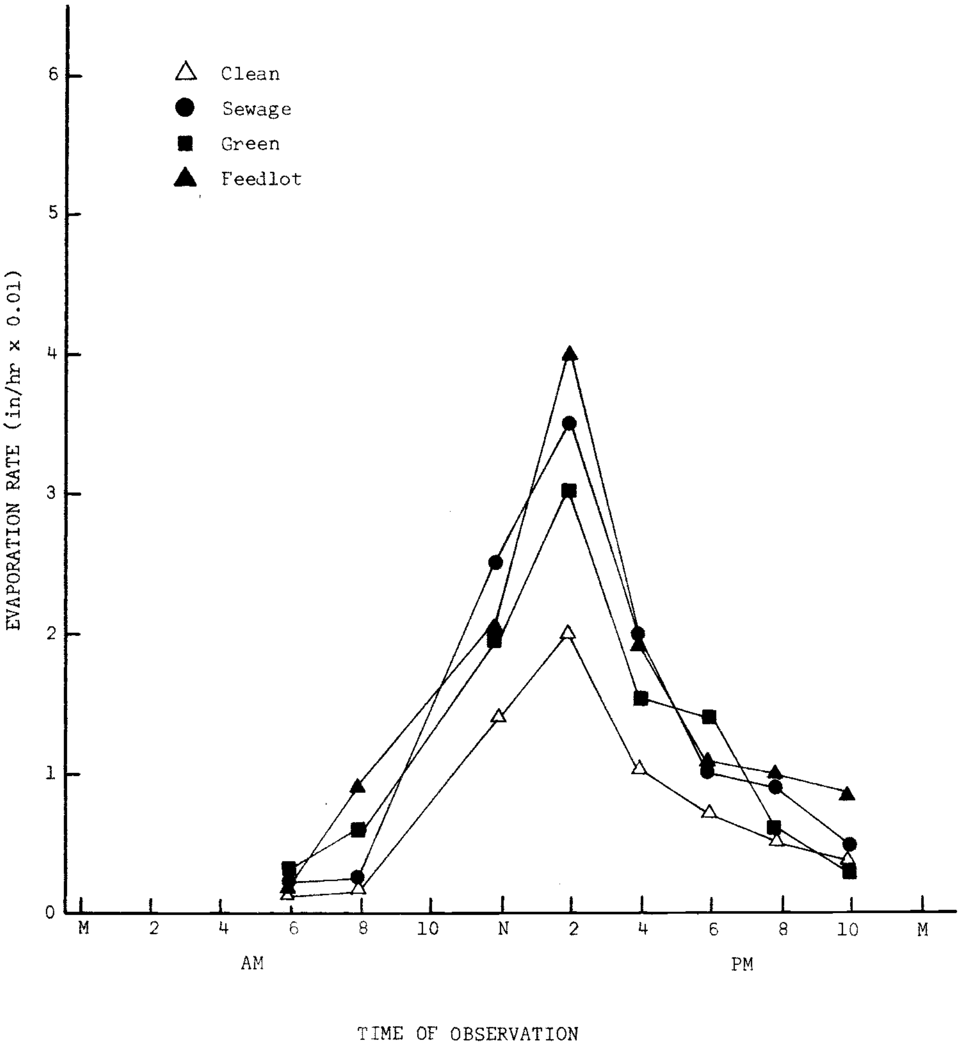


Figure 7. Evaporation rates for select waters at 2-hour intervals during January, 1970.

# SUMMARY AND CONCLUSIONS

A total of five (5) different colored waters were studied during the period 1966 thorugh 1970 to determine their evaporation rates. Color was the one parameter which was significantly different for all waters as the color ranged from none or clear for the municipal water to a very dark brown for the beef cattle feedlot runoff. Other waters were surface runoff (reddish-brown), sewage effluent (dark-black-green) and colored municipal (dark green).

Observations indicate that different rates exist for the different waters with the darker or colored waters possessing the highest rates. This was consistent for all periods of observation leading one to the conclusion that color is a parameter of influence. The color seems to affect the amount of adsorbed radiation as well as the extent of back radiation.

The latter observation is supported when 2-hour interval evaporation rates are noted. The increasing rate during daylight hours for the colored waters and the reduced or lesser rate during hours of darkness leads one to conclude that back radiation might be significantly different. In all observations the trend for a higher daily rate of evaporation existed for the colored waters except during periods when air temperatures were low.

Additional study is required to firmly establish that color can be used in the development of an empirical relationship for estimation or prediction of evaporation from free water surfaces.

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# SOME LEGAL PROBLEMS OF URBAN RUNOFF*

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We are entering an era where increasing pressure is being brought to bear on our national resources--air, earth and water. More and more mileage is being squeezed from existing resources, and heretofore "wasted" materials are being turned into valuable commodities. One of these is urban runoff.

Presently water resources for growing cities in the arid Southwest are developed at great cost involving deep wells or great aqueducts. It is not uncommon to see vast quantities of pumped water being used to irrigate and maintain park areas while storm runoff is drained past or even through the parks without being put to use. The University of Arizona has undertaken an interdisciplinary study of urban runoff to determine what type of water is where and when. My part of the study was to examine the legal questions involved in capturing urban runoff and putting it to a beneficial use--in particular for a park.

Preliminary findings show that urbanization of a watershed results in a three to five fold increase in the amount of water that will run off, especially during an intense summer storm.[1] Thus an arroyo that in a natural state would have carried ten acre-feet of water in an average year may be discharging

*The work upon which this paper is based was supported by funds provided by the United States Department of the Interior, Office of Water Resources Research, as authorized under the Water Resources Research Act of 1964.

forty or fifty acre-feet per year after the watershed has been urbanized.[2]

The legal questions which surround urban runoff and its uses are numerous. They include problems of tort liability from floods, water rights to the increased flows, land use restrictions along the banks and flood plains to prevent encroachment, possible condemnation of privately owned land for park development and flowage easements, financing problems, zoning applications to single-ownership tracts vs multi-ownership areas, and coordination of various governmental bodies responsible for management of parks, storm drainage systems and other related municipal services.

The legal questions applicable to a specific proposal are largely shaped by locations of the project, the ownership of the land in the project area, and the history of development within the project area. In the Urban Hydrology study, three basic types of watersheds are being investigated, with one type being further divided for legal purposes. These include completely urbanized watersheds such as the High School Wash and Railroad Wash areas. A partially urbanized watershed such as the Arcadia Wash area, and non-urbanized "control" watersheds such as two sub-catchments of the Atterbury Watershed. The Atterbury-type watershed can be further defined as non-urbanized wholly owned by one person or corporation, such as one finds in large ranches or in new town developments such as the proposed Empire City, or non-urbanized in multi-ownership such as is common in the eastern areas of the Tucson Basin.[3]

In the totally urbanized watersheds, the legal problems have largely been ones of tort liability for flood damage and injury. In such areas the opportunities for park development have been largely foreclosed by existing development, though possibilities appear to remain for some bicycle path construction where the washes have not been totally placed underground. The Arcadia Watershed poses the most legal problems in that tort liability questions arise from grade crossings of streets, substantial tracts of undeveloped land are still available for parks and greenbelts, water rights questions must be answered to supply the parks, and zoning and condemnation could be employed to restrict subdivision of park sites. In the non-urbanized watersheds the questions are largely zoning matters, with water rights being the important issue to resolve. The best possibilities to develop an urban runoff management system with park development with the least legal problems are in a wholly-owned watershed, such as one finds in the "new town" type of development.

The most obvious legal problem of urban runoff, is in the tort liability area. In this area urban runoff is a nuisance, to be gotten rid of as quickly as possible with the least amount of property damage and injury to life. The probelm was discussed at length in City of Tucson v Sanderson, a 1969 case decided by the Supreme Court of Arizona. In that case, involving a wrongful death action on behalf of a woman washed from her husband's car and drowned at the Arroyo Chico crossing of Tucson Boulevard, the court described the urban runoff problem:

> Nearly every summer the Tucson area is subjected to "high intensity" storms--i.e. rainstorms in which the amount of rain received in a very short time is comparatively high. In these storms, water cannot be carried away fast enough to prevent it from flooding certain areas....
>
> In any rainstorm, the amount of "runoff" is determined not only by the amount of rain that falls, but also by the nature of the ground on which it falls. Soft earth, covered with vegetation, will absorb and hold much of the rain that falls. Buildings, paved streets, parking lots, airplane runways, etc., reduce the area that would otherwise absorb water, and replace them with impervious materials. The more such improvements increase, the more runoff increases.

> In the parts of Arizona where rainfall is sparse and infrequent--including some urban areas such as Tucson--it formerly was felt that the damage and inconvenience of some flooding was more acceptable than the expense of extensive sewer and drainage systems.[4]

In this case, the court upheld the County's non-liability for the flood as found by the trial court. The City settled by paying plaintiff $60,000 for dismissal of the appeal and a covenant not to sue. The non-liability of the County was based on the County having no duty to relieve the hazard of the flooding where the City had deliberately used the street as an open storm sewer to carry off flood waters during high-intensity storms.[5]

In a case involving property damage to several store buildings adjacent to Arroyo Chico at Park Avenue, the state Supreme Court defined flood waters and municipal liability:

> Inherent in the view herein expressed are certain obvious limitations to the city's liability. We have heretofore defined flood waters as being those waters which escape from a watercourse in great volume and flow over adjoining lands in no regular channel, Southern Pacific Co. v Proebstel, 61 Ariz. 412. Since the city, by covering over the arroyo and providing culverts has only attempted to control the flow of waters within the arroyo, it has assumed no responsibility for water which naturally leaves the channel and, therefore, is not liable for damages caused by flood waters. The city's immunity from liability is absolute unless some other act or acts of the city caused or contributed to the overflow.[6] City of Tucson v Koerber.

In that case the city had built a culvert for the arroyo under Park Avenue, with a capacity for 1,700 cubic feet per second (cfs) when the arroyo itself had a capacity of 2,500 cfs. The city's liability in that case was somewhat limited because it had originally sold one of the lots subject to a flood easement. Thus it was not liable for damages to any structure placed in the easement.[7]

In the Koerber case, the court made a statement basic to a city's liability for damages from urban runoff:

> ...even though a municipal corporation has no duty to keep a stream flowing in a safe condition and protect private property from overflow, if it assumes to act it must act without negligence....[8]

This language came to haunt the city in City of Tucson v Wondergem.[9] In that case an elderly tourist was caught in rising water at Cherry Avenue and Arroyo Chico in his auto. He climbed to the top of the car and waited for rescue. While he was waiting, the water flipped his car and he was drowned. His widow sued for wrongful death and was awarded $60,000. One of the findings was that the culvert was underdesigned and the city had reason to expect larger flows in the arroyo due to urbanization upstream. The court noted the city could have escaped liability had it placed warning barricades and lights at the crossing prior to Mr. Wondergem's happening on the scene. The city claimed Wondergem had assumed the risk by driving into the flooded crossing, but the court rejected that argument because Wondergem was from Wisconsin and had never seen a flooded arroyo and could not be expected to know of the danger.[10]

While city officials deny the Wondergem decision had any effect on their flood warning program, a vigorous effort to mark all grade crossings with warning signs and to place barricades at dangerous crossings where a high traffic volume and high water volume can be expected to coincide has been undertaken.[11] Construction of extremely large culverts at major street crossings of city arroyos has also been pursued virorously.[12]

The Urban Hydrology study involved diverting some of this storm runoff into park areas with some of it being retained for recreational lakes and the remainder released slowly into the natural drainage system. The effect of this would be to flatten the peak of the flood and spread the flow out over several days. Present circumstances are that most of the water which falls on a watershed is quickly funneled into the washes by the street drainage system, and down the wash to the rivers. It is not uncommon for walls of water to be seen in these washes immediately after or during an intense summer storm. It is these walls of water which catch motorists unaware when they cross a wash during a storm. The net result of the study is that extremely large culverts to pass the flood would not be needed, and further that some runoff would be captured and used beneficially.

The major beneficial use of urban runoff proposed is the diversion of runoff into park areas and lakes. The runoff would be used to irrigate parts of the park and, following removal of sediment, would be stored in lakes such as now exist in El Encanto Park in Phoenix and Randolph Park in Tucson. The proposed size of the lakes for this plan does not exceed five surface acres. Runoff use has been proposed for parks near the intersection of Broadway and Swan in Tucson (on land owned until recently by Howard Hughes) and on vacant land north of the Tucson Medical Center near Glenn and Craycrost.[13]

Common to all watersheds where some park development is possible is the question of water rights. As the City of Tucson has previously learned, one cannot spend vast sums of money developing a new water resource without having a good grasp on the water rights issues involved. Under the Arizona law of appropriation for surface water it is not clear whether urban runoff is "salvaged" water subject to appropriation in a wash, or "diffused surface water" which can be captured and used at the discretion of the landowner. The statutes are silent on the matter of water generated by paving over a watershed with homes, shopping centers, streets and parking lots. Thus the question must be approached by analogies.

A valid appropriation can be made from arroyos similar to those found in urban areas. In England v Ally Ong Hing, 105 Ariz. 65, a 1969 case, the state Supreme Court said seasonal waters of a creek would be subject to appropriation even though the flow was intermittent.[14] For a stream or wash to be subject to appropriation it generally must have a well defined channel with a bed and banks, and the water must be put to some beneficial use.[15]

There has been some dispute in Arizona over the size of a wash and the relation to appropriability, but for the purposes of the urban arroyos, it can be assumed they would require the City or County to follow ARS 45-141 et seq., the statutory formula for obtaining a valid appropriation of the water.[16]

The real question is the relation of the "new" water to prior downstream uses. It is assumed that flood rights have been obtained along the Rillito, Santa Cruz and Gila River systems. All of the urban washes in the Urban Hydrology project feed these systems ultimately, and any diversion of runoff would no doubt raise objections from downstream users with historic flood rights such as the Buckeye Irrigation District. In a case dealing with appropriation of flood waters from 1914, the state court said:

> In determining the amount of water appropriated for irrigation, the court may determine the area of lands to which the water has been beneficially applied, the frequency with which floods occur, the waters from which reached the lands of the appropriator, and other facts bearing on the question of the quantity of water actually appropriated to a beneficial use. Santa Cruz Reservoir Co. v Ramirez.[17]

Applying that formula, one could assume that the increased runoff in areas long urbanized may serve as the source for downstream flood rights which could be proved. Many factual questions would have to be resolved, such as whether the urban runoff has actually increased flood flows fifty or a hundred miles downstream from the city and whether such increased flows are regular enough to support use of the water beneficially for a lengthy period of time.

In an area such as the Arcadia Watershed, where some parts have been urbanized for many years, the problem of conflicting with downstream users could be avoided by relying only on newly created runoff for diversion into parks and lakes, allowing existing flows to continue albeit over longer periods of time. Thus an anticipated amount of increased runoff from the construction of a new subdivision on the upper end of Arcadia could be appropriated for use in one park, and the anticipated amount from parking lot and other consturction between Broadway and Grant Road filed on for the park at the Tucson Medical Center.

The question of water rights is simpler in the non-urbanized areas, especially those in single ownership. Water that falls on vacant land, which is captured and put to use by the owner before reaching an appropriable water course, is generally conceded to belong to the person on whose land it falls and with whose efforts it was put to use.[18] In Colorado, for example, all water in a watershed is considered part of the stream for the purpose of appropriation.[19] There is no specific statute dealing with this "diffused surface water," but Professor Clark of the University of Arizona College of Law contends that the common law doctrine of the water rights belonging to the landowner who develops the water is in effect.[20] A Stanford law student writing in the Stanford Law Review contends that in the absence of statute or case authority, the issue is unsettled.[21] Certainly if every drop of water, that would normally flow into a stream with a large number of appropriators on it, was diverted into stock tanks or urban runoff parks and lakes under the diffused surface water doctrine, the matter would be resolved by either statute or court action.

In the case of a single-ownership watershed, such as is possible in a new town like Empire City, all of the increased runoff generated by urbanization could be captured and diverted before entering the washes and arroyos and thus put to use under the diffused surface water doctrine. Or a projection could be made of the amount of increase expected from the urbanization and the landowner could file an appropriation on the difference between the pre-urbanization historic flow and the post-urbanization average.

An interesting case which has some application here is Vantex Land & Development Co. v Schnepf, 82 Ariz. 308, a 1957 case involving respective water rights to waste water from irrigation.[22] In that case tail water from A's land flowed across B's land into an old wash or arroyo where it was appropriated by C at the point where it left B's land. B later intercepted the water where it entered his land and used it for irrigation without applying for a permit to appropriate it. The court held that C could not enjoin B, who was privileged to capture and use the water since the upper owner had no right to drain his artificially introduced water onto his land, nor did the lower owner have any easement to conduct his appropriated water across B's land.[23] The court did not pass on the question of whether C had a right to appropriate the waste water. The dissent stated that "...all water found flowing in natural channels belongs to the public and may be appropriated. No distinction is made (in the statute) between natural and artificially produced water; the only condition is that it be found in a natural channel."[24]

It would be possible for a landowner just downstream from a newly urbanizing area to appropriate the increased flow for his beneficial use if the urban development followed the present pattern of using streets to collect the runoff and directing it into the first natural watercourse that is handy.

Another case which may pose some legal problems to the use of urban runoff is Salt River Valley Water Users v Kavacovich, 3 Ariz App 28 (1966). In that case Kavacovich employed water saving techniques and used the water thus salvaged on additional acreage not included in his original appropriation. The court required him to cease using the waters on the newly opened land and to allow it to become return flow.[25] It has not been interpreted yet as to "new" water not previously in the appropriable supply, though it has been cited as a discouragement to water salvage practices among irrigators.[26]

There are several ways to unravel the legal aspects of water rights to urban runoff. The first is to create by statute a new class of water for urban runoff and spell out priorities and uses for appropriation. Another would be for a city to develop a park and commence diverting runoff and file for an appropriative right. If, as would be expected, a downstream holder of flood rights challenged the city's use, the matter would be adjudicated and resolved. Another possibility, though remote, is for Arizona to adopt the Colorado view that all waters, regardless of their location, are subject to appropriation (and thus regulation and control by the state) eliminating the possibility of a new town capturing all runoff under the diffused surface water doctrine.

In conclusion it appears feasible to divert what amount to small quantities of water from urban area washes and arroyos for park and recreational uses. Such diversions would provide similar flood control benefits and mitigation of tort liability as are now gained by culvert construction while providing beneficial uses of the runoff. The tort liability probelms of street runoff would be shifted to problems of ownership water rights.

In the water rights area, under the existing laws of appropriation it appears that municipalities could appropriate the increased flows in arroyos caused by urbanization. Large-scale land developments could similarly divert increased flows caused by urbanization under either the appropriation doctrine, or that of use of diffused surface waters. The ultimate legal questions must be resolved either by legislative designation of urban runoff for appropriation by cities, by litigation between claimants of urban runoff water rights, or by the appropriatve system being extended to include diffused surface waters.

NOTES

[1]K. J. DeCook, WRRC Project Data

[2]Ibid

[3]The eastern areas are owned in part by private persons, the State of Arizona, and the federal government.

[4]City of Tucson v Sanderson, 104 Ariz. 151 (1969)., at 152.

[5]104 Ariz 151, 156

[6]City of Tucson v Koerber, 82 Ariz. 347 (1957)

[7]82 Ariz 347

[8]82 Ariz 347

[9]City of Tucson v Wondergem, 105 Ariz 429, 6 Ariz App 570, 10 Ariz. App 267. (1957-69)

[10]105 Ariz 429, 6 Ariz App 570.

[11]Interview with City of Tucson Public Works Director Verne Tregonis, 1972.

[12]Ibid.

[13]Extensive plans were drawn up by two landscape architecture students Wade and Sonberg for the Hughes' site.

[14]England v Ally Ong Hing, 105 Ariz 65 (1969)

[15]Fourzan v Curtis, 43 Arz 140 (1934); City of Globe v Shute, 22 Ariz 280 (1921)

[16]Vantex Land & Development Co. v Schnepf, 82 Ariz 54 (1957) and Southern Pac Co v Proebstel, 61 Ariz 412 on the differences between Sonoqui Wash and Coyote Wash.

[17]Santa Cruz Reservoir Co v Ramirez, 16 Ariz 64 (1914).

[18]I Waters and Water Rights 52.1 (A), at 302 (R.E. Clark ed 1967) and 93CJS. Waters S 113 (1956).

[19]Dehaas v Benesch 116 Colo 344 (1946)

[20]Clark, R. E. "Background and Trends in Water Salvage Law" 15 Rocky Mountain Mineral Law Institute 421. Note 64 at page 445.

[21]"Ownership of Diffused Surface Waters in the West." 20 Stanford Law Review 1205,1212.

[22]Vantex Land & Development Co v Schnepf, 82 Ariz 54 (1957).

[23]Summary taken from Trelease, Water Law, Cases and Materials 95 (1967), also cited in Clark, "Trends in Water Salvage Law" 15 RMLI 421

[24]82 Ariz 54, 58.

[25]Salt River Valley Water Users v Kavacovich, 3 Ariz App 18 (1966).

[26]Clark, Ibid. 456.

# CONVERTING CHAPARRAL TO GRASS TO INCREASE STREAMFLOW

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## INTRODUCTION

Chaparral covers about 4 million acres in Arizona, with about half on National Forest land. Approximately 400,000 acres are on the Prescott National Forest (about 29% of the Forest area). The Rocky Mountain Forest and Range Experiment Station is interested in determining how much the chaparral-covered lands contribute to surface water supply, and how much this contribution could be changed by conversion of the chaparral cover to grass or a grass-forb type.

In the 1950's the Station established several research watersheds on National Forest lands to augment chaparral studies at the Sierra Ancha Experimental Forest north of Globe, Arizona. Two clusters of these new watersheds are located on the Prescott National Forest. The Mingus cluster of 3 watersheds is about 20 miles northeast of Prescott and 2 miles south of the highway to Jerome and Clarkdale. The two Whitespar watersheds are approximately 6 miles southwest of Prescott on drainages crossed by the highway to Wickenburg.

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[1]Central headquarters maintained at Fort Collins, Colorado, in cooperation with Colorado State University; author stationed at Tempe, Arizona, in cooperation with Arizona State University.

Both clusters were instrumented in 1958 to measure streamflow from the watersheds, precipitation within them, and continuous temperature and relative humidity at a point on or near each. The Mingus watersheds, which are in lower density chaparral, have just been calibrated. Calibration of the Whitespar watersheds, with their medium density cover, was completed in 1967. Drainage B was treated in March of that year. Results from the first treatment in watershed B are the subject of this paper.

## EXPERIMENTAL AREA

The Whitespar watersheds are located in the headwaters of the Hassayampa River at 6,000 to 7,000 feet elevation (Figure 1). Watershed B is 246 acres in size and watershed A 303 acres. Soils in the watersheds are nearly all derived from Bradshaw granite which intruded into Yavapai schist. Soil depths range from 19 to more than 42 inches. Surface horizons are open and porous fine gravelly loams or gravelly sandy loam, and there is little evidence of overland flow.

Depth to firm bedrock is unknown. Roadside cuts outside the watersheds indicate that bedrock weathering may penetrate several feet (precipitated clay films show to depths of 12 to 15 feet) and that deep fractures into the granite are possible.

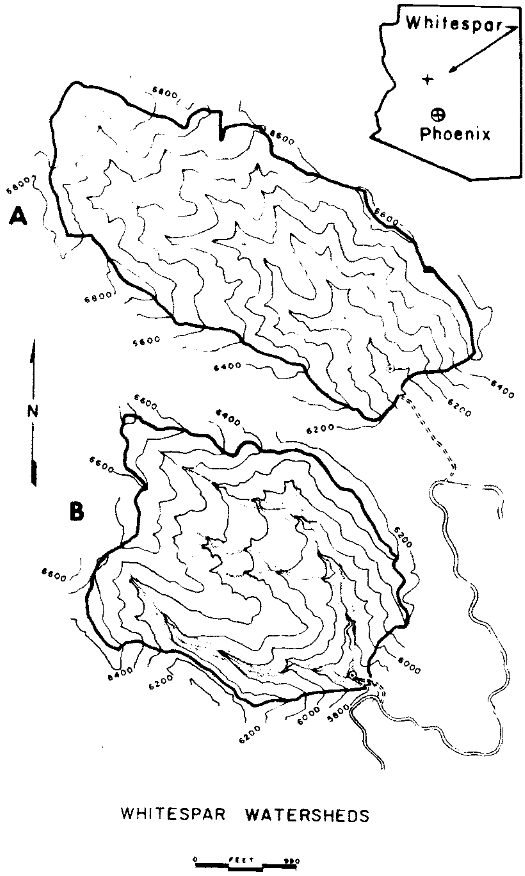


Fig. 1. Whitespar experimental watersheds near Prescott, Arizona. March 1967 treatment of 38 acres (15% of the 246-acre catchment) is shown along channels of watershed B.

Dominant cover in the watersheds is oak-mountainmahogany chaparral. The two most abundant shrubs are shrub live oak (*Quercus turbinella*) and true mountainmahogany (*Cercocarpus montanus*), but other shrub species are common. Scattered alligator junipers (*Juniperus deppeana*) ranging up to 4 feet in stem diameter are found mostly in swales and drainageways. Crown cover (vertical crown projection) in the chaparral type averages 51% on watershed B. Islands of Gambel oak (*Quercus gambelii*) are common on cool northerly slopes. Crown cover is 75% in the Gambel oak type. Herbaceous plants are sparse. Cover in the channel bottoms is largely an extension of shrub types found on adjacent slopes.

Annual precipitation during the calibration period (July 1, 1958 - June 30, 1967) averaged 22.7 inches on watershed B, with the low 14.9 and the high 31.4 inches. Forty-two percent fell in the winter period (December 1 - April 30) (Figure 2). During the 4 years since treatment, precipitation has averaged 21.2 inches with about 47% during the winter.

During the 9-year calibration period, timing and seasonal fluctuations in streamflow in both watersheds were similar (Figure 2). Streamflow averaged 1.62 inches annually in A and 1.20 inches in B. The range in annual flow in watershed B was 0.01 to 4.95 inches. Nearly 90% of the average yearly total came during the winter period, December 1 - April 30.

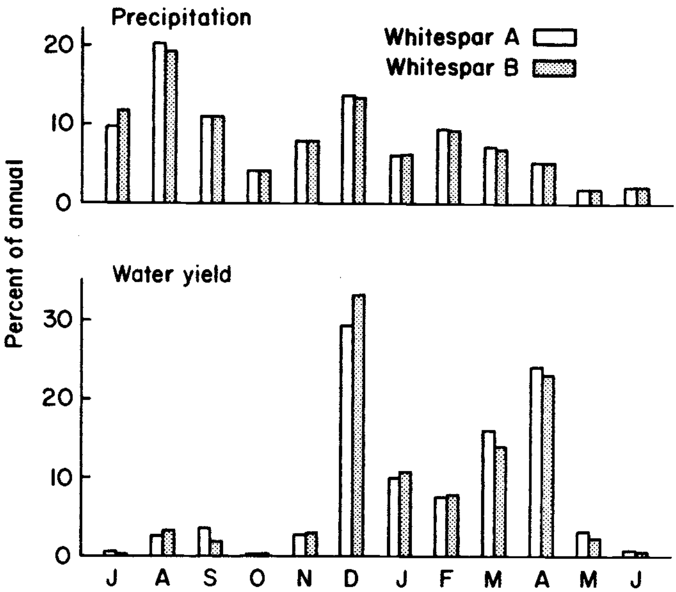


Fig. 2. Mean monthly precipitation near the streamgage and water yield as percent of annual, for Whitespar experimental watersheds A and B.

## TREATMENT

The first treatment to convert chaparral cover to grass was made on Whitespar B during March 1967. Treatment was confined to areas along well-defined channels (Figure 1). The intent was to treat all shrubs and trees on both sides of the channel that use water from the moist channel environment. By eliminating the deep-rooted plants within this zone, water concentrating downslope along the stream channel would be subject to lower transpiration loss before becoming streamflow. Bounds of the zone were set at 30 feet vertical but no more than 75 feet horizontal distance on each side of the channel. These limits were exceeded in a few places to eliminate small islands and narrow stringers of brush where channels were close together. Cover in the zone was not a "riparian" type. Sycamore, alder, cottonwood, and other species often found along streams were not present.

Pelleted fenuron[2] (25% active ingredient) was hand-placed underneath the shrubs and around the smaller junipers in March 1967. No chemical was placed in the openings between shrubs and trees. The application on the 38-acre zone averaged 23.2 pounds active ingredient per acre.

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[2]Trade names and company names are used for the benefit of the reader, and do not imply endorsement or preferential treatment by the U. S. Department of Agriculture.

Larger junipers were either cut or girdled with a power saw in June of the same year. Felled junipers were cleared from the stream channels. Litter was not deliberately disturbed.

## RESPONSE TO TREATMENT

By June it was obvious that the brush was reacting to the chemical treatment. Leaves were losing their vigor and appeared to be drying out. Infrared pictures taken a year later clearly defined the treated areas.

In June another effect of treatment became obvious. Streams were flowing in both watersheds at time of treatment as a result of heavy rainfall in December 1966. Streamflow in watershed A (the control watershed) became intermittent in late June 1967. In contrast, flow on treated watershed B did not stop (Figure 3) and its rate of recession was less than in previous years.

Prior to treatment, streamflow in the two watersheds was quite well synchronized. The difference in number of days with streamflow was greatest in the year ending in June 1967 where there were 62 more days with flow from watershed B than from watershed A. A minimum difference of only 3 more from watershed A occurred in the year ending June 1965, while the pretreatment annual average was 8 more

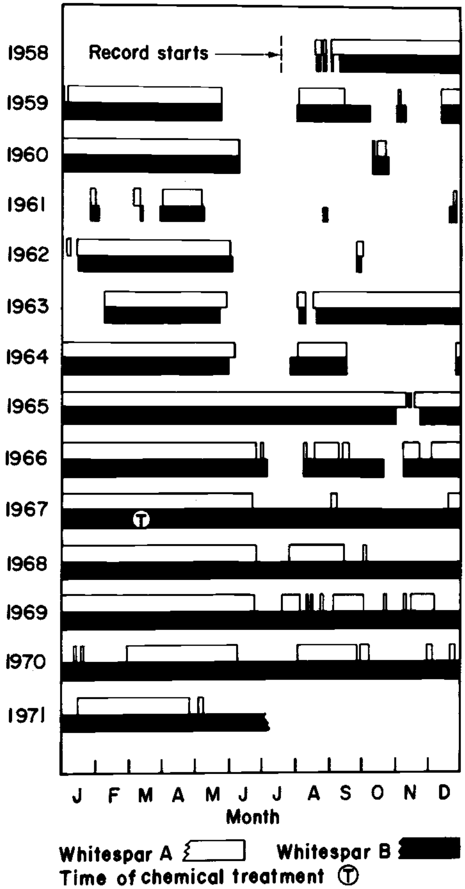


Fig. 3. Flow duration on the Whitespar watersheds. Flow, normally intermittent on both watersheds, became continuous on watershed B following treatment in March 1967.

days with flow in watershed B than in watershed A. In 4 of the 9 pretreatment years watershed B had the greater number of days with flow; in 5 it had less than watershed A.

Since treatment, streamflow in the two watersheds is no longer synchronized. Watershed B has had a continual flow since chemicals were first applied, while A's runoff remains intermittent. The average annual difference between watersheds in days with streamflow increased from 8 in the pretreatment period to 167 for the four posttreatment years.

The volume of streamflow from watershed B has also increased due to treatment. During calibration, the relationship (estimating regression) between annual streamflows on the watersheds was determined so that annual runoff from watershed B could be estimated from watershed A runoff (Figure 4). The consistent "plus" differences for the four posttreatment years represent the increased streamflow resulting from treatment. These increases average 0.61 inch, and because they result from treatment of only 15% of the watershed area, indicate an average annual increase of 3.9 inches from the treated area. About 77% of the increase came during the winter period.

During the posttreatment years, the increase in yield from watershed B has ranged between 0.27 and 0.97 inch. Generally the size of increase varied with the amount of precipitation, which implies a dependency of

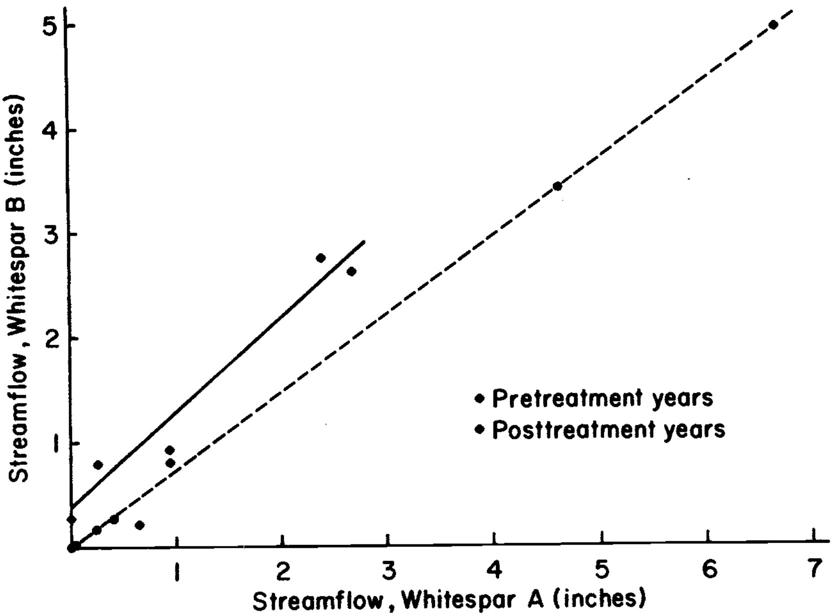


Fig. 4. Annual July through June streamflow from Whitespar watersheds A and B, July 1958 through June 1971. Dashed line based on regression for calibration period 1958-1967. Solid regression line is for posttreatment period 1967-1971.

increased yields on precipitation. To verify, annual streamflow was regressed on annual precipitation. Two regression lines were plotted (Figure 5); one for the control or pretreatment period, the other for the posttreatment years. As the amount of precipitation increases, the space between the two regressions widens. This indicates that, in the wetter years and within the limits experienced to date, the increase in yield due to treatment becomes greater, hence the dependence of yield increase on amount of precipitation.

Similar trends have been noted in the effect of brush cover treatment on water yields at the Three Bar and Natural Drainage watersheds. The degree of effect has varied, however, and further study is needed to determine the cause of the varied response to treatment.

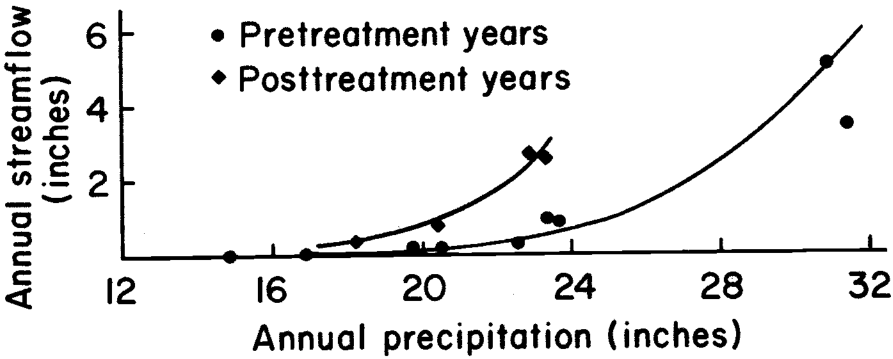


Fig. 5. Pretreatment and posttreatment regressions show effect of precipitation on runoff from Whitespar B. Difference between the regression lines reflects influence of treatment on runoff. Lines were fitted by linear regression on logarithmic transformation of the data.

WEATHER MODIFICATION IN

ARIZONA, 1971

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## INTRODUCTION

In recent years there have been many efforts to modify thunderstorms through cloud seeding. Most of these programs and experiments were designed to increase rainfall, although a significant number were efforts to suppress hail or reduce lightning. Thunderstorm (convective cloud) seeding programs and experiments in Arizona all have been efforts to increase rainfall, since much of Arizona, and particularly southern Arizona, is dependent directly or indirectly on thunderstorm rainfall for water supply and forage production. For example, in southeastern Arizona increasing thunderstorm rainfall by 25% would almost double annual runoff, while decreasing it by 25% would about halve annual runoff (Osborn, 1972). This paper includes a review and discussion of convective cloud seeding efforts in Arizona before 1971, and a hydrologic analysis of the operational convective cloud seeding program carried out in Arizona in the summer of 1971. The results of the study further emphasize need for more controlled experiments on a large areal scale.

## SANTA CATALINA EXPERIMENT

One of the most comprehensive randomized cloud seeding experiments in this country was conducted by the Institute of Atmospheric Physics, University of Arizona over the Santa Catalina Mountains from 1957 through 1964 (Battan and

Kassander, 1960). Since there were significant changes in operational procedures between 1960 and 1961, the experiment was officially described by Battan (1966) as two experiments, from 1957 through 1960, and in 1961, 1962, and 1964 (there was no seeding in 1963). Silver iodide was seeded upwind from the Santa Catalinas at about 20,000 feet above MSL during the first experiment, and at about 12,000 feet above MSL (the cloud base) in the second. Also, different silver iodide generators were used and experimental days were determined somewhat differently after 1960. The experiments were randomized with seeding days determined as those with more than a predetermined minimum level of moisture aloft. Seeding began at 12:30 pm and continued for over 2 hours and up to 4 hours. In the first experiment, rainfall was estimated from a network of 26 recording rain gages unevenly scattered over and near the Santa Catalinas. In the second experimen several gages were added in the Santa Catalinas, but gages near the mountains were removed. Therefore, estimates of rainfall were more accurate for the second experiment because of the greater gage density, but applied to a considerably smaller area.

Battan (1966) analyzed the experiments in considerable detail. His analyses of seeding effects on rainfall, which is most pertinent to hydrologists and others interested in water use and supply in Arizona, was restricted to a 5-hour period, 1:00 p.m. to 6:00 p.m., which seemed reasonable at the time. Battan reported that there was no evidence from these experiments to support the hypothesis that seeding with silver iodide increased rainfall amounts or areal extent. He added that there was 30% less rainfall on seeded days, but that this decrease was not significant. Finally, he concluded that additional research was needed on convective cloud seeding "to resolve the conflicting views on the physics of convective precipitation and on the efficacy of various cloud seeding techniques

The possibilities of more widespread effects of seeding were not analyzed at that time, because there were few recording rain gages within a 20-to 30-mile radius of the Santa Catalinas, and because more widespread effects were not anticipated. However, subsequent indication of possible widespread effects of cloud seeding over longer periods in the Whitetop Experiment (1961-1965) in Missouri, reported by Lovasich and others (1969), led to analyses of recording and standard rain gage records throughout south-central and southeastern Arizona for 1957-1964 in an effort to identify any possible changes in rainfall amounts or patterns from seeding in the Santa Catalinas.

Included in the analyses were recording rain gage records from a dense network of gages located on the USDA, ARS Walnut Gulch Experimental Watershed 65 miles southeast of the Santa Catalinas. Neyman and Osborn (1971) reported apparent effects of seeding on Walnut Gulch rainfall based on records from 26 recording gages that were in continuous operation from 1957 through 1964. They found that when Walnut Gulch was roughly "downwind" from the Santa Catalinas, there was 70% less rainfall for the 24-hour period after seeding as opposed to not-seeded days, which was statistically highly significant ($P = .008$). When Walnut Gulch was roughly "upwind" there also was less rainfall on seeded as opposed to not seeded experimental days, but the differences were not significant. For all experimental days, there was 40% less rainfall on seeded days, which was statistically significant ($P = .03$). The categories of "upwind" and "downwind" are based on dividing the data into 2 sets with wind directions across 180° in each set. Although admittedly crude, the subdivisions seemed reasonable, particularly since any seeding effects at 65 miles probably would be postulated to be indirect rather than from direct contamination of silver iodide nuclei.

Average hourly rainfall amounts for 14 hours from noon to 2:00 a.m. are plotted for two situations--downwind and all experimental days (Figure 1). On seeded days when Walnut Gulch was downwind from the Santa Catalinas, the characteristic double peak of rainfall was missing and rainfall was less than on nonseeded days until about midnight. On upwind days (not plotted) both peaks were evident for both seeded and nonseeded days, although both peaks were lower on seeded days. On all experimental days, seeded rainfall was less than nonseeded from about 3:00 p.m. to about 2:00 a.m. If the seeding did indeed affect rainfall on Walnut Gulch, the timing indicated in the two plots in Figure 1 seems reasonable.

Further analyses of the Santa Catalina data for 24 hours following seeding, indicated a greater difference in rainfall amounts on seeded as opposed to nonseeded days for the second experiment (Figure 2). This suggests that changes in experimental procedures between 1960 and 1961 did make a difference and that, as Battan (1966) suggested, analyses of the 1957-1964 data should be on two sets of data. However, the differences between the two experiments were not statistically significant.

Comparison of Santa Catalina and Walnut Gulch data suggest that seeding may have affected rainfall less on the target than 65 miles from the target. There are two obvious explanations of the contradictions in Walnut Gulch and Santa Catalina analyses. First, the differences may have been entirely by chance and have had nothing to do with seeding; second, the overall effects of convective cloud seeding may have been indirectly greater in some areas surrounding the target than on the target. Also, the Walnut Gulch rain gages represented a denser network and therefore provided a more accurate estimate of rainfall over a limited area than the Santa Catalina gages, particularly in the first experiment; so direct comparison of differences may be incorrect. However, enough questions

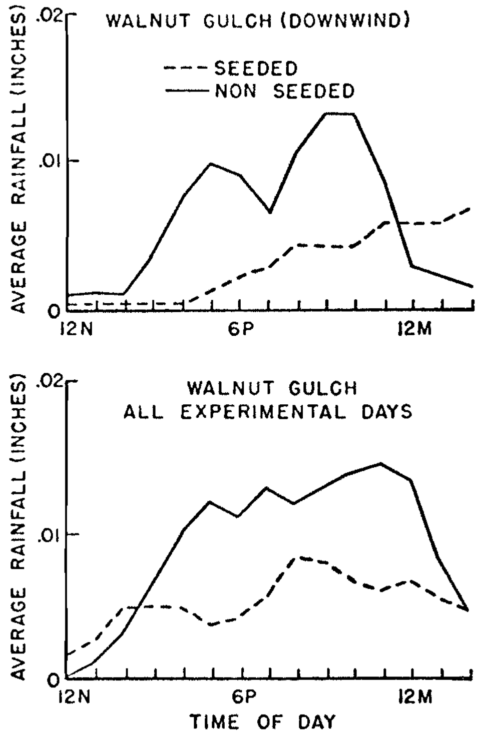


Figure 1. Hourly average rainfall on selected days, Walnut Gulch watershed, 1957-1964.

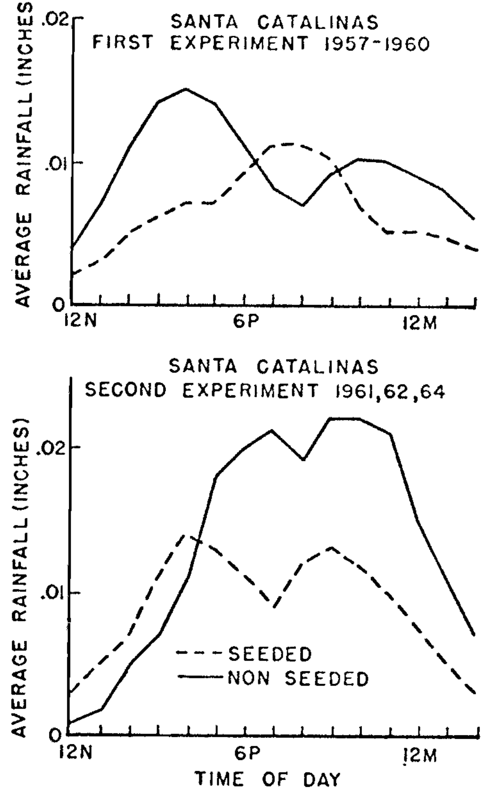


Figure 2. Hourly average rainfall for first and and second experiments, Santa Catalinas.

have been raised by analyses of Santa Catalina and Walnut Gulch data to justify further analyses of other rainfall records in Arizona, and to warrant further experimentation.

## OTHER ARIZONA CLOUD SEEDING (before 1971)

From 1966 through 1970, individual convective clouds were seeded under the supervision of the U. S. Bureau of Reclamation near Flagstaff, Arizona. The program was based on the belief that under certain conditions adding silver iodide nuclei to a convective cloud would promote more rapid freezing, thus releasing added heat that would cause the cloud to grow more rapidly and release greater amounts of precipitation. Weinstein and McCready (1969) reported on early results of these experiments based on their one-dimensional model of a cumulus cloud. In their paper, and in later papers by Weinstein (1970, 1971, and 1972) in which model results were extrapolated over large areas, conclusions were based on changes in reflectivity after seeding as estimated by 3-cm radar, with these changes related to predicted but unmeasured changes in rainfall amounts during the experiments. The validity of the model and the claims of success for the experiment have been challenged by several investigators. Warner (1970) challenged the reality of the model and Dennis and Schock (1971) stated "results of seeding experiments aimed at individual clouds or small target areas can not be extrapolated to predict accurately the effects of seeding over large areas."

The experiment failed to answer several questions. Among these are (1) what might have happened to a specific cloud if it hadn't been seeded; (2) if there were significant changes in the seeded cloud, did these changes mean an appreciable increase in rainfall at the ground; and, (3) if there were significant changes in the seeded cloud, did these changes affect the potential for cloud growth and resulting rainfall in surrounding clouds. To date, these questions have not been answered satisfactorily.

## 1971 ARIZONA CLOUD SEEDING

An intensive program of seeding individual cumulus clouds with silver iodide from airplanes was carried out in the summer of 1971 in central and eastern Arizona under the supervision of the U. S. Bureau of Reclamation. The program was patterned after the earlier Arizona experiments (cited previously) and similar experiments in Florida reported by Woodley (1970) and Simpson and Woodley (1971). Three planes were stationed in Safford, one in Flagstaff, and one in Show Low with seeding concentrated along the Mogollon Rim on the Salt River Watershed (Figure 3). U. S. Weather Bureau climatological divisions are also indicated in Figure 3. Clouds were seeded daily from July 20 through August 15. In August, it was reported that by the end of the period the seeding would have produced 400,000 acre-feet of additional summer rain over 20,000 square miles (Tucson Daily Citizen, August 14, 1971). This would amount to about 3/8 inch or 15% of the estimated summer rainfall of 2.5 inches for that period. An increase of 15% is well within any reasonable statistical confidence limits for prediction based on the highly variable thunderstorm rainfall. The method used to estimate 400,000 acre-feet of additional rain (most recently reported in the Tucson Daily Citizen, March 19, 1972, as 200,000 acre-feet, or less than 1/4 inch) has not been reported officially, but it was reported that the estimate was based on greater than average natural occurrence of rainfall during the period.

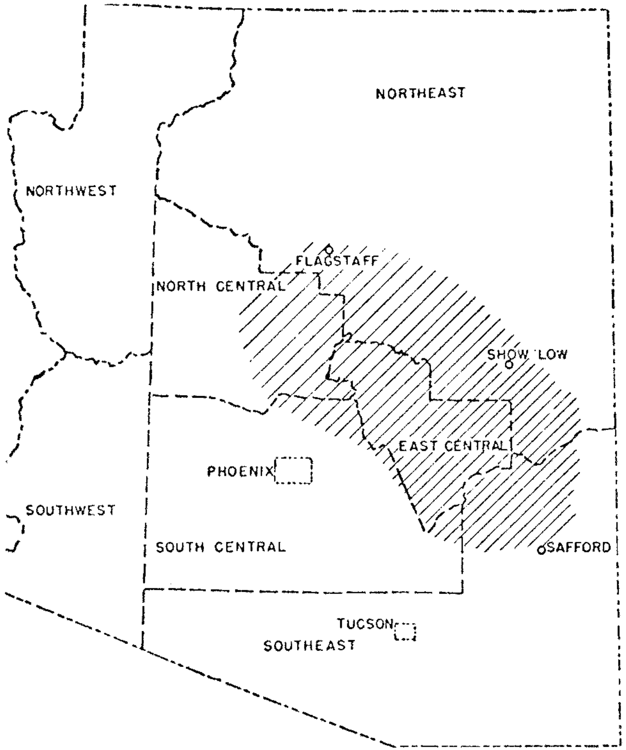


Figure 3. Principal seeded region in Arizona, 1971.

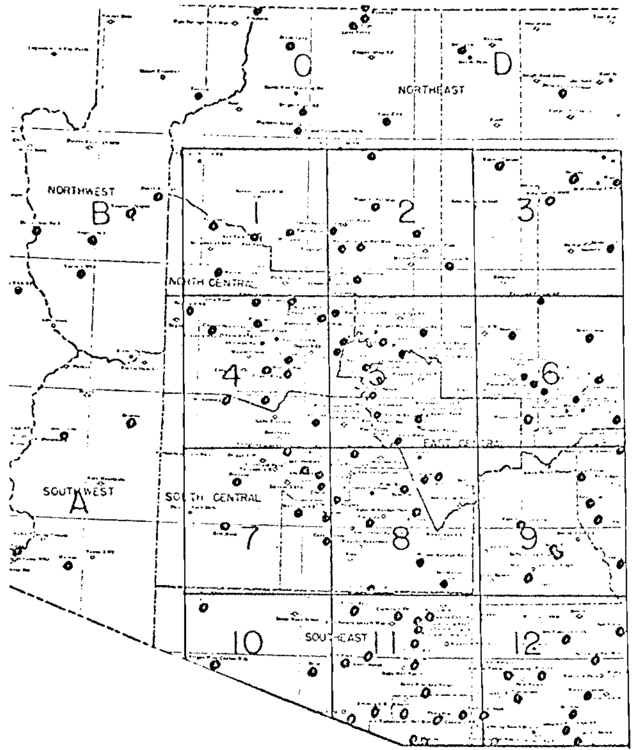


Figure 4. Stations used in an alyses of rainfall in Arizona, July 20–August 15, 1971.

Data from 133 U. S. Weather Bureau stations with at least 10 years of continuous record prior to 1971 were used in an attempt to analyze the rainfall pattern throughout Arizona for the seeded period in 1971 (Figure 4). Twelve equal quads were drawn to better illustrate the irregular rain gage network and to facilitate possible statistical analysis. Average annual amounts of rainfall for July 20-August 15 were determined for 10 years of record, 1961-70 (Figure 5). Rainfall in 1971 (July 20-August 15) was compared with the 10-year average rainfall for that period. In general, rainfall appeared above average in southern and western Arizona, about average through central Arizona, and below average in northern and eastern Arizona (Figure 6). In Arizona, summer winds are usually from the south, so north-central and northeastern Arizona normally would have been "downwind" from the seeding. Because of the extreme variability of thunderstorm rainfall and the uncertainty of point averages based on only 10 years of record, any stations within $\pm$ 20% of average arbitrarily were assumed average in designating regions that were above average, average, and below average during the period of seeding (Figure 7). Although obviously approximate, the figure again illustrates the large differences in expected versus actual rainfall in Arizona.

Finally, some feeling for the magnitude of the differences and the reliability of the analysis was attempted by first averaging the differences in each of the 12 quads and second by using a simple statistical sign test (Lindgren, 1968) for above or below average rainfall for each of the 12 quads. Assuming that the stations in each quad are representative of that quad, rainfall was above average in quads 1, 7, 11, and 12; below average in quads 2, 3, and 9; and uncertain in quads 4, 5, 6, and 8 (there were only three stations in quad 10, so it was not included in the analysis) (Figure 8). Statistically, rainfall was highly significantly above average in quads 7 and 11 (using zero as average), significantly

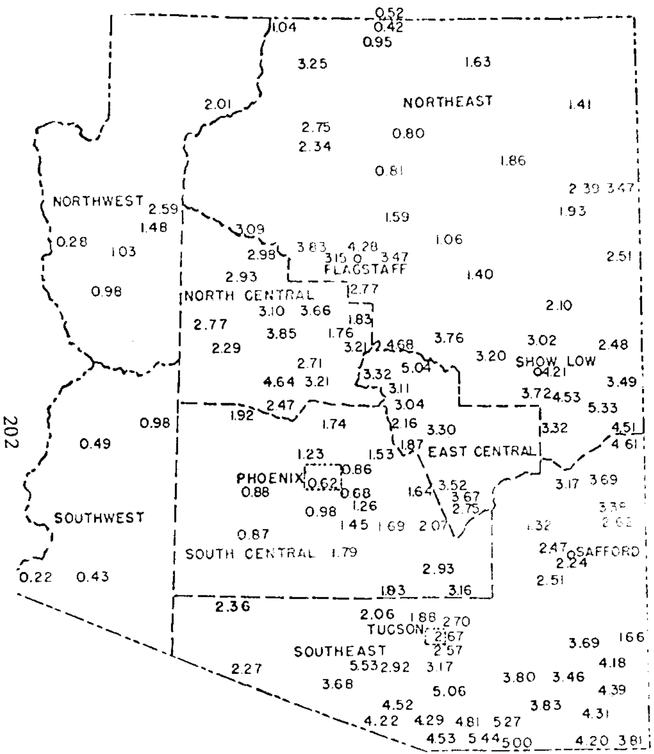


Figure 5. Average rainfall at selected Arizona stations, July 20–August 15, 1961–1970.

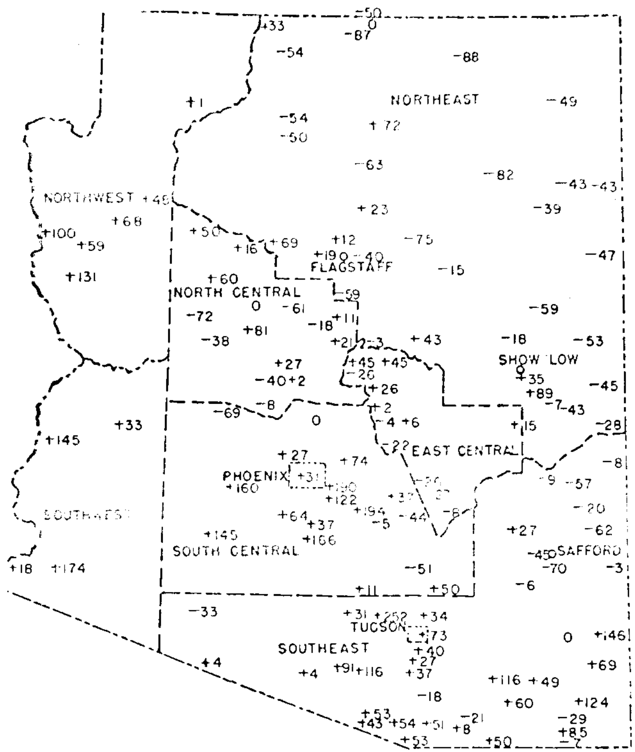


Figure 6. Percentage comparison of average and 1971 rainfall, July 20–August 15.

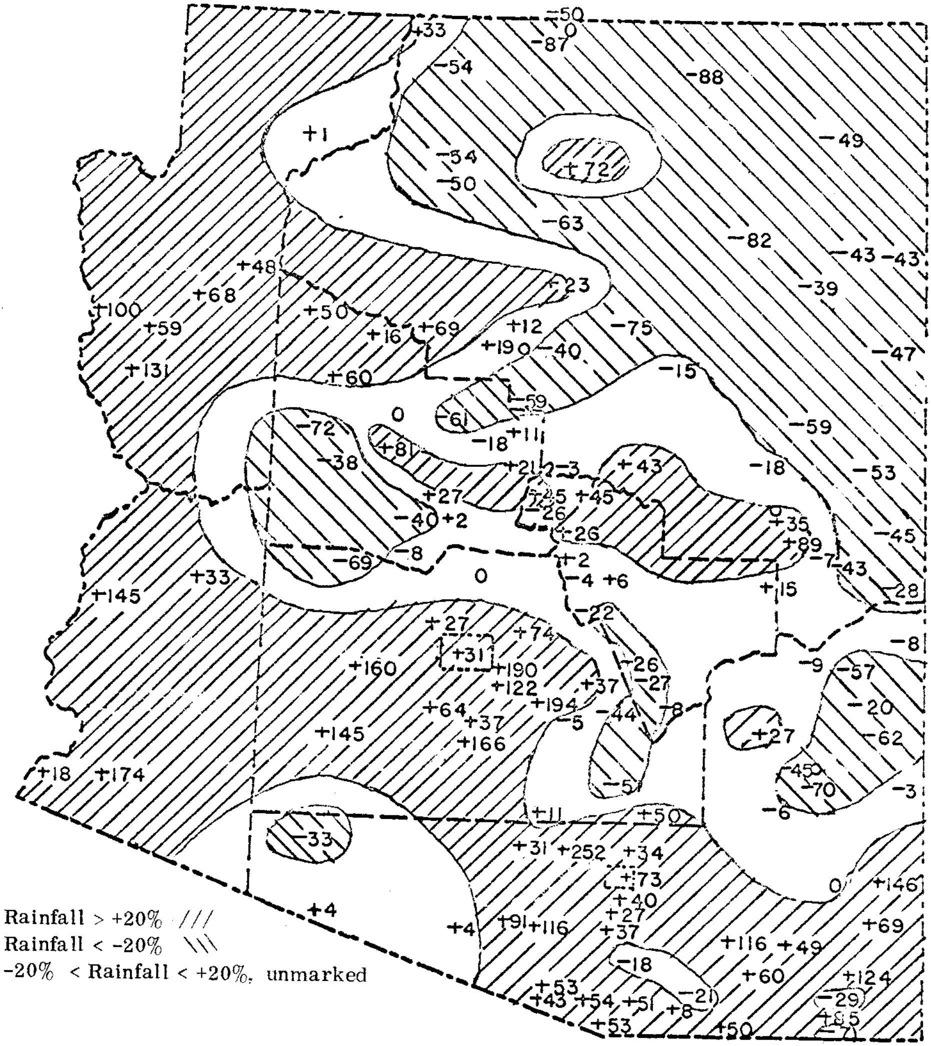


Figure 7. Comparison of average and 1971 Arizona rainfall, July 20-August 15, based on 133 VSWB standard rain gage records.

above average in the far west, and significantly below average in quads 3 and 9 (Figure 9). There was no statistical evidence that rainfall was other than average in quads 2, 4, 5, 6, and 10. Because of the scarcity of gages in most areas, a more rigorous analysis may be impossible.

## SUMMARY AND CONCLUSION

There have been two experimental and one operational convective cloud seeding programs in Arizona. The Santa Catalina experiment carried out by the Institute of Atmospheric Physics, University of Arizona, from 1957 through 1964 was by far the most comprehensive and only randomized experiment in Arizona. Early results from this experiment were inconclusive, but there was a suggestion that seeding with silver iodide decreased rainfall on the target. More recent analyses of Santa Catalina and other rain gage records suggested strongly that seeding decreased rainfall on and downwind from the target. Some of these results were statistically significant, whereas earlier results had not been.

Analyses of experimental seeding of individual convective clouds from 1967 through 1970 in the vicinity of Flagstaff were inconclusive because of inadequate experimental control and limited rainfall measurements at ground level. Analyses of a widespread program of seeding individual convective clouds in central Arizona in 1971 were also inconclusive because of insufficient experimental control and scarcity of rainfall data. However, rough analyses of available rainfall data did not support the hypothesis that widespread seeding of individual convective clouds increased rainfall over the seeded region. Furthermore, these analyses raised several questions concerning possible negative effects which can only be answered through more comprehensive randomized convective cloud seeding experiments, with ground measurements of actual rainfall.

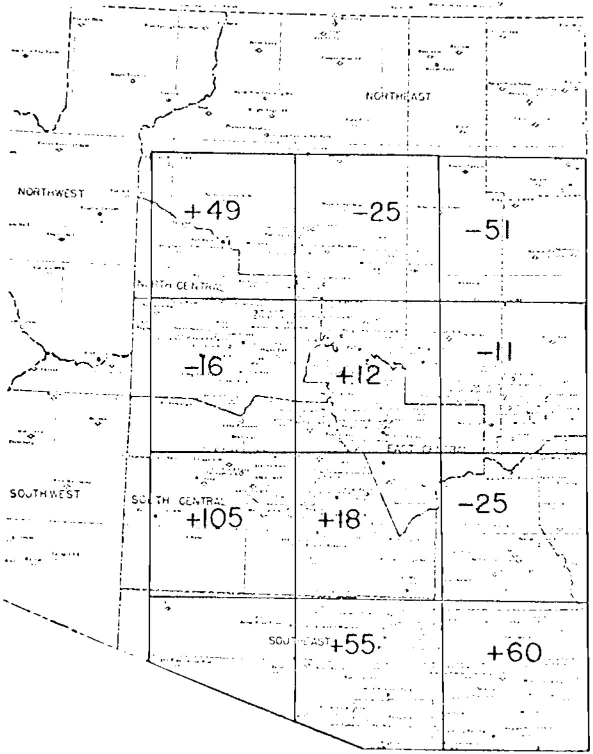


Figure 8. Comparison of average and 1971 rainfall by equal zones, July 20-August 15.

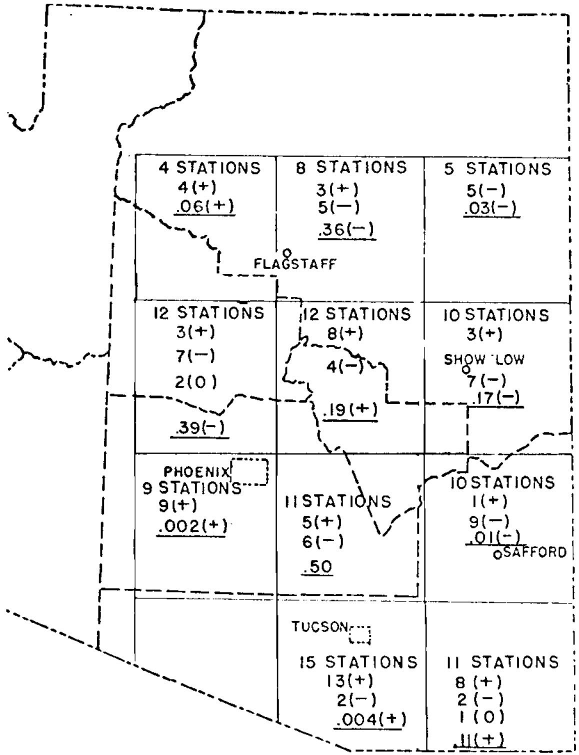


Figure 9. Probability of chance occurrence of rainfall, July 20-August 15, 1971.

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# SALINE AND ORGANIC WATER POLLUTION

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Future utilization of the earth's water resources will take into account the compound problem of increasing human population and decreasing availability of water due to pollution. Pollution, by decreasing the utility of water, decreases the amount of available water. Primary attention has been focused on pollution by heat, organic substances, and nitrogen and phosphate compounds. The importance of these pollutants is undeniable, but polluting fresh water by dumping it into the sea is an even graver, yet rarely considered problem.

The effect of saline pollution of water is that organisms must expend more energy to absorb water from saline than from fresh sources. Salt water is thermodynamically less suitable for life. While many organisms are adapted to saline conditions, they might be better described as tolerating salinity or making the best of a bad situation.

Salt pollution is a far more serious problem than the pollution resulting from the other components because once water is contaminated with salt, external energy is necessary to separate the water from the salt. The only energy source at present for desalting large amounts of water is the sun.

Organic water pollutants, in contrast, are inherently unstable chemical compounds. Give sufficient time and suitable conditions, these substances will disappear from the water by chemical and microbial means. The suitable conditions generally entail only circulation of the water, access to oxygen, and microbial catalysis. The oxidation of organic wastes is spontaneous, hence requires no energy input. The oxidation products are a little energy and carbon dioxide which is volatilized to the atmosphere. Ironically, dumping organically-polluted water into the sea not only contaminates the water with salt, it slows the rate of organic oxidation. Microbial reactions are slowed by the salinity and low temperatures of the sea, the slow circulation of sea water, and the depth of the oceans which minimizes access to oxygen.

The slight degree of pollution in organic pollution is also obvious from another, superficial aspect. The effluent from secondary stage sewage treatment contains 15 ppm BOD, is 99.9985% pure water, and the impurity is transitory. When mixed with sea water, however, this water is only 96.5% pure and the impurity is stable.

Man is unaccustomed to think of salt water as polluted water. The oceans are part of the natural environment and therefore "good", but oceans have been rightfully called watery deserts in part because salts inhibit organic activity. We do not propose here that the biological activity of the seas should somehow be increased, or even meddled with, but rather that better use and recycling of fresh water would reduce man's disruption of nature to increase his supply of potable water.

## GLOBAL WATER DISTRIBUTION

The oceans contain 97.3% of the earth's water; another 2.1% is frozen; only 0.6% is liquid, low in salt content, and a potential source of water for terrestrial organisms. Perhaps half of this liquid fresh water is deeply underground or unavailable for most purposes so that only 0.3% of the earth's photosynthesis and organic life that occurs on land, for almost all of man's sensible environment, and for his food supply.

Man is powerless to change this distribution of water even slightly: the energy required to desalinize significant amounts of water, for example, is staggering. The earth's fuel supplies are inadequate except for local desalination and then only if nuclear fusion can be harnessed. An increase in the supply of fresh water requires better use and better reuse of local supplies of that small, but manageable, 0.3% of the earth's total water supply.

The world now relies almost totally upon solar energy for the recycling and redistribution of water. The resultant precipitation is erratic and unevenly distributed over the earth's surface. In addition most of this solar desalted water falls back directly into the sea and is again contaminated with salts without entering the terrestrial water cycle. Attempts to modify the distribution of precipitation have been unsuccessful in Arizona (Osborn, 1972).

## DESALINATION

Changing the distribution of water by desalination has received much attention as a means of increasing our water supplies (Young, 1970).

Desalination requires the physical separation of salt from water. Whether accomplished by distillation, freezing, reverse osmosis or electodialysis, the basic energy requirements of separation are considerable and expensive even if any of these processes could be carried out in the most efficient way -- the infinitely slow and unattainable, thermodynamically reversible process. The large energy requirement brings with it the strong likelihood of wasted heat and thermal pollution. Nonetheless, desalting has been hailed as a means of increasing fresh water supplies by such groups as the U.S. Atomic Energy Commission and the Department of Interior. Man has long dreamed of converting sea water into fresh water, but the dream has unfortunately stifled his ingenuity to devise alternative ways of providing fresh water.

In some situations, however, desalting may be worthwhile. Where fuel is cheap and the only water available is saline, such as along the Persian (Arabian) Gulf, desalting may be economically feasible. Desalting may also be a useful way of utilizing waste heat from power stations and industry, thereby decreasing thermal pollution. The Environmental Research Laboratory of The University of Arizona uses waste heat from a power plant for desalting. The coastal desert town of Puerto Peñasco, Mexico, using this process receives electrical power and potable water with little wasted heat or thermal pollution.

Desalting may be profitable when it removes valuable substances from some industrial waste waters. The profit impetus may encourage the cheese, dairy, pulp and paper, iron and steel, plating and metal finishing, nuclear power, and coal mining industries to clean up their waste water (Chem. Eng.

News, 1970). Their goal would be to remove valuable chemicals but would also result in the production of better water.

## WATER RECLAMATION AND RECYCLING

A more feasible alternative than desalination for increasing our water supplies is to reclaim and reuse our waters. Water reuse receives too little attention in relation to its potential. Governmental agencies have no vested interest in it, only a small industry sees profit in it, and the stigma attached to reuse of waste is difficult to overcome in a squeamish, cleanliness-is-next-to-godliness society. Our reflex action is to flush away wastes immediately so we dump our water into streams, lakes, and the oceans.

But by organically polluting water, we do not destroy it. The added organic substances only temporarily reduce water's utility because they are unstable. Through microbial catalysis in sewage treatment, streams and soils, organic compounds literally self-destruct through fermentation and oxidation.

The Mescatal and Lerma Valleys of Mexico use as their irrigation water the untreated sewage water of Mexico City (H. K. Qashu, personal communication). This raw sewage water travels 60 km through an open aqueduct and arrives free of odors, sediment and colloidal matter. During their travel in contact with the air, the urban wastes oxidize to carbon dioxide and a little soluble salt. To insure complete oxidation, however, the water is slightly chlorinated prior to irrigation. The irrigated area is now 40,000 hectares. The project has been successfully underway for 70 years. The water leaves these valleys by evapotranspiration and by percolation into the porous limestone bedrock of the sea. Before recharging the groundwater reserve, the water has been returned to its unpolluted state through oxidation by air and nutrient adsorption and uptake by soil and plants. A sewage treatment is unnecessary; the only capital cost is that of the canal and its cost can be apportioned between sewage disposal and irrigation development.

The analogy between this scheme and the proposed exchange of Tucson's sewage water for Avra Valley groundwater seems obvious.

## ROLE OF SOILS AND PLANTS IN WATER RECYCLING

Sewage treatment affects only the carbon content of the water. The nitrogen, phosphate and other nutrients in the water are untouched. Dumping sewage water into fresh and marine waters represents mining of the soil's resources. If this water were returned to the soil-plant system, water pollution would be diminished, soil fertility maintained, and less fertilizer required for food production. A lower fertilizer requirement would also mean less pollution from the industrial production of fertilizers.

Present-day waste disposal discards waste water without even intending to return it to the nutrient and water cycles. Our sewage treatment and waste disposal technology has considered itself as an end and not as a part of a cycle. Nature does not reject our wastes, it converts them into useful forms for the world's biomass, including man. When placed in the sea, however, wastes and water are returned only very slowly to the terrestrial cycles. In addition, the transformations of pollutants into carbon dioxide soil organic matter, plant nutrients and good water are much more rapid in the soil.

The interaction of soil, water, and air is that which our sewage treatment technology tries to emulate. Soil treatment is so effective in purifying water that conventional sewage treatment may in some cases be unnecessary. The Campbell Soup Company in Paris, Texas, for example, removes 95-99% of the organic matter, 95% of the nitrogen and 60% of the phosphate from its raw cannery waste water by simply spraying the water on hillsides and allowing the water to trickle downslope over the soil surface before passing into a stream (Thornthwaite, 1969). The phosphate content would undoubtedly be lowered even more if soil-water contact were increased. One might hope that the treatment would be even more extensive so that the water could be immediately recycled into the cannery. But progress comes only one step at a time.

By removing nitrogen and phosphate from water, soil-plant treatment far surpasses sewage treatment. Soils and plants can also complement conventional sewage treatment by efficiently extracting pollutants at Pennsylvania State University (Kardos, 1967); The University of Arizona; USDA Water Conservation Laboratory, Tempe, Arizona, (Bouwer, et al., 1971); The University of Guelph (Heywood, et al., 1969) and elsewhere (Wilson and Beckett, 1968). In these studies, the secondary-treated water percolates through or flows over soils. Irrigation with waste water is an accepted practice in Germany, France, and Great Britain. In intimate contact with the large reactive surface of soil and the soil microbial population, carbon compounds are adsorbed in great amounts by soil. Nitrogen compounds are adsorbed by plants and soil microorganisms; the latter even remove some of it from the system by forming nonpolluting nitrogen gas and expelling it to the atmosphere. The phosphate and nitrogen are available to plants as essential, often growth-limiting nutrients. This plant and microbial uptake of nitrogen is the only effective means in nature of recycling nitrogen and of preventing nitrogen pollution of water.

URBAN MISUSE OF WATER WATER

Los Angeles is one of the many examples of water misuse -- dumping waste water into the sea rather than reclaiming it. The water taken by Los Angeles from groundwater, the Colorado River, Owens Valley and Northern California could be reclaimed rather than discarded into the Pacific Ocean. The sandy soil and high temperatures of nearby Ontario and Coachella Valley, for example, are ideal for reclaiming the waste water. The waste water could also be used to create green belts for fire breaks in the chapparal-covered mountains which periodically burst into flame. Perhaps the best choice would be to return the water to the areas from whence it came.

The amount of water available for recycling is more than considerable. The greater Los Angeles sewage system will ultimately have to dispose of 1,600,000 acre-feet annually (California Department of Water Resources, 1961). Because of some arbitrary cost allocations, the present method of first and partial secondary sewage treatment plus ocean disposal is the only method now considered for almost all of this water. The cost of sewage treatment ($10 to $15 per acre-foot) is allocated to the waste water even though the treatment would be done for any disposal method. In our opinion, this cost should be considered separately. Sewage treatment may even be unnecessary if the water is conveyed any distance and if the water is given access to oxygen along its route. The cost of conveyance plus recycling is about $10 to $15 per acre-foot so that recycling the water is likely to be no more expensive than the rudimentary sewage treatment plus ocean dumping now done and envisioned for the greater Los Angeles area. The soil-plant reclamation of sewage water could a) end much of the pollution of Los Angeles' shores, b) increase crop production by enriching and irrigating the Ontario and Coachella Valley soils, c) decrease fire hazards, d) regenerate the groundwater supply, and e) decrease the amount of water which Los Angeles now imports from the other areas.

Water recycling involves some problems so that a closed system with regard to water is not feasible. Each cycle of water through municipal water systems adds 100 to 300 ppm salts. Higher amounts of salts often come from industrial wastes and from sea water entering leaky sewage systems. These salty flows are easily isolated from the main flow. Some water reclamation in Los Angeles is already underway, but the cost of this reclamation has raised some complaints. When this treatment of water includes such obvious benefits as the prevention of salt water intrusion into the groundwater, however, the cost of water recycling is hardly considered. The complaints also ignore the real cost of the water which Los Angeles now imports and the benefits which the water would yield in the areas now sacrificed to Los Angeles' water demands. Los Angeles pays only the cost of water distribution and pays nothing for the imported water itself.

Much of the water is taken from the already water-short Colorado River Basin including Sonora and Baja California, Mexico.

In an effort to increase the water supply of the Colorado River to meet the demands of the state in the upper and lower Colorado Basins and Mexico, the U.S. Bureau of Reclamation wishes to straighten the river's channel and to remove or control the bank vegetation. This plan is under attack by conservation groups. The project and controversy might be unnecessary if Los Angeles would better use and reuse its water supply.

Los Angeles is only one obvious example of the municipal misuse of easily reclaimable water by pouring it into the sea. This practice is worldwide among coastal cities. Recent water shortages in the north-eastern U.S. point out the results of carelessness in the use of our water supplies and the shortsightedness of using the seas for sewage water disposal. New York City throws away its waste water rather than reclaiming it. Water reclamation would decrease the drain of the city's water sources and could simultaneously prevent underground salt water encroachment on Long Island. The abandoned farmland of upstate New York and the nearby Hackensack Meadows of New Jersey could also be used for waste water reclamation. Large areas of wooded, pasture and open land surround other major cities, including those along the Great Lakes and in the northeastern U.S. Water will be reclaimed by soil-plant treatment in Muskegon, Michigan (Shaeffer, 1970). Hopefully this treatment will soon be expanded to other areas. The land required is negligible -- it is used simultaneously for parks, golf courses, woodlands, green belts, wildlife sanctuaries, and agriculture. Metropolitan New York City would require 100,000 to 150,000 acres of this multiple-use land; greater Los Angeles, perhaps 50,000 acres.

In areas where irrigation is necessary only sporadically for intensive agriculture, plants can make use of large amounts of additional water (Sopper, 1971), The sewage could be spread on woodland and pastures by large rotating sprinklers in much the same way as the pilot project at Pennsylvania State University. The sprinklers spread the water in pulses which maximizes aerobiasis. Plant growth would be enhanced by the plant nutrients in the waste waters, by an unlimited water supply, and in cold climates by lengthening the growing season.

Waste water reclamation by soil causes little ecological change because the pollutants and waters are being returned to the soils from which they originate. The change in soil is local and is far more easily measured, manipulated and controlled than are the changes caused by the same substances in water or air.

Recycling and a nutrient balance imply living off our wastes. Such an idea is hard to sell to a society which is already subjected to a hard sell about soap and deodorants. Our society needs to be reminded that the materials used to keep itself clean and pleasant smelling are often as serious pollutants as the dirt or odor removed by them.

The earth has been involved with the recycling of wastes into nutrients for some 3,500,000,000 years, at the last estimate. Man's pollution represents a tilt in the nutrient balance. By correcting this imbalance, we can mitigate pollution problems and, not insignificantly, increase our supply of food and good fresh water.

The intimate involvement of soils and plants in the initial production of fresh water and food is clear. Just as clear, but as yet only poorly recognized, is that soils and plants should be equally involved in covering our waste water into potable water and useful nutrients.

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# COLLECTIVE UTILITY OF EXCHANGING TREATED SEWAGE EFFLUENT FOR IRRIGATION AND MINING WATER

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## ABSTRACT

This paper applies the concept of Collective Utility to a case study of alternative water resource utilization. Collective Utility is a theory that provides a basis for comparing alternative uses of resources from the point of view of aggregate welfare. It compares alternative natural resource systems that are marginally different by means of calculating a change of Collective Utility. In order to demonstrate the usefulness of the Collective Utility approach, data from a case study of water reuses scheme in Tucson, Arizona are used. Two alternatives are considered, namely, the exchange of sewage effluent for groundwater that is presently used by the farmers as irrigation water and the exchange of sewage effluent for groundwater that is presently used by the mining companies as processing and milling water.

## INTRODUCTION: THE CONCEPT OF COLLECTIVE UTILITY

The concept of collective utility has been imbedded in many economic thoughts since the 18th century (*Duckstein and Dupnick*, 1971). However, a systematic presentation of collective utility theory had not been made until 1964 (*Lesourne*, 1964).

Primarily, collective utility theory considers an aggregated social well-being, i.e. calculates a collective utility function under a set of well-defined hypotheses. Consequently, with an initially optimal economic state, any pertubation to this equilibrium will result in a change in collective utility. The question is whether or not this change, which constitutes a marginal transformation from an initial economic state (initially optimal) to a transformed one (perturbed state), increases the well-being of the society. If the answer is affirmative, then this marginal transformation is desirable from the standpoint of collective utility. Therefore, a change of collective utility, dU, should provide an objective index (subjective from Big Brother's point of view) for decision-making in marginal transformations. This, in a way, is a complement to the cost-effectiveness approach, which compares a set of distinct alternative systems designed to satisfy certain goals (*English*, 1968; *Ko and Duckstein*, 1972).

## CASE PROBLEM

Due to the lack of perennially flowing streams in Southern Arizona, the diminishing groundwater resource is regarded as a scarce natural resource. The crisis of future water shortage, the increasing pumping costs, the wasteful dumping of treated effluent, a valuable resource, in the Santa Cruz river and the resulting potential pollution of the nearby groundwater have received considerable attention in recent years. Several possible schemes to alleviate the problem have been proposed, four of which are described below.

*System I.* The sewage effluent is exchanged for the groundwater that is presently used for irrigation in the Avra-Marana Valley area. Several advantages can be associated with this exchange scheme:

1. The plant nutrient value that exists in the secondary effluent can be utilized, which saves fertilizer expense for the farmers.
2. The solid waste resulting from the sewage treatment process can be applied to the farmland as a soil conditioner. Currently, this solid waste is transported to various city parks to be used as soil conditioner. The transportation cost for Randolph Park alone is $33,000 annually.
3. The potential of polluting the groundwater in the receiving ground can be avoided. *Cluff et al.* (1971) reported that in recent years the nitrate content of the groundwater in the area receiving secondary effluent discharge had increased. Should this effluent be applied to the farmlands, the nitrates and phosphates will be assimilated by the plants. As a matter of fact, a court suit has been filed against the City of Tucson for high nitrate content in the groundwater because it allegedly endangers public health.

However, the following disadvantages can also be associated with the exchange scheme:

1. There is no assurance the City can pump whatever it needs from the groundwater under the farmlands. In other words, the risk of water supply shortage is three-fold:

a. The City is upper bounded by a contract,

b. There is no guarantee that a contract will be signed after the present contract will have expired, and

c. The exchange price for a diminishing resource will increase indefinitely.

2. There is no safeguard against mismanagement of water by farmers.

*System II.* The sewage effluent is exchanged for the groundwater that is presently used as mining and milling water in the Sahuarita area. In this system, only advantage (3) listed under System I applies. However, one additional advantage can be listed for this system: there is almost no discharge from the mines; therefore, the danger of groundwater pollution is avoided. Furthermore, the water demand is constant, i.e., there is no seasonal fluctuation as in System I. On the other hand, there are many disadvantages:

1. As the extent of water shortage increases, the mines become potential competitors in buying the farmlands to ensure their own future water supply; certainly, the mines have the capital to do so.

2. Due to the mining process, the water quality requirements are higher than those of secondary effluent. Consequently, additional treatment is needed.

*System III.* The City of Tucson purchases the farmlands in the Avra Valley area and lease it back to farmers. Here the City can dispose of the effluent optimally and has assured water supply in the future. In this exchange system, the advantages listed under System I can still be listed and at the same time the disadvantages listed for System I are eliminated. However, considerable negotiations and strategic planning are needed in order to implement this scheme, which is described in greater detail in the cost-effectiveness study by *Ko and Duckstein* (1972).

*System IV.* The used water is recycled after advanced treatments. The efficiency of water use is maximum; so is the cost. Anyway, unless additional sources are available in the future, the available groundwater resource will sooner or later become depleted.

There are obviously many other alternatives, such as a dual distribution system, one for potable and one for nonpotable water, or any combination of the above systems. For the purpose of illustrating the technique of Collective Utility, it will be assumed that System I and System II are to be compared. and that the transition from one system to the other is marginal, i.e., does not cause a radical change in the economy of the region under consideration. The latter hypothesis is certainly reasonable in terms of the foreseeably moderate impact of exchange Scheme I versus Scheme II upon the Tucson Basin economy.

## DEFINITION OF THE PROBLEM

The economic state for each exchange system will be compared to the initial state obtained if the current operational scheme is continued, i.e., no exchange scheme is implemented. The economic agents under consideration include: (1) the City of Tucson, (2) the farmers in Avra Valley, and (3) the mines in the Sahuarita area. The goods involved are: (1) groundwater, (2) secondary sewage effluent, and (3) other goods.

The following hypotheses are necessary in order to formulate our mathematical model (*Lesourne*, 1964; *Duckstein and Kisiel*, 1971; *Metler and Duckstein*, 1971):

*Hypothesis I.* In the initial state, the distribution of income is optimal, everybody knows what he wants and maximizes his satisfaction function.

*Hypothesis II.* A fair market exists for the exchange of goods and services: each firm pays the same price for the same goods or service.

*Hypothesis III.* Each firm is a price taker instead of a price maker, there are no quantity discounts.

*Hypothesis IV.* No additional water can be imported into the economic area under consideration: the economy is closed, with partial utilization of natural resources (because the possibility of saving groundwater is included in our model).

## FORMULATION OF THE MODEL

Under the above hypotheses, the proposed single-period deterministic model will be formulated using the folowing economic criteria:

1. Latent benefits.
2. Latent costs.
3. The environmental impact due to the use of effluent.
4. The amount of water saved due to exchange.

All these items will first be defined, then quantified whenever possible. Items which are not quantifiable will be lumped under a symbol G so that a mathematical expression can be obtained and the value of G which causes a decision reversal, i.e., a sign change of the collective utility dU, can be calculated.

*Latent Benefit.* The latent benefit is defined as the difference between the benefit provided by the current supply system and the benefit provided by the future exchange system under the set of hypotheses defined earlier. In other words, the benefits provided by the present farming water supply scheme and the present mining water supply scheme will not be considered; only the benefits resulting from the change of economic state will be taken into account.

The latent benefit can include items such as: (a) plant nutrient value for the farm, (b) agricultural utilization of the solid waste, and (c) better water quality.

The plant nutrient value corresponds to the fact that nitrogen and phosphorus which are regular ingredients of the secondary effluent would be purchased anyway by the farmers in the form of fertilizers. Therefore, this additional (latent) benefit stems from the difference in fertilizer cost between the initial and the transformed economic state. Next, the solid waste, which is a by-product of sewage treatment, can be used as soil conditioner; saving on its transportation costs provides futher latent benefits.

*Latent Costs.* Latent costs are defined similarly to latent benefits under the same set of hypotheses and include the following items: (a) change of pumping cost between the two economic states, (b) additional costs of conveyance systems, (c) differential operation and maintenance costs, (d) change of treatment cost, and (e) added cost of storage of excess effluent, if any.

*Environmental Impact.* The environmental impact due to reuse of the effluent can constitute either a negative or a positive contribution to collective utility. For instance, use of the effluent to irrigate farmland may improve the permeability of the soil on one hand and increase its salinity on the other. Furthermore, some of the impact cannot be quantified, such as, for instance, the unpleasant odor created by the use of solid waste, which lowers the quality of living of nearby residents and of the farmers themselves; thus, this reduction in quality of living varies from individual to individual.

*Amount of Water Saved.* Assume that in the initial state (i.e., no exchange), the City pumps X a.f. of water each year. In the transformed state (i.e., with exchange), the City will pump Y a.f. per year, with $X > Y$. The amount of water exchanged will be $(X-Y)$. Let the amount of reclaimable water be K% of the water transported (due to losses during the transportation); then the amount of water saved will be

$$Q = (X - Y) \sum_{j=0}^{n} K^j \qquad (1)$$

where n is the horizon of the study. We shall assume that $n = \infty$ to simplify calculations. Since each system has a different transportation method, the value of K will be different.

## MARGINAL CHANGE OF COLLECTIVE UTILITY

*Definitions and Basic Expression for dU.* The following symbols will be used in this analysis:

<u>*Index*</u>

| | |
|---|---|
| *i* | *Goods or services. Let i = 1 for groundwater, i = 2, for sewage effluent, and i = 3 for others.* |
| *h* | *Firms that produce or consume goods or services. Let h = 1 for the City of Tucson, h = 2 for the farmers and h = 3 for the mines.* |

<u>*Quantities*</u>

| | |
|---|---|
| *B(i,h)* | *Latent benefit produced by h due to the consumption of good i.* |
| *CC(i,h)* | *Latent construction cost required by h due to the consumption of good i.* |

*CM(i,h)* *Latent cost of operation and maintenance.*

*CT(i,h)* *Latent treatment cost.*

*CS(i,h)* *Latent storage cost.*

*S(i,h)* *Amount of water saved.*

*Q(i,h)* *Quantity of water consumed or exchanged.*

*G* *Unquantifiable factors, especially the reduction in quality of living due to the odor of solid waste for the farming exchange system.*

*E(i,h)* *Environmental impact, such as saving on tertiary treatment costs.*

Based upon the previous set of hypotheses and the above symbols, the change of collective utility dU is

$$
\begin{aligned}
dU &= \sum_i [p(i)dQ(i,h)] - G \\
&= \sum_i [B(i,h) - CC(i,h) - CM(i,h) - CT(i,h) - CS(i,h) \qquad (2) \\
&\qquad + E(i,h)]\ dQ(i,h) - G
\end{aligned}
$$

*Transformation for the Farming Exchange System*

The total demand for the agricultural irrigation was projected by the City of Tucson at 45,000 a.f. per year. Therefore dQ(1) = dQ(2) = 45,000 a.f./yr.

*Latent Benefits.* Latent benefits for the farming system will be the added plant nutrient value and the saving on the solid waste transportation; the plant nutrient value was reported to be \$4 to \$5 per a.f. (*Cluff et al.*, 1971). An updated value of \$7 per a.f. will be used. The savings on transporting the solid waste is estimated from the transportation cost to Randolph Park at \$2 per a.f.

*Latent Costs.* Latent costs consist of pumping construction, operation and maintenance costs. The amount reported by *Cluff et al.* (1971) was updated at \$198,000 per year for 13,000 acres of irrigatable farmland.

*Environmental Impact.* Analysis of this item may be divided as follows:

(a) *Change of Groundwater Quality.* As indicated earlier, if the sewage effluent is discharged directly into the Santa Cruz River, there is the potential of polluting the nearby groundwater due to the nitrate and phosphorus content of the effluent. In order to avoid such hazard, additional advanced treatments should be applied to the secondary effluent before it can be discharged into the river. On the other hand, if the exchange scheme is implemented, this hazard can be avoided. Advanced treatment costs were given by *Smith* (1969); an updated value of such advanced treatment is estimated at \$11.50 per a.f. for a 45 mgd plant.

(b) *Change of the Quality of Living.* As said earlier, this factor cannot be assigned a monetary value and will be represented by G.

(c) *Change of Soil Quality.* The increase of soil salinity is a negative contribution to the change of collective utility dU and the increase of soil permeability is a positive contribution to dU. It is assumed that these effects are secondary and nullify each other.

*Amount of Water Saved.* The percentage of reclaimable water can be estimated from the ratio of total annual groundwater pumped to total annual sewage treated, which is equal to 50%. If the transportation losses are assumed to be 10% of the total water transported, then, using equation 1 the amount of water saved is: 1.7 dQ(1,2).

Assume the effluent is sold at \$1.00 per a.f. Then the total amount of water saved is equal to \$1.7 dQ(1,2) per year.

Now it is possible to calculate dU for the farming exchange system.

$$\begin{aligned} dU &= [B(2,2) - CC(2,2) - CM(2,2) - CS(2,2) \\ &\quad + E(2,2) + S(2,2)]\, dQ(2,2) - G \\ &= [\$9.0 - \$4.4 + \$11.5 + \$1.7] \times 45{,}000 - G \\ &= + \$800{,}500 - G \end{aligned}$$

*Transformation for the Mining Exchange System*

*Latent Benefits.* Should the mining exchange system not be implemented, then the effluent has to be discharged into the Santa Cruz River, and a potential danger of polluting the groundwater exists. If the exchange system is implemented, this hazard can be avoided. Consequently, the latent unit benefit is equivalent to the cost of treating the effluent. This cost is the same as in the farming exchange system.

*Latent Cost.* According to the City, the total cost for constructing the exchange system is estimated at \$264,000 per year. In addition to this cost, the mines require a certain water quality standard so that treatment beyond the secondary treatment is needed. While no definite quality specifications are available at this time, estimations are based on *Smith* (1969) for a 45 mgd treatment plant.

*Environmental Impact.* We expect this element to be negligible for the system considered.

*Amount of Water Saved.* Assume there is no loss during the transportation through the pipe line. Then, using equation 1, the amount of water saved is:

$$\sum_{j=0}^{n} (0.5)^j \, dQ(1,3) \; .$$

Over a sufficiently large horizon n, at an effluent cost of \$1.00 per acre-foot, the total water saved is worth \$2.00 x dQ(1,3) per year.

It is now possible to calculate the change of collective utility for the mining exchange system

$$\begin{aligned} dU &= [B(2,3) - CC(2,3) - CM(2,3) - CT(2,3) + S(2,3)] \; dQ(2,3) \\ &= [\$11.5 - \$5.9 - \$11.5 + \$2.0] \times 45,000 \\ &= - \$175,000 \end{aligned}$$

## CONCLUSIONS

The change in collective utility for the farming exchange and the mining exchange systems are respectively:

$$dU_I = 800,500 - G$$

$$dU_{II} = - 175,000$$

Comparing the two systems, it appears that System I is more desirable than System II. However, the cost of lowering the quality of social living is not defined at this point. If G = \$975,500 per year, then System I is equivalent to System II. If G > \$975,500 per year, then the mining system is more desirable. On the other hand, the solid waste is to be spread over an area of low population density (so far). Therefore, it is unlikely that G would have a magnitude of the order of \$1 million per year.

It is hoped that this case study shows the applicability of the concept of Collective Utility to compare natural resources systems that are marginally different. The method clearly defines the interrelationship among variables and prevents double-counting through the use of a mathematical expression that was derived under a set of well-defined hypotheses.

## ACKNOWLEDGMENT

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# EFFECT OF A GRASS AND SOIL FILTER ON TUCSON URBAN RUNOFF: A PRELIMINARY EVALUATION

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Abstract. Storm runoff from the Tucson metropolitan area is unsuitable for most uses without processing. A lysimeter comprised of a grass and soil filter was constructed and is being evaluated as a water-quality treatment facility. The lysimeter is 200 feet long, 4 feet wide and 5 feet deep, and contains homogeneous calcareous loam covered by common grasses. Experimental apparatus was installed to divert less than a cubic foot per second of runoff from urbanized Arcadia Watershed. Runoff flows by gravity over the lysimeter, where surface inflow, surface outflow and subsurface outflow are measured and sampled. Four trials, each associated with a discrete runoff event, were conducted in the fall of 1971. Water samples were analyzed for inorganic chemical constituents, chemical oxygen demand (COD), coliforms, turbidity and sediment contents. Subsurface-outflow samples from initial trials were high in COD and total dissolved solids, representing soil flushing or leaching. Concentrations of inorganics reached a maximum value within a few hours of initial seepage, and then decreased. The peaking represents a salt build-up between trials. Concentrations of COD, coliforms, turbidity and sediment in subsurface-outflow samples decreased significantly during each trial. Surface-outflow samples had lower turbidity, COD, bacteria and sediment contents than surface-inflow samples. Turbidity, suspended and volatile solids, coliforms and COD in runoff samples may be reduced by grass and soil filtration. Increased grass development and soil settling work to produce a better quality effluent. Quantification of the lysimeter's effectiveness will be useful for urban watershed management.

## INTRODUCTION

Water in the semiarid and arid environment is a limited resource which must be managed efficiently and economically. Urban water problems in such an environment are unique in that the watershed changes rapidly in time with urbanization. Hydrologic effects of urban growth include increase in runoff volume and flood hazard, degradation of streamflow quality and increase in demand for water. Many southwestern cities are growing at rates that make these hydrologic effects significant. Increases in construction of flood- detention and routing structures, water and waste-water treatment plants, and fire-protection and water-conveyance units are apparent in recent years.

The University of Arizona has undertaken an interdisciplinary Urban Hydrology Study with the support of a grant from the Office of Water Resources Research, U. S. Department of the Interior. Multipurpose objectives of this research are the following:

1) To collect and interpret hydrologic data on the Tucson, Arizona, metropolitan region.
2) To develop management procedures for urban runoff in arid lands.
3) To apply a systems approach with interdisciplinary objectives to the problems defined.
4) And to aid in training students as hydrologists.

Through this study, a management procedure which reduces summer flood peaks by diverting channel flow following removal of sediment to surface ponds and/or recharge wells is being considered. Initial benefits will be flood control and resulting decrease in economic loss function. Surface ponds and detention areas could be used simply for water detention and subsequent release to stream channels or to recharge structures, or to be developed as recreational areas. There are many benefits here--benefits of flood control, of recharge and of recreation. Recharging could be achieved by wells, shafts, pits or channels. Recreation could include water-related activities, or simply open spaces with or without ponds. Reclaimed flood water could supplement irrigation water for a park. Use of flood water for recharge of recreation would reduce demands for limited ground-water or costly surface-water reservoirs. The total management scheme would be under the constraints of total costs, nature of predictability of streamflow quality and quantity, political wisdom and legal factors. Such a plan requires comprehensive analysis.

Water quality is an important consideration under any management procedure for both recharge and recreation. Recharge water should be free of algae, sediment, turbidity, organic material, bacteria and dissolved gases to be efficient. It should be of a caliber which will not degrade the water stored in the aquifer if ground-water quality is to be maintained. Ponded water has quality restrictions depending on intended use. Water for public consumption, swimming, fishing and types of irrigation must meet legal standards.

Runoff in the Tucson metropolitan area is typically high in fecal and total coliforms, organic matter, sediment and turbidity, while low in total dissolved solids and most inorganic constituents. It may at times have a high temperature and contain varying amounts of pesticides and phenols. Methods of water-quality treatment, in a spirit consistent with urban arid hydrology, need evaluation.

This paper presents a preliminary analysis of the effectiveness of a grass and soil filter as a water-quality treatment for Tucson urban runoff. Quantification of this process's effectiveness will be useful for urban watershed management.

## METHODS

### Experimental Facilities

A grass and soil filter water-treatment pilot plant (Figure 1) was constructed by the Water Resources Research Center in the summer of 1971 on Tucson

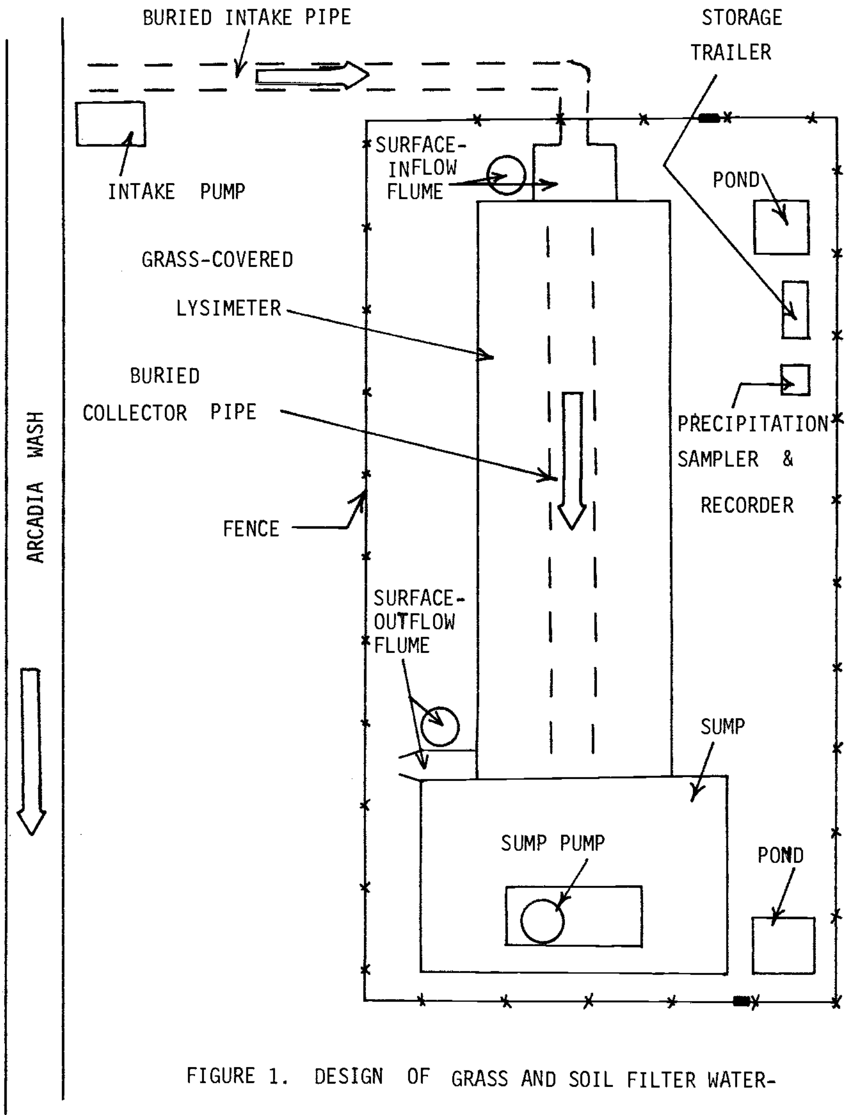


FIGURE 1. DESIGN OF GRASS AND SOIL FILTER WATER-TREATMENT PILOT PLANT NEAR ARCADIA WASH, TUCSON, ARIZONA,

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*SEE "EXPERIMENTAL FACILITIES" IN TEXT FOR DIMENSIONS.

Medical Center property in northeast Tucson, 200 feet west of ephemeral Arcadia Wash (Figure 2). The wash drains a 6-square mile urbanized basin. This site was selected because of watershed properties and potential recreational development. The pilot plant is a 200-foot long, 4-foot wide and 5-foot deep lysimeter consisting of native calcareous loam and covered with common rye grass in fall and winter and Bermuda grass in spring and summer. A one-percent slope is maintained on the top and bottom of the filter. Lysimeter sides and base, below the ground surface, are lined with sprayed asphalt and two layers of 10mm-thick black plastic sheeting, installed in strips 16 feet wide and 100 feet long. The lined base is gravel-packed around a 6-inch diameter saw-slotted polyvinyl chloride (PVC) pipeline to accept and transport seepage water to a 5-foot long, 3-foot wide and 10-foot deep concrete block sump. Walls of the filter, above the ground surface, consist of two tiers of 8 x 8 x 16 inch block, covered with plastic, chicken wire and plaster. Sump blocks and wall plaster are painted with colorundum sealer to prevent seepage.

A gasoline-powered centrifugal pump with a capacity less than a cubic foot per second is installed on the west bank of the wash to divert storm runoff through a 3-inch diameter PVC pipeline. The pipeline ends at a 3-inch Parshall flume at the surface inlet. Surface outflow from the filter is measured by a one-foot H flume at the surface outlet. Both flumes have continuous stage recorders and staff gages. Subsurface outflow from the filter is collected in a sump and measured by a staff gage and by pumping to calibrated barrels by means of a gasoline-powered submersible pump. A plastic non-recording raingage, plastic funnels for precipitation sampling, and an equipment-storage trailer are impounded by the 230-foot long, 50-foot wide and 6-foot high long chain linked fence which surrounds the pilot plant.

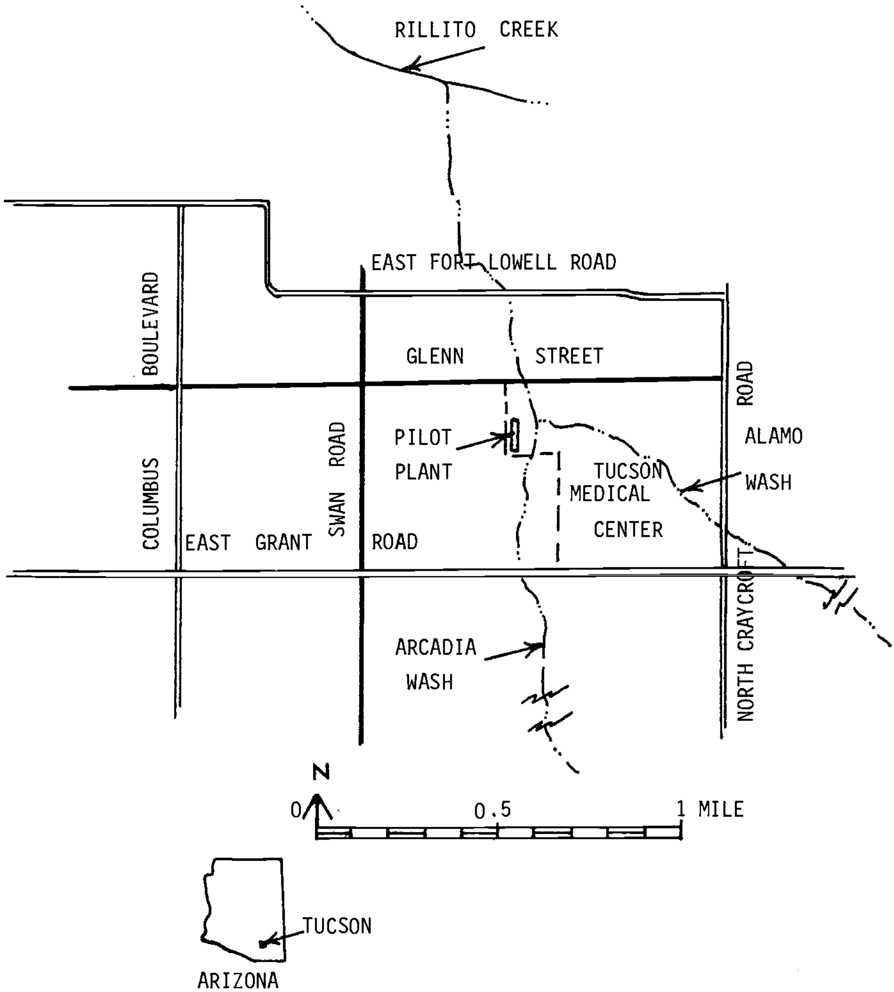


FIGURE 2. LOCATION OF GRASS AND SOIL FILTER WATER-TREATMENT PILOT PLANT IN NORTHEAST TUCSON, ARIZONA

## Data Collection

A runoff producing storm, sufficient to supply the intake pump with water, is called a trial. Four trials, each associated with a discrete runoff event, summarized in Table 1, occurred in the fall of 1971. The procedure was to pump water to the lysimeter, noting time and flume stage. Water samples and temperatures were taken from the wash, and surface- inflow and outflow flumes, and seepage discharge pipe. Subsurface outflow was measured with buckets and calibrated barrels. Precipitation was measured for all trials, and sampled for the last two trials. Grass-filtered sediment was measured for each trial.

TABLE 1: Summary of Fall 1971 Trials at Grass and Soil Filter Water-Treatment Pilot Plant

| | TRIAL 1 | TRIAL 2 | TRIAL 3 | TRIAL 4 |
|---|---|---|---|---|
| Date of flow event | Oct. 17 | Nov.15-16 | Dec.7-8 | Dec.13-14 |
| Surface-inflow volume (gal) | 18,350 | 88,000 | 16,600 | 6,900 |
| Surface-outflow volume (gal) | 16,150 | 77,200 | 14,750 | 6,200 |
| (percent of inflow) | 88 | 88 | 89 | 90 |
| Subsurface-outflow volume and abstractions (gal) | 2,200 | 10,800 | 1,600 | 700 |
| (percent of inflow) | 12 | 12 | 11 | 10 |
| Duration of surface application (min) | 303 | 1,770 | 1,125 | 195 |
| (hrs) | 5 | 30 | 19 | 3 |
| Duration of seepage (min) | 1,710 | 20,205 | 7,965 | 13,170 |
| (hrs) | 29 | 337 | 133 | 220 |
| Grass-filtered sediment (cu ft) | 20.5 | 30.0 | 3.7 | 2.8 |
| Precipitation at plant (in) | 0.9 | 0.8 | 0.8 | 0.3 |

## Data Analysis

Discharge data were calculated to within about 10 percent with the help of the Center's Wang desk computer to determine mass flow. Water samples were refrigerated to inhibit bacterial activity. Samples were selectively analyzed for chemical oxygen demand (COD), total and fecal coliform, turbidity, and volatile and suspended solids by the University of Arizona Sanitary Engineering Laboratory. Selective samples were analyzed for inorganic mineral constituents by the University's Soil and Water Testing Laboratory.

Various time series were plotted for biological and chemical data using dimensionless, arithmetic, semilogarithmic and logarithmic scales.

## RESULTS

Fourteen arithmetic and semilogarithmic graphs were used to convey information concerning turbidity, suspended and volatile solids, total and fecal coliforms, COD and total dissolved solids (TDS) for surface- inflow and outflow, and subsurface outflow samples, see Figures 3 to 16.

Turbidity graphs, Figures 3 and 4, indicate that grass and soil filtration, after a few hours, reduce turbidity into the range of precipitation water samples. This is virtually complete turbidity removal. As the grass developed, and the soil settled, treated water decreased in turbidity: that is, turbidity values were lowest for the later trials.

Suspended solids, Figures 5 and 6, similarly reduced in concentration when runoff ran through the plant. Soil filtering was more effective than grass filtering. Most of the soil-filtered samples had suspended-solid concentrations in the range of precipitation samples. Again, suspended solids decreased with grass development and soil settling.

Coliforms, Figures 7 to 10, generally decreased with succeeding trials, but may increase in the early part of any trial. Total and fecal coliforms in seepage samples, Figures 8 and 10, generally show a decrease in time

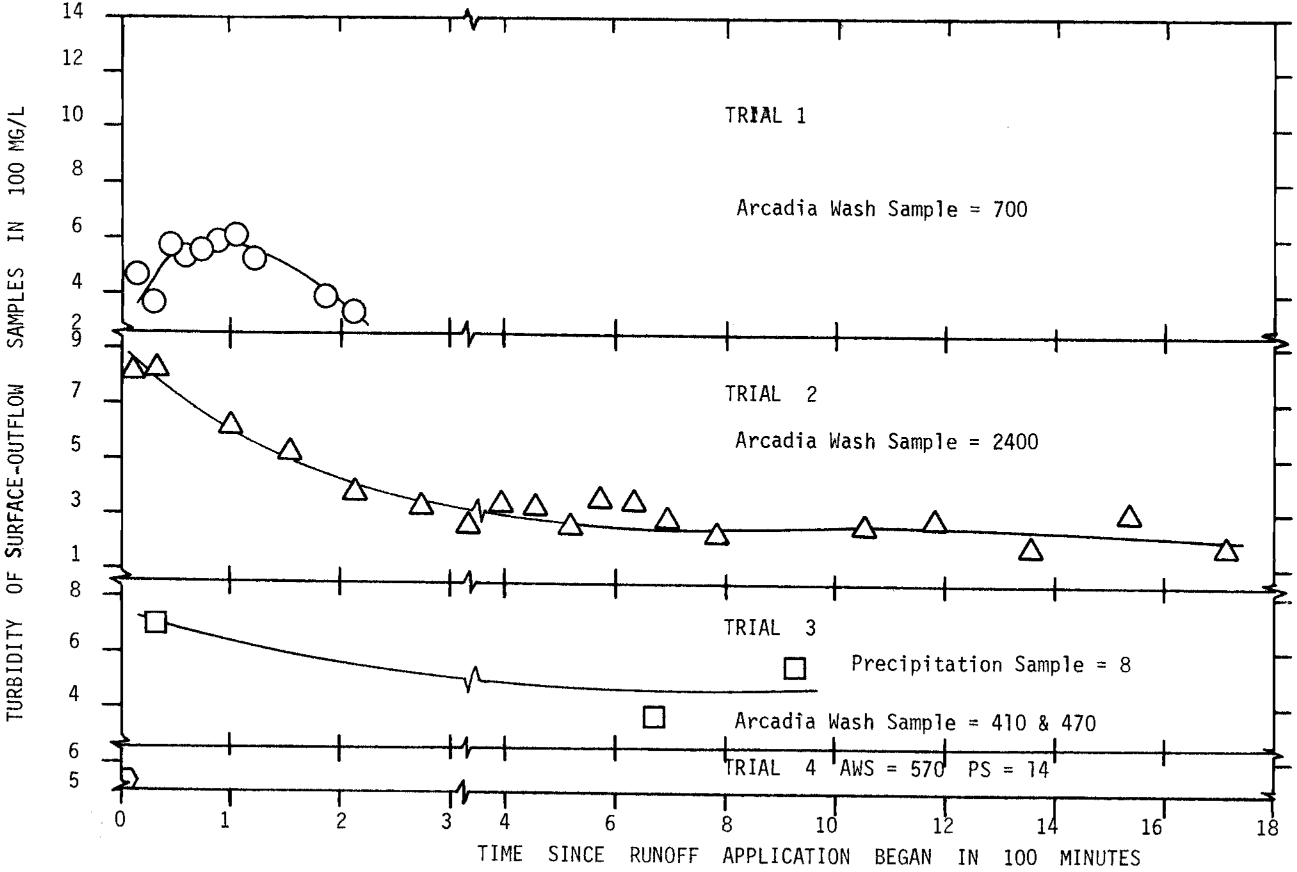


FIGURE 3. TURBIDITY SERIES OF SURFACE-OUTFLOW SAMPLES

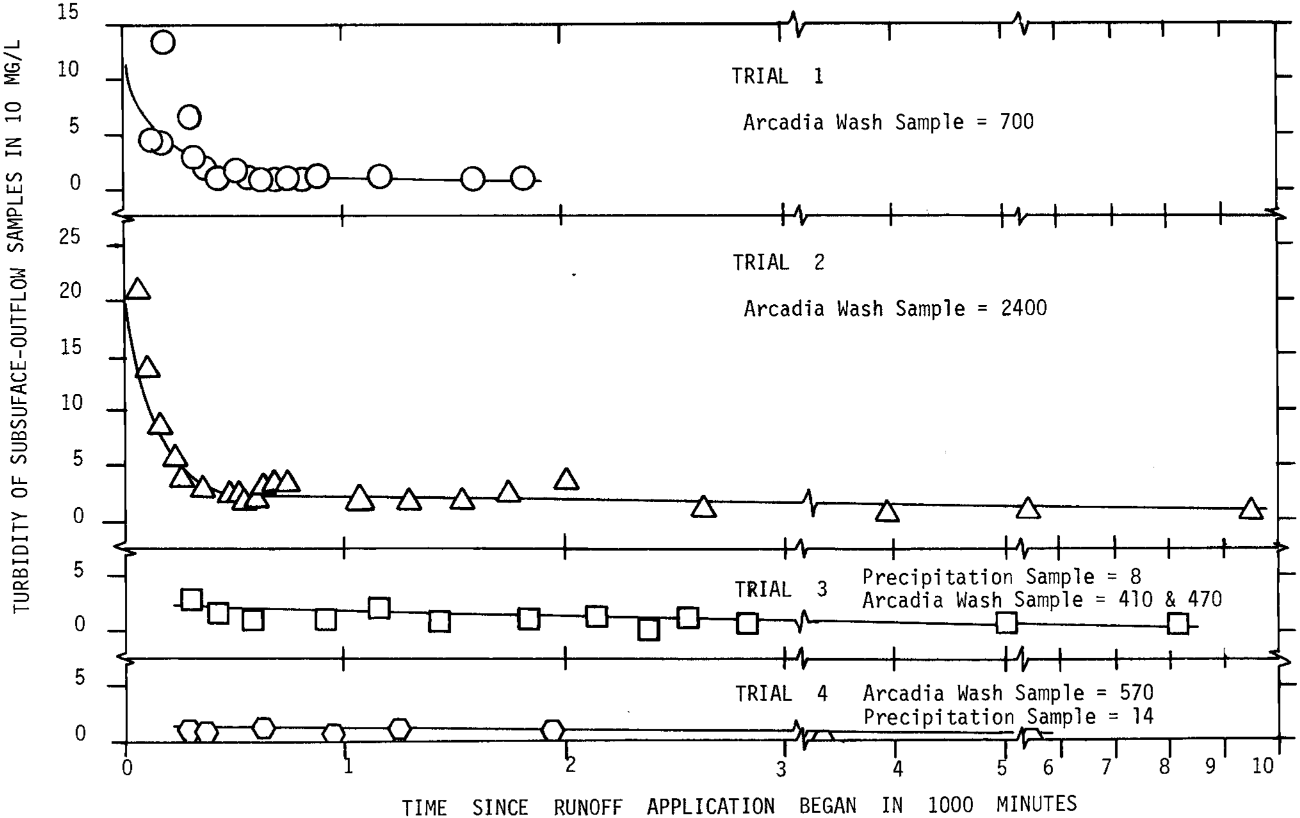


FIGURE 4. TURBIDITY SERIES OF SUBSURFACE-OUTFLOW SAMPLES

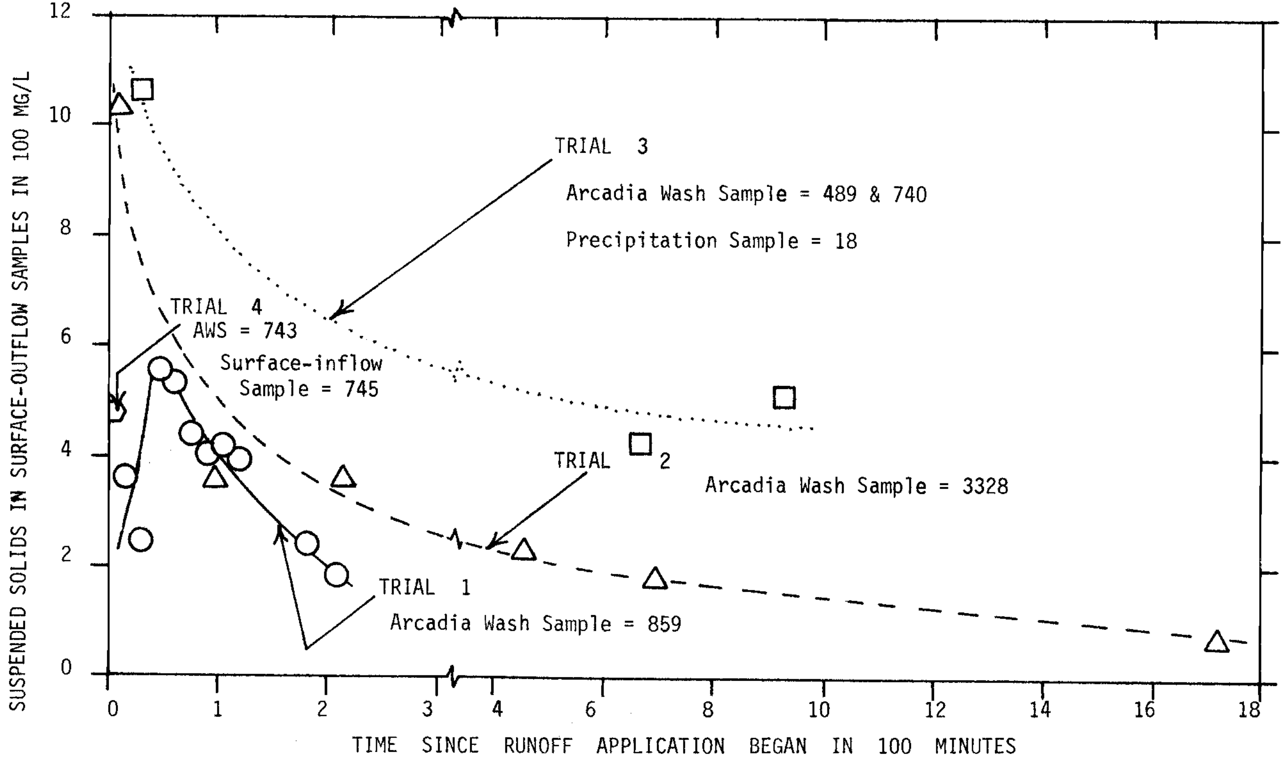


FIGURE 5. SUSPENDED-SOLIDS SERIES IN SURFACE-OUTFLOW SAMPLES

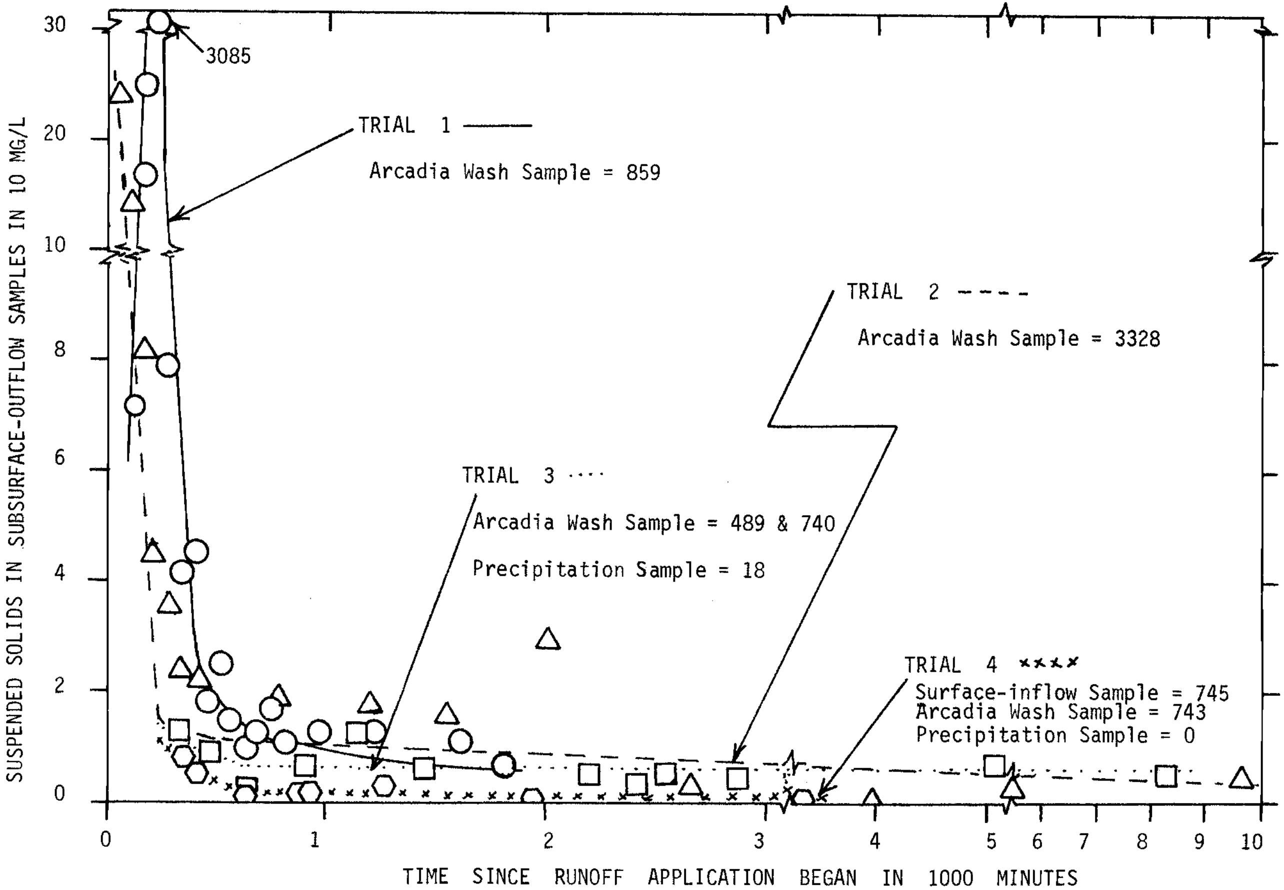


FIGURE 6. SUSPENDED-SOLIDS SERIES IN SUBSURFACE-OUTFLOW SAMPLES

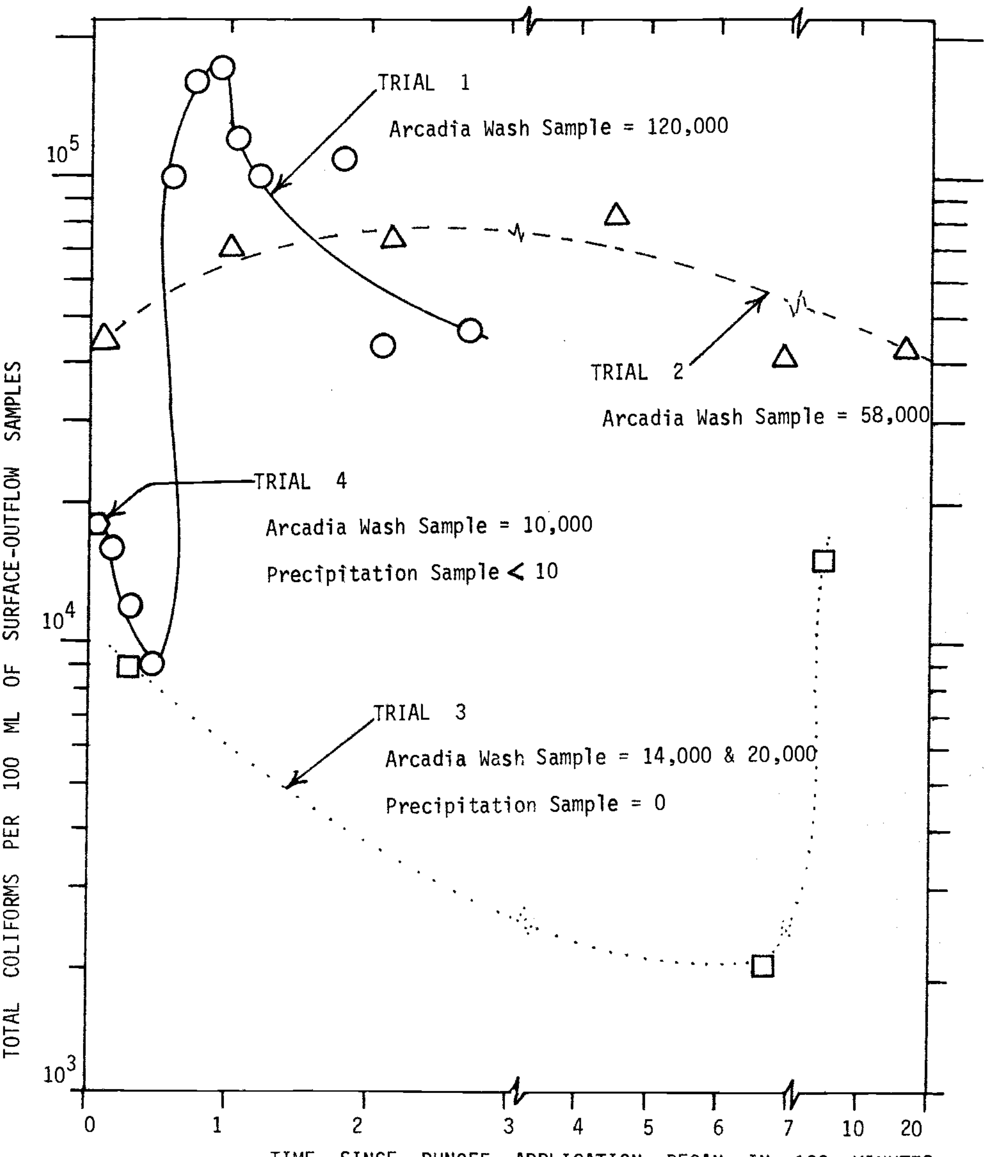


FIGURE 7. TOTAL-COLIFORM SERIES OF SURFACE-OUTFLOW SAMPLES

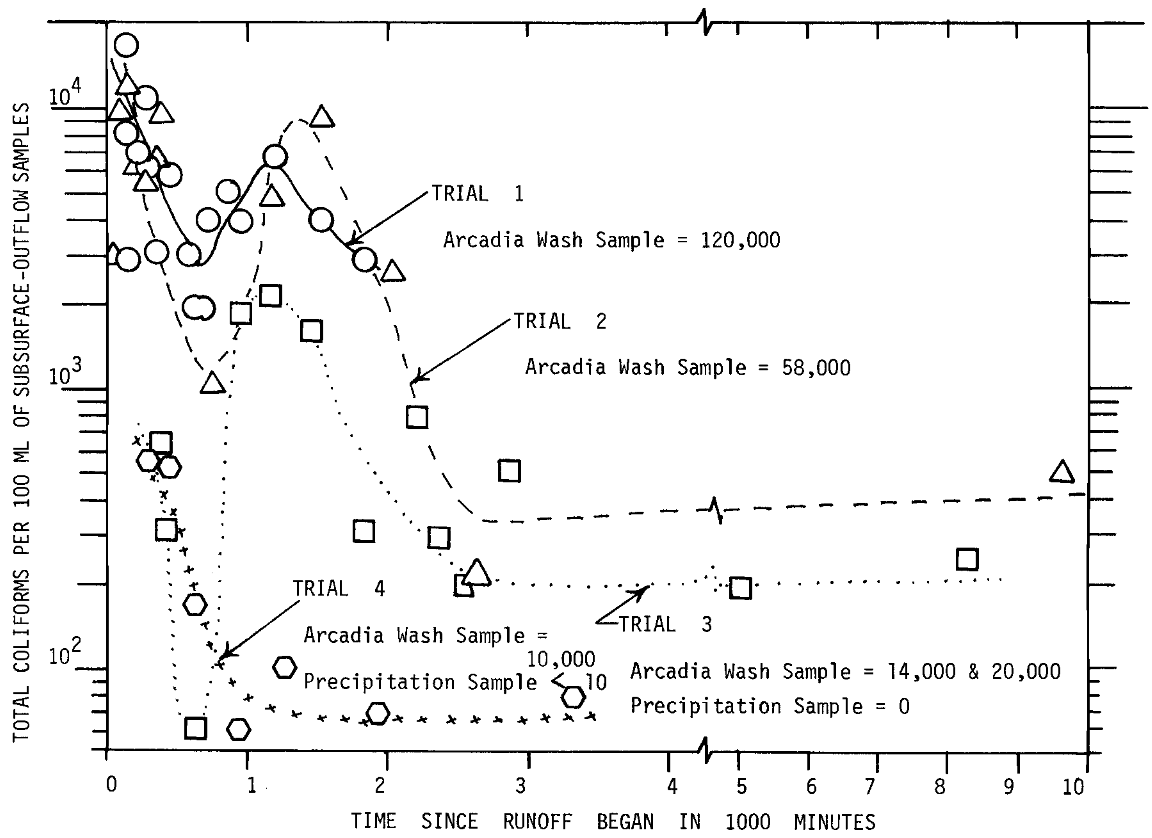


FIGURE 8. TOTAL-COLIFORMS SERIES OF SUBSURFACE-OUTFLOW SAMPLES

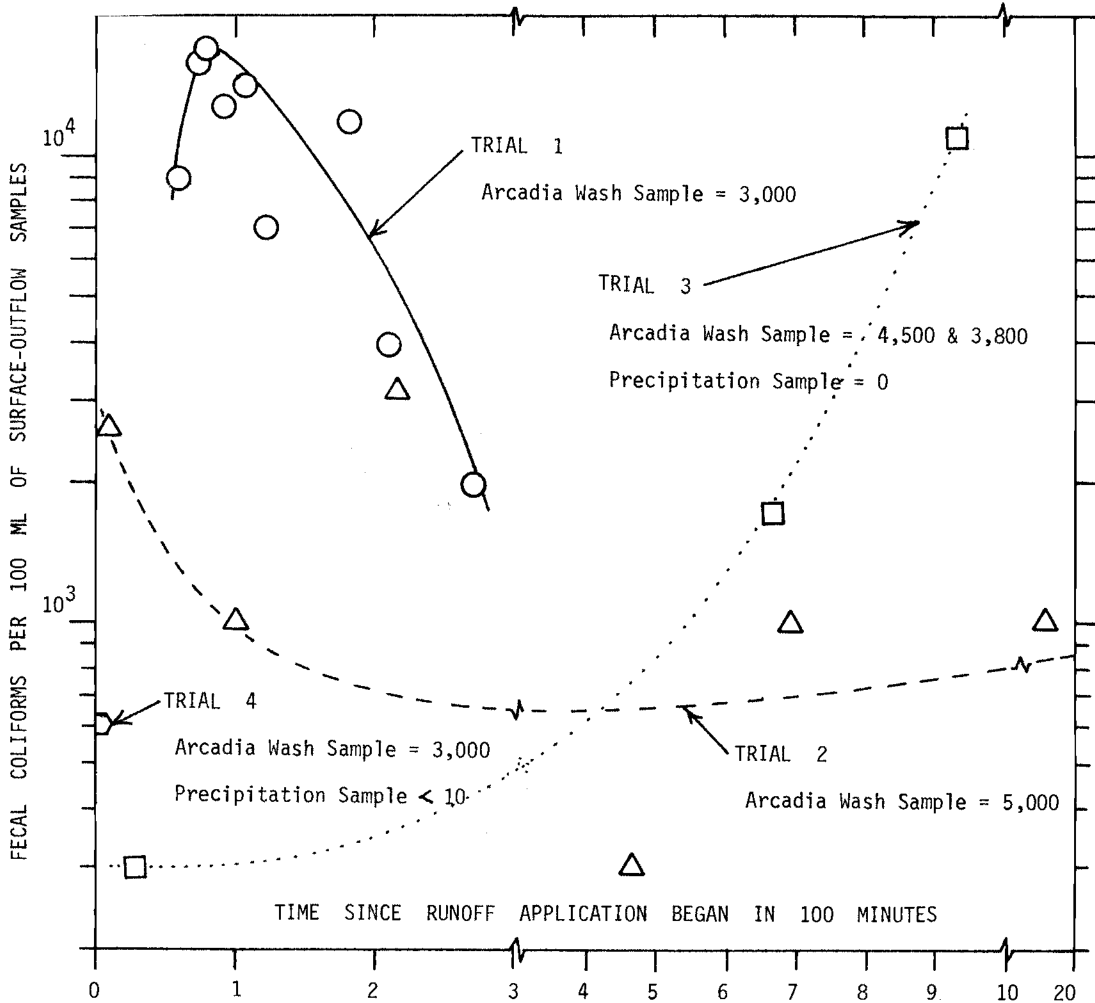


FIGURE 9. FECAL-COLIFORMS SERIES OF SURFACE-OUTFLOW SAMPLES

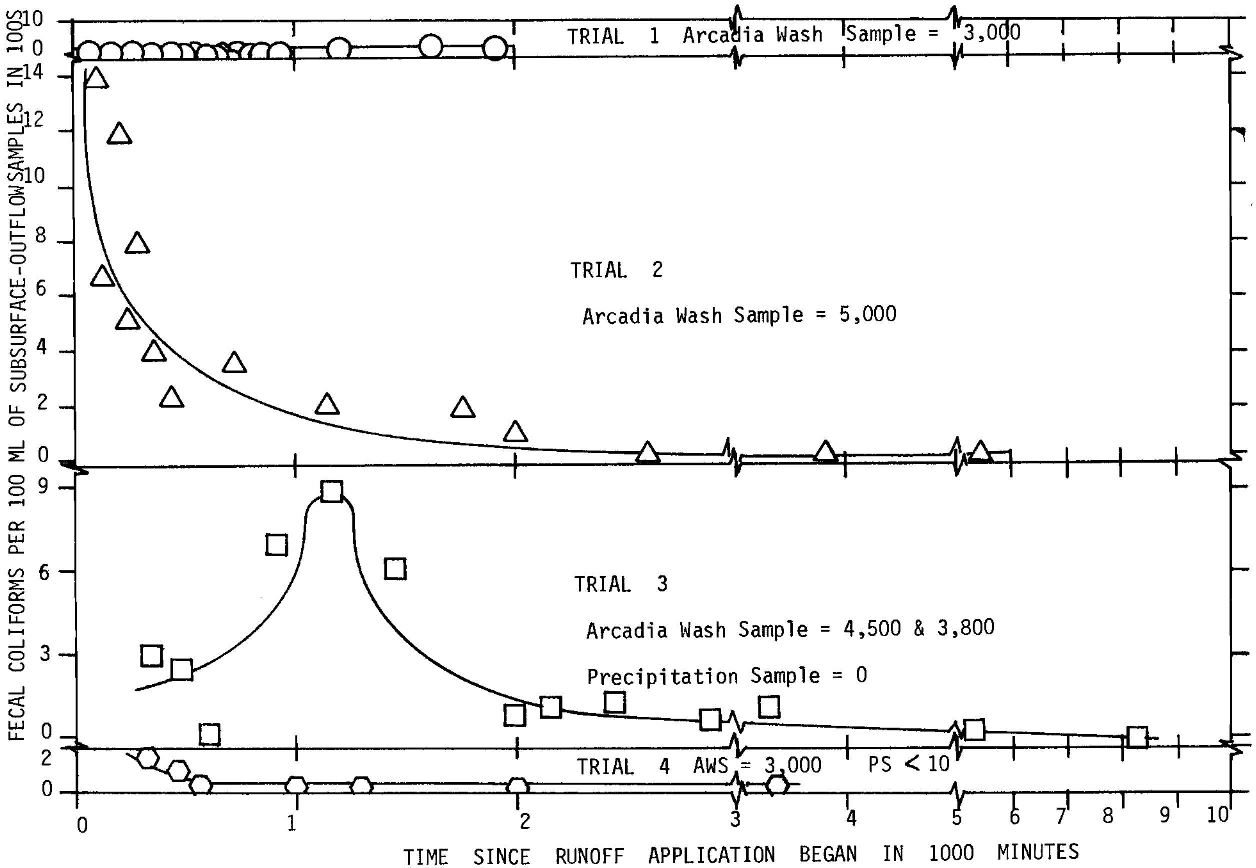


FIGURE 10. FECAL-COLIFORMS SERIES OF SUBSURFACE-OUTFLOW SAMPLES

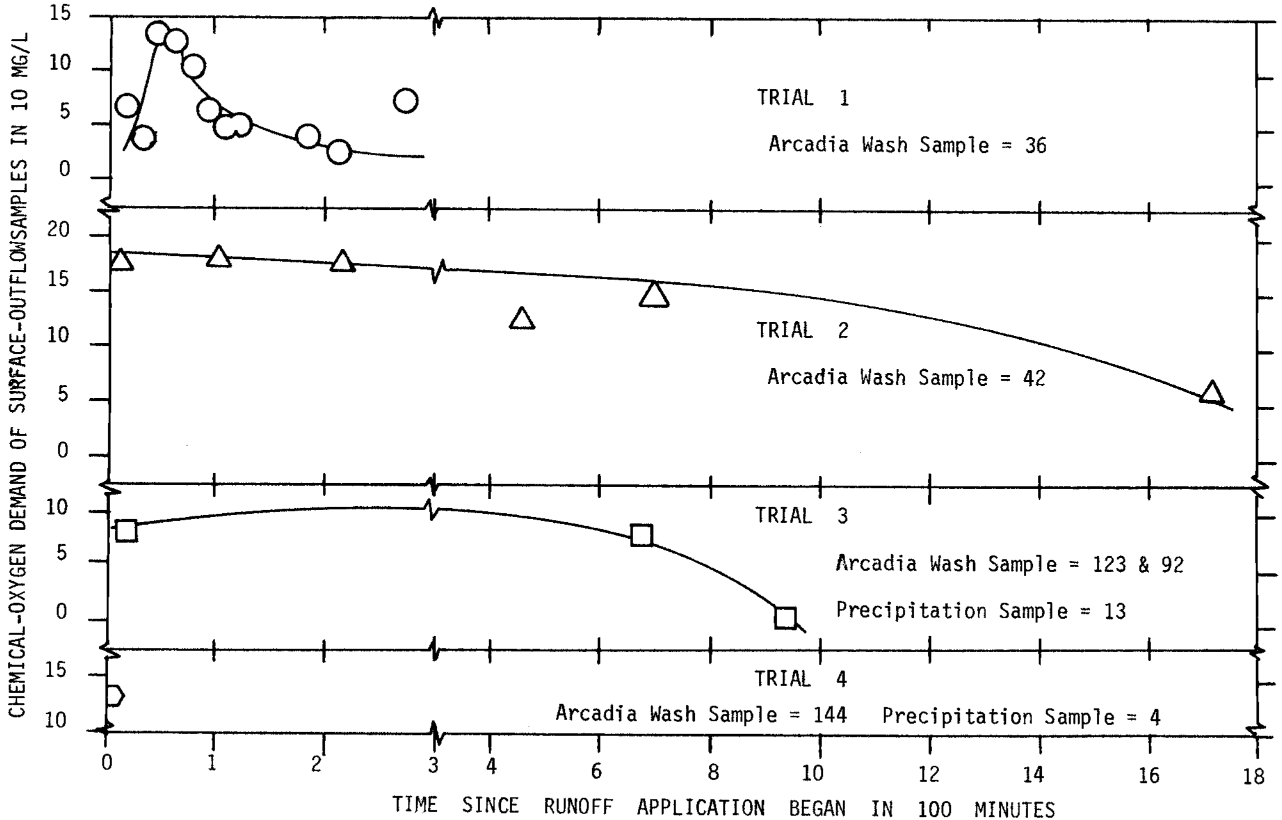


FIGURE 11. CHEMICAL-OXYGEN DEMAND SERIES OF SURFACE-OUTFLOW SAMPLES

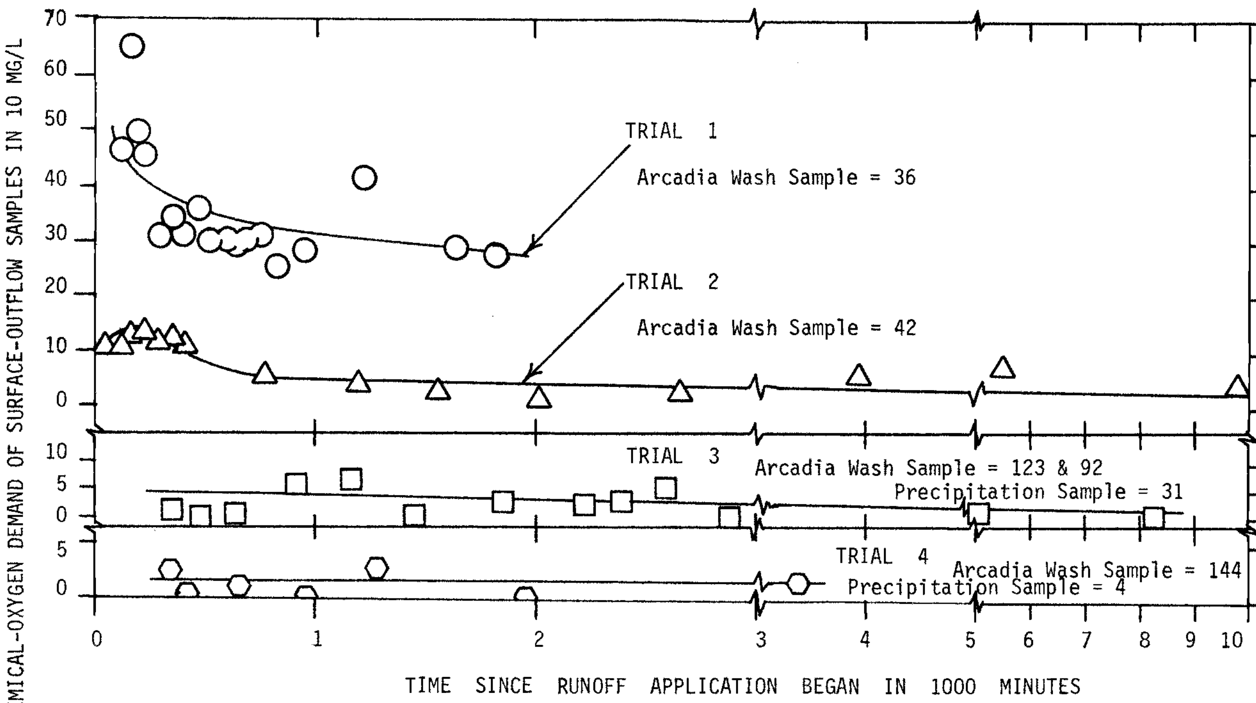


FIGURE 12. CHEMICAL-OXYGEN-DEMAND SERIES OF SUBSURFACE-OUTFLOW SAMPLES

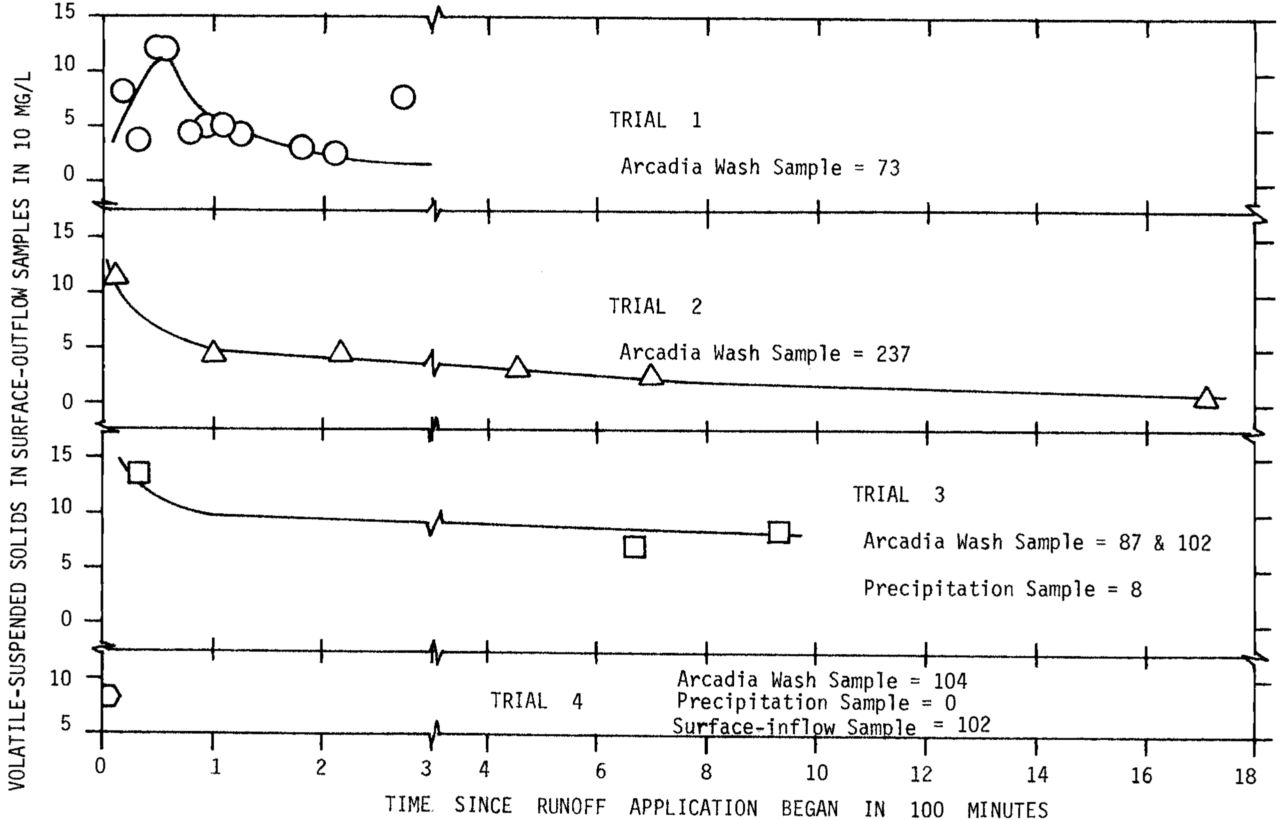


FIGURE 13. VOLATILE-SUSPENDED-SOLIDS SERIES OF SURFACE-OUTFLOW SAMPLES

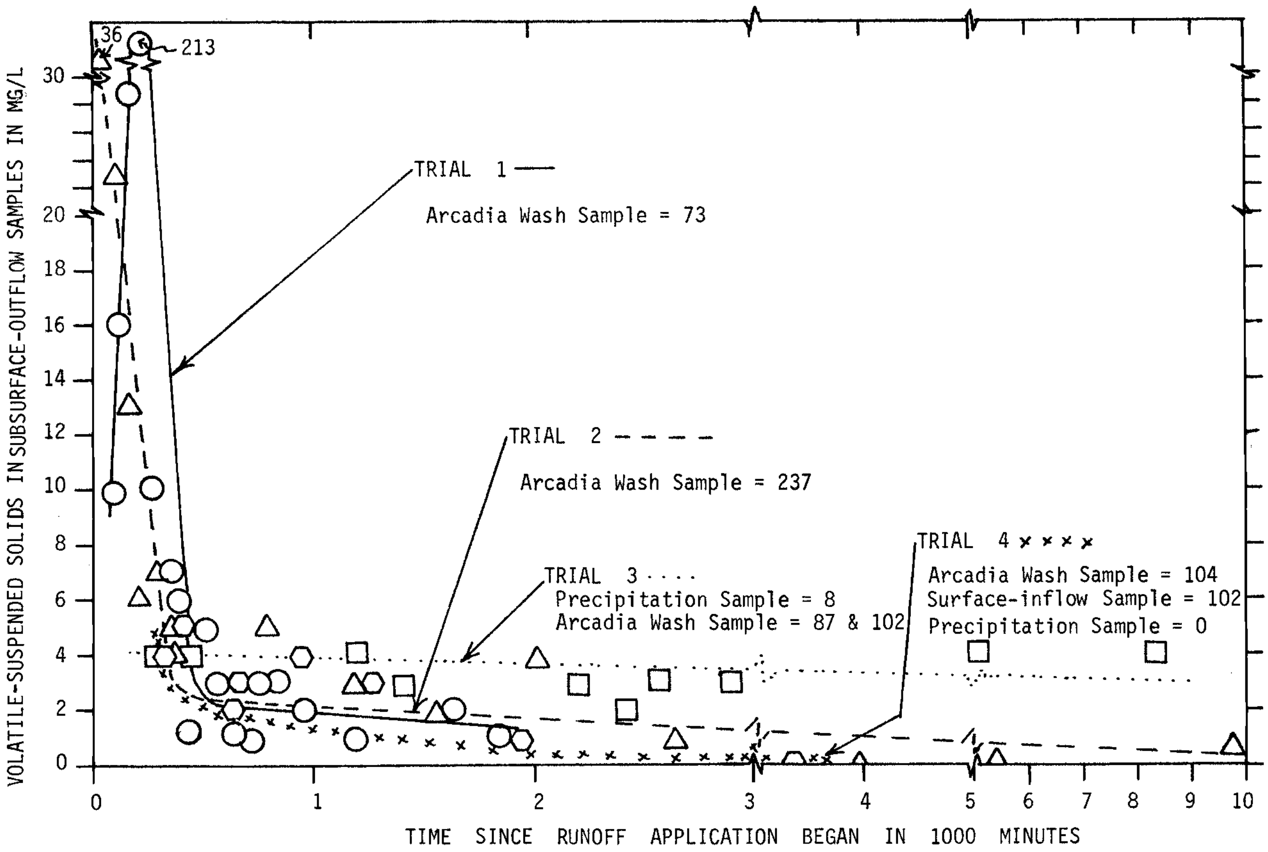


FIGURE 14. VOLATILE-SUSPENDED-SOLIDS SERIES OF SUBSURFACE-OUTFLOWSAMPLES

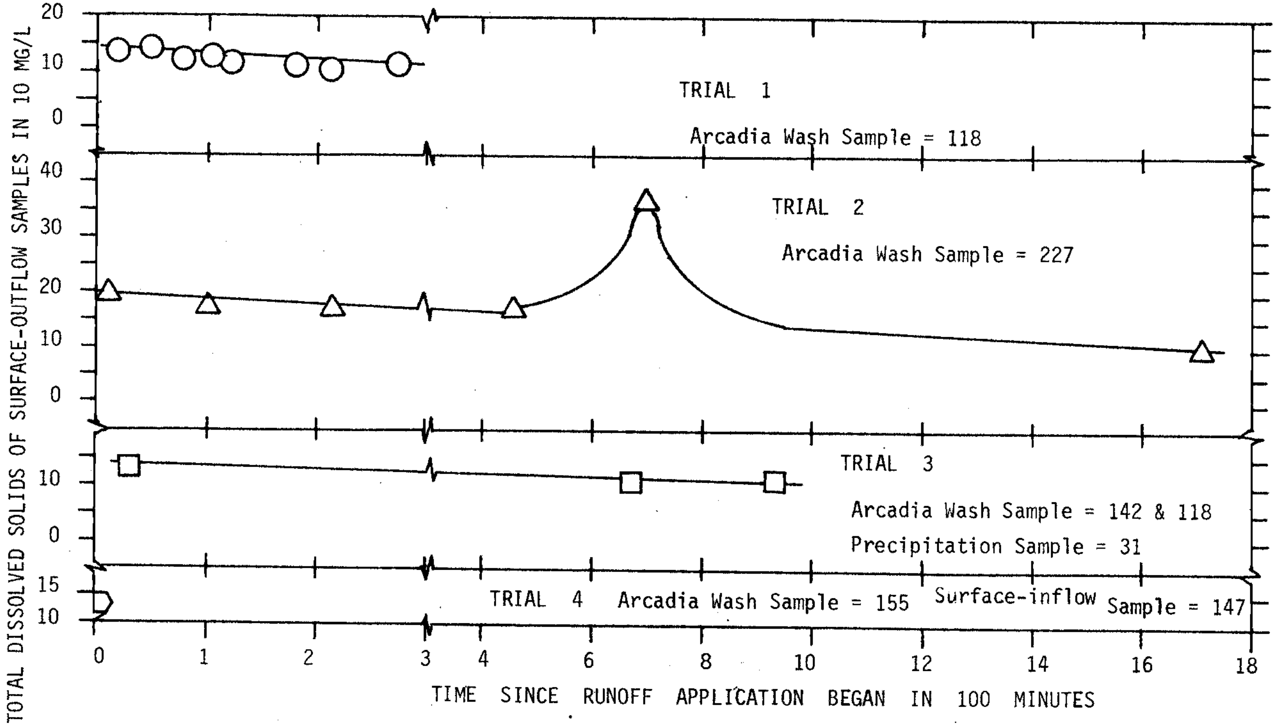


FIGURE 15. TOTAL-DISSOLVED-SOLIDS SERIES OF SURFACE-OUTFLOW SAMPLES

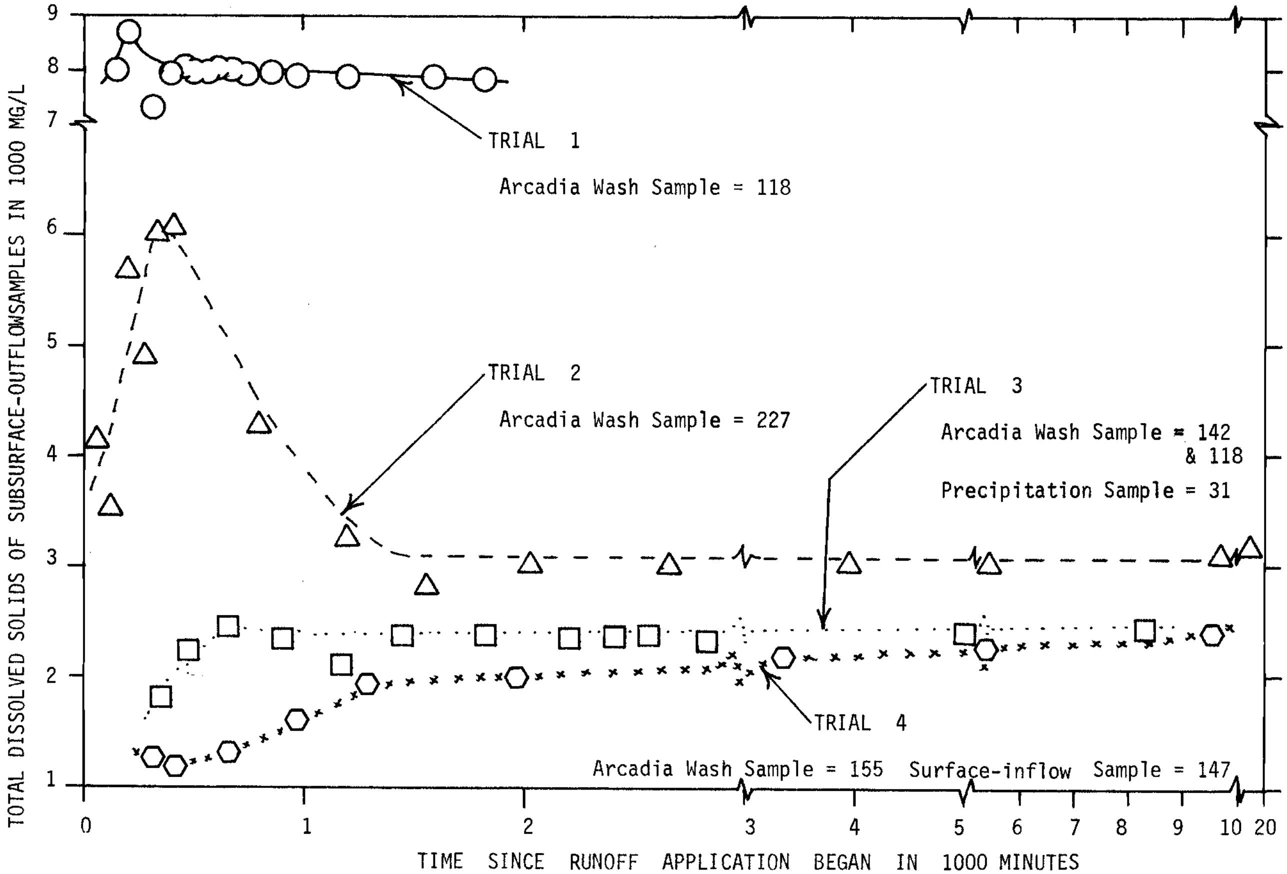


FIGURE 16. TOTAL-DISSOLVED-SOLIDS SERIES OF SUBSURFACE-OUTFLOW SAMPLES

with each trial and from early to later trials. Bacterial analyses of surface-outflow samples, Figures 7 and 9, produced more scattered results. Increasing grass and soil establishment again seemed to be more effective in bacterial removal from runoff.

Chemical-oxygen demand, Figures 11 and 12, follows the general pattern of reduction in both grass and soil filtering, with increased filtering in the later trials. This pattern also holds for volatile suspended soilids, Figures 13 and 14. The volatiles, however, initially increased at the early part of the first trial.

Total dissolved solids, Figures 15 and 16, remain relatively constant, and slightly below the inflow value, in each trial run over grass. Subsurface-outflow data show very high salt concentrations, diminishing with later trials. The first two seepage trials show early increases, followed by decreases in salts, while the last two trials show later increases in salts. The trend in dissolved solids content in seepage water decreases with later trials.

## SOURCES OF ERRORS

Processes at work in this experiment are very dynamic. Absorption, adsorption, diffusion, dispersion, hydrolosis, ion exchange, migration, miscible displacement, nitrification and solution are kinetic phenomena which vary in time and space, and according to flow rates, soil-water potential and temperature. This measuring and sampling regime probably is not sensitive to all the physio-chemical activity at play. The following examples apply:

1) Likely initial increases in TDS and COD may not appear in the surface-outflow samples because of timing of data collection.
2) Leaching effects are most pronounced with unsaturated flow regimes

of small seepage water volumes.

3) Effects of temperature variations in the lysimeter are lumped in bacterial and COD concentrations of all outflow samples.

## INTERPRETATION

Turbidity, suspended and volatile solids, coliforms and COD in runoff samples may be reduced by grass and soil filtration. These water-quality parameters are related, as bacteria may fall out with solids and may comprise some of the organics showing up in COD. The initial increase in coliforms in the early part of the trials over grass probably reflects a grass washing, as high TDS in seepage samples likely represents leaching of soluble minerals. The collection of more water quality data is requisite to concluding that soil-runoff equilibrium is achieved. Increased grass development and soil settling works to produce a better quality effluent.

## FUTURE STUDIES

Small ponds were constructed in spring 1972 to hold treated and untreated runoff for observation of biological activity. These ponds may be stocked with algae- or mosquito-eating fish over the summer. A recording rain gage was installed.

Summer runoff events will be diverted to the treatment facility to produce more trials. Bermuda grass will replace rye for the summer. A more detailed analysis will be made of basic data, with a view toward quantifying relationships and identifying biological, chemical and physical soil-water and plant-water processes.

Possibly unsaturated soil-water samples and more spatially distributed samples will be collected by suction cups to delineate zones of water-quality changes, and to evaluate some models of water-quality dynamics through soils.

Pesticide, phenol and trace-element analysis may be performed.

Quantified effects of the water-treatment pilot plant will be used in evaluating the feasibility of the urban runoff management scheme suggested. Perhaps other treatments will be studied. Architectural and legal aspects of constructing a recreational facility on this site are being investigated. Efficiency of treatment will be evaluated by examining water-quality improvement versus unit cost of treatment.

## ACKNOWLEDGMENTS

Gratitude is expressed to the following: Mr. Blencowe Daniel and the Directors of the Tucson Medical Center for use of their land; Prof. C. Brent Cluff, Drs. Kenneth J. DeCook and Lorne G. Wilson, and the late Al Hurley for early planning and design; Ms. Susan H. Kuyper for personal support; Messrs. Dennis Scheall, William J. Staggs, their assistants and other friends for construction and maintenance; John and Mary Falk for irrigation supply; Prof. Sol D. Resnick and Drs. Robert A. Phillips and Eugene S. Simpson for constructive counseling; and Mr. and Mrs. Walter Popkin for personal encouragement.

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## RELEVANT LITERATURE

Procedures for managing urban runoff in arid lands are discussed by University of Arizona and U.S. Geological Survey (1959), Resnick and DeCook (1970), Mische (1971) and Terlizzi (1972). These studies recognize the influence of water quality on utilization of captured runoff.

Grass filtration for sediment removal is suggested by Wilson and Lehman (1966), Wilson (1967), Lehman (1968) and Wilson (1971). Soil filtration for bacterial removal is suggested by Stone (1953), Bocko (1965) and Merrell and others (1965).

Babcock (1963) synthesized the principles of chemical transfer of waters in contact with soil colloidal systems at equilibrium. Dutt and others (1972) developed computer simulation models of dynamic bio-physiochemical processes in soils.

Some other related work includes Jorden (1962), McCarty (1965), Marsh (1968), Atala (1969), Boyd and others (1969), DeCook (1970), Dharmadhikari (1970) and Mishe and Dharmadhikari (1970).

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# NITROGEN SPECIES TRANSFORMATIONS OF SEWAGE EFFLUENT RELEASES IN A DESERT STREAM CHANNEL

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Abstract. A preliminary study was made with the objective of examining nitrogen species transformations of treated sewage effluent releases within the channel of an ephemeral stream, the Santa Cruz River of southern Arizona. Water quality samples were taken at established locations in sequence so that peak daily flows could be traced as the effluent moved downstream. Results indicate that increased nitrification, coinciding with changing stream characteristics, starts in the vicinity of Cortaro Road (6.3 river miles from the Tucson Sewage Treatment Plant discharge). Through physical-chemical changes in streamflow, nitrate-nitrogen values reach a maximum at approximately 90-95 percent and 60-80 percent of total flow distance for low flows and high flows, respectively. Concentrations of ammonia-nitrogen and total nitrogen decrease continuously downstream with both high and low flows. Therefore, the rate of nitrification within sewage effluent releases in a desert stream channel evidently is related to flow distance and physical characteristics of the stream.

## INTRODUCTION

Treated sewage effluent from the Tucson metropolitan area has been utilized as a source of irrigation water for farmlands near the City sewage treatment plant. With rapid growth of the urbanized area, the rate of production of effluent has grown to an average flow of more than thirty million gallons per day (mgd), far exceeding the irrigation requirement of the local farms; consequently, increasing quantities of the effluent have been released to the channel of the Santa Cruz River, an ephemeral stream. Studies are currently in progress on technical, economic, and legal aspects of allocation or exchange of the bulk of the effluent to more extensive farmlands in Avra Valley (Cluff et al., 1971).

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Pending further studies and decisions by local water agencies to implement this or alternative uses of the effluent, releases to the river channel will continue. It appears that these sewage effluent flows may be a contributing factor to the excessive nitrate concentration in the ground-water supply northwest of Tucson. This preliminary study was proposed to obtain information about flow characteristics and the nature of nitrogen species transformations occurring in surface effluent flow. Ultimately, this initial information would provide basic data for an overall nitrogen balance survey of the lower Santa Cruz groundwater basin.

## PROCEDURE

Records of the City of Tucson Sewage Treatment Plant were examined for information concerning quantities and quality characteristics of the treated effluent released at the plant. In order to trace downstream reduction of flow by infiltration and other losses, a four-foot H-L flume was installed in 1971 at Rillito Narrows near the Avra Valley Road crossing, approximately eleven miles downstream from the plant. Supplemental measurements and estimates of flow were made at other locations, and records of flow measured by the U. S. Geological Survey at Cortaro Road, six miles downstream, also were utilized.

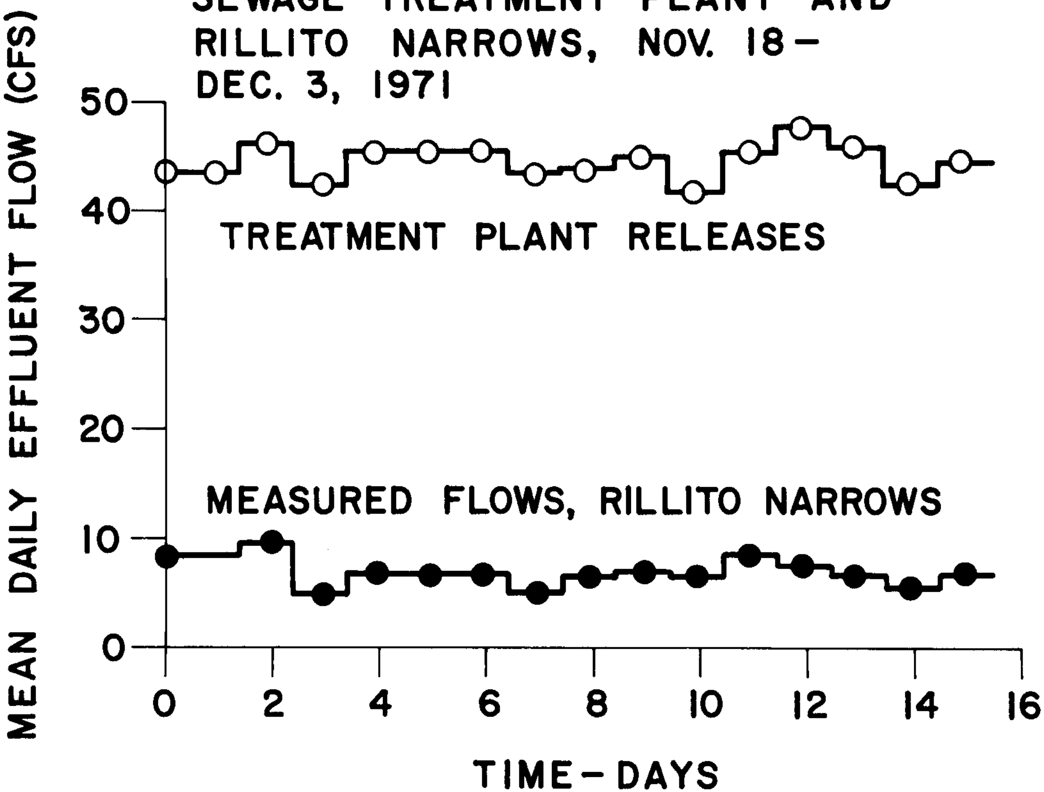
It was noted that a large diurnal fluctuation of influent sewage flows into Tucson's sewage plant was reflected in a large daily advance and recession of the wetted front of effluent in the Santa Cruz River channel, whereby the surface flow was depleted and disappeared at a flow distance of about eleven miles at low ebb. However, at peak flow the effluent extended down river from the plant approximately twenty-five miles. As a means of tracing nitrogen species transformations in an increment of flow, the peak was followed and sampled intermittently as it moved downstream. Likewise, the point of low flow from the plant was followed and sampled at the same locations, for as far as flow extended. At these collection points, one-liter samples of the flow were taken and measurements on water temperature, dissolved oxygen, and pH were made. The samples were analyzed in the laboratory for content of total dissolved solids, nitrate-nitrogen, ammonia-nitrogen, organic nitrogen, and total nitrogen.

## RESULTS AND DISCUSSION

In Figure 1, channel loss between the Tucson Sewage Treatment Plant and Rillito Narrows, approximately eleven miles downstream, from November 18 to December 3, 1971, is shown with a resulting water loss of approximately 85 percent at an average infiltration rate of 3.5 cubic feet per second (cfs) per mile. These measurements were taken following a flood flow in the Santa Cruz River with a peak of 500 cfs as measured at Cortaro Road, U.S.G.S. gaging station.

More recently, with construction of an additional four-foot H-L flume near Cortaro Road, measurements on effluent flows before and after a summer flood with a peak of approximately 3,000 cfs indicate that significant changes in infiltration rates occur following flood flows. As shown in Table 1, this summer flood flow changed only the infiltration rate of the stream section between the City sewage treatment plant and Cortaro Road (and therefore between the plant and Rillito Narrows). No change is seen in the stream section between Cortaro Road and Rillito Narrows. Matlock (1966) recorded an average infiltration rate of three cfs/mile for the reach between the sewage treatment plant and Cortaro Road. His measurements were made in the fall of 1964, about six weeks after a major flow event. As indicated by the preliminary data and suggested by Matlock (1966), occasional flood flows increase infiltration rates and the recharge

FIG. 1. CHANNEL LOSSES BETWEEN TUCSON SEWAGE TREATMENT PLANT AND RILLITO NARROWS, NOV. 18 – DEC. 3, 1971



condition by scouring and moving deposited materials in the channel.

TABLE 1. EFFLUENT INFILTRATION RATES FOR VARIOUS STREAM SECTIONS BEFORE AND AFTER A SUMMER STORM

| | Stream Sections | | |
|---|---|---|---|
| | Treatment Plant to Cortaro Road | Cortaro Road to Rillito Narrows | Treatment Plant to Rillito Narrows |
| Before | 2.2 cfs/mile | 3.4 cfs/mile | 2.7 cfs/mile |
| After | 3.1 cfs/mile | 3.4 cfs/mile | 3.2 cfs/mile |

The change in values of the various nitrogen species tested, as a function of flow distance, is illustrated in four graphs (Figures 2 through 5), which represent sampling of high and low flow in both summer and winter. It is observed that a decrease in content of total nitrogen occurs fairly consistently with flow distance. Changes in measured organic nitrogen content are relatively minor. Quite significant, on the other hand, is the pronounced and persistent decrease in ammonia-nitrogen in the downstream direction; correspondingly, there is a gradual increase in nitrate-nitrogen, notably below the Cortaro Road crossing where aeration apparently is enhanced.

Nitrogen transformations of low flow during June (Figure 2) seem to indicate that some type of ammonia loss as well as associated total nitrogen loss did occur in the first three miles. There are five possible causes for the ammonia-nitrogen decrease as suggested by Bower and Lance (1971): (1) volatilization of ammonia; (2) adsorption by the cation exchange complex; (3) fixation by clays; (4) fixation by the organic fraction in the soil; and (5) incorporation into microbial cell tissue. Although arguments can be made for the various causes of this ammonia loss, laboratory experiments conducted on effluent samples in the current study seem to indicate that the ammonia loss is due to adsorption by the cation exchange complex and fixation by clays and organic particles. Some ammonia volatilization also may have occurred. Preul and Schroepfer (1965) suggested that physical adsorption in soil is an important mechanism inhibiting the travel of nitrogen, as in the form of ammonium ion. More recently, Shaffer (1972) indicated that a saturated sandy soil can have more than 25 percent of the total ammonia-nitrogen as an exchangeable form. However, additional experiments must be completed, so that the cause of initial ammonia-nitrogen loss can be determined.

In Figure 2, low summer flow samples also show that total nitrogen content increases slightly downstream, indicating that residual nitrogen is being assimilated into the water system due to the decreased water velocity and increased contact time. Similar results were found by Preul and Schroepfer (1965), for soils in which lower rates of water movement enhanced the physical adsorption process with respect to ammonia. In addition, results show that nitrification in the river proceeds rapidly beyond Cortaro Road, as indicated by the reduction of ammonia and the corresponding increase of nitrate-nitrogen as flow distance increases. Nitrate-nitrogen attains its highest value within one mile from maximum reach of low flow. Therefore, the rate of nitrification is associated with the increased surface area, the increased infiltration, and the increased oxygen transfer related to reduced flow in a widening channel.

In Figure 3, high summer flows show an increase in nitrification at Cortaro Road (6.3 river miles) which corresponds with channel widening. The

FIG. 2. NITROGEN TRANSFORMATIONS IN EFFLUENT, SANTA CRUZ RIVER DURING LOW FLOW, JUNE 23, 1971

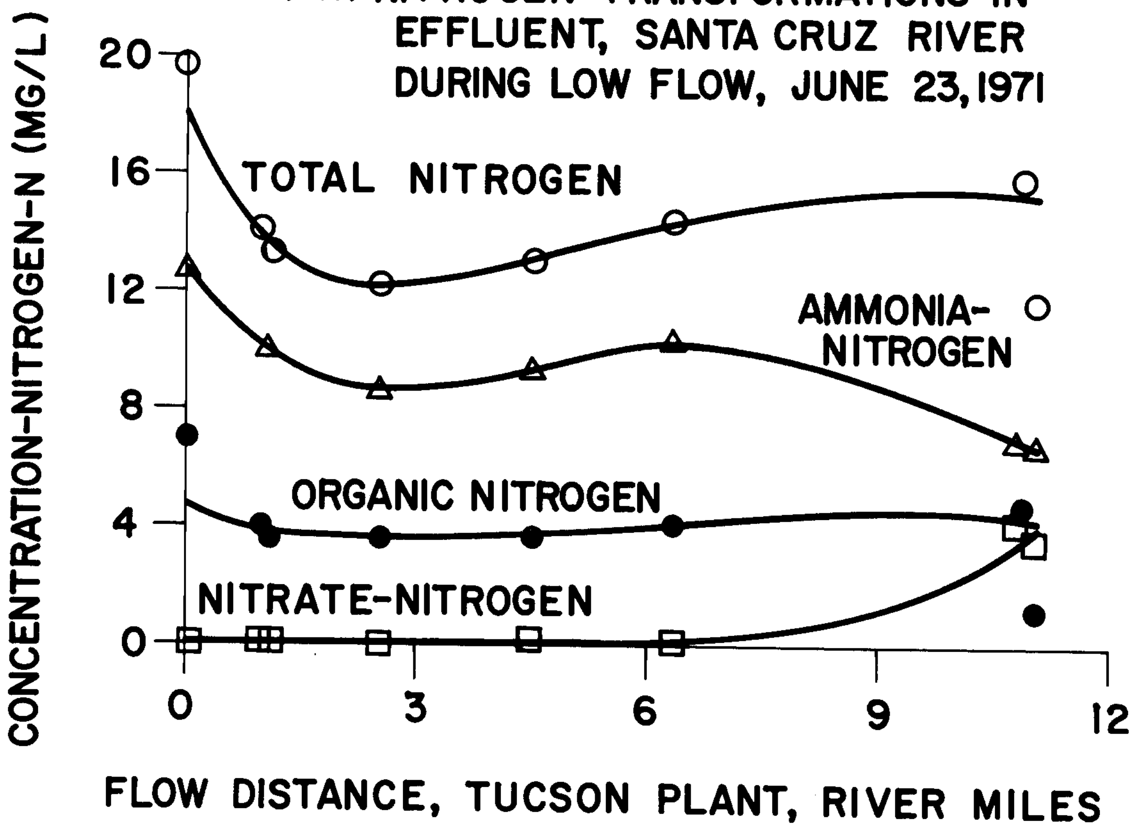
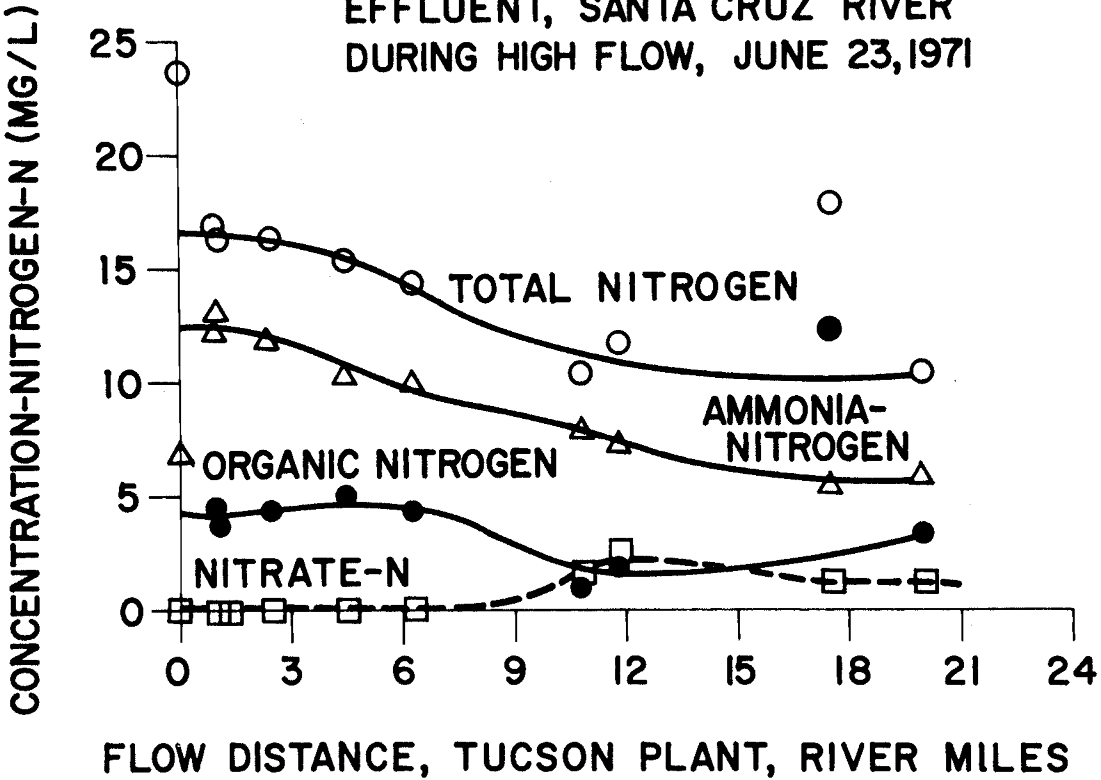


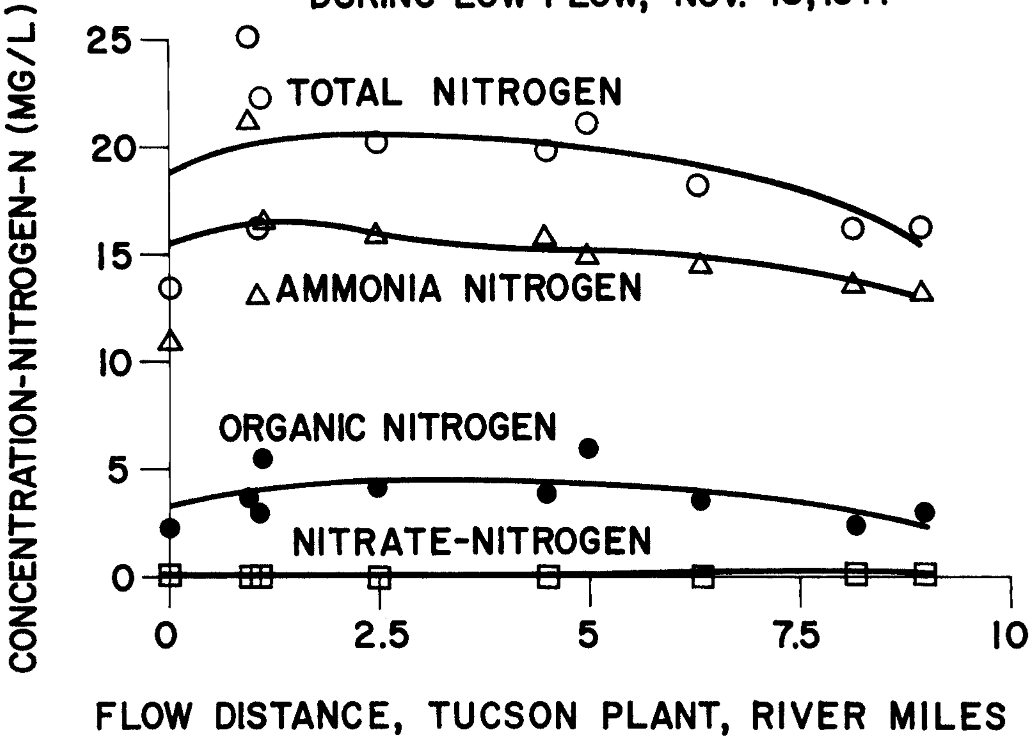
FIG. 3. NITROGEN TRANSFORMATIONS IN EFFLUENT, SANTA CRUZ RIVER DURING HIGH FLOW, JUNE 23, 1971



maximum value of nitrate-nitrogen occurred at Rillito Narrows or approximately two-thirds of the total flow distance. In addition, it appears that total nitrogen content decreases rapidly with distance, indicating that denitrification and some ammonia volatilization occur, although the total nitrogen loss may also be associated with solids removal and fixation by clay and organic particles. It is hypothesized that the denitrification process may occur in the upper soil-water interphase layer in which a three-inch-thick, black, odoriferous layer exists, which is assumed to be anaerobic as found in many sewage reservoirs. Laverty et al. (1961) also found that a similar anaerobic layer reduced flow by 88 percent in seven weeks; this buildup of an illuviated layer may be the major factor affecting infiltration rates. However, within the flowing body of water nitrification does occur, as shown by the measurements, since aeration and turbulence do exist. Nommik (1965) found that nitrification and denitrification can occur concurrently in the same profile in an irrigated field. Therefore, it seems that both nitrification and denitrification occur simultaneously within the same stream-streambed profile.

In Figure 4, low winter flows show a slight decrease in total nitrogen content which is probably due to plant use, solids removal, fixation by clay and organic particles, and some denitrification in the lower soil-water zone. Yet, nitrification does not seem to take place at a significant rate, since only a small increase in nitrate-nitrogen occurred - approximately one mile from the maximum reach of low flow. Lower winter temperatures affecting the microbial reaction rates, as well as reduction in effluent flow, may have had significant effect on both nitrification and denitrification. No significant trends in nitrogen transformation were found in total nitrogen or ammonia-nitrogen

FIG. 4. NITROGEN TRANSFORMATIONS IN EFFLUENT, SANTA CRUZ RIVER DURING LOW FLOW, NOV. 18, 1971



concentrations, both of which had low linear correlation coefficients with distance.

In Figure 5, high winter flows show the characteristic nitrification at Cortaro Road and total-nitrogen loss as high-flow peak proceeds downstream. Nitrogen loss is attributed as before to denitrification, solids removal, fixation of ammonia by clay and organic particles, and some ammonia volatilization. Maximum nitrate-nitrogen concentration occurred at approximately three-fourths of the total flow distance.

## CONCLUSION

It can be concluded that the rate of nitrification in sewage effluent releases is related to flow distance and physical characteristics of the stream, with nitrification and denitrification processes evidently occurring in the moving effluent stream. Nitrogen loss processes such as solids removal, fixation of ammonia by clay and organic particles, and ammonia volatilization in sewage effluent releases to the Santa Cruz River need to be further investigated. Once nitrogen loss mechanisms are known, it may be economically feasible to manage the sewage effluent regime to stimulate increased nitrogen loss, thereby reducing the nitrate-nitrogen concentrations in the lower Santa Cruz Groundwater Basin.

## ACKNOWLEDGMENTS

The work upon which this paper is based was supported by funds provided by the United States Department of the Interior, Office of Water Resources Research, as authorized under the Water Resources Research Act of 1964.

The assistance of Professor Sol Resnick, Director of the Water Resources

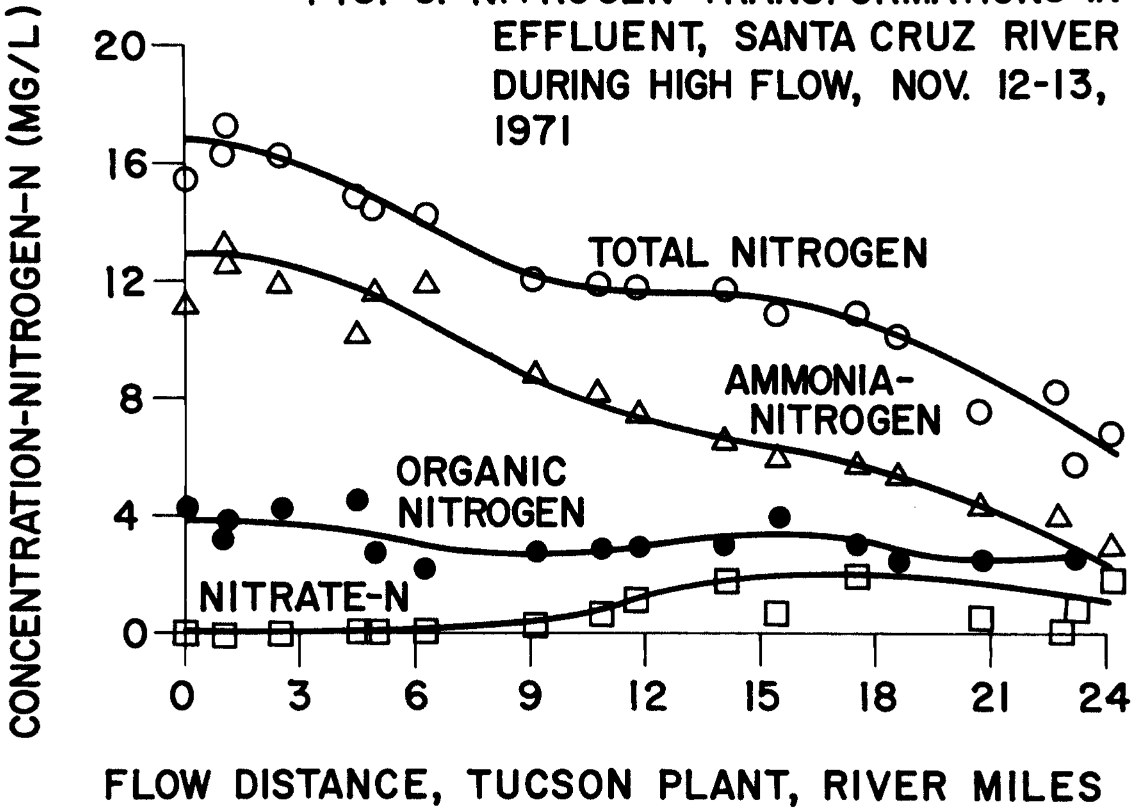



FIG. 5. NITROGEN TRANSFORMATIONS IN EFFLUENT, SANTA CRUZ RIVER DURING HIGH FLOW, NOV. 12-13, 1971

Research Center, The University of Arizona, is gratefully acknowledged. The authors wish also to thank Mr. Dennis Scheall, Superintendent of the Water Resources Field Laboratory, and associated individuals who provided assistance in this study. The installation of flumes was financed in part by the City of Tucson Water and Sewers Department, and construction equipment was generously provided by the Pima County Department of Sanitation.

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# EVALUATION OF A TURFGRASS - SOIL SYSTEM TO UTILIZE AND PURIFY MUNICIPAL WASTE WATER[1]

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## Introduction

The value of sewage effluent for the irrigation of agronomic crops has been well established. The plant-soil system is capable of utilizing the nutrient in the effluent and at the same time provides quality treatment on the waste water. Most of the current use of sewage effluent is directed toward the production of food and fiber crops.

In studies conducted by Day, Tucker and Vavich, it was shown that municipal effluent can be effectively utilized for the irrigation of small grains. Kardos (1968) at Pennsylvania State University found high nitrogen (N) utilization by reed canarygrass irrigated with sewage effluent. A farm near Lubbock, Texas (Gray, 1968) has been using nearly all of the city's wastewater for the irrigation of small grains and cotton since 1937. This operation has reported that yields with effluent irrigation are up to twice that obtained by normal irrigation farming.

The reclamation of municipal wastewater, emphasizing ground water recharge rather than crop production, has been studied by Bouwer and Lance (1970) at the Flushing Meadows Project west of Phoenix, Arizona. Secondarily treated effluent is given tertiary treatment by soil filtration before reaching the groundwater table. Similar work done at Whittier Narrows in California by McMichael and McKee (1965) indicated that the soil was an effective wastewater

[1]Journal Paper No. 1966 of the Arizona Experiment Station

treatment system. Wells (1968) has stated that some potential problems associated with sewage effluent recharge of ground water are: chemical mixing of recharge effluent and existing ground water; introduction of disease carrying organisms to the ground water; soil clogging; and aesthetic acceptance.

Nitrogen compounds in sewage effluent or fertilizer applied to the soil can contribute to ground water pollution. Keeney and Gardner (1970) reported instances of increasing ground water contamination from nitrate as a result of effluent and fertilizer applied to the soil. Cases of high nitrate content in ground water have been cited (McGauhey and Krone, 1967) when wastewater was recharged through the soil. Ground water nitrate concentrations increased 1.0 mg/liter/month at the Texas Tech Farm when sewage effluent was used for deliberate recharge (Wells, 1963). Ramati and Mor (1966) also reported increased nitrate concentrations resulting from wastewater irrigation of field crops on shifting sand dunes in Israel.

Under arid and semi-arid conditions turfgrass can utilize large amounts of N. Bermudagrass may require 8 to 16 pounds of N per 1,000 $ft^2$ during a 12 month growing season (Keen, 1969). Annual ryegrass needs about one pound of N per 1,000 $ft^2$ per month during the winter season (Schmidt and Blaser, 1969). It is evident that these grass species which are common to the southwest have a high nutrient demand for N and may be used effectively in a year-round system for sewage effluent purification.

This study was undertaken to determine the capacity of selected turfgrass soil systems to purify municipal sewage effluent and to measure the degree of utilization of N in the effluent by turfgrass. These factors, together with potential recharge rates, were examined for sandy loam and silt loam soils.

## Materials and Methods

Secondarily treated sewage effluent which had been chlorinated was obtained from the sewage treatment plant of the City of Tucson. The effluent varied from 13 to 26 ppm in total N, 70% of which was generally in the form of $NH_4^+$-N. Turfgrass was grown in sandy loam (70% sand, 17% silt, 13% clay) and silt loam (22% sand, 61% silt, 17% clay) soils contained in 24 20-liter pots. Twelve of the pots were irrigated with tap water supplemented with N and phosphate (P) in the predominate forms and amounts present in the sewage effluent. The remaining pots were irrigated with sewage effluent. Three levels of irrigation were used: low = 0.71 in.; medium = 1.42 in.; high = 2.13 in. Due to a N deficiency in the turfgrass these levels were later increased to low - 1.28 in., medium = 2.56 in. and high = 2.84 in. Treatments were prepared in duplicate and arranged in a complete factorial combination.

Each pot was instrumented with two suction probes located about 25 centimeters below the soil surface to simulate sub-soil moisture tensions and to extract soil water quantitatively. One of the probes, which was designed to remove gravitational water, consisted of a perforated plastic tube packed with glass wool. The other probe was a ceramic tensiometer cup for removal of soil water at tensions up to 1/3 bar. Both of the probes were connected to suction lines leading to a collection bottle packed in ice (Fig. 1).

Irrigations were made when approximately 60% of the available soil moisture had been depleted as determined from soil moisture blocks three inches below the surface on the low irrigation level treatment. All irrigation levels were applied at the same time for a given soil and quantitative soil water extracts were obtained through the suction probes following each irrigation.

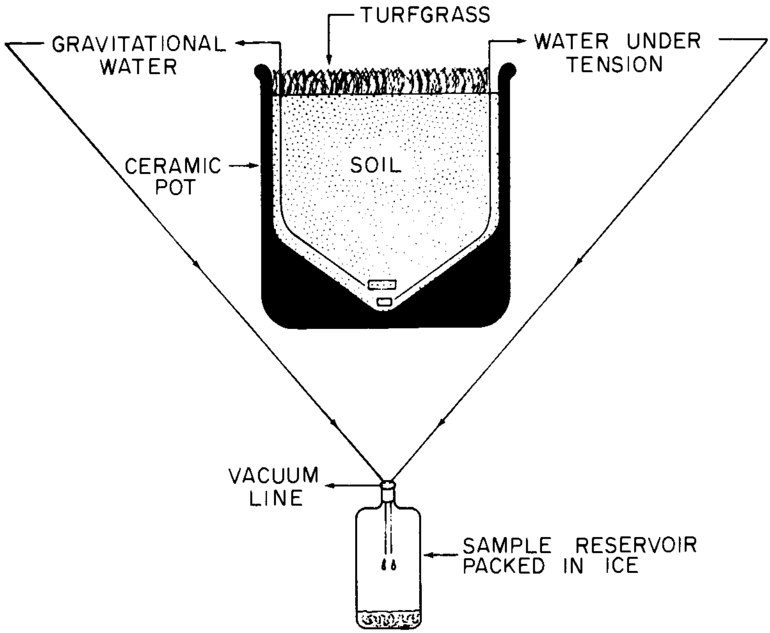


Figure 1. Diagram of experimental system

Soil water samples and turfgrass clippings were combined for three week intervals throughout the study. The volume of leaching water and its content of $NH_4^+$-N, $NO_3^-$-N, organic-N and total-N were measured for each period. Turfgrass clippings were oven dried at 87°C, weighed, and analyzed for total-N. Total-N $NH_4^+$-N, $NO_3^-$, and organic-N analyses were performed on the initial and final soil samples. All analytical methods used have been described by Bremner (1965).

The study was conducted at the University of Arizona Turfgrass Research Center over a period of 30 weeks, from September 3, 1971, to March 30, 1972. Common bermudgrass (Cynodon doctylon) was grown for the first month of the study and was then overseeded with annual ryegrass (Lolium multiflorum) for the remainder of the period.

The main objectives of the study were achieved by evaluating purification efficiency, utilization, and percent recharge at the end of each three week period and for the entire 30 week study. Purification efficiency, utilization, and percent recharge are defined by the following equations:

$$E = \left[1 - (\Sigma N_1 / \Sigma N_e)\right] 100 \qquad (1)$$

where: E = purification efficiency (for N) of system

$\Sigma N_1$ = total N removed in leachate

$\Sigma N_e$ = total N applied in effluent or nutrient solution

$$U_t = (\Sigma N_t / \Sigma N_e) 100 \qquad (2)$$

where: $U_t$ = utilization of N by turfgrass

$\Sigma N_t$ = total N removed in turfgrass clippings

$\Sigma N_e$ = total N applied in effluent or nutrient solution

$$\% R = (\Sigma V_1 / \Sigma V_e) 100 \qquad (3)$$

where: $\%R$ = percent recharge

$\Sigma V_1$ = total volume of leachate collected

$\Sigma V_e$ = total volume of effluent or nutrient solution applied

All data were analyzed statistically using Student-Newman-Keul's multiple range test using the .05 level of significance.

## Results and Discussion

The purification efficiency parameter was developed to account for N in solution moving below the root zone. Since a completely closed N balance is nearly impossible to obtain in a field study because of the difficulty in measuring gaseous losses, it was felt that purification efficiency would give the best estimate of potential N pollution of ground water. As used here, purification efficiency is a measure of the amount of N in the recharge water compared to the amount of N applied. There was no significant difference in purification efficiency among any of the treatments in the sandy loam soil or between the two soils. The high irrigation level on the silt loam soil resulted in a significantly lower purification efficiency than either the low or medium irrigation levels. Apparently under higher irrigation rates the ability of the system to remove N was reduced. A similar tendency was found for the sandy loam soil, (Table 1), although the difference was not significant.

The concentration of $NH_4^+$-N in the leachate was not significantly different among treatments within either of the soils, however, averaged over all irrigation levels and sources the $NH_4^+$-N concentration was significantly greater for the silt loam (1.38 ppm) than the sandy loam soil (0.36 ppm). Concentrations of $NO_3^-$-N in the leachate from the sandy loam soil were not significantly different among treatments, whereas, the high irrigation level

TABLE 1. MEAN VALUES FOR PURIFICATION EFFICIENCY, NITROGEN UTILIZATION AND PERCENT RECHARGE.

| SOIL TYPE | IRRIGATION SOURCE | IRRIGATION LEVEL | PURIFICATION EFFICIENCY | UTILIZATION OF N BY GRASS | RECHARGE |
|---|---|---|---|---|---|
| | | | % | % | % |
| SL | EFFLUENT | L | 94.4 | 115.5 | 22.9 |
| SL | EFFLUENT | M | 94.6 | 83.0 | 36.9 |
| SL | EFFLUENT | H | 94.0 | 86.7 | 40.6 |
| SL | CONTROL* | L | 95.4 | 98.0 | 21.0 |
| SL | CONTROL* | M | 94.7 | 86.0 | 36.9 |
| SL | CONTROL* | H | 94.8 | 79.4 | 42.8 |
| SiL | EFFLUENT | L | 96.9 | 82.4 | 16.1 |
| SiL | EFFLUENT | M | 95.6 | 70.5 | 27.4 |
| SiL | EFFLUENT | H | 91.5 | 68.8 | 39.5 |
| SiL | CONTROL* | L | 99.1 | 70.0 | 11.8 |
| SiL | CONTROL* | M | 96.3 | 83.3 | 15.7 |
| SiL | CONTROL* | H | 94.9 | 94.9 | 33.3 |

*TAP WATER + $(NH_4)_2HPO_4$

resulted in significantly higher $NO_3^-$-N concentrations than either the low or medium levels in the leachate from the silt loam soil. The mean $NO_3^-$ leachate concentrations did not differ significantly between soils. Nitrate concentrations never exceeded 4.0 ppm for any treatments at any time during the experiment.

There was a significantly greater concentration of organic-N in the leachate under low irrigation treatment than in the medium or high levels for the sandy loam soil. The concentration of organic-N in the leachate from the silt loam soil was not affected by treatments. Significantly greater concentrations of organic-N were found in the leachate from the sandy loam than in the silt loam soil. This may be a result of increased mobility of microbial debris in the coarser textured soil. The total-N concentration in the leachate was significantly greater under low irrigation than the medium or high irrigation levels for the sandy loam soil. Apparently a higher percentage of the applied solution was evaporated at this low irrigation level on the coarse textured soil causing the leachate to be more concentrated in dissolved solids. There were no differences in total-N concentration among treatments in the silt loam soil nor between the two soils.

Utilization of N by the turfgrass represents a ratio of N removed in clippings N applied in the effluent or nutrient solution (Table 1). There was no significant difference in the utilization rates among treatments for either soil. The utilization rate was significantly higher for turfgrass grown on sandy loam than on silt loam soil. This may have been due to slightly higher dry weight yield on the sandy loam treatments. Irrigation with nutrient solution resulted in a significantly greater percentage of N in the dry grass clippings than irrigation with effluent for both soils. For both soils the high irrigation treatment resulted in significantly more N in clippings than the medium level, which in turn was more than the low level. No differences in N content of clippings was found between soils.

The potential ground water recharge (expressed as a percentage of the irrigation volume) was higher under the sandy loam soil than the silt loam soil (Table 1). This could be a result of higher infiltration and permeability rates in the coarser textured soil. Low irrigation levels resulted in significantly lower recharge than the other levels for both soils. There was a significantly greater recharge from the high irrigation level than the medium level in the silt loam, but not in the sandy loam soil. Also the effluent treatments were associated with significantly greater recharge than the nutrient solution treatments in the silt loam soil.

Major differences were observed in purification efficiency in relation to time. These differences, illustrated by the data for the sandy loam soil irrigated with effluent (Fig. 2) were not related to specific treatments of soil type. Initially the purification efficiency was high since the bermudagrass was well established when treatments were first applied. The small drop in efficiency during the first part of October was probably due to N leaching caused by the application of excess tap water to aid in establishing the newly seeded ryegrass. The continuing drop in efficiency up to November may have resulted from excessive rain during the period which required leaching extractions to be withdrawn even though treatments were not applied. After exhibiting a higher efficiency level near the end of November, the rate once again dropeed due to excessive rain. The growth of ryegrass was hampered

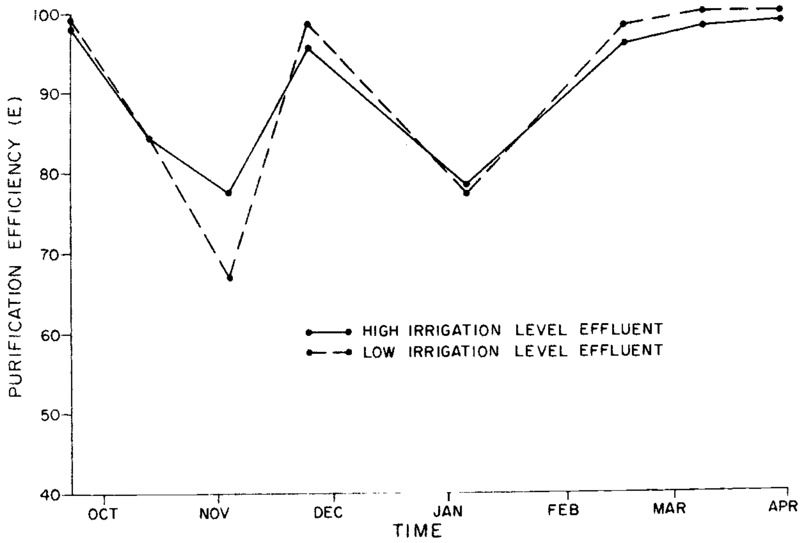


Figure 2. Sewage effluent purification by turfgrass - sandy loam soil system.

by disease. As a result of these low treatment levels and the absence of rain, the efficiency increased during this period. Once the ryegrass recovered from the disease, it grew well and efficiency rates remained very high.

Utilization of N by the turfgrass tended to vary with factors such as cutting height and disease. These variations, shown by the data for the sandy loam soil irrigated with effluent (Fig. 3), were not related to treatments or soil type. The utilization peaks in October and January were associated with scalping the grass plots in preparation for seeding and due to occurrence of disease respectively. The low utilization in February was probably a result of slow recovery of the ryegrass from disease.

## Summary and Conclusions

The capacity of turfgrass-soil systems to utilize N and purify wastewater was examined. Results indicate that turfgrass can be irrigated with sewage effluent at common rates without hazard of N ground water pollution

Purification efficiency was greater than 90% for each of three irrigation levels and two different textured soils. Irrigation with effluent in excess of plant water requirements on coarse textured soil resulted in up to 41% recharge with high quality water in terms of N content. This was demonstrated by the fact that $NO_3^-$-N concentrations in the leachate from the sandy loam soil irrigated with effluent averaged about 40 times less than the U.S. Public Health Service limits. Periods of substantial rain and low N utilization by the turfgrass tended to reduce purification efficiency.

Nitrogen utilization by the turfgrass was higher for treatments on sandy loam than silt loam soil. Average utilization rates varied from 79 to 115% for the coarser textured soil. Factors causing major fluctuations in utilization rates throughout the study appeared to be cutting height and disease of the turfgrass.

Acknowledgements:

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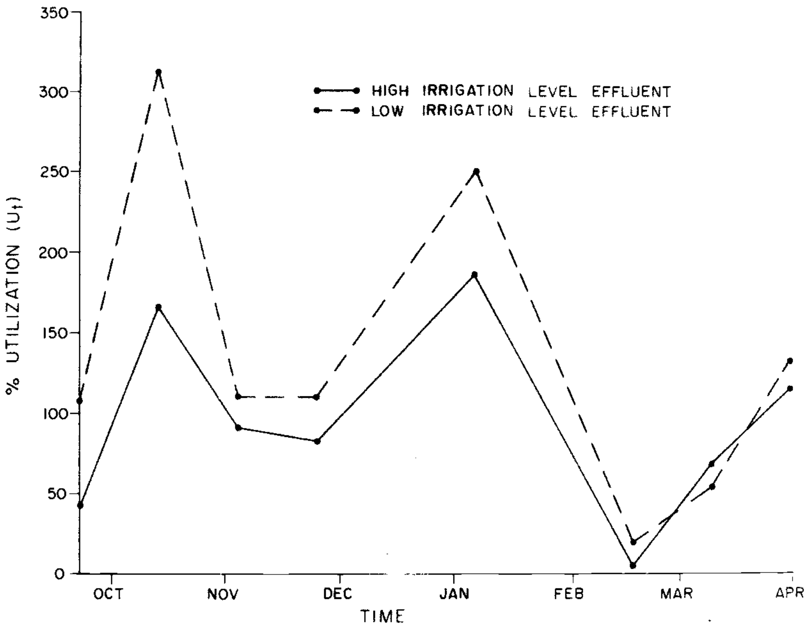


Figure 3. Utilization of nitrogen from sewage effluent by turfgrass grown on sandy loam soil.

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# INPUT SPECIFICATIONS TO A STOCHASTIC DECISION MODEL

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## ABSTRACT

The use of discrete conditional dependency matrices as input to stochastic decision models is examined. Some of the problems and initial assumptions involved with the construction of the above mentioned matrices are discussed. Covered in considerable detail is the transform used to relate the gamma space with the normal space. A new transform is introduced that should produce reasonable results when the record of streamflow (data) has a highly skewed distribution. Finally, the possibility of using the matrices to provide realistic inputs to a stochastic dynamic program is discussed.

## Introduction

Much progress has taken place in the past decade in devising new and improved methods of determining optimal operating rules for reservoirs. By optimal operating rules for reservoirs we mean those decisions concerning the amount of water to be released during specified time periods that maximizes the utility of the releasedwater. Several different techniques have been proposed and used to model the operation of reservoirs. The most basic techniques are 1) deterministic linear programming, 2) deterministic dynamic programming, 3) stochastic linear programming, 4) stochastic dynamic programming, and 5) simulation. The purpose of this study is not to propose a new technique, but to suggest a type of input that could be used with a stochastic dynamic programming model. The type of input that is proposed is a monthly conditional probability dependency matrix that represents the probability of a certain flow occuring during the current month based on the flow that occured during the previous month. The conditional probability dependency matrix for the pair of months of January and February are graphically shown in Figure 1.

A typical matrix would consist of mxn conditional probabilities, $P_{ij}$, where i = 1,m and j = 1,n with the constraint that $\sum_{j=1}^{n} P_{ij} = 1$. Information could be contained from the matrix as follows: assuming we are presently in the month of February, we would know the magnitude of the flows observed

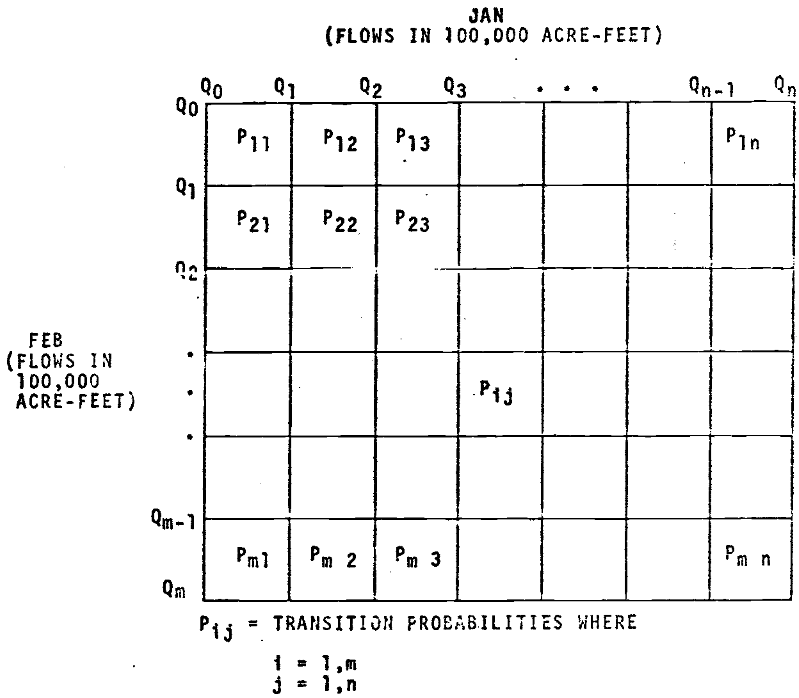



Figure 1. Conditional Probability Dependency Matrix for January-February

in January. Let us say then, for example, that for a given flow in January between $Q_1$ and $Q_2$, we might want to know the probability of experiencing a flow in February between $Q_1$ and $Q_2$. In this case, the probability that is described is $P_{22}$.

After the conditional probability dependency matrix is constructed for the pair of months of January and February, a similar matrix would be constructed for the pair of months of February-March. Following February-March, a matrix would be created for March-April, and so on until twelve sets of matrices are created. These tweleve sets of conditional probability dependency matrices would comprise the input to a stochastic dynamic programming model.

Loucks [1969] and Butcher [1971] have considered the use of conditional probability dependency matrices as input to stochastic dynamic programming models. Butcher used sets of matrices as input to a stochastic dynamic program in order to obtain optimal operating rules for the Watasheamu Dam, which is near the California-Nevada state line. Butcher reported that the system he studied was ergodic in nature since the final state of the system was independent of the starting state. In other words, his report is equivalent to stating that regardless what the state of the reservoir is at tht start of the computations, the steady state of the system will be independent of that starting state. This is an important observation since it is the basic assumption that Butcher made in order to determine that an optimal policy can be determined in the first place.

It was also noted by Butcher that the probabilities of the various flows in a river were dependent on the flows in the previous time period. Hence the probability of being in a given state following another given state is a fixed value. This sequence of events, or course, is known as a Markov chain and the conditional probabilities, in this case, are stationary. Hence the conditional probability dependency matrices that this study is concerned with are stationary Markov chains.

The sets of matrices that are constructed in this study are similar to those used by Butcher. However, the matrices constructed herein are more precise than any others constructed, to the best of the writers knowledge, for the purpose of obtaining more precise results.

## Streamflow Synthesis Technique Selected

The stochastic synthesis technique that was selected for the purposes of this study was Beard's method [Monthly Streamflow Simulation, 1967]. This method was not selected because it was necessarily the best technique available, but because it is at least a reasonable method and has been shown to work. The reader should bear in mind that the "best" streamflow synthesis technique is mainly in the eyes of the beholder. It is extremely difficult, if not impossible, for one person to select the "best" data generation method and have all of his colleagues concur with his decision. After all, the problems that practitioners of stochastic hydrology attempt to solve are usually so complex that it is difficult to pick the best technique based on the results. At any rate, it is felt that Beard's method is satisfactory, since it had been used by the U.S. Government and the results, for practical purposes, were satisfactory [Monthly Streamflow Simulation, 1967]. Therefore, considering several factors, Beard's method is best suited for the purposes of this study. There are, however, some problems that were encountered when Beard's technique was used. The two problems are 1) the logarithmic transformation that is used to "smooth" the original streamflows, and 2) the Wilson-Hilferty transform that is used to transform gamma deviates to normal deviates and vice-versa. With respect to the first problem, Weber and Hawkins [1971] noted that the format, or organization, of the data is important when a logarithmic transformation is performed. If the data are not in a rectangular hyperbolic form, then Weber and Hawkins indicate that a logarithmic transformation will introduce a significant amount of error. The degree of error will depend on how closely the organization of the data adheres to the rectangular hyperbolic form. Beard's technique does not attempt to determine how closely the original data compares with a rectangular hyperbolic form. Unfortunately, to the best of our knowledge, no technique that utilizes a logarithmic transformation considers this particular problem. Hence, until implementable methods of working in the non-transformed domain are found, we have to resort to such methods as used here to solve practical reservoir operation planning problems.

The second problem encountered with Beard's technique was the use of the Wilson-Hilferty transform. This problem will be discussed in detail later in this paper.

## Background Information

The data that was used in this study was supplied by the Hydrologic Engineering Center, U.S. Aruy Corps of Engineers at Sacramento, California. The average monthly streamflows, that constitutes the data, were observed at the Sacramento River in the vicinity of the Shasta Dam, which is in Northern California. The length of record is forty-eight years of continuous observations, commencing with 1928.

## Assumptions

In order to proceed with this study, a number of assumptions were made. The first assumption was that the statistics, specifically the mean, standard deviation, coefficient of skewness, and the first order serial correlation coefficient, that were generated by Beard's routine are correct.

In the case of the mean and the variance, the assumption is completely justified. However, the assumption that the skew coefficient is correct may be questionable to some practioners of stochastic hydrology. Since the skew coefficient is a function of the third moment about the mean, many researchers believe that any measure of skewness is unreliable. According to Matalas and Benson [1968], however, the error related to the coefficient of skewness can be determined. They acknowledge that for a finite number N of observations the true value of the coefficient of skewness $\sqrt{\beta_1}$ is unknown, but that an estimate of this coefficient, denoted by $\sqrt{b_1}$, can be determined. The estimate of $\sqrt{\beta_1}$ is defined as

$$\sqrt{b_1} = \left(\frac{\sum_{i=1}^{n} (x_i-\bar{x})^3}{N}\right)\left(\frac{\sum_{i=1}^{n} (x_i - \bar{x})^2}{(N - 1)}\right)^{-3/2}$$

where $\bar{x}$ is the sample mean for the hydrological variable x. To determine how good an estimate of the coefficient of skewness $\sqrt{b_1}$ is, Fisher, [1931], showed that for samples obtained from a normal distribution the standard error of $\sqrt{b_1}$ is

$$\sigma_1 = \left(\frac{6N(N - 1)}{(N - 2)(N + 1)(N +3)}\right)^{1/2}$$

where $\sigma_1$ is independent of the central moments of x. Matalas and Benson [1968] suggested that although the above equation is based on assumption of normality, it may still be used as an indicator of the magnitude of the standard error. As far as this study is concerned, there are 48 observations (monthly flow values for forty-eight years of record). Substituting the value of N into the above equation, the standard error is approximately equal to 0.34. Since we have some measure of the magnitude of the standard error of the estimate of skewness, and we consider this error to be acceptable, it is felt that this assumption is supportable.

The assumption that the monthly serial correlation coefficient is correct may also be questionable to some since it was computed by considering only the preceding monthly flows, not all of the annual flows. However, it was again Matalas [1967], who showed that even though all preceding flows could have an influence on streamflows in a given month due to recession, a good estimate of the phenomena involved is given if it is assumed that the streamflows are serially correlated with a lag of one. It is in view of Matalas' research that it is assumed that the first order serial correlation coefficient is correct.

The second assumption that was made in this study was that the base method for the observed flow frequencies is the log-Pearson Type III distribution. This assumption was prompted by the Water Resources Council, which consists of technical staff members of the Federal departments represented in the Council and of the Tennessee Valley Authority. In 1966, the Chairman of the Water Resources Council, Stewart L. Udall, urged the adoption of the log-Pearson Type III distribution as a base method for flow frequencies for two reasons: 1) the log-Pearson Type III distribution is a sufficiently accurate representation of flow distributions, and 2) it is felt that a uniform technique for determining flow frequencies would be desirable for research agencies.

The third assumption is related to the type of reservoir that is used in this study. Beard, in his streamflow simulation routine, assumed a multi-station, multi-reservoir system. For the purposes of this study, a single-station, single reservoir system has been assumed. Since a single-station set-up has been assumed the formulas that Beard presented will be changed to some extent. The formulas, in their somewhat different form, will be discussed next.

## Construction of the Conditional Probability Matrices

Beard's routine essentially consists of two parts: The statistical analysis portion and the synthetic streamflow generation portion. Since the assumption was made that only a single-station, single reservoir system would be considered in this study, Beard's procedure, which assumed a multi-station, multi-reservoir system, would be altered as follows:

1) add a small increment (one percent of the average monthly flow) to each streamflow to preclude negative infinite logarithms;
2) take the common logarithm of the result of step 1;
3) for each monthly logarithmic vector, compute the mean, standard deviation, and skew factor; the skew factor is defined as:

$$g = \frac{N \sum_{i=1}^{N} (X_i - \bar{X})^3}{(N - 1)(N - 2)S^3}$$

where $X_i$ is the result of step 2, S is the standard deviation, $\bar{X}$ is the mean, and N is the number of observations;

4) the standardized gamma deviate y is then computed by

$$y = \frac{X_i - \bar{X}}{S}$$

5) the result of step 4 is then transformed to a normal deviate K using the Wilson-Hilferty transform as shown below:

$$K = \frac{6}{g} \{[(\frac{gy}{2}) + 1]^{1/3} - 1\} + \frac{g}{6}$$

where g and y are as previously defined:

6) the correlation coefficient r is then computed between the normal deviates for adjacent months:

) a uniform (0,1) random number is generated;

8) the uniform random number is transformed to a standard normal deviate;

9) a stochastic process is generated by using the following relationship

$$K_{t+1} = rK_t + (1 - r^2)^{1/2} \; Z$$

where $K_t$ and $K_{t+1}$ are correlated normal deviates at time t and t+1, respectively; $K_1$ is given $K_t$, where t<1 is undefined, and Z is the random normal deviate;

10) the inverse of the transform of step 5

$$y = \{[(\frac{g}{6})(K - \frac{g}{6}) + 1]^3 - 1\}\frac{2}{g}$$

is used to produce a standardized gamma deviate from a standardized normal deviate;

11) the standardized gamma deviate y is then multiplied by the standard deviation S and added to the sample mean $\bar{X}$ as shown below:

$$X = yS + \bar{X}$$

12) the antilogarithm of X is taken;

13) the small increment that was added in step 1 is subtracted.

Steps 1 through 6 comprise the analysis portion of the routine and steps 7 through 13 define the generation portion of the routine. The generation portion of the routine might, when the routine is used in a simulation mode, be repeated several hundred or thousand times. However, what concerns one here is not the use of this routine in a simulation mode, but construction of a conditional dependency matrix.

The construction of a discrete conditional probability dependency matrix between streamflows, not gamma or normal deviates, is desired for each pair of sequential months (e.g., Jan-Feb., Feb-Mar., Mar.-Apr., etc). Therefore, using the definition of conditional probability, the conditional probability $P(A|B)$ is desired where

$$P(A|B) = P(A,B)/P(B)$$

To solve the above equation for the conditional probability, one must first determine the unconditional probability of the January flows and the joint probability of the January and February flows.

## Computation of the Unconditional Probabilities

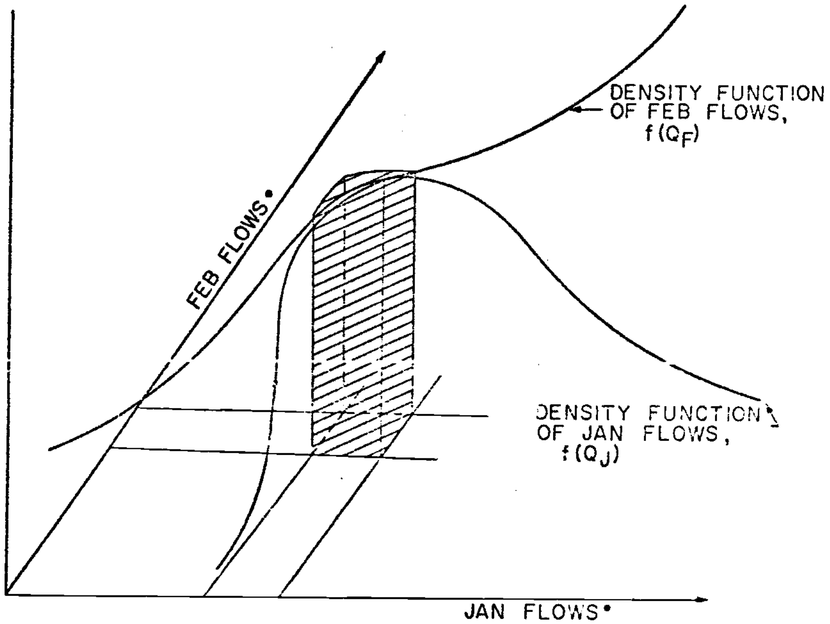
The computation of the unconditional probabilities of the prior monthly streamflows is straightforward. A procedure that could be used to determine them is as follows:

1) take the common logarithm of each streamflow in question;
2) determine the standardized gamma deviate of the value in step 1;
3) transform the standardized gamma deviate of step 2 to a standardized normal deviate;

4) then using a table of normal probability function or a numerical integration scheme and a digital computer, determine the unconditional probability of each streamflow in question.

## Computation of the Joint Probabilities

Determining the joint probability, however, is not so straightforward. The joint probability is expressed by means of bivariate normal distribution. Graphically , the joint probability is represented by the shaded volume as shown in Figure 2. Of course, one could obtain tables of the bivariate normal distribution and obtain the joint probabilities if the flow intervals were long enough. But, the flow interval size selected for the study was 100,000 acre-feet, which is too small for a bivariate normal table to be of help. Unfortunately, no more accurate tables could be found. Another method for computing the joint probability, as shown in Figure 2, is to perform double numerical integration. However, considering the practical side of the problem, if a 50x50 dependency matrix were desired, it would mean that 2500 conditional probabilities, $P_{ij}$, would have to be computed for each pair of months. Since there are tweleve pairs of months, a total of 30,000 conditional probabilities would have to be computed. If it took five seconds to perform each double numerical integration on a digital computer, one can readily see that approximately, forty-two hours of computer time would be required. Obviously, an analysis of numerical methods must indicate another method of determining the conditional probabilities that preserves accuracy but reduces the computational time. Roefs and Clainos




VOLUME = JOINT PROBABILITY OF JAN AND FEB FLOWS

VOLUME = $P(Q_J, Q_F) = \iint f(Q_J)\, f(Q_F)\, dQ_J\, dQ_F$

• ALL FLOWS ARE EXPRESSED AS STANDARD NORMAL DEVIATES.

Figure 2. Joint Probability Density Function $P(Q_J, Q_F)$

[1971] determined that if the flow interval of the previous month, January in this case, were sufficiently small it would be represented by a single point. More specifically, the interval could be more accurately represented by the midpoint of the interval than any other point. If this numerical approximation is made, the computation is greatly reduced since only single numerical integration is required. But what about the accuracy of this method? Roefs and Clainos [1971] showed that if the interval that is represented as a point were sufficiently small then all of the probabilities of the same class would sum to unity. Expressed in more familiar form we have

$$\sum_{j=1}^{n} P_{ij} = 1 \qquad \text{as } \underset{i=1,2,\ldots,m}{n \to \infty}$$

It was determined by an iterative procedure that an interval of 100,000 acre-feet was sufficiently small to represent an interval as a point. The reader should remember, however, that the size of an interval will depende on the magnitude of monthly streamflows; there is nothing sacred about 100,000 acre-feet.

Now that both the unconditional and joint probabilities can be determined, the conditional probabilities can be computed. A computer program was devised for this purpose; it will be discussed later. The conditional probability dependency matrices were computed for twelve pairs of months. The error for each computation of a conditional probability ranged from zero to approximately twelve percent.

Throughout the preceding paragraphs, the months of January and February have been used as examples. The same procedure is used for the other twelve pairs of months.

## The Problem with Transforms

In Beard's routine, the Wilson-Hilferty transform is used to transform a Pearson Type III deviate to a normal deviate, and vice-versa for the synthetic streamflow generation portion. This particular transformation is possibly the weakest part of Beard's routine. At best the transformation is an approximation. McGinnis and Sammons [1970] studied the Wilson-Hilferty transform in detail and evaluated, along with other transforms, its accuracy with respect to the coefficient of skewness that was used. Of the four transformations that were evaluated, the Fisher transform could be eliminated immediately since it produced values that were obviously inferior to the remaining transforms. The three remaining transforms were 1) the Fisher-Cornish transform, 2) the Wilson-Hilferty transform, and 3) the Severo-Zelen transform. A comparison of these three transforms evaluated when the coefficient of skewness is 1.0 or less shows no significant difference. As the coefficient of skewness is increased to 2.0, the Severo-Zelen transform seems to perform the best (relative error of approximately 0.25 percent), the Wilson-Hilferty transform was second (relative error of approximately 2.19 percent), and the Fisher-Cornish transform was third (relative error approximately 19.1 percent). However, McGinnis and Sammons point out that as the coefficient of skewness increases beyond 2.0, all of the above mentioned transforms seem to produce poor estimates of the upper and lower tails of distribution. It should be made clear, however, that at low levels of skewness $g<1.0$ there does not seem to be any significant error (relative error $<.01$ percent). Therefore, the question is now what does one do when the coefficient of skewness exceeds 2.0?

At this time, one should acknowledge that the above mentioned decision is not purely technical in nature, but also managerial. For example, McGinnis and Sammons [1970] gathered information on seventy-four watersheds in the state of Pennsylvania that ranged in size between 2 square miles and 200 square miles and in length of record from 10 years to 59 years. Of these, two thirds of the watersheds ahd coefficients of skewness between -0.5 and +0.5 and only seven had coefficients of skewness outside of the range of -2.0 to +2.0. Hence the water resources manager, the same as the business manager must determine what his return-on-investment will be if he elects to create a mew method for the specific prrpose of solving a small fraction of the total, or he must determine the penalty for not creating that special method. His alternatives are three-fold. First, he can do nothing and accept the errors introduced into the conditional probabilities, $P_{ij}$. Second, he can establish some arbitrary cut-off level of skewness; for example g = +1.8. This cut-off level of skewness was determined subjectively. Since McGinnis and Sammons [1970] showed that a significant amount of error was introduced by the Wilson-Hilferty transform when skewnedd exceeded 2.0, the cut-off level of skewness was arbitrarily set at 1.8. Using this method whenever the coefficient of skewness exceeded +1.8, it would automatically be truncated to + 1.8, depending on if the distribution is positively or negatively skewed. Finally, a new method, or transform, could be devised to handle the transformation when the coefficient of skewness exceeds 2.0. A decision must be made.

## A New Transform

In this study it became necessary to investigate the possibility of devising a new transform because 1) one of the monthly coefficients of skewness exceeded 2.0, and 2) it is felt that when this new transform is perfected, or at least greatly improved, it will be able to transform variables from the Pearson Type III distribution to the normal distribution, and vice-versa, under highly skewed conditions. Such a transform would certainly aid water resources planners in an arid-land environment such as Arizona, where the rivers and streams are characteristically highly skewed.

Although this transform is a new technique in the sense that is was devised by this writer the idea was provided by Harter [1969] who developed the table of percentage points to the Pearson Type III distribution arranged in categories of different levels of skewness. Basically, Harter computed the percentage points of the chi-square distribution, which he then modified to percentage points of the Pearson Type III distribution. One procedure that Harter used can be described as follows:

1) Compute the probability integral $P(\chi^2;\nu) = I(u,p)$ of $\chi^2$ with $\nu$ degrees of freedom such that

$$I(u,p) = \frac{1}{\Gamma(p+1)} \int_0^{u\sqrt{p+1}} v^p e^{-v} dv$$

where $u = \chi^2 \sqrt{2\nu}$, $p = \nu/2 = 1$, and $I(u,p)$ is the incomplete gamma function ratio;

2) Obtain the percentage points of $\chi^2$ by performing inverse interpolation in the table of the probability integral in order to obtain the percentage point of u, then multiply by $\sqrt{2\nu}$;

3) Relate the chi-square distribution with $\nu$ degrees of freedom to the Pearson Type III distribution with mean $\mu = \nu$ , standard deviation $\sigma = \sqrt{2\nu}$, and skewness $\gamma = \sqrt{8/\nu}$.

The result of Harter's study is a table of percentage points to the Pearson Type III distribution for a specified level of skewness. At this point, one could build on Harter's work and construct an algorithm suited for a digital computer that would produce standard normal deviates. The procedure is as follows:

1) the gamma deviates are computed from the streamflows;

2) given the level of skewness, the gamma deviate is related with the appropriate percentage point as determined by Harter, using linear interpolation (Lagrangian interpolation could be used if additional accuracy is required).

3) by means of areas, the percentage point of the Pearson Type III distribution is then related to the percentage point of the standard normal distribution,

4) iterative numerical integration is performed until convergence (wtihin specified error bounds) is achieved;

5) the result is a normal deviate.

The question now is how well does this "new" transform work. The transform has been subjected to two tests. The first test involved actually computing conditional probabilities when the coefficient of skewness exceeded 2.0. This occurred during the pairs of months of September-October and October-November. When the Wilson-Hilferty transform was used, unrealistic conditional probabilities were produced. The probabilities that were generated appeared to be hydrologically unreasonable because a high flow in September virtually dictated that a flow two or three times higher would occur in October and the probability of lower flows was essentially nonexistent. In addition, the sequence of the conditional probabilities caused a disturbance since a small conditional probability would be followed by a slightly higher probability, only to be next followed by a probability of zero. In other words, there was no visible trend followed by the probabilities, as one might expect, but merely an erratic sequence of numbers. These errors were so pronounced that the dependency matrix that was generated was considered to possess more noise than meaningful information. However, using the new transform, the conditional probabilities were reasonable in as much as the rows approximately equalled unity and there were no pronounced inconsistencies. Although encouraging, this test certainly does not show that the "new" transform works well.

Another test was performed. The purpose of the second test was to determine how the new transform would perform when skew coefficients less than 1.0 were encountered. In this case, the new transform could be directly compared with the Wilson-Hilferty transform. Since McGinnis and Sammons [1970] determined the accuracy of the Wilson-Hilferty transform, one could determine the accuracy of the new transform by comparing the two. The pair of months of June-July was arbitrarily selected. The coefficient of skewness for the streamflows in June and July are 0.183 and 0.240, respectively. One hundred different conditional probabilities were computed by the new transform and by the Wilson-Hilferty transform. THe results were compared. The average difference between the conditional probabilities was 1.28 percent. These results show that, at least, the new transform performs as well as the Wilson-Hilferty transform. The results of the first test indicate that the new transform performs well when the skew coefficient exceeds 2.0. Although encouraging, the new transform should undergo several additional tests before it is accepted.

In this section it has been shown that the conditional probability dependency matrices can be constructed. The actual matrices that have been constructed are too voluminous to be included in this report. However, they are available [Clainos, 1972] by special request. In the next section the possibility of using these matrices to provide realistic inputs to a stochastic dynamic program will be discussed.

## Application

In order to demonstrate the application of the conditional probability dependency matrices as inputs to a stochastic dynamic programming model, such a model will be formulated in this chapter. At this point, it is important to make the assumption that only a single-purpose reservoir will be considered. This is an important assumption because with a single-purpose reservoir the economic return in each time period is a function of the water released only. Before the model is formulated, however, we shall specify what subscripts will be used to identify the different time periods involved. Basically, the formulation will start at some arbitrary time period in the future. Then each set of computations (in each time period) will look forward in time but a step backwards in time will be made in order to determine the next set of computations. The two state variables, amount of reservoir storage and inflow, will be defined as s and q, respectively. The decision variable, amount of release, is defined as r. It might be helpful, at this point, to refer to Figure 3 to determine the time relation of the computations involved in the formulation.

We are presently at some arbitrary future time period. If the storage in the reservoir at this time period is $s_1$ and the flow that has taken place in the preceding time period is $q_2$, then the value of the reservoir at this time is equal to the value of the release only. At this point, one should realize that the initial value of the reservoir is necessarily set to be equal to zero. The value of the reservoir is shown mathematically as:

$$f_1(s_1,q_2) = \max_{r} \, [V(r)]$$

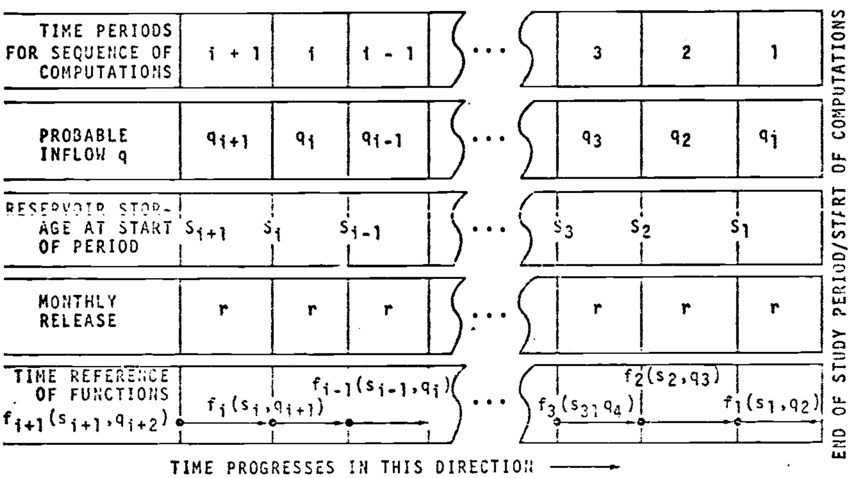



Figure 3. Time Relation of Computations

where $f_1(s_1,q_2)$ is the expected return from the optimal operation of the system in the first time period; and $V(r)$ is the value of the release in the time period. We are now ready to take another step backwards in time. Following Bellmans' Principle of Optimality [Bellman, 1957], which is loosely defined as any optimum policy has the property that regardless of initial state or initial decision, the resulting decisions will form an optimal policy with regard to the state that resulted from the initial decision. Use will be made of one of the optimal policy determinations made for the last stage in the next stage. Stated mathematically we have,

$$f_2(s_2,q_3) = \max_r \left[V(r) + \sum_{q_2=0}^{\max q_2} P(q_2|q_3) \cdot f_1(s_1,q_2)\right]$$

where $\Sigma P(q_2|q_3)$ are the transition probabilities that relate the flow in the second time period, $q_2$, with the flow in the third time period, $q_3$. Also the limits of the summation of the transitionprobabilities range from zero to the maximum flow in the second time period. If this procedure is repeated $f_3$ can be determined from $f_2$, $f_4$ can be determined from $f_3$, and so on. This recursive procedure then can be generalized as:

$$f_i(s_i,q_{i+1}) = \max_r \left[V(r) + \sum_{q_i=0}^{\max q_i} P(q_i|q_{i+1}) \cdot f_{i-1}(s_{i-1},q_i)\right]$$

with the recursive relationship:

$$s_{i-1} = s_i - r + q_i$$

where $f_i(s_i,q_{i+1})$ is the expected return from the optimal operation of a system in the ith time period; $s_i$ is the volume of storage at the start of the ith time period; $q_i$ is the flow into the reservoir in the ith time period; and $\Sigma P(q_i|q_{i+1})$ are the transition probabilities that relate the flow in the ith time period, $q_i$, with the flow in the $(i + 1)$th time period, $q_{i+1}$.

Using the above described relationship, which would start in some future time period and utilize the conditional probability dependency matrices in conjunction with data from an actual reservoir, it is possible to calculate values of release for each time period as a function of the state variables, storage and previous inflow. These releases then determine the optimal operating policy of that reservoir when the policy converges. A policy is said to converge when the values of release that are used to evaluate the function $f_i(s_i, q_{i+1})$ are repeated for all values of i as i increases. In his study at the Watasheamu Dam, Butcher [1971] observed that the release policy did converge. In this case, convergence took place after 30 iterations, but Butcher computed an additional 24 iterations to insure convergence.

It is felt that if the conditional probability dependency matrices constructed in this study and actual data from a reservoir were used as input to the stochastic dynamic programming model developed in this section, one could obtain an optimal operating policy for that reservoir. This statement is based on the assumption that such a solution exists in the first place.

## Conclusions

A number of conclusions can be made when creating conditional dependency matrices based on a stochastic streamflow synthesis technique. First, not all assumptions of current streamflow synthesis techniques produce results which are hydrologically reasonable. Because of the high value of the skew coefficient, the transformation from the gamma space to the normal space and back to the gamma space is not at all accurate. Hence, the result is that the conditional probabilities that are generated are clearly not accurate.

Second, double numerical integration is not computationally feasible because of the large amount of computer time required. An approximation, such as the midpoint method, is required to solve the problem of double numerical integration in a practical manner.

Third, it may be possible to use transforms efficiently when streamflows are high skewed. Although this new transform was used successfully in this study, the real value of such a transform may be realized when studying watersheds in an arid region where the coefficient of skew of the original data may be as large as 7.

Finally, it has been shown that conditional probability dependency matrices can be constructed. It has been further shown that conditional probability dependency matrices can be used as input to a stochastic dynamic programming model in order to obtain an optimal operating policy for a reservoir.

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# ROLE OF MODERN METHODS OF DATA ANALYSIS FOR INTERPRETATION OF HYDROLOGIC DATA IN ARIZONA

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## INTRODUCTION

Mathematical models of hydrologic and water resource systems have been the subject of intensive investigation in recent years. Their construction requires substantial data. Our goal in this paper is to interrelate the processes of data analysis and model building. Implied in this goal is the use of models to forecast for either scientific reasons or decision making (ultimately).

The digital computer has led to the rapid development of methods for extracting more information from available hydrologic data and for more comprehensive management of real systems. However, there has been a substantial roadblock in developing standardized computer packages, in orienting hydrologists and engineers to the advantages of computer-based data analysis, and in achieving compatibility between existing data bases and computer models developed for private and government contractors. To achieve maximum information (dependent on one's definition) from the available data, a skillful amalgamation is necessary of knowledge about methods of data analysis, hydrologic processes in space and time, process modeling, and identification of properties of process models from the data. To demonstrate this synthesis, examples are drawn from previous and current research on modeling aquifers,

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watersheds, streamflow and precipitation patterns in Arizona. A critical review of the state of the art in data analysis is presented. In this sense, this paper is more programmatic of work to be done and more a promise of the future.

## CLASSES OF PROBLEMS

An experienced hydrologist entering a new hydrologic basin is forced with the need to answer a number of myriad questions:

(a) What is the past hydrologic history of the basin?

(b) How quantitative and reliable are past observations?

(c) How adequate are these observations to define present conditions and in turn to forecast future states of the basin?

(d) To what engineering or social uses are the forecasts to be put? How sensitive is the use to goodness of forecast?

(e) Are the forecasts so unreliable that the hydrologist must recommend an investment in more data? If so, what are the opportunity costs (in benefits foregone) as a consequence of waiting?

(f) Is the purpose of the study of such importance that a computer-based approach is justified?

(g) Are the available hydrologic data punched onto cards such that computer-based data analysis can be efficiently pursued? That is, does a comprehensive hydrologic data base exist?

Perhaps many of these questions are only hazily perceived or subjectively considered at present by the profession. We suggest that future environmental management requires answering them more consciously.

An individual's approach to hydrologic practice is conditioned by his formal education, field and office experience and refresher courses. His conceptual basis for problem analysis may be in terms of empirical rules learned from older hydrologic texts or, at the other extreme, in terms of advanced knowledge of today's deterministic and stochastic methods that require computers. Much hydrologic practice is empirically based and constrained by available time (deadlines), budgets, data, and knowledge. In this context, conceptual foundations established in earlier formal and informal education tend to persist throughout a lifetime and actually are in conflict with more recent concepts imparted to the younger generation. However, even among the younger generation there exists philosophical disagreement on the merits of deterministic versus stochastic methods or of formal versus empirically-based methods. Given this situation, there in fact exists a model choice problem for each component of the hydrologic process in a basin: aquifer, overland flow, channel flow, evaporation, transpiration, precipitation, infiltration, and so on.

Hence, the "solution" to all subsequent problems depends on the hydrologist's personal solution to the model choice problem. The model may be mental and not quantifiable even in a fuzzy sense. Or the models may be quantitatively conceived as a consequence of an understanding of the demands for logic by a computer or an understanding of conservation (mass, energy, momentum), thermodynamics and probability concepts. All subsequent data analysis is obviously conditioned by the above choice. Sometimes no choice exists (Fogel, et al, 1972) or is conditioned by tradition.

A useful basis for classifying hydrologic problems is in terms of the sequence of events occurring in a basin: Inputs, their transformation by components of the hydrologic process, state transition, and the resulting system response as manifested by changes in states (water table, soil moisture, river flow, chemical quality, etc.) and by actual outflow from the basin. Within this context: the following problems are discernable:

(a) Model choice: Models for inputs, state transition and output may be specified depending on goals of the data analysis.

(b) Parameter estimates: The choice of model gives birth to the parameters to be estimated for later use of the model.

(c) Initial condition: If a model for state transition (watershed model, aquifer model, flood routing model) has been constructed, then the initial condition over the space of the system must be estimated from the available data to permit forecasts.

(d) Input identification: To operate or exercise the state transition model, it is necessary to identify the nature of inputs in space and their past time patterns. For example, in chemical modeling of an aquifer, required knowledge includes space-time patterns of mountain-front recharge, natural recharge through beds of ephemeral streams, artificial recharge (including wastes) and precipitation.

(e) Forecasting: Having chosen the state transition model, estimated its parameters (like watershed or aquifer properties), and identified both the initial condition and past inputs, forecasts of future behavior are attempted. Note that this is the main business of science or of model building. Two classes of forecasts arise:

  (1) Short-term: Predicting aquifer water levels in the next year, flood behavior in the next few hours, or effects of watershed management within the next year are examples.

  (2) Long-term: Prediction of climatic change, of the probability of floods and droughts in the next 1-50 years and of hydraulic and chemical behavior of an aquifer in the next 50 years are examples (Gates, 1972).

(f) Validation: To validate a model, be it managerial or non-managerial, is indeed a complicated task. It is a fact that a model of an aquifer or watershed becomes a part of system I containing the water and the built or eventually-built system II for delivering the water to the consumer. If, over a period of time, the systems I and II more or less meet the requirements established by society for water, then we might conclude that the model did the job. But life is not so simple because decisions must be made with the aid of various models about system plans and designs before the investment and construction are undertaken. With this constraint we either appeal to the plausibility of the model's assumptions, or evaluate the sensitivity of the proposed system (and model!) to perturbations in assumptions, or use a formal method for model evaluation (Davis, et al, 1972b; Kisiel and Duckstein, 1972). It is our feeling that model evaluation can never be comprehensive to the point of absolute validation in all detail. The degree of confirmation should be consonant with the measures of effectiveness used to evaluate system (including model) performance.

(g) Control: To control is to manage. Spatial scales of the system to be controlled and the time horizon over which the control action is effective determine the severity of the problem. Classes of control problems are:

(1) Deterministic: In this special case no stochastic properties are attributed to the inputs, state transitions within the system and demands on the system. In a recent example, the stability of a combined surface reservoir and aquifer system, that

allows for recirculation of reclaimed waste waters into either the lake and aquifer, has been evaluated and the optimal control determined when the water demand varies in a sinusoidal manner, (as over a day or over a year), (Duckstein and Kisiel, 1972).

(2) Stochastic: A more complicated but more realistic problem formulation arises when the inputs and demands on the system are stochastic. The problem is to identify from available data the input and water demand. But the control variable is usually chosen to anticipate future water demands and so these demands must be forecasted by socio-economic methods. Obviously more work and data are required to do this.

(3) Adaptive: Sequential control over short time periods may be deterministic or stochastic. Most human activity is of this type.

(h) Presence of multiple objectives: Most control or management problems require consideration, either concurrently or sequentially, of a set of multiple objectives that pertain to costs, taxation, adequate water supply, quality of water, environmental impact, and others. Provided data analysis has permitted at least a partial resolution of many of the previous problems, then recent computer developments allow a sequential evaluation of this set of objectives, much in the manner of the actual decision process (Monarchi, 1972). Many of the above-mentioned elements of the multiple objective problem arise in the management of Arizona's aquifers, watersheds and reservoirs. The key question is whether the available data on these systems is adequate for future computer-based management of Arizona's water resources. It is our feeling that model evaluation can never be comprehensive to the point of absolute validation in all necessary detail. The degree of confirmation should be consonant with the measures of effectiveness used to evaluate system (including model) performance.

(i) Uncertainty: Uncertainties may be technological or structural and be present in all of the above problems. Technological uncertainties pertain to model structure, data, (including measurement errors and sampling errors in space-time), parameter estimation, identification of inputs and initial conditions, and forecasts. In part, modern methods for identifying systems encompass these uncertainties (Astrom and Eykhoff, 1970). Structural uncertainties pertain to our inability to know future social, economic, and political states of the world; these are more difficult to anticipate than the technological uncertainties. A common human response to uncertainty is to overdesign as in the case with the engineer or to insure ourselves commercially so that the future is pseudocertain (at least as it affects our loved ones).

By dissecting the problem areas as above, we focus on the use to which the data is to be put rather than on data analysis for its own sake, on data analysis with a limited view, or on data analysis without an awareness of the assumptions in the data or method of massaging the data.

## CLASSES OF HYDROLOGIC DATA ANALYSIS

In general, data analysis proceeds by assuming some model for the input process, the state transition, the system response, or even the instrument or measuring system that gave us the data. The classes of problems identified previously should structure our thinking about possible data analyses. Likewise, the nature of the hydrologic component to be evaluated with the aid of the data will dictate our mode of analysis.

It must be emphasized strongly that good models are a proxy for data and serve as a healthy guide to future data collection activities. This requires considerable empirical judgment preferably by a collection of professionals with diverse backgrounds. Computers can help greatly in designing more efficient hydrologic data collection systems. Thus, methods of analysis will vary for aquifer, watershed, or atmospheric processes. The space and time scales of motions within each of these systems are distinct

In the atmosphere, the synoptic pattern of winter storms is not the same as the mesoscale behavior of thunderstorms; these features dictate the kinds of chastic space-time models that might be constructed from available field data as inputs to watershed models.

In the watershed, heterogeneity of surface roughness, land slope, soil type, and vegetative patterns mitigates the tendency for water to flow in orderly ways; nonetheless, the watershed does tend to smooth out the high frequency fluctuations of precipitation and to produce memory of past precipitation by way of time delays in runoff. Thus, on small watersheds more data may be required because damping of inputs is minimal in contrast to very large river basins. Transferability of information from gaged to ungaged basins presents severe problems as a result of this heterogeneity.

In aquifers, water movement is highly damped but the system is not as easily observed as the atmosphere or watershed. If aquifers were homogenous and isotropic, then only one observation well would be necessary to establish transmissivity and the storage coefficient as aquifer properties. Of course, this is not so. Cost of well drilling limits the number of observation wells and the number of pump tests. Absence of a legal requirement to record pumpage at all wells in the State of Arizona creates problems in aquifer modeling (Gates, 1972; Lovell, et al, 1972) as has been attempted in the Tucson Basin.

Illustrative of the effect of a narrow-based approach to data analysis are the various reports of natural and mountain front recharge in the Tucson Basin (see Table 1).

## Methods of Data Analysis

We are concerned with processes in space and time. Methods of hydrologic data analysis are dictated by the size of the basin or watershed, the length of record, management goals, and computational facilities. Thus, deterministic watershed models are used to reproduce historical flow records on watersheds up to 1,000 square miles. Computer constraints limit the size of these models. Time series models of flow or precipitation are built for any size watershed, although the tendency is to build them for larger watersheds. Multivariate time series models forecast flows at different sites in rather large basins (see Table 2 for classification of models). To forecast flow properties on ungaged sites, a multiple linear regression model (Moosburner, 1970) has been used in Arizona and all other States as a basis for deciding on adequacy of the streamflow data program; these models are a concession to the impossibility of simulating large sectors of the State by a deterministic watershed model.

Methods of data analysis are either subjective or objective. Objective methods include:

(a) Correlation methods (Kendall and Stuart, 1968).

(1) Space and time series analysis (Kisiel, 1969; Jenkins and

TABLE 1.--SUMMARY OF RECHARC ..TIMATES IN TUCSON BASIN

| INVESTIGATOR | METHOD | RESULTS |
|---|---|---|
| H. C. Schwalen, University of Arizona, Agricultural Engineering Department (1960) | Water budget for winter 1959-60 on Rillito Creek. | Recharge from Rillito Creek flows computed to be 14,500 AF. This is recharge for Rillito Creek only, for a single season. Consumptive use and evaporation must be estimated; some tributary inflow to the Rillito must be estimated. The final recharge estimate is limited in its usefulness by the reliability of all these estimates. Tells us nothing about the variability in space and time. |
| W. Clyma and R. J. Shaw Progressive Agriculture in Arizona, 20:2, March-April, pp. 14-17 (1968) | Analysis of water levels along Rillito Creek and Santa Cruz River during high runoff year 1965-66. | Recharge from spring 1965 to spring 1966 was approximately 150,000 AF. Calculated from assumed storage coefficient for the aquifer and observed water level rise. Areas of equal groundwater level rise along the stream channels were delineated, thus showing areas of maximum recharge. These are areas of high transmissivity and would be good well locations. This study begins to examine the problem of spatial variability of recharge along the stream channels. This study does not attempt to deal with the time variability of recharge. |
| W. G. Matlock, H. D. Schwalen and R. J. Shaw, University of Arizona Agricultural Experiment Station Report No. 233 (1965) | Water budget for period 1961-65 in Tucson Basin. | The average annual recharge to ground water in the period 1961-65 was about 70,000 AF, varying from 56,000 to 79,000 AF. Again, evaporation must be estimated, rainfall estimates are based on inadequate data, consumptive use is estimated or calculated from crop-use factors, and thus the accuracy of the resultant recharge estimate is difficult to evaluate. Spatial variability of recharge is not approached. Some idea of the time variability of recharge is gleaned from the fluctuations observed in these four years. |

Table 1. (continued)

| INVESTIGATOR | METHOD | RESULTS |
|---|---|---|
| A. Moench and C. Kisiel, Water Resources Research, 6:4, August (1970) | Convolution relation applied to water levels in observation wells near influent streams | For assumed storage coefficient (S) of 0.15, recharge was computed to be 2,470-2,840 AF/mi.; for S = 0.10, recharge was 1,770-2,270 AF/mi. for a single flow event in Rillito Creek. The accuracy of this calculation is difficult to determine because of the constraints on any other independent method used, but the results do seem to agree well with other computed estimates. Water level observations from existing wells along influent stream channels could be used to compute recharge from flow events if homogeneous reaches of channel could be identified. Potential problems are pumping effects of other wells and seasonal or long term water level trends would have to be corrected for. This method would yield some knowledge of the spatial variability, and indirectly of the time variability, if records of sufficient length at a particular station exist. |
| K. E. Foster, University of Arizona unpublished thesis (1969) | Calculated Recharge based on estimates of stream channel permeability, average flow duration, average channel width and depth. | Average annual recharge for Tucson Basin is 70,000 AF. Spatial variability along the stream channel is ignored. Variability of recharge from year to year is smoothed by averaging; what is the range in expected value? |
| D. E. Burkham, U.S.G.S. Water Supply Paper 1939-B (1970) | Average annual volume of infiltration for the period 1936-63 computed for seven alluvial channels in Tucson Basin. Average relation between inflow rate and infiltration rate was | Average infiltration volume for the period was 47,000 AF. Annual variation is large, and ranges from near zero to more than four times the average annual volume. What confidence is attached to the estimates, and to the results? Spatial variability along an individual stream channel is not treated. What probability can |

Table 1. (continued)

| INVESTIGATOR | METHOD | RESULTS |
|---|---|---|
| | computed or estimated based on flow records. Flow duration curves derived from flow records. | be attached to the "average" volume of infiltration? |
| R. Belan (1972, in press) | Flow-net analysis of mountain-front recharge. | Tentative results indicate mountain-front recharge along the Rillito Creek amounts to approximately 1,000 AF/year. The flow-net delineates the distribution of recharge in space, but average water levels over a period of years must be used to arrive at an average annual figure. The time-variability of water levels would determine the time variability of recharge; this has not been examined. |
| C. Kisiel and D. Supkow, Progress Report No. 3 to City of Tucson (1972) | Calibration of digital model of Tucson basin aquifer. | Spatial variability of recharge among individual stream channels is considered and incorporated in model. Spatial variability is determined by model adjustments, not independent studies. Temporal variation in recharge over 19 year period (1947-66) is smoothed. Recharge (52,000 AF stream channel plus 57,000 AF mountain front) over 5 year period (1966-71) is approximately 20% higher than mean of previous 19 year period. |

TABLE 2.--CHOICE OF HYDROLOGIC MODELS FOR STREAMFLOW FORECASTING

| GENERAL CASE | DRAINAGE AREA | LENGTH OF RECORD AVAILABLE FOR MODEL | SITE | | PERIOD | USE OF MODEL (DETERMINISTIC OR STOCHASTIC) |
|---|---|---|---|---|---|---|
| 1 | SMALL(< 1,000 $MI^2$)<br>LARGE(>1,000 $MI^2$) | SMALL(< 10-YRS.)<br>MEDIUM(>10-YRS.)<br>LARGE(>50-YRS.) | SINGLE | | SINGLE → ANNUAL | SHORT TERM MEMORY (FORECASTING)<br>FLOODS<br>VOLUMES |
| 2 | SMALL<br>LARGE | SMALL<br>MEDIUM<br>LARGE | SINGLE | | MULTI - MONTHLY<br>- PERENNIAL<br>- EPHEMERAL<br>- DAILY<br>- PERENNIAL<br>- EPHEMERAL | DROUGHTS<br>QUALITY<br>LONG TERM MEMORY (FORECASTING)<br>FLOODS<br>VOLUMES |
| 3 | SMALL<br>LARGE | SMALL<br>MEDIUM<br>LARGE | MULTI | # OF GAGING STATIONS<br>SMALL (5-10)<br>LARGE (15-30) | SINGLE → ANNUAL<br>- PERENNIAL<br>- EPHEMERAL<br>MULTI - MONTHLY<br>- DAILY | DROUGHTS<br>QUALITY |

Watts, 1968).

(2) Multiple linear regression (Moosburner, 1970).

(3) Principal components analysis (Stockton and Fritts, 1971).

(4) Factor analysis.

(5) Canonical correlation.

(b) System identification (Astrom and Eykhoff, 1970).

(c) Tests for stationarity (Kolmogorov-Smirnov test, split-sample tests).

(d) Tests for independence or randomness (correlograms, runs tests).

(e) Tests for seasonality (Woodyer, et al, 1972).

(f) Event based approach (Kisiel, et al, 1971).

(g) Analysis for runs, range, and crossing levels.

(h) Tests for fitting of probability distributions (chi-squared).

In all cases, a model must be postulated to pursue the data analysis. This fact cannot be overstressed because the users of the above methods too often dismiss their responsibility in the implied models that purport to approximate natural conditions. Quite often, the analysis is terminated after the model parameters have been estimated from the available data. No further check of model predictions against another set of independent data is pursued as required in the framework of model evaluation. In the above classification, tests are given for checking various properties of time series. It cannot be over-emphasized that these tests imply a model structure; hence, a model is being used to check a model! This is acceptable only if the model reasonably corresponds to the main features of the real world. This realization can be quite important in certain decisions and not others. Experience and more research can help us to better make this judgment.

## Parameter Estimation

To repeat, models give rise to parameters. Methods of parameter estimation are dictated by available data, nature of model, and computational facilities. Table 3 outlines an important distinction between linear systems and linear regression (Astrom and Eykhoff, 1970). A system can be linear in its dynamic behavior but the mathematical form of the equation describing this behavior may be nonlinear. However, a logarithmic or other transform may induce linearity-in-the-parameters of the equation; the nonlinear function is then intrinsically linear. But as noted in the table, such transformations are not always possible.

TABLE 3.--LINEARITY OF PROCESS AND ITS PARAMETERS

| Dynamic Behavior of Process | Model Form in Relation to its Parameters | |
|---|---|---|
| Linear | Intrinsically linear | Intrinsically nonlinear |
| Nonlinear | Intrinsically linear | Intrinsically nonlinear |

Given the above distinctions, it is easily seen that parameter estimation depends crucially on process dynamics and success in linearization of the mathematical form of the process model. The most common method for parameter estimation is least squares which minimizes the sum of the squared differences between the data and model estimates. There is no guarantee that the data "conform" to the assumptions of the model; but, as a consequence of coupling the data to the model, the data tend to be straight-jacketed into the mold of the model. Thus, any subsequent graphical plots of model estimates versus the data simply give an internal consistency check on one's use of the least squares algorithm, unless the model closely approximates reality. It may be that the model and the data represent similar or homomorphic systems but other checks are necessary to more or less establish this possibility. If no waiting time is available to acquire more data for these checks, then the user seemingly appeals to the plausibility of the assumptions underlying the model and must be willing to suffer any losses generated by predictions of an inadequate model. This entire matter is part of the validation issue. But it cannot be overemphasized that least squares does not justify the model.

If the analyst is willing to make assumptions about the probability distribution of measurement errors, fixedness or randomness of model parameters and a loss function (for errors of overestimation or underestimation), then more powerful methods of parameter estimation, other than least squares, can be used (Astrom and Eykhoff, 1970). The problem, of course, is the acquisition of necessary data or experience that permit the analyst to infuse more prior knowledge into the estimation procedure (Davis, et al, 1972).

Time Series Methods

Research hydrologists have vigorously pursued the implementation of methods of time series analyses over the past 15 years. In general, the methods assume linear system dynamics (See Table 3). If this assumption does not hold, then interpretation of results is confounded. The analysis relies primarily on data-based statistical moments through the second order (mean, variance, and serial and cross-correlation structure).

Time series methods may be of value to:

(a) Detect the structure of hydrologic time series (oscillations, short-term memory, long-term memory, crossing properties, range properties, and so on).

(b) Build short-term or long-term forecast models.

(c) Give insight on the space-time sampling of large-scale natural systems.

(d) Detect nonstationarities and the effects of human control of natural or man-made systems (like reservoirs).

It is, however, important in (a) to evaluate subjectively the structure of the time series prior to the computer analysis. In this way, we should be able to determine our net gain (or loss in time!) from using the time series methods if detection is the only use to which the analysis is put. That is, what is it that we know after the analysis that we did not know beforehand?

An understanding of time series methods prompts the analyst to raise many questions, in a quantitative way, about the real world that normally would not be posed. In this sense, these methods and models condition our intuition and help us to better discern the gaps between reality and our conceptualization of that reality. For example, in trying to evaluate the results of bivariate time series analyses we must first ascertain which of the following situations holds:

(a) The input time series causes the output time series: In this case the time series are not on an equal footing. For example, stream infiltration causes water level fluctuations in nearby wells.

(b) The two time series are on an equal footing. One series does not cause the other. For example, salinity and discharge fluctuations at the same cross-section on the Colorado River.

(c) The two time series, like two streamflow or precipitation records in a region, have common causal factors (for example, forces that determine the annual hydrologic cycle).

(d) Multiple inputs, like precipitation, produce a single output, like flow at the outlet of the basin.

(e) Single input produces a multiple output (because of spatial dispersion of single-source pollutants).

(f) Combinations of the above.

If we are also willing to assume something about the hydrodynamics of the hydrologic processes that generate the above time series, then we further strengthen the interpretive use of time series methods. It is suggested that these procedures "let the data speak for themselves" more so than older hydrologic analyses.

Event Based Approach

We single out this approach because of its special merits even though it is in reality a time series method. In the absence of computers, data, and lack of quick access to what data was available, earlier generations of hydrologists relied on the more accessible annual, monthly, weekly and, in some instances, daily data. In each case, averaging has smoothed the data. For certain purposes like total volume of water supply this is adequate. But when events occur over only a fraction of the time during any of these time intervals, serious misinterpretation can result (Kisiel, *et al*, 1971).

For example, the standard intensity-duration-frequency is of value in climatic description but it is poor in defining rainfall probability. A one-inch storm in 15 minutes might also be interpreted by the classical procedures as being indicative of other events that really didn't occur in the storm: 4 in/hr for 15 minutes, 2 in/hr for 30 minutes, 1 in/hr for 60 minutes or 0.5 in/hr for 120 minutes. In the event-based approach, we count only those events that actually occurred; counting more than once is disallowed.

In aridlands not only is precipitation sporadic but so are flow events in most streams. By considering the number of events occurring in each season as a random variable, we can structure a probabilistic model of the sequence of events. In this way, data interpretation is strengthened.

### Regression Methods

As noted before, regression is based on least squares optimization of parameters in an assumed model. Hydrologic uses of this method are many because complexity of the system mitigates use of detailed process models over large areas and for long-term and regional forecasting. Many questions can be raised concerning the use of these methods:

(a) Is the model structure a reasonable approximation of reality?

(b) Are the variables compatible and completely specified?

(c) How sensitive is the model to intercorrelation among variables, serial correlation of residuals (errors), and normality of errors?

(d) How biased are estimates of coefficients of the model?

(e) Does the model forecast well in unmeasured regions?

(f) Does confidence in the model increase as more data is acquired?

(g) Does the model give proper forecast of the turning point or direction of change even if the squared correlation coefficient is high, like 0.95?

(h) How well does it forecast the magnitude of change?

The last two questions are crucial because the real test of a model is its forecasting ability

## Examples of Data Analysis

The following hydrologic studies, conducted primarily within Arizona, are illustrative of some of the issues discussed previously.

### Tree Ring Analysis

A recent paper by C. Stockton (1971) proposed a relation to forecast mean annual flows from tree ring sequences using a regression relation as a calibration of tree rings with flow. If this were a satisfactory relation, then the tree rings could be used as a proxy for flow. A flow record (1896-1962) is used to estimate model parameters. Then the model is used to forecast retroactively the Colorado River flows at Lee's Ferry from tree ring sequences over the past few hundred years (Fritts, 1972). The model predicts retroactively that the long-term mean annual flow of the Colorado River is 13 MAF in contrast to the 15 MAF recorded in what is termed a wet period (in the short record), and used for allocation of flow to the Lower Colorado River Basin States. Theoretical analysis by Matalas (1964) has shown that the cross correlation $\rho_{xy} \geq \frac{1}{N_1-2}$ if the calibration is to produce a net gain in information (Fisher's measure in terms of variance of mean flow). For record lengths $N_1$ = 20 and 40 years, the critical $\rho_{xy}$ are, respectively, 0.25 and 0.16. Thus, the $\rho_{xy} \simeq 0.7$ obtained by Stockton is well above these requirements, provided the model assumptions hold: namely, no serial correlation in annual flows or tree rings and adequacy of the assumed linear relation between flow and tree ring indices. But water resource planners must take cognizance of the variance of the mean annual flows (as well as other stochastic properties); if the calibration model is to forecast future variability, then $\rho_{xy} \geq 0.84$ if the record is to be extended 30 years. The conclusion is self-evident. The case for bringing into question the calibration model can further be strengthened by recognizing that for a lag-one autoregressive scheme the variance of the average of the mean annual flows can actually be <u>greater</u> than the variance of the average derived from an uncorrelated process. A crucial test for tree ring methodology may be its ability to forecast variability of flows during the design period and to provide forecasts of future critical periods. This has not been shown. The tree ring-flow relation is adequate only for averages--only one of many time series properties of interest to stochastic hydrologists. But mean value reasoning in water resource projects is dangerous. Aside from statistical issues, important questions can also be raised about the physical basis for the tree ring-flow relation.

Another example of use and misuse of regression is given in the next section.

Streamflow Gaging Stations in the Tucson Area

On the basis of recommendations made by Moosburner (1970) in a U.S. Geological Survey report on evaluation of Arizona's streamflow-data program, there was a possibility that the following important gaging stations in the Tucson Area might be discontinued:

1. Sabino Creek near Tucson--record length as of 1970 is 45 years.
2. Santa Cruz River at Tucson--record length as of 1970 is 65 years.
3. Santa Cruz River at Continental--record length as of 1970 is 25 years.
4. Santa Cruz River at Cortaro--record length as of 1970 is 25 years.
5. Rillito Creek near Tucson--record length as of 1970 is 62 years.

The primary criterion for this recommendation was the standard error of estimate as given by regression analysis of existing Arizona data. Regression analysis was used to relate each of nine statistical streamflow characteristics to nine drainage basin and climatological properties of gaged sites. The nine statistical characteristics are:

1. Mean annual discharge.
2. Standard deviation of annual discharge.
3. Mean monthly discharge (average).
4. Standard deviation of monthly discharge (average).
5. Seasonal discharge (July-September).
6. 50-year flood.

7. 7-day 50-year high flow.

8. 7-day 2-year low flow (perennial).

9. 7-day 20-year low flow (perennial).

and the nine drainage basin and climatologic properties are:

1. Total drainage area, A.

2. Mean annual precipitation, P.

3. Shape factor (of drainage basin), S.

4. Mean annual snowfall, SM.

5. 10-year 24-hour precipitation, I.

6. Main channel slope, SI.

7. Soil index, SO.

8. Mean seasonal precipitation, PS.

9. Mean basin elevation, E.

All available data was used to develop regression equations for each of the nine statistical characteristics of recorded streamflow. In turn, the regression equations were used to estimate each of the nine characteristics for each gaged stream whose record was used to derive the regression equation. The difference between the estimated characteristics (from the regression equation) and the computed characteristics (from the actual record) were used to compute the standard error of regression estimate for each characteristic. The standard error is the square root of the mean of the square of the above differences for all gaging stations included in the computations. The standard error was used in the report as the prime basis for judging whether a station should be discontinued or not. Table 2 in the Moosburner report gives accuracy goals in terms of standard error for each of the 9 statistical streamflow characteristics; the shorter the record, the higher the standard error allowed. Apparently the accuracy goals were specified by the Geological Survey in Washington, D.C.

Hydrologic Considerations: Hydrologic, statistical and economic considerations enter into our critique of the above methodology. On hydrologic grounds it may be argued that hydrologic data analysis is strongly tied to consideration of basic events such as hydrographs arising from specific storms. Stochastic and deterministic models of the rainfall-runoff process increasingly require such information. The nine statistical streamflow characteristics are in the mainstream of traditional hydrology and were born in the days when digital computers and recent mathematical developments did not exist. Their choice implicitly represents a value judgment on the potential use of the data. Modern data analysis in hydrology focuses on a continuous spectrum of descriptors, more general and inclusive than the nine, arising from the recent modeling of event-based hydrologic systems rather than averages over arbitrary time periods and with arbitrarily chosen return periods.

For the Tucson area, increasing urbanization and controlled or accidental watershed manipulation result in changes in some, if not all, of the possible statistical descriptors. Evaluation of changes in volume of runoff and in flood hydrograph properties require more data than would be the case for long time series of flow that exhibit no apparent trends. Pumping of the aquifer has definitely modified the flow regime. Recent digital computer studies on the Tucson aquifer (Second Progress Report, City of Tucson Contract, February 1972) show that stochastic variation of natural recharge from streamflows has a definite effect on better prediction of aquifer water levels. Such information is pertinent to water litigation and to anticipation of the effects of future operation and location of wells along the recharge zones in the aquifer; streamflow data are necessary for assessing the risk factor in recharge associated with future droughts. Given the apparent value of the digital model for aquifer management, modeling of the stochastic recharge is necessary at many points in the system and this requires more data than has been collected in the past. As more computer-based management of the aquifer is evolved, more data is required. As hydrologic processes become more nonstationary in time and space, more data is required. Once again, how much data is enough?

Statistical Considerations (Streamflow Gaging Stations in Tucson Area)

Statistical methods have evolved to help in answering the above question. One method relies on the variance or mean square error concept as a basis for judging the standard error of hydrologic estimate (as described in the Introduction). In the case of the Moosburner report, the basis for deriving this error is the multiplicative model as illustrated in Table 3 of that report. This model is not based on basic reasoning about hydrologic processes. It is used strictly in the spirit of classical statistical prediction and is a rather simple substitute or proxy for the very difficult task of modeling the entire rainfall-runoff process (in terms of watershed models). The approach is strictly data-based and does not hesitate to mix "apples with oranges" as is exemplified by the nine physical and climatic factors.

The multiple linear regression model is proposed as a basis for predicting any of the nine statistical streamflow parameters at ungaged sites in the region. But the Moosburner report gives no evidence of having validated this model. Instead, the report describes an internal check on computations because the comparisons are made between model estimates and actual estimates (from the record) of the nine flow parameters. One such independent check was reported recently in England by Nash, a noted Irish hydrologist, and the comparison was poor (discussed in the 1969 WMO/IHD report on hydrologic network design). In addition to the above issue, it is interesting to note how dissimilar record lengths (25 versus 62 years) were mixed to derive model coefficients (powers of the equation), how dissimilar physical and climatic factors were mixed, how nonstationary properties or trends in the record were not evaluated, how dissimilar basins and watersheds in Arizona were mixed, and how the dissimilar meteorological origins of the events over the seasons may have been disregarded.

Economic Considerations (Streamflow Gaging Stations in Tucson Area)

Probably more important than the previous issues is the social and economic value of hydrologic data. It is self-evident that not all such values can be quantified. But there is merit in quantifying whatever is feasible at this time as we, in our research, have recently pursued (Davis, et al, 1972a). The basic premise of this work is that hydrologic data has no value unless it affects a decision. We found the annual sequence of flood peaks on the Rillito were longer than necessary to arrive at a decision about installation of a flood levee (the decision was not to build the levee according to the analysis). Actual economic data, supplied by the Corps of Engineers, was used. We set up a procedure for evaluating the worth of one additional year of flood data for that particular use. After the 1929 flood of record it became increasingly apparent that each year's data would not affect the flood levee decision. Note that nothing is being said about discontinuance of the gaging station because the analysis considered only one feature of the flow record, namely the annual flood peaks, and only one use of that flood peak data. This serves to emphasize the many sides to the problem of evaluating the worth of hydrologic data.

The accuracy goals set up by the Geological Survey effectively involved some subjective measure of data worth. But it was not made explicit. Discussion of the various uses of the data was not given. Their procedure in deciding on stations to discontinue would have been rendered more palatable if some explicit measure of data worth for each of the nine streamflow characteristics were evaluated for a set of uses. Actually no one in the world has resolved such questions. Nonetheless, efforts are underway at the University of Arizona and elsewhere to get some answers to these questions (Metler, _et al_, 1972). The analysis builds on the available multiple linear regression models and now is extended to the decision situation as in culvert design, where the data would be used. This is the spirit of things to come in hydrologic research.

## Data Analysis for Digital Modeling of Small Watersheds

Simulating the hydrologic response of a watershed with the use of mathematical models is a fertile field for hydrologists. A major problem to overcome before these procedures are readily accepted concerns estimating the parameters of a model for a particular watershed. There are two general procedures for evaluating these parameters. One approach is to obtain values by some optimizing technique involving the computer. The other method makes use of prior knowledge gained from studying watershed subsystems (infiltration, overland flow, interception, etc.) with parameters being inserted into the model as known quantities.

Since the first approach requires simultaneous records of inputs and outputs, for example, rainfall and surface runoff, it cannot be used on ungaged watersheds. If more than 5 or 6 parameters are to be optimized, computer time may approach real operating time of a runoff event from a small watershed (Ibbitt, 1970; O'Hayre, 1972). In addition to high computer costs for obtaining parameters that result in the best match of observed and estimated runoff hydrographs, uniqueness of parameter values may present a problem (Ibbitt and O'Donnell, 1971). An additional difficulty arises when the optimized parameters do not have any physical significance, an all too often occurrence in watershed models. Under such conditions, it is nearly impossible to use these models for predicting the effects of land use treatments or modifications on runoff.

The second method for estimating parameters is not without its difficulties. Sensitivity analyses by Huggins and Monke (1966) indicate that their model is most responsive to the infiltration parameters and to the hydraulic roughness and the surface retention volume parameters in the roughness function. Making prior estimates of parameters that describe these processes may introduce considerable error. Nevertheless this approach is the only one open for simulating the hydrologic response from ungaged watersheds.

In some models data are required that are not readily available from experimental watersheds, to say nothing of operational watersheds. For example, knowledge of the soil moisture content at all times is a necessary condition for such models as the one developed by the Agricultural Research Service of the U.S. Department of Agriculture (Holtan and Lopez, 1971). Soil moisture accounting is usually accomplished by keeping record of precipitation and estimating the evapotranspiration and other water losses between rainfall events. Since initial infiltration rates are highly dependent on the water content of soils, estimating this value prior to the occurrence of a particular event can lead to a serious error in predicting surface runoff.

The development of watershed models is in its infancy. There is every reason to believe that future efforts will lead to solutions of value to both researcher and practitioner.

Data Analysis for Digital Modeling of the Tucson Aquifer

In preparing either an analog or digital computer model of an aquifer, considerable evaluation is necessary in the use of data on recharge, pumpage, transmissivity, storage coefficient and water level. Information from various sources like the U.S. Geological Survey and the Department of Soil, Water and Engineering of the University of Arizona allowed development of two digital models: The large-scale model with each node covering a 1/4 square mile area and the small-scale model with each node covering a one square mile area. The initial purpose of these models was to evaluate the worth of data on the earlier-mentioned variables (see Gates and Kisiel, 1971). More recently, these models have been adapted to the operational needs of the City of Tucson (Kisiel & Supkow, 19

The models are developed to forecast water levels at each node over a one-year time period. Before future forecasts can be made, the model must be calibrated. As in watershed modeling two approaches are possible: To optimize automatically the parameter values at each node or to use a trial and error procedure. The latter has been employed by Gates (1972). The results are summarized in Table 4. Even with the sizable volume of data available on the Tucson aquifer, no perfect matching of observed and model estimated water levels is achieved. This is not unusual for there is no unique solution just as in watershed models. The average 5-6 foot nodal error is the best achievable after 38 computer runs no matter what adjustments are made in the parameters. The parameters so estimated are presumed to define field conditions and become the starting point for forecasting aquifer behavior in response to proposed pumping and recharge policies. At present, no theoretical method is available to give reliability of these forecasts. We must rely on the confidence that we can place in our data and computer model.

## Conclusions

Modern methods of data analysis capitalize on the computational speed of digital computers. But these tools are no substitute for careful evaluation of the underlying assumptions in the tools and models. Such awareness allows the researcher and practitioner to make allowances for the gaps between model and reality. This is the essence of professional practice. In this sense, computers can strengthen such practice. Because of the pressures for sounder environmental management, the digital computer can integrate the many steps, outlined

TABLE 4.--COMPARISON OF INITIAL AND FINAL ERRORS IN THE LARGE-SCALE AND SMALL-SCALE MODELS (FROM GATES, 1972)

| | Average Nodal Error, in Feet | % Nodes w/ Errors < 10 Feet | % Nodes w/ Errors < 20 Feet | % Nodes w/ Errors < 30 Feet | % Nodes w/ Errors < 40 Feet | Maximum Error, in Feet |
|---|---|---|---|---|---|---|
| Large-Scale Model (1,890 nodes) | | | | | | |
| Initial Run | 24.4 | 26 | 48 | 65 | 82 | 190.6 |
| Final Run | 5.3 | 87 | 99.5 | 100 | -- | 28.6 |
| Small-Scale Model (509 nodes) | | | | | | |
| Initial Run | 6.2 | 80 | 89 | 99.5 | 100 | 34.4 |
| Final Run | 5.7 | 82 | 99.5 | 100 | -- | 24.7 |

in this paper, between data collection and its ultimate use in evaluating a large number of alternatives for managing aquifers, watersheds, and streams.

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# A SOLUTION TO SMALL SAMPLE BIAS IN FLOOD ESTIMATION

by

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ABSTRACT

In order to design culverts and bridges, it is necessary to compute an estimate of the design flood. Regionalization of flows by regression analysis is currently the method advocated by the U.S. Geological Survey to provide an estimate of the culvert and bridge design floods. In the regression analysis a set of simultaneous equations is solved for the regression coefficients which will be used to compute a design flood prediction for a construction site. The dependent variables in the set of simultaneous equations are the historical estimates of the design flood computed from the historical records of gaged sites in a region. If a log normal distribution of the annual peak flows is assumed, then the historical estimate of the design flood for site i may be computed by the normal as

$$\log Q_{d,i} = \bar{x}_i + k_d \hat{s}_i .$$

However because of the relatively small samples of peak flows commonly used in this problem, this paper shows that the historical estimate should be computed by

$$\log Q_{d,i} = \bar{x}_i + t_{d,n-1} \sqrt{\frac{n+1}{n}} \; \hat{s}_i$$

where $t_{d,n-1}$ is obtained from tables of the Student's t. This t-estimate when used as input to the regression analysis provides a more realistic prediction in light of the small sample size, than the estimate yielded by the normal.

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## INTRODUCTION

In the construction of culverts and bridges, the design is specified by the design flood. If the design flood were the 25 year flood, then the culvert or bridge is to pass that flood which is equaled or exceeded once in 25 years. The prediction of the design flood for a construction site may be found by some method which considers the estimates of the design flood computed from the records at the gaged sites within the region.

Regression analysis is one such method of regionalization which may be used to predict the design flood at a construction site. The USGS (Thomas and Benson, 1970) has assumed that the design flood, a streamflow characteristic, is related to the basin and climatic characteristics for the drainage site by the formula

$$Q_d = aA^{b_1} E^{b_2} Si^{b_3} \tag{1}$$

for example. In (1) $Q_d$ represents the design flood, A represents the drainage in square miles, E represents the elevation in thousands of feet, and Si represents a soil index figure. By taking logrithms in (1) we have a linear relationship of topographical and climatic characteristics with a streamflow characteristic

$$\log Q_d = b_0 + b_1 \log A + b_2 \log E + b_3 \log Si \ . \tag{2}$$

Consider for example that there are m gaged sites in a region where the predictor variables are area, elevation, and soil index. Then the set of simultaneous equations for the 25 year flood would be

$$\log Q_{25,1} = b_0 + b_1 \log A_1 + b_2 \log E_1 + b_3 \log Si_1$$

$$\log Q_{25,2} = b_0 + b_1 \log A_2 + b_2 \log E_2 + b_3 \log Si_2 \qquad (3)$$

$$\vdots$$

$$\log Q_{25,m} = b_0 + b_1 \log A_m + b_2 \log E_m + b_3 \log Si_m.$$

The equations in (3) could then be solved for the $b_i$'s. For a construction site the topographical and climatic characteristics for that site along with the regression coefficients, the $b_i$'s, would provide an estimate of the design flood for the ungaged construction site. The procedure could be repeated and a set of regression coefficients for a region could be computed for the 2,5,10,25,50, and 100 year floods or any other streamflow characteristic. This method of regionalization permits one from the data already on hand to infer a streamflow characteristic, e.g., the 25 year design flood given the readily measurable topographical and climatic characteristics of a particular site.

It is obvious here that the determination of the prediction of the design flood for the construction site depends on the regression coefficients. The regression coefficients depend on the estimates of the design flood computed from the historical record. This paper is concerned with the computation of the historical estimate which must serve as input to the regression analysis.

## THE HISTORICAL ESTIMATE

If a log normal distribution of the annual peak flows is assumed for a gaged site i, then the logs of the peak flows are distributed normally. Hence the sample mean for site i is

$$\bar{x}_i = \sum_j \frac{x_{ij}}{n_i} \tag{4}$$

and the sample variance is

$$\hat{s}_i = \sum_j \frac{(x_{ij} - \bar{x}_i)^2}{n_i - 1} \tag{5}$$

where $x_{ij}$ is the log of the discharge for the $j^{th}$ year of record for the $i^{th}$ site and $n_i$ is the length in years of record for the $i^{th}$ site. Thus the historical estimate of the streamflow characteristic, say the 25 year flood, might be computed by

$$\log Q_{25,i} = \bar{x}_i + k_{25}\,\hat{s}_i \tag{6}$$

where $k_{25}$ is the number of standard normal deviates from the mean and may be obtained from tables of the standard normal distribution for the probability level 1/25 = .04 ($k_{25}$ = 1.75). However, the small sample sizes commonly encountered in streamflow estimation introduces a bias into the estimate (6). It is shown in Appendix A that

$$\log Q_{d,i} = \bar{x}_i + t_{d,n_i-1}\sqrt{\frac{n_i+1}{n_i}}\;\hat{s}_i \tag{7}$$

provides an estimate from the historical record which removes the bias introduced by the small sample size. In (7) $Q_{d,i}$, $\bar{x}_i$, $n_i$, and $\hat{s}_i$ are as previously defined and $t_{d,n_i-1}$ is found in tables of the Student's t with $n_i - 1$ degrees of freedom and the cumulative probability level equal to 1/d where d is the recurrence interval of the design flood. It will be shown in this paper that the estimate as computed by (6) will always underestimate the value computed by (7) for the design floods commonly used in construction specifications. Because the historical estimate must serve as input to the regression analysis for defining the regression coefficients, the regression predictions based on the inputs by (6) will always be less than the regression predictions based on the inputs from (7). The amount of underestimation for the design floods is defined as being significant in terms of percentage error.

## EXAMPLE AND RESULTS

The example for this paper is the plains region in Missouri and the sites of record are restricted to those areas of less than 30 square miles. The topographical and climatic characteristics are listed in Table 1 and further details can be found in Homyk (1971). The historical records were provided by the U.S. Geological Survey.

Table 2 gives the coefficient for the sample variance when using (7). This table is used to compute the estimate of a design flood by (7). For example suppose the sample mean is 5.85 in log units and the sample standard deviation is .69 in log units. Then the estimate of the 25 year flood (1/d=1/25=.04) for a record length of 16 years is from Table 2

$$\log_e Q_{25} = 5.85 + (2.099)(.69) = 7.3$$

$$Q_{25} = 1480 \text{ ft}^3/\text{sec.}$$

The normal estimate would be

$$\log_e Q_{25} = 5.85 + (1.75)(.69) = 7.05$$
$$Q_{25} = 1153 \text{ ft}^3/\text{sec.}$$

Thus the percentage error based on the normal is $\frac{1480-1153}{1153} \cdot 100 = 28\%$.
That is, the normal because of the small sample size underestimates the 25 year design flood by as much as 28%.

Tables 3,4,5,6,7, and 8 were computed in the following manner. The regression coefficients for a given design flood were computed using the inputs from 30 sites by (6). These coefficients were used to compute a prediction for each of the 30 sites. This was repested using the t-estimates as input and the percentage error of the normal versus the t was computed as listed in Table 9. Table 9 also includes the average percentage error of the normal versus the t estimates computed from the historical record. The regression analysis gives a smoothing effect as the percentage error for a given flood is less for the regression prediction than for the historical estimation.

Table 9 also lists the $R^2$ for each analysis. $R^2$ is a number between zero and one; the closer $R^2$ is to one, the better is the fit of the regression line. It will be noticed that the $R^2$ for the normal case is slightly better than for the t inputs. It will be also noticed that as the recurrence interval of the flood increases the $R^2$ decreases. These results indicate that although one measure of our confidence, $R^2$, in the predictions computed with the t inputs is slightly poorer than the predictions computed by the normal inputs, the t inputs provide a significantly more conservative, i.e., realistic, prediction than the normal. The conclusion is that in order to compute an estimate of the design flood for construction projects the estimate should include the effect of the small sample size, and the use of the Student's t accomplishes this task.

TABLE 1

Site index with associated topographical and climatic characteristics

| Site Index | Area | Elevation | Forest Cover | Precipitation | 2 yrs/24 hrs. Precipitation | Soil Index |
|---|---|---|---|---|---|---|
| 54977.0 | 2.4 | .8 | 4.3 | 11.0 | 3.3 | 2.4 |
| 55020.0 | 31.0 | .7 | 7.9 | 11.0 | 3.4 | 2.5 |
| 55136.5 | 3.1 | .6 | 23.5 | 11.0 | 3.4 | 2.6 |
| 68200.0 | 6.0 | 1.1 | 2.0 | 9.0 | 3.3 | 3.2 |
| 68945.0 | 20.0 | 1.0 | 11.0 | 11.0 | 3.5 | 3.5 |
| 69013.0 | .1 | .9 | 14.8 | 11.0 | 3.3 | 2.4 |
| 69075.0 | 16.6 | .9 | 7.1 | 13.0 | 3.5 | 2.8 |
| 69102.0 | 1.0 | .8 | 1.0 | 13.0 | 3.4 | 3.5 |
| 54951.0 | .7 | .6 | 23.3 | 11.0 | 3.3 | 2.6 |
| 55030.0 | 2.6 | .8 | 7.8 | 11.0 | 3.4 | 2.2 |
| 55136.0 | 1.5 | .6 | 36.3 | 11.0 | 3.4 | 2.6 |
| 55142.0 | .5 | .9 | 1.0 | 11.0 | 3.4 | 2.2 |
| 68160.0 | 4.9 | 1.1 | 6.0 | 9.0 | 3.4 | 3.5 |
| 68203.0 | 1.3 | 1.1 | 1.0 | 9.0 | 3.3 | 3.2 |
| 68210.0 | 2.7 | 1.0 | 1.0 | 9.0 | 3.4 | 3.1 |
| 68961.8 | .4 | .9 | 1.0 | 9.0 | 3.3 | 2.4 |
| 68965.0 | 5.6 | 1.0 | 6.8 | 9.0 | 3.3 | 2.4 |
| 68967.0 | .8 | 1.0 | 2.5 | 9.0 | 3.4 | 2.4 |
| 68972.0 | 4.7 | 1.0 | 17.4 | 9.0 | 3.3 | 2.4 |
| 68996.0 | .2 | .8 | 1.0 | 9.0 | 3.3 | 2.4 |
| 69025.0 | 2.5 | .9 | 5.4 | 11.0 | 3.4 | 2.4 |
| 69028.0 | 1.0 | .8 | 2.0 | 11.0 | 3.4 | 2.4 |
| 69047.0 | 1.0 | .9 | 21.4 | 11.0 | 3.3 | 2.8 |
| 69057.0 | .8 | .7 | 5.6 | 11.0 | 3.4 | 2.8 |
| 69072.0 | 1.6 | .8 | 1.2 | 13.0 | 3.5 | 2.3 |
| 69083.0 | 1.0 | .8 | 9.0 | 13.0 | 3.5 | 2.8 |
| 69085.0 | 2.9 | .8 | 2.3 | 13.0 | 3.5 | 2.8 |
| 69094.0 | .3 | .7 | 6.7 | 13.0 | 3.5 | 3.5 |
| 69097.0 | .5 | .8 | 4.0 | 13.0 | 3.5 | 3.5 |
| 69102.5 | .6 | .8 | 11.5 | 13.0 | 3.5 | 3.5 |

TABLE 2

Values of $t_{d,n-1}\sqrt{\frac{n+1}{n}}$ for use in (7).

| Record length | 2 yr. flood | 5 yr. | 10 yr. | 25 yr. | 50 yr. | 100 yr. |
|---|---|---|---|---|---|---|
| 6 | 0.000 | 1.107 | 1.658 | 2.343 | 2.842 | 3.354 |
| 7 | 0.000 | 1.096 | 1.632 | 2.282 | 2.747 | 3.217 |
| 8 | 0.000 | 1.088 | 1.612 | 2.238 | 2.677 | 3.117 |
| 9 | 0.000 | 1.081 | 1.596 | 2.204 | 2.624 | 3.041 |
| 10 | 0.000 | 1.076 | 1.584 | 2.177 | 2.582 | 2.981 |
| 11 | 0.000 | 1.072 | 1.574 | 2.155 | 2.549 | 2.933 |
| 12 | 0.000 | 1.069 | 1.566 | 2.138 | 2.521 | 2.894 |
| 13 | 0.000 | 1.066 | 1.559 | 2.123 | 2.499 | 2.862 |
| 14 | 0.000 | 1.063 | 1.553 | 2.110 | 2.479 | 2.834 |
| 15 | 0.000 | 1.061 | 1.548 | 2.099 | 2.463 | 2.810 |
| 16 | 0.000 | 1.059 | 1.543 | 2.090 | 2.449 | 2.790 |
| 17 | 0.000 | 1.057 | 1.540 | 2.082 | 2.436 | 2.772 |
| 18 | 0.000 | 1.056 | 1.536 | 2.074 | 2.425 | 2.756 |
| 19 | 0.000 | 1.054 | 1.533 | 2.068 | 2.415 | 2.743 |
| 20 | 0.000 | 1.053 | 1.530 | 2.062 | 2.406 | 2.730 |
| 21 | 0.000 | 1.052 | 1.528 | 2.057 | 2.398 | 2.719 |
| 22 | 0.000 | 1.051 | 1.526 | 2.052 | 2.391 | 2.709 |
| 23 | 0.000 | 1.050 | 1.524 | 2.048 | 2.385 | 2.700 |
| 24 | 0.000 | 1.049 | 1.522 | 2.044 | 2.379 | 2.692 |
| 25 | 0.000 | 1.049 | 1.520 | 2.040 | 2.373 | 2.684 |
| 26 | 0.000 | 1.048 | 1.518 | 2.037 | 2.368 | 2.677 |
| 27 | 0.000 | 1.047 | 1.517 | 2.034 | 2.364 | 2.671 |
| 28 | 0.000 | 1.047 | 1.516 | 2.031 | 2.359 | 2.664 |
| 29 | 0.000 | 1.046 | 1.514 | 2.028 | 2.356 | 2.659 |
| 30 | 0.000 | 1.046 | 1.513 | 2.026 | 2.352 | 2.654 |
| 31 | 0.000 | 1.045 | 1.512 | 2.024 | 2.349 | 2.650 |
| 32 | 0.000 | 1.045 | 1.511 | 2.022 | 2.345 | 2.644 |
| 33 | 0.000 | 1.044 | 1.510 | 2.020 | 2.342 | 2.641 |
| 34 | 0.000 | 1.044 | 1.509 | 2.018 | 2.340 | 2.636 |
| 35 | 0.000 | 1.044 | 1.508 | 2.016 | 2.337 | 2.633 |
| 36 | 0.000 | 1.043 | 1.508 | 2.014 | 2.334 | 2.629 |
| 37 | 0.000 | 1.043 | 1.507 | 2.013 | 2.332 | 2.626 |
| 38 | 0.000 | 1.042 | 1.506 | 2.011 | 2.329 | 2.623 |
| 39 | 0.000 | 1.042 | 1.505 | 2.010 | 2.327 | 2.620 |
| 40 | 0.000 | 1.042 | 1.505 | 2.008 | 2.326 | 2.617 |
| 41 | 0.000 | 1.042 | 1.504 | 2.007 | 2.324 | 2.615 |
| 42 | 0.000 | 1.041 | 1.503 | 2.006 | 2.322 | 2.612 |
| 43 | 0.000 | 1.041 | 1.503 | 2.005 | 2.320 | 2.610 |
| 44 | 0.000 | 1.041 | 1.502 | 2.004 | 2.318 | 2.607 |
| 45 | 0.000 | 1.041 | 1.502 | 2.002 | 2.317 | 2.605 |
| 46 | 0.000 | 1.040 | 1.501 | 2.002 | 2.316 | 2.603 |
| 47 | 0.000 | 1.040 | 1.501 | 2.001 | 2.314 | 2.601 |
| 48 | 0.000 | 1.040 | 1.500 | 2.000 | 2.313 | 2.599 |
| 49 | 0.000 | 1.040 | 1.500 | 1.999 | 2.311 | 2.598 |
| 50 | 0.000 | 1.040 | 1.500 | 1.998 | 2.310 | 2.596 |
| ⋮ | | | | | | |
| ∞ | 0.000 | 0.849 | 1.282 | 1.750 | 2.055 | 2.326 |

TABLE 3

Comparison of 2 year flood by (6) and (7)

| Site Index | Record Length | Normal Regression Prediction | Student's t Regression Prediction |
|---|---|---|---|
| 549770 | 13 | 475 | 475 |
| 550200 | 25 | 1616 | 1616 |
| 551365 | 13 | 343 | 343 |
| 682000 | 19 | 866 | 866 |
| 689450 | 17 | 2113 | 2113 |
| 690130 | 13 | 133 | 133 |
| 690750 | 14 | 2190 | 2190 |
| 691020 | 13 | 233 | 233 |
| 549510 | 12 | 148 | 148 |
| 550300 | 12 | 578 | 578 |
| 551360 | 13 | 240 | 240 |
| 551420 | 11 | 236 | 236 |
| 681600 | 18 | 811 | 811 |
| 682030 | 15 | 333 | 333 |
| 682100 | 18 | 426 | 426 |
| 689618 | 13 | 136 | 136 |
| 689650 | 12 | 976 | 976 |
| 689670 | 13 | 299 | 299 |
| 689720 | 13 | 987 | 987 |
| 689960 | 13 | 76 | 76 |
| 690250 | 12 | 643 | 643 |
| 690280 | 13 | 272 | 272 |
| 690470 | 13 | 385 | 385 |
| 690570 | 13 | 176 | 176 |
| 690720 | 13 | 437 | 437 |
| 690830 | 13 | 348 | 348 |
| 690850 | 13 | 553 | 553 |
| 690940 | 10 | 109 | 109 |
| 690970 | 13 | 180 | 180 |
| 691025 | 10 | 217 | 217 |

## TABLE 4

### Comparison of 5 year flood by (6) and (7)

| Site Index | Record Length | Normal Regression Prediction | Student's t Regression Prediction |
|---|---|---|---|
| 549770 | 13 | 933 | 929 |
| 550200 | 25 | 3390 | 3362 |
| 551365 | 13 | 812 | 811 |
| 682000 | 19 | 2280 | 2262 |
| 689450 | 17 | 4325 | 4288 |
| 690130 | 13 | 232 | 232 |
| 690750 | 14 | 3309 | 3282 |
| 691020 | 13 | 531 | 529 |
| 549510 | 12 | 357 | 357 |
| 550300 | 12 | 1023 | 1019 |
| 551360 | 13 | 560 | 560 |
| 551420 | 11 | 437 | 436 |
| 681600 | 18 | 2161 | 2146 |
| 682030 | 15 | 929 | 924 |
| 682100 | 18 | 1236 | 1228 |
| 689618 | 13 | 359 | 359 |
| 689650 | 12 | 2065 | 2051 |
| 689670 | 13 | 673 | 671 |
| 689720 | 13 | 1987 | 1977 |
| 689960 | 13 | 221 | 221 |
| 690250 | 12 | 1136 | 1131 |
| 690280 | 13 | 562 | 560 |
| 690470 | 13 | 721 | 720 |
| 690570 | 13 | 433 | 433 |
| 690720 | 13 | 706 | 702 |
| 690830 | 13 | 590 | 590 |
| 690850 | 13 | 998 | 993 |
| 690940 | 10 | 253 | 254 |
| 690970 | 13 | 385 | 385 |
| 691025 | 10 | 437 | 437 |

TABLE 5

Comparison of 10 year flood by (6) and (7)

| Site Index | Record Length | Normal Regression Prediction | Student's t Regression Prediction |
|---|---|---|---|
| 549770 | 13 | 1343 | 1427 |
| 550200 | 25 | 4810 | 5024 |
| 551365 | 13 | 1095 | 1164 |
| 682000 | 19 | 3314 | 3491 |
| 689450 | 17 | 5767 | 6027 |
| 690130 | 13 | 304 | 330 |
| 690750 | 14 | 4581 | 4798 |
| 691020 | 13 | 831 | 889 |
| 549510 | 12 | 480 | 516 |
| 550300 | 12 | 1423 | 1511 |
| 551360 | 13 | 735 | 786 |
| 551420 | 11 | 674 | 724 |
| 681600 | 18 | 2943 | 3105 |
| 682030 | 15 | 1401 | 1492 |
| 682100 | 18 | 1889 | 2003 |
| 689618 | 13 | 554 | 596 |
| 689650 | 12 | 2824 | 2978 |
| 689670 | 13 | 953 | 1019 |
| 689720 | 13 | 2571 | 2715 |
| 689960 | 13 | 345 | 373 |
| 690250 | 12 | 1592 | 1690 |
| 690280 | 13 | 845 | 903 |
| 690470 | 13 | 930 | 994 |
| 690570 | 13 | 622 | 667 |
| 690720 | 13 | 1094 | 1166 |
| 690830 | 13 | 813 | 869 |
| 690850 | 13 | 1491 | 1582 |
| 690940 | 10 | 359 | 388 |
| 690970 | 13 | 555 | 596 |
| 691025 | 10 | 592 | 635 |

TABLE 6

Comparison of 25 year flood by (6) and (7)

| Site Index | Record Length | Normal Regression Prediction | Student's t Regression Prediction |
|---|---|---|---|
| 549770 | 13 | 1645 | 1848 |
| 550200 | 25 | 8206 | 8882 |
| 551365 | 13 | 2377 | 2685 |
| 682000 | 19 | 4470 | 5065 |
| 689450 | 17 | 8143 | 8976 |
| 690130 | 13 | 347 | 411 |
| 690750 | 14 | 4863 | 5263 |
| 691020 | 13 | 867 | 978 |
| 549510 | 12 | 855 | 982 |
| 550300 | 12 | 2197 | 2488 |
| 551360 | 13 | 1623 | 1858 |
| 551420 | 11 | 940 | 1096 |
| 681600 | 18 | 5040 | 5793 |
| 682030 | 15 | 1962 | 2287 |
| 682100 | 18 | 3678 | 4274 |
| 689618 | 13 | 1016 | 1211 |
| 689650 | 12 | 4277 | 4854 |
| 689670 | 13 | 1911 | 2271 |
| 689720 | 13 | 3911 | 4452 |
| 689960 | 13 | 740 | 892 |
| 690250 | 12 | 2138 | 2424 |
| 690280 | 13 | 1335 | 1538 |
| 690470 | 13 | 1012 | 1156 |
| 690570 | 13 | 1160 | 1343 |
| 690720 | 13 | 1414 | 1597 |
| 690830 | 13 | 1064 | 1214 |
| 690850 | 13 | 1902 | 2125 |
| 690940 | 10 | 568 | 662 |
| 690970 | 13 | 739 | 853 |
| 691025 | 10 | 786 | 905 |

TABLE 7

Comparison of 50 year flood by
(6) and (7)

| Site Index | Record Length | Normal Regression Prediction | Student's t Regression Prediction |
|---|---|---|---|
| 549770 | 13 | 2060 | 2894 |
| 550200 | 25 | 10196 | 11361 |
| 551365 | 13 | 3054 | 3547 |
| 682000 | 19 | 5987 | 9458 |
| 689450 | 17 | 10477 | 12306 |
| 690130 | 13 | 453 | 642 |
| 690750 | 14 | 5865 | 6334 |
| 691020 | 13 | 1058 | 1820 |
| 549510 | 12 | 1089 | 1651 |
| 550300 | 12 | 2828 | 2821 |
| 551360 | 13 | 2107 | 2451 |
| 551420 | 11 | 1238 | 1240 |
| 681600 | 18 | 6972 | 9193 |
| 682030 | 15 | 2686 | 4259 |
| 682100 | 18 | 5132 | 6066 |
| 689618 | 13 | 1416 | 1732 |
| 689650 | 12 | 5735 | 6970 |
| 689670 | 13 | 2713 | 2547 |
| 689720 | 13 | 5257 | 6391 |
| 689960 | 13 | 1040 | 1274 |
| 690250 | 12 | 2754 | 2975 |
| 690280 | 13 | 1741 | 1885 |
| 690470 | 13 | 1284 | 2081 |
| 690570 | 13 | 1519 | 1894 |
| 690720 | 13 | 1763 | 1600 |
| 690830 | 13 | 1337 | 1454 |
| 690850 | 13 | 2352 | 2551 |
| 690940 | 10 | 726 | 970 |
| 690970 | 13 | 937 | 1251 |
| 691025 | 10 | 995 | 1328 |

TABLE 8

Comparison of 100 year flood by (6) and (7)

| Site Index | Record Length | Normal Regression Prediction | Student's t Regression Prediction |
|---|---|---|---|
| 549770 | 13 | 2959 | 3860 |
| 550200 | 25 | 12147 | 14382 |
| 551365 | 13 | 3666 | 4765 |
| 682000 | 19 | 10356 | 13883 |
| 689450 | 17 | 13452 | 16629 |
| 690130 | 13 | 626 | 915 |
| 690750 | 14 | 6611 | 7817 |
| 691020 | 13 | 1852 | 2485 |
| 549510 | 12 | 1667 | 2295 |
| 550300 | 12 | 2868 | 3700 |
| 551360 | 13 | 2505 | 3348 |
| 551420 | 11 | 1228 | 1686 |
| 681600 | 18 | 10111 | 13762 |
| 682030 | 15 | 4551 | 6477 |
| 682100 | 18 | 6540 | 9023 |
| 689618 | 13 | 1770 | 2584 |
| 689650 | 12 | 7434 | 9777 |
| 689670 | 13 | 2634 | 3736 |
| 689720 | 13 | 6798 | 9000 |
| 689960 | 13 | 1289 | 1927 |
| 690250 | 12 | 3045 | 3963 |
| 690280 | 13 | 1902 | 2562 |
| 690470 | 13 | 2125 | 2907 |
| 690570 | 13 | 1928 | 2656 |
| 690720 | 13 | 1582 | 2016 |
| 690830 | 13 | 1450 | 1915 |
| 690850 | 13 | 2589 | 3278 |
| 690940 | 10 | 968 | 1362 |
| 690970 | 13 | 1258 | 1737 |
| 691025 | 10 | 1338 | 1839 |

TABLE 9

A Summary of the Regression Analyses

| Year Flood | $R^2$ for Normal | $R^2$ for t | Average Percent Error based on Normal | |
|---|---|---|---|---|
| | | | Historical | Regression |
| 2 | .92 | .92 | 0.0 | 0.0 |
| 5 | .889 | .889 | 2.5 | - .3 |
| 10 | .825 | .809 | 11.3 | 6.5 |
| 25 | .739 | .705 | 21.9 | 14.4 |
| 50 | .683 | .627 | 33.0 | 24.5 |
| 100 | .635 | .556 | 47.8 | 34.4 |

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## APPENDIX A

### The t-estimate

It can be shown (Hogg & Craig 1971) that

$$\frac{ns^2}{\sigma^2} \sim \chi^2_{(n-1)}$$

where

$$s^2 = \frac{\Sigma(x_i - \bar{x})^2}{n}$$

and $x_i$ = log $y_i$ for $y_i$ the ith year maximum annual flow. Now since

$$\hat{s}^2 = \frac{\Sigma(x_i - \bar{x})}{n - 1}$$

then

$$\frac{(n - 1)\hat{s}^2}{\sigma^2} \sim \chi^2_{(n-1)}$$

Letting r = n-1 (the degrees of freedom) then

$$\frac{r\hat{s}^2}{\sigma^2} \sim \chi^2_r$$

Since $X \sim N(\mu, \sigma^2)$

and $X \sim N(\mu \ \frac{\sigma^2}{n})$

then

$$\bar{U} = X - \bar{X} \sim N(0, \sigma^2 + \frac{\sigma^2}{n}) \tag{A.1}$$

and the standard normalizing $\bar{U}$ to Z

$$Z \sim N(0,1).$$

Thus from (1) $Z = z_i = \dfrac{x_i - \bar{x} - 0}{\sqrt{\sigma^2 + \frac{\sigma^2}{n}}} = \dfrac{x_i - \bar{x}}{\sqrt{\sigma^2 + \frac{\sigma^2}{n}}}$ (A.2)

Since the Student's t distribution is defined by

$$\frac{Z}{\sqrt{V/r}} \sim t_r \tag{A.3}$$

where $Z \sim N(0,1)$ and $V \sim \chi_r^2$ then from (A.2) and (A.3)

$$\frac{\dfrac{x_i - \bar{x}}{\sqrt{\sigma^2 + \dfrac{\sigma^2}{n}}}}{\sqrt{\dfrac{\dfrac{(n-1)\ \hat{s}^2}{\sigma^2}}{r}}} \sim t_r \tag{A.4}$$

Simplifying (A.4) we get

$$\frac{x_i - \bar{x}}{\hat{s}} \sqrt{\frac{r+1}{r+2}} \sim t_r$$

and the random variables

$$T = \frac{Z}{\sqrt{V/r}} = \frac{x_i - \bar{x}}{\hat{s}} \sqrt{\frac{r+1}{r+2}} \tag{A.5}$$

Thus for $P(T \geq t) = P(T \geq \dfrac{x_{50} - \bar{x}}{s} \sqrt{\dfrac{r+1}{r+2}}) = .02$,

The exact 50-year flow for small samples n is

$$\frac{x_{50} - \bar{x}}{\hat{s}} \sqrt{\frac{r+1}{r+2}} = t_{.02,r}$$

$$x_{50} = \bar{x} + t_{.02,r} \sqrt{\frac{r+2}{r+1}}\ \hat{s},$$

or

$$x_d = \bar{x} + t_{d,n-1} \sqrt{\frac{n+1}{n}}\ \hat{s} \tag{A.6}$$

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# THE CONSTRUCTION OF A PROBABILITY DISTRIBUTION FOR RAINFALL ON A WATERSHED BY SIMULATION

by

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## ABSTRACT

A raingage reading is a sample from the point rainfall population of an area. The actual average rainfall on the area (watershed) is a conditional probability distribution. For the case of thunderstorm rainfall this distribution is simulated by looking at all storms that could have produced the raingage reading. The likelihood of each storm is a function of its center depth. The amount of rain dumped on the watershed by each storm is weighted by the likelihood of its occurence and the totality of such calculations is used to produce a probability distribution of rainfall on the watershed. Examples are given to illustrate the versatility of the program and its possible use in decision analysis.

## INTRODUCTION

Areal rainfall, average rainfall over a watershed, is the consequence of a rainfall event and is estimated from raingage readings. In this paper we shall describe how simulation may be used to conduct experiments on a model of a watershed in lieu of direct experimentation with the system itself. Such experiments will be used to construct an empirical probability distribution function (p.d.f.) on a watershed.

When the number of raingages on or near a watershed is small the estimate of areal rainfall may be in considerable error, especially if the rainfall event is a thunderstorm. Watershed models such as the Kentucky model (James, 1970) and the U.S. Geological Survey's flood discharge model (Dawdy and Bergman, 1969) give runoff predictions that become increasingly uncertain as the number of raingages on a watershed declines and as the location of the raingage moves away from the approximate center of the watershed.

A raingage reading may be viewed as a sample of the rainfall on the watershed. The amount of areal rainfall on a watershed can vary between extremely wide limits while the point rainfall over a single gage remains constant. Given a rainfall model this variability in the areal rainfall for a given raingage reading and location may be quantified as a p.d.f., by simulation. The quantification of the uncertainty in areal rainfall enables the use of decision theoretic methods (Davis et al, 1972) in the solution of water resource problems where areal rainfall, as estimated by a small number of raingages, is a factor.

In this model, we are going to determine the areal rainfall over a watershed by simulating the interaction among the variables of raingage reading, raingage location, and storm location.

The practice of simulation is rapidly growing and yet many people who rely on the results of simulation studies are not aware of exactly what is involved. For this reason, we are going to describe the simulation approach to this model in some detail so that it may be seen how the individual equations and operations are built into a simulation model. This procedure must be understood to insure proper use of simulation results and is of course necessary if one wants to define their own model. Once the concept is understood, it may be readily applied to a system of most any complexity.

## THE MODEL ENVIRONMENT

We will consider the relations between a raingage reading and its location, the storm location and height, and the areal rain on the watershed. The objective in doing so might be to most accurately determine runoff, determine peak flows, to minimize some economic criteria, or many other possibilities. The objective for this paper is to determine a probability distribution for the areal rainfall on the watershed, given the raingage reading at a particular location.

If this were to be done in the field first the watershed would have to be densely gaged in order to determine the areal rainfall from each storm. Then we would have to wait until a storm came along, take measurements and record the raingage reading and the areal rainfall. When we had enough data to represent all possible storms for all possible raingage readings an empirical p.d.f. of areal rainfall conditioned on raingage reading could be calculated. If this information was desired for a location that was not gaged the study would have to be repeated. Such an approach is clearly horrendus so we turn to the technique of simulation to approximate the watershed system.

At this point it must be stressed that simulation is only as good--meaning accurate, reliable, and useful--as the model and program behind it. Too many people are mislead into believing that the techniques of simulation coupled with a large computer will somehow validate the results. The model andresultant simulation is only a conception of how the system behaves.

In our simulation we could have mimicked reality directly and allowed the simulation to proceed as follows:

1. Choose a random location and random height for a storm from some probability density function.
2. Determine the raingage reading given the distance from raingage to storm center.
3. Determine the average raingall on the watershed.

As in the field situation, we might repeat the process an indefinite number of times until the required statistics stabilized. Then we could move the raingage in our model and repeat the sequence. All that we would gain is the ability to examine one storm after another while remaining dry in the computer lab.

This approach is called Monte Carlo Simulation (Emshoff and Sisson, 1970). Values are chosen from probability density functions and are entered into the model. Repeated runs are analagous to repeated sampling in an experimental situation.

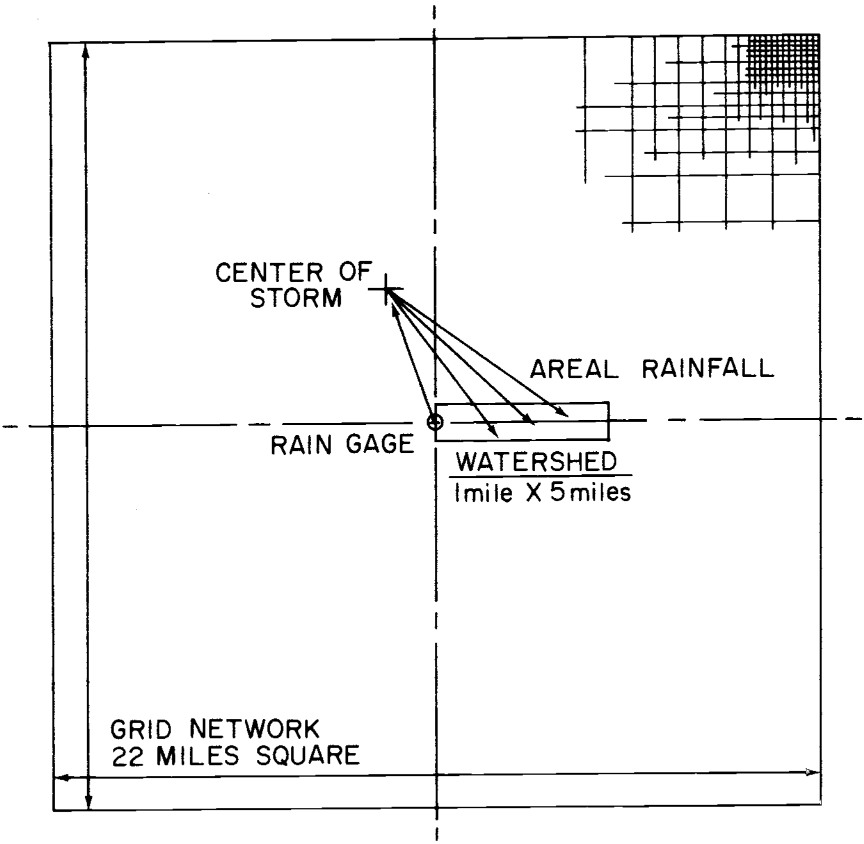
To save computer time and to make the problem more tractable, we have constrained the simulation to perform in a predefined manner. Given the same parameters, equations, and criteria, the simulation will always give the same output. This is not Monte Carlo Simulation but what is sometimes referred to as deterministic simulation.

## THE SIMULATION

We examined a rectangular watershed of dimensions one mile by five miles. First the raingage was located on the long axis of the watershed at its western boundary. Storms were allowed to occur in a square 22 miles on a side centered on the raingage.

To reduce the effort involved we constrained the sampling. First we arbitrarily chose a raingage reading. For this reading there are an infinite number of storm heights and locations that may produce the raingage reading, each of which has its own effect on the watershed of interest. To keep calculating time under control we reduced the number of allowable storm locations. The 22 mile square was divided into a grid of 64 units on a side, as shown in Figure 1. For calculating purposes storm centers were located only on grid points. It was now easy to calculate the distance from the raingage to the storm center, the height of the storm at the center and the areal rainfall on the watershed. The first calculations were made for an 8 x 8 grid. Second calculations were made for a 16 x 16 grid. The number of equally distant grid lines on the side of the 22 mile square was doubled until the variance of the empirical p.d.f. obtained from the calculations had a variance within one percent of the variance calculated with the previous grid, or until a 64 x 64 grid was used.

For each grid point we calculated the height of the storm necessary to produce the raingage reading. The areal rainfall on the watershed was also




LAYOUT OF WATERSHED

FIGURE 1

calculated. The empirical p.d.f. of areal rainfall is nota histogram of the areal rainfalls so calculated, because each storm is not equally likely. The empirical p.d.f. is a histogram of areal rainfalls weighted by the likelihood of storm from which they came.

In the calculations presented here we have assumed the watershed to increase in height from 2,500 feet of the western end to 7,500 feet at eastern. The surrounding area in the east-west and north-south directions is level with the watershed.

We have used the Fogel-Duckstein (1969) model of thunderstorm rainfall:

$$R = R_0 e^{-b[(x - x_0)^2 + (y - y_0)^2]}$$

where R is the rainfall amount at the raingage coordinates (x,y) and $R_0$ is the rainfall at the storm center $(x_0,y_0)$ and $b = 0.27\ e^{-.67R_0}$

The model is modified to reflect the higher relative likelihood of storm occuring above the base level of the watershed. Assuming a base level of 2500 feet, the likelihood previously calculated was multiplied by the factor

$$1.0 + 0.0002727(h - 2500)$$

where h is the height as defined by a function intersecting the western boundary at 2500 feet and the eastern boundary at 7500 feet.

Otherwise, the depth of storm centers was assumed to be Gumbel distributed with a mode at 0.9 inches. Rainfall at the raingage assumed to be distributed exponentially with a mean of 0.25 inches.

These assumptions are made to model a typical small watershed in the foothills of Tucson, Arizona.

For the raingage reading, the height of a storm on the grid point is determined by the numerical analysis technique of Newton-Raphson. Double integration with Simpson's Rule is used to establish the area rainfall. The rainfall at each point within the watershed is calculated by substituting the distance to the storm center in the rain equation. A schematic representation of this sequence is shown in Figure 2. Two additional steps which are discussed later are also shown.

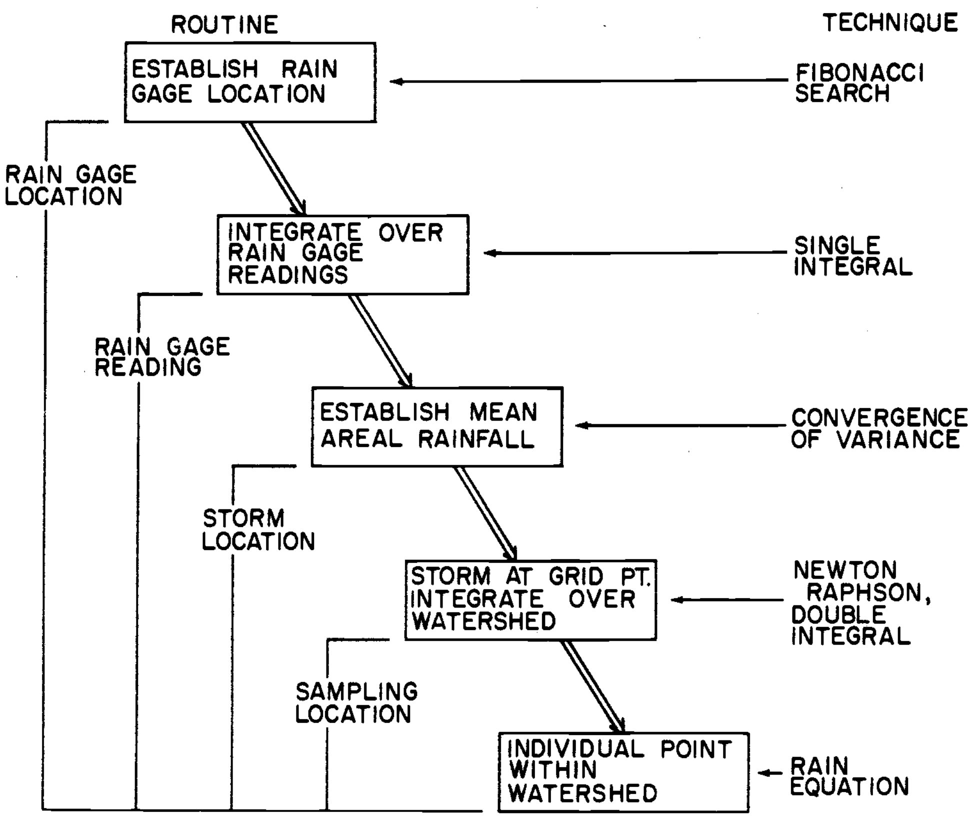
## DATA DISPLAYS

For each point on the grid that is processed, we can record and plot the storm height, the likelihood, and the areal rainfall on the watershed. This is for a single reading on the raingage at a specific location.

We arbitrarily limited the maximum storm height to 4.0 inches. Given a raingage indicating 0.4 inches located at the western boundary of the watershed a plot of the storm heights would produce a convex surface with the raingage in the center. The maximum height of 4.0 inches occurs at a distance of just under six miles from the gage.

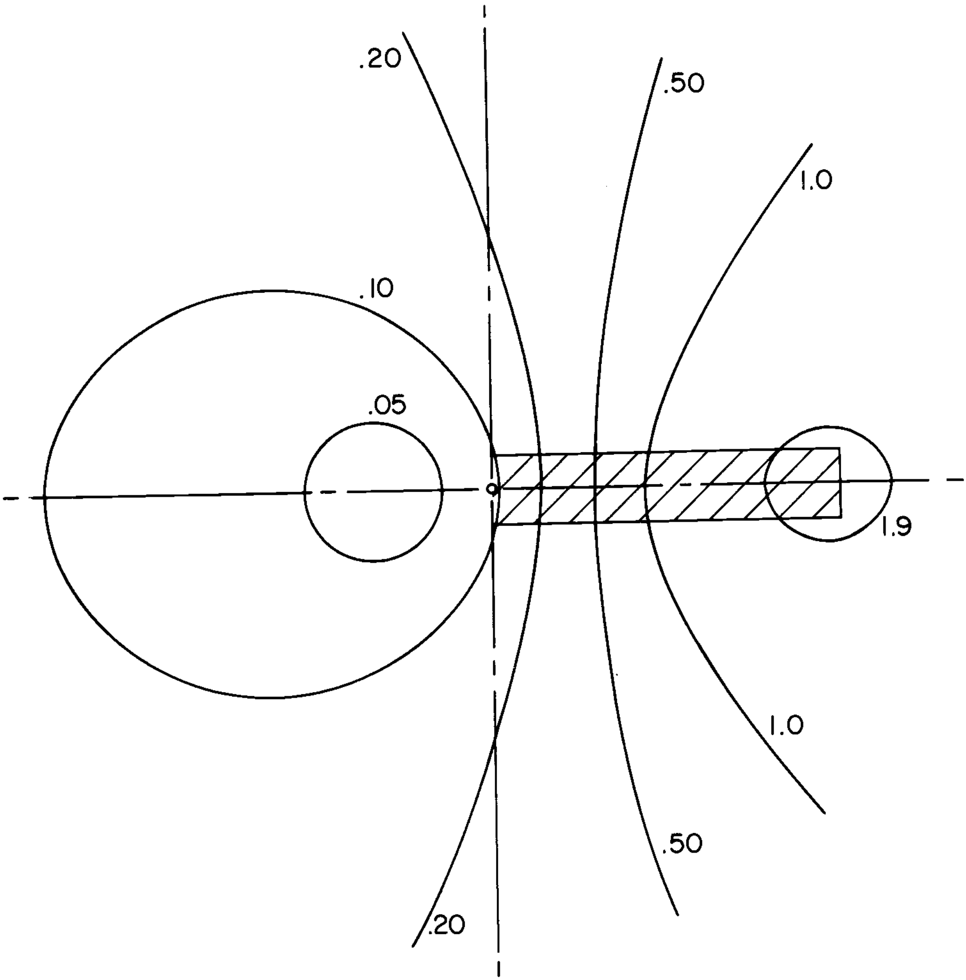
Figure 3 shows a topographical map of the areal rainfall of the watershed for this example. The countour lines represent lines of equal average rainfall over the watershed. The large circular shape to the left represents an average rainfall of 0.1 inches. Although the storms along this line grow larger as they move away from the raingage, they have a decreasing effect on the watershed. The converse effect occurs over the watershed. Near the right boundary, a storm must be quite large to produce a raingage of 0.4 inches and the effect of such a storm on the watershed is seve

SEQUENCE OF PROGRAM CALCULATIONS




VARIABLE IS FIXED FOR STEP TO THE RIGHT

FIGURE 2




CONTOUR LINES OF EQUAL AREAL RAINFALL ON WATERSHED

FIGURE 3

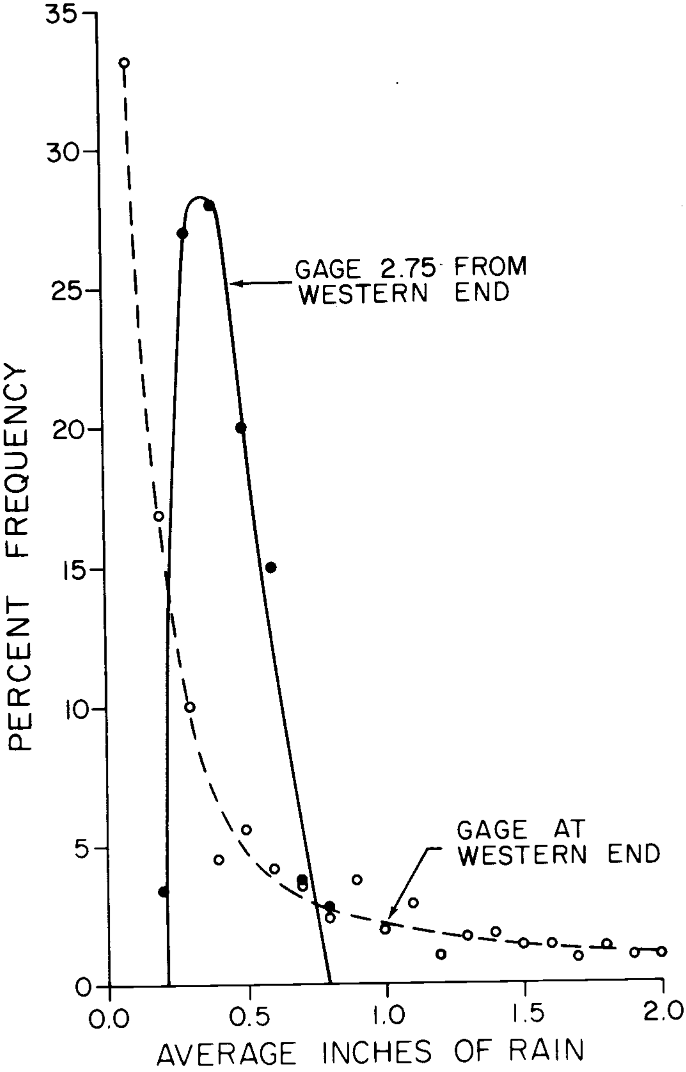
A plot of the likelihoods is more complex. A "crater" effect is seen where the most likely values are located at approximately 1.30 miles from the raingage. The likelihood falls off in both directions away from this radius. This is slightly biased over the watershed due to the adjustment for elevation.

Areal rainfall, given a raingage reading, is a conditional probability distribution function. Figure 4 presents the p.d.f. for the average rain on the watershed in the example.

## EXTENSIONS

The empirical p.d.f. we have obtained may be used in conjunction with other techniques, such as watershed models, to evaluate the results of the rainfall (Parmele et al, 1972, for a closely related example). It is desired to locate the raingage so as to minimize the uncertainty in this evaluation (uncertain rainfall implies uncertain runoff). Since the variance of the p.d.f. is a crude but easily calculated indicator of the uncertainty we have calculated the variance of the p.d.f. obtained at various locations on the watershed assuming 0.4 inches in the raingage. The results are in Table 1.

We have found the optimum location for the raingage assuming the raingage reading. In reality we don't know what the raingage will read. However we usually have enough information to infer a probability distribution for the raingage reading. We have used an exponential with mean of 0.25 inches, with mean increasing for altitude in the same manner as the likelihood of a storm. Using this distribution we calculate the expected variance at each location. The expectation is calculated by Gaus-LeGuerre integration.




PROBABILITY DENSITY FUNCTION OF AREAL RAINFALL FOR 0.4 INCHES IN RAINGAGE

FIGURE 4

When this integration is performed, we have a single value for some location of the raingage. Now we move the raingage to a new location and repeat the simulation in its entirety to obtain a new value. Using a Fibonacci search, we can find the location of the logitudal axis where the variance is a minimum. This is shown in the scheme presented in Figure 2.

For the watershed that has been previously described, this location of minimum average variance is 2.66 miles from the western boundary. This result is hardly surprising because of the altitude correction factor. But our primary purpose was not to find this location, but to develop the single gage watershed model. With very little effort, we can define watersheds of different and possibly irregular shapes, different elevation factors including a contour way if necessary, and any storm model that we chose. Somewhat more difficult modifications would include multiple gaging and allowing the storm to move over some trajectory. Various objective functions might also be expected.

TABLE 1

| Distance from Western Edge | Mean | Variance |
|---|---|---|
| 0 | .43 | .230 |
| 1.25 | .44 | .093 |
| 2.50 | .40 | .018 |
| 2.75 | .39 | .017 |
| 3.00 | .37 | .020 |
| 3.75 | .33 | .050 |
| 5.00 | .27 | .120 |

Mean and Variance of Simulated p.d.f.'s with raingage reading of 0.4.

## CONCLUSIONS

We have shown how a rainfall model may be broken into segments and implemented on a computer. In doing so we developed probability density functions for areal rainfall given the raingage reading and storm location. The criteria of minimum expected variance was then used to find the optimum raingage location.

This simulation was designed to provide information needed for a decision theoretic analysis. From a sample, a raingage reading, we developed a p.d.f. representing the possible amounts of areal rainfall on a watershed. We then found the optimum location to minimize the expected variance of this p.d.f. The raingage location could have been chosen to minimize other functions of the p.d.f.

Simulation is an extremely evaluable tool, especially when other methods of analysis fail. The process must be understood to insure that a model is defined and implemented correctly.

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# SIGNIFICANCE OF ANTECEDENT SOIL MOISTURE TO A SEMIARID WATERSHED RAINFALL-RUNOFF RELATION [1/]

D. L. Chery, Jr. [2/]

## INTRODUCTION

A series of reports from the Southwest have claimed or been interpreted to claim that soil moisture prior to a rainfall-runoff event has no influence, or possibly an inverse effect, on the resulting flow volumes and peak rates [Keppel (1965), Schreiber and Kincaid (1967), Osborn and Lane (1969), Hickok and Osborn (1969), Fogel and Duckstein (1970), Kisiel, Duckstein, and Fogel (1971), and Osborn, Lane, and Kagan (1971)].

These reports cast doubt on the significance of antecedent soil moisture in the semiarid convective rainfall-runoff relationship, but the evidence presented is not conclusive. Further, the unimportance of soil moisture is not consistent with the theory of the soil-watershed system (for an example, consult Smith and Woolhiser, 1971) or with observation of the records from the Safford and Albuquerque experimental watersheds operated by the Agricultural Research Service, (USDA, ARS). Runoff occurs from many storms that would not be expected to produce runoff and the explanation

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1/ Contribution of the Southwest Watershed Research Center, USDA, Agricultural Research Service, Soil and Water Conservation Research Division.

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appears to lie in the occurrence of antecedent rains. One way to test the significance of antecedent rainfall events is to divide the runoff events into two subsets, one with no rain within the preceding 120 hours (5 days) and another with some amount of rain within the preceding 24 hours and then test the null hypothesis ($H_0$):

> Selected parameters of the flow hydrographs for two subsets [(1) no rain within 120 hours and (2) rain (greater than selected depths) within 24 hours] are of the same population.

If $H_0$ is rejected when the rainfall associated with both sets is similar, then the contention that antecedent rainfall contributes to differences in runoff would be supported.

## PROCEDURE

The hypothesis was tested with rainfall and runoff data from a 40-acre ARS watershed located 22 miles west of Albuquerque, New Mexico. This watershed is identified by ARS location code 47 and site number 002; thus, it will be identified by the number 47.002. The watershed is described in "Monthly Precipitation and Runoff for Small Agriculture Watersheds in the United States," 1957. All single-response runoff events were selected from the runoff tabulation of Watershed 47.002 for the analysis. The other category of events is multiple-response, which means that several distinct runoff events resulted from a long, complex rainfall. Each multiple-response could be associated with some part of the rainfall, but this complication was avoided in a first test of the proposition.

The date, beginning time, peak rate of flow (cfs, in/hr), volume (in), duration of flow (minutes), first moment of the stage record, and time to peak (minutes) of each flow were tallied. With the date and beginning time of each selected runoff event, the rainfall data were searched and selected rainfall parameters were tallied for all rainfall events within 120 hours preceding each runoff event. The selected rainfall parameters were: average duration (min), Thiessen-weighted total depths (in), average maximum 5-min. intensity (in/hr), average maximum 15-min. intensity from an individual gage, average histogram centroid, and time (hours) before a runoff event began.

The runoff events were divided into two sets—one with rainfall in the preceding 24-hour period and one without any rainfall in the preceding 120-hour period. Rainfall parameters corresponding to events in these two sets were compared with a Wilcoxon's Rank Sum Test to determine if the two subsets of runoff came from the same rainfall population. If the rainfall of the two runoff events were of the same population, then the null hypothesis with respect to each of the runoff parameters was tested using Wilcoxon's Rank Sum Test.

## WILCOXON'S RANK SUM TEST

The distributions of the rainfall-runoff parameters were not determined but it is known that most of them are not normal distributions. The Wilcoxon's Rank Sum Test is a distribution-free test for identical populations that is sensitive to unequal location. This statistical test was used to test the stated hypothesis. Definition of the test, statistics, and discussion of the test procedure may be found in Bradley (1968). The only condition that must be met for use of this test is that each population contain an infinite number of units, or that the sampling is with individual replacement (Bradley, p. 107). An infinite population will be assumed for the conditions of the reported tests inasmuch as rainfall and runoff events were selected from 30 years of historical records. Some sets of data had many ties and they were treated as recommended (Method A) by Bradley (1968, p. 49-50).

## RESULTS

The results of the Wilcoxon statistical tests are summarized in Tables 1 and 2. The parameters are identified by the following symbols:

| | |
|---|---|
| D | Total rainfall depth (inches) |
| R5 | Average maximum 5-minute rainfall intensity (in/hr) |
| R15 | Average maximum 15-minute rainfall intensity (in/hr) |
| Dur | Duration of rainfall or flow event (min.) |
| RM1 | First moment of rainfall histogram (min.) |
| $Q_t$ | Total runoff volume (in.) |
| $Q_p$ | Maximum flow rate (in/hr) |
| $T_p$ | Time from beginning of flow event to maximum flow rate (min.) (time to peak) |
| QM1 | First moment of flow stage record (min.) |

A two-tailed probability of $\alpha = .05$ of making a Type I error was selected as the threshold at which to accept or reject the null hypothesis.

## DISCUSSION

### Comparison of Rainfall Sets.

The first statistical test (A-1 of Table 1) made on the rainfall parameters rejected the hypothesis that the two rainfall subsets (wet and dry) were the same population with respect to rainfall depth. When the frequency distributions of rainfall depths for the two subsets were examined (see Figure 1), the wet subset had ten events with rainfall depths of 0.10 inch or less that were not in the dry set. Upon eliminating all events with depths of 0.10 inch or less, the statistical test accepted the hypothesis that the two subsets were from the same population (Test A-2, Table 1). The frequency distributions of the two subsets of Test A-2 are also shown in Figure 1.

The hypothesis that the two subsets were from the same population was next tested with respect to the 5-minute rainfall intensity (R5), and it was rejected (Test A-3, Table 1). The frequency distributions for these two subsets (see Figure 2) showed that the dry subset had only one event with R5 less than .70 inch/hour and that was a value of .30 inch/hour. The occurrence of runoff for that event (September 22, 1952) is questionable. So all events with R5 less than 0.70 inch per hour were eliminated and the statistical test again made with respect to the parameter, R5. The test (Test A-4) accepted the hypothesis that the two subsets (wet and dry) were of the same population.

Comparison of Tests A-5 and A-6 shows that eliminating depths of 0.10 inch or less did not necessarily make the two sets the same with

Table 1. Test results for 24-hour antecedent rainfall of any amount greater than 0 ($D_{24} > 0$) of the Null Hypothesis: $\underline{X}$ parameter of the event with selected rainfall depth ($D_0$) and 5-minute intensity (R5) causing runoff with (1) no rain within 120 hours and (2) rain of any amount greater than 0 ($D_{24} > 0$) within 24 hours are of the same population.

| Test | Rainfall or Runoff Set ID | Threshold of 24-hour Rain Depth | Threshold of RO Rain Depth | Threshold of RO Rain 5-min. int. | Parameter Tested | Number in Subset | | Reject @ α = 5% | Also reject @ | Accept @ α = 5% | Also accept @ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | $D_{24}$ | $D_0$ | R5 | X | (1) | (2) | | α | | α |
| A 1 | P | >0 | 0 | 0 | D | 51 | 80 | X | 1.6 | | |
| A 2 | | | .10 | 0 | D | 51 | 70 | | | X | 11.0 |
| A 3 | | | 0 | >0 | R5 | 44 | 70* | X | 0.3 | | |
| A 4 | | | 0 | .70 | R5 | 43 | 54 | | | X | 15.6 |
| A 5 | | | 0 | .70 | D | 43 | 54 | X | 4.0 | | |
| A 6 | | | .10 | 0 | R5 | 44 | 66 | X | 0.8 | | |
| A 7 | | | .13 | .70 | D | 43 | 47 | | | X | 18.0 |
| A 8 | | | | | R5 | | | | | X | 67.0 |
| A 9 | | | | | R15 | | | | | X | 70.4 |
| A 10 | | | | | Dur | | | X | 3.4 | | |
| A 11 | | | | | RM1 | | | | | X | 21.9 |
| A 12 | Q | | | | $Q_t$ | | | | | X | 5.6 |
| A 13 | | | | | $Q_p$ | | | | | X | 10.3 |
| A 14 | | | | | $Q_t$/D | | | X | 0.1 | | |
| A 15 | | | | | $Q_p$/R5 | | | X | 3.5 | | |
| A 16 | | | | | $Q_p$/R15 | | | X | 2.7 | | |
| A 17 | | | | | $T_p$ | | | | | X | 41.0 |
| A 18 | | | | | Dur | | | X | 0.4 | | |
| A 19 | | | | | QM1 | | | X | 0.3 | | |

*Sets reduced by 17 events with no measurement of R5.

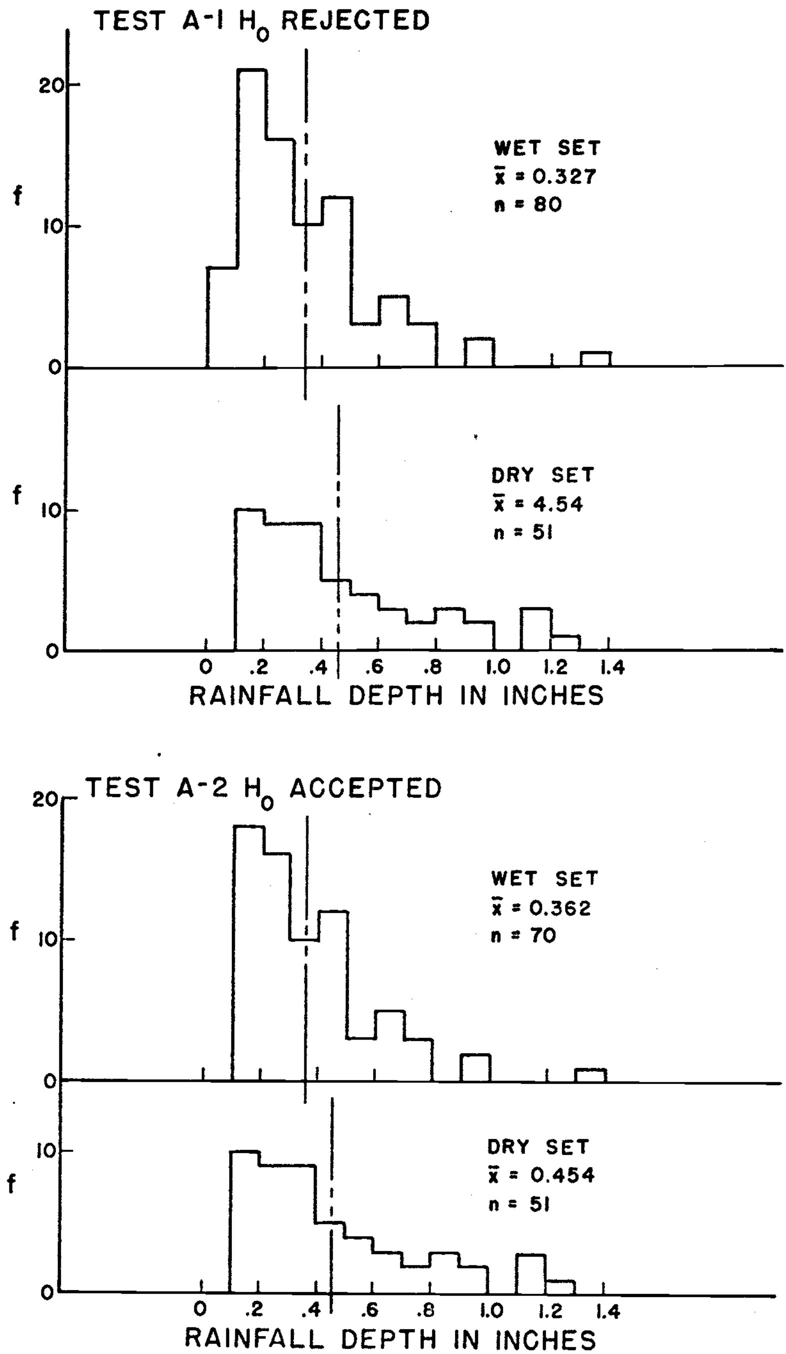


Figure 1. Distributions of Rainfall Depths for Tests A-1 and A-2.

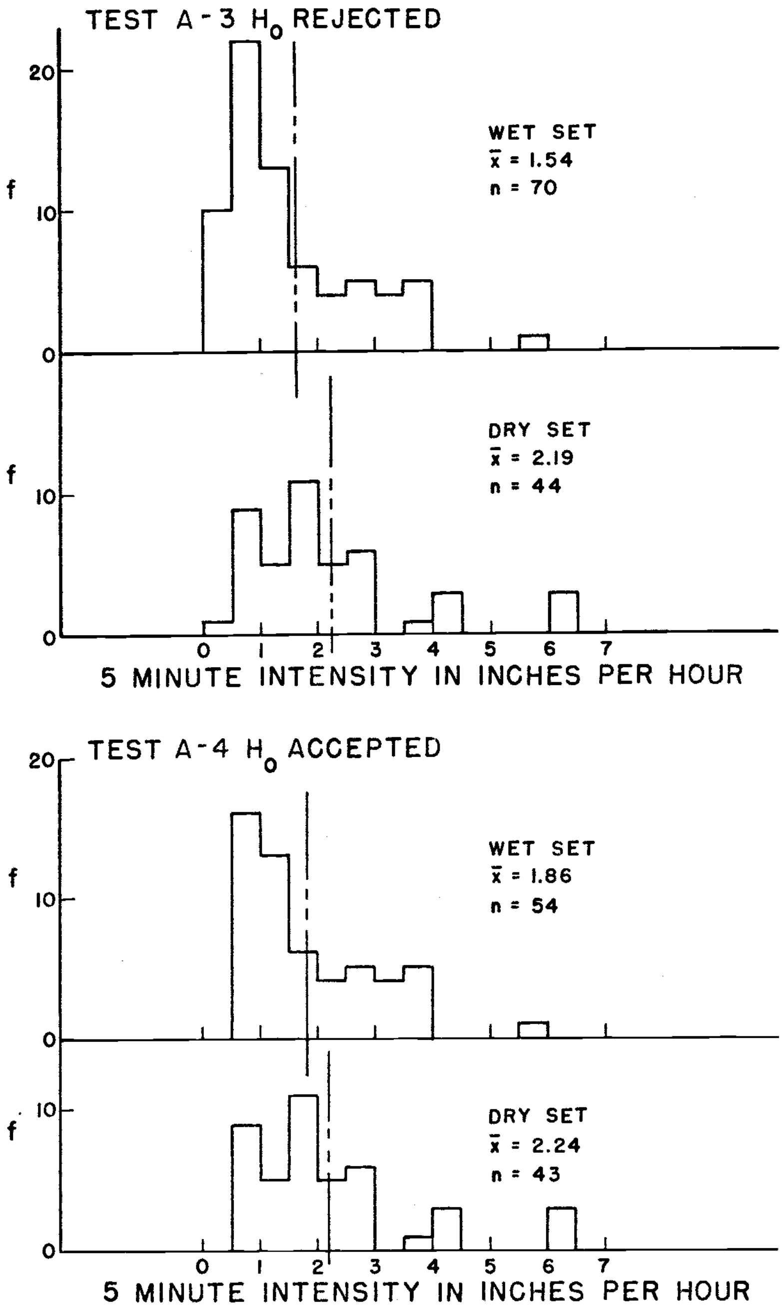


Figure 2. Distributions of 5-minute intensities for Tests A-3 and A-4.

respect to 5-minute intensities, and vice versa, eliminating all events with 5-minute intensities of less than 0.70 inch per hour did not make the two sets the same with respect to total rainfall depth. As a consequence, a double elimination based on both depth and 5-minute intensities must be made to obtain a set of rainfall events in which the subsets may be from the same population with respect to all rainfall parameters.

The data for Test A-5 had six events with rainfall depths between 0.09 inch and 0.13 inch—all in the wet subset. No events were in this range in the dry subset. Three dry events with depths of 0.12 inch had no measurement of the intensities, and these events were eliminated from the test data when the elimination of R5 $< 0.70$ in/hr was made. Because of the missing record, events with rainfall depths less than 0.14 inch and 5-minute intensities less than 0.70 in/hr were eliminated from the test data and the residual events were tested as being of the same population.

The residual sets of rainfall events were accepted as of the same population with respect to four of the five parameters. The null hypothesis was rejected with respect to rainfall duration. However in total, the two subsets of rainfall events can be considered to be of the same population and the runoff events tested for the influence of antecedent rainfall events. Before these tests are discussed, the obvious conclusions of the tests of the rainfall will be summarized.

The tests on the rainfall data alone demonstrate that rainfalls of less than 0.10-inch depth and 5-minute intensities of less than 0.70 inch per hour will cause surface runoff if there is rainfall in the preceding 24 hours. These limits on depth and rainfall intensity apply to the particular 40-acre watershed tested. Two rainfalls of 0.10-inch depth and one of 0.09 inch with 5-minute intensities greater than 0.70 inch/hour caused runoff when there had been antecedent rainfall. Also one event (July 23, 1954) with 0.08-inch depth and 0.52-inch/hour 5-minute intensity caused considerable runoff when it followed a rainfall-runoff event by 3 hours.

Comparison of Runoff Sets for any Amount of Antecedent Rainfall.

When the A set (Table 1) parameters of the runoff events were tested for being of the same population, total runoff volume and peak discharge rate were accepted as being of the same population. However, the ratios of total runoff to total rainfall volume and peak discharge rate to the 5-and 15-minute average rainfall rates (Tests A-14, A-15, and A-16) were rejected as being of the same population. Also the tests rejected the null hypothesis for the two subsets (wet and dry) with respect to total flow duration and first moment of the stage hydrograph (Tests A-18 and A-19). But the null hypothesis was accepted with respect to time to peak. These tests were made for any depth of rainfall within the preceding 24 hours determining a "wet" event, and they show definitely that runoff volume from a given rainfall rate will be greater with antecedent rainfall. This trend is shown more definitely when rainfall depths above a given threshold are used to define a "wet" antecedent event. A threshold of 0.09 inch was used in the tests summarized in Table 2.

Table 2. Test results for 24-hour antecedent rainfall amount greater than 0.09 inches ($D_{24}$ > 0.09) of the Null Hypothesis; $\underline{X}$ parameter of the event with selected rainfall depth ($D_0$) and 5-minute intensity (R5) causing runoff with (1) no rain within 120 hours and (2) rain of an amount greater than 0.09 inches ($D_{24}$ > 0.09) within 24 hours are of the same population.

| | Rainfall or Runoff Set ID | Threshold of 24-hour Rain Depth | Threshold of RO Rain Depth | Threshold of RO Rain 5-min. int. | Parameter Tested | Number in | Subset | Reject @ $\alpha$ = 5% | Also reject @ | Accept @ $\alpha$ = 5% | Also accept @ |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Test | | $D_{24}$ | $D_0$ | R5 | X | (1) | (2) | | $\alpha$ | | $\alpha$ |
| B 1 | P | .09 | 0 | 0 | D | 51 | 54 | X | 4.1 | | |
| B 2 | | | .10 | 0 | D | 51 | 47 | | | X | 23.0 |
| B 3 | | | 0 | >0 | R5 | 44 | 49 | X | 0.5 | | |
| B 4 | | | 0 | .70 | R5 | 43 | 39 | | | X | 10.1 |
| B 5 | | | 0 | .70 | D | 43 | 39 | | | X | 5.7 |
| B 6 | | | .10 | 0 | R5 | 44 | 45 | X | 1.4 | | |
| B 7 | | | .13 | .70 | D | 43 | 34 | | | X | 26.0 |
| B 8 | | | | | R5 | | | | | X | 39.0 |
| B 9 | | | | | R15 | | | | | X | 58.2 |
| B 10 | | | | | Dur | | | | | X | 5.4 |
| B 11 | | | | | RM1 | | | | | X | 61.7 |
| B 12 | Q | | | | $Q_t$ | | | X | 2.2 | | |
| B 13 | | | | | $Q_p$ | | | Inconclusive | | | |
| B 14 | | | | | $Q_t$/D | | | X | 0.04 | | |
| B 15 | | | | | $Q_p$/R5 | | | X | 0.7 | | |
| B 16 | | | | | $Q_p$/R15 | | | X | 0.7 | | |
| B 17 | | | | | $T_p$ | | | | | X | 13.4 |
| B 18 | | | | | Dur | | | X | 0.4 | | |
| B 19 | | | | | QM1 | | | X | 0.4 | | |

Comparison of Runoff Sets for $D_{24} > 0.09$ inch.

The same procedure was followed, testing the rainfall parameters of the two subsets (wet and dry). After the double elimination, the two subsets were accepted as being of the same population with respect to all five rainfall parameters.

The depths and time distribution of the antecedent rainfall for the wet subset are shown in Figure 3. On the average the accumulated rainfall was 0.29 inch (Figure 3a) occurring 14 hours (Figure 3c) before the runoff event. The distribution of times that an antecedent rainfall occurred prior to a rainfall-runoff event is rather uniform whereas the distribution of individual antecedent rainfall amounts (Figure 3b) is skewed with a mode in the 0- to 0.10-inch interval, markedly less than the mean depth of 0.17 inch.

When the antecedent rainfall was greater than 0.09 inch, the null hypothesis was rejected very conclusively with respect to runoff volume, ratio of runoff volume to rainfall depth, ratios of peak flow rate to average 5- and 15-minute intensities, flow duration, and first moment of the stage hydrograph. The test results were inconclusive with respect to peak flow rate, and they accepted the null hypothesis with respect to time to peak.

The comparison between distributions of the wet and dry subsets for the Tests B-13 and B-19 are shown in Figures 4 through 11. These tests were made and the conclusions drawn knowing that there were discharge rating inaccuracies, but since the statistical tests were making relative

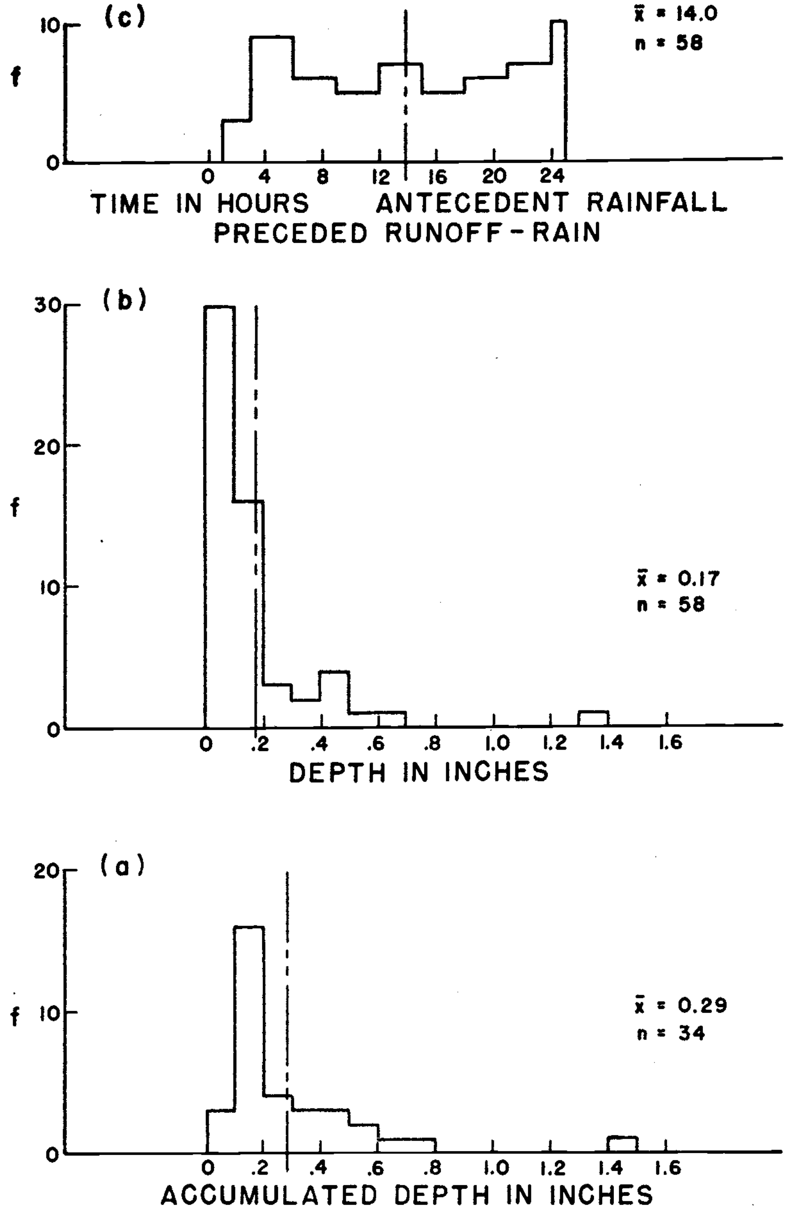


Figure 3. Distribution of Depths and Time for Antecedent Rainfalls greater than 0.09 inch.

comparisons it was assumed that these rating errors would not affect the conclusions.

The distributions of runoff volumes (Figure 4) show the mean runoff volume of wet events is greater than that of dry events. Dry events overwhelmingly produce very little runoff. The first interval has been divided in half to further illustrate this tendency, but the greatest runoff volume resulted from a dry event (August 24, 1957).

The null hypothesis could neither be rejected nor accepted with respect to peak discharge rate. The distributions of Figure 5 show the occurrence of more small events under dry conditions (the 0-10 interval has been divided at 1 to illustrate this point), but again the four largest peak discharges of either set were from dry events.

When runoff volume relative to the associated rainfall volumes are compared, the null hypothesis is strongly rejected. The distributions of Figure 6 show the mean of the wet events to be considerably greater than that of the dry events. Thus, in general, more runoff results from a given rainfall depth when there has been antecedent rainfall. Again, the dry events have a significant cluster in the lowest interval (0.00-0.05), but two dry events stand out as producing a large ratio of runoff volume to rainfall volume. For one of the wet events (August 14, 1964), runoff was greater than rainfall. An examination of the original records reveals no apparent problems other than the obvious question of how could such a large flow have resulted from such a small event. It is known that the rating relations for the flow measuring weirs are inaccurate, but even

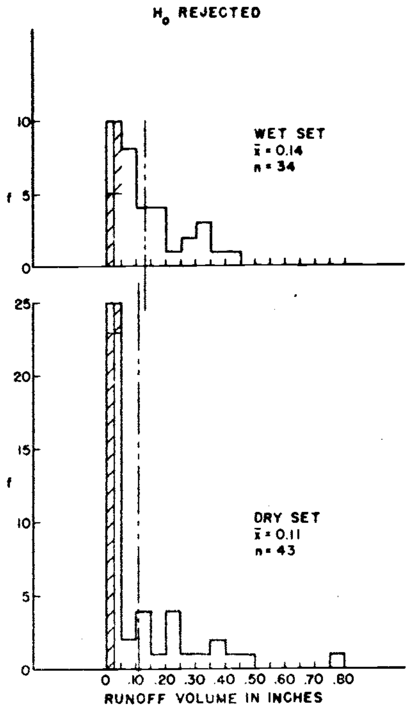


FIGURE 4. RUNOFF VOLUME DISTRIBUTIONS
(TESTS B-12)

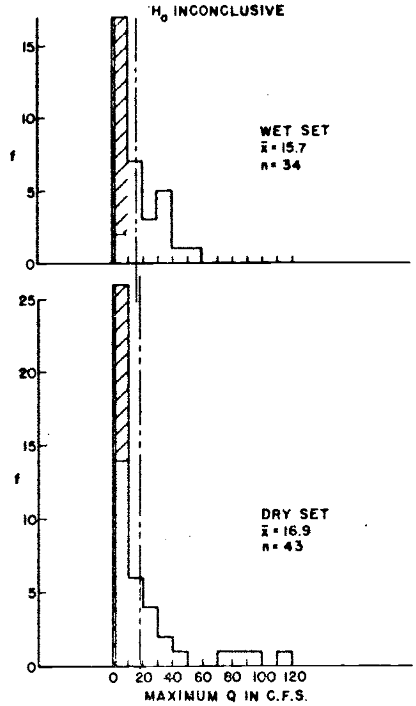


FIGURE 5. MAXIMUM FLOW RATE DISTRIBUTIONS
(TESTS B-13)

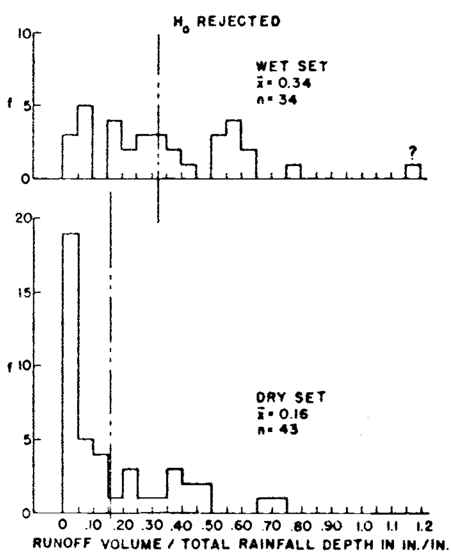


FIGURE 6. VOLUME RATIO DISTRIBUTIONS
(TESTS B-14)

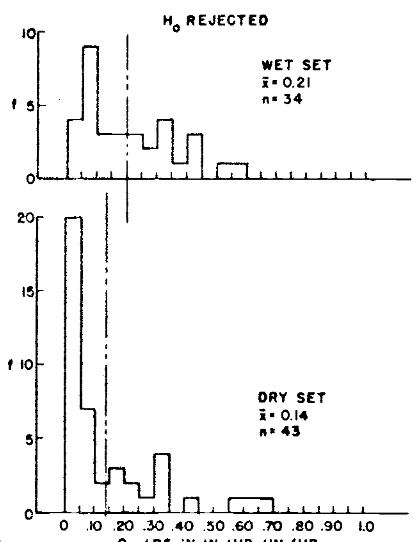


FIGURE 7. RATE RATIO DISTRIBUTIONS
(TESTS B-15)

these rating errors may not explain this anomalous event.

The ratios of peak flow to average 5- and 15-minute intensities were tested and their distributions are shown in Figures 7 and 8. Again the mean of the wet sets is significantly greater than that of the dry sets. Thus, in general, runoff rates relative to rainfall rates of the associated rainstorm will be higher when there has been antecedent rainfall. Again, the ratio distribution for dry events has a large cluster in the lowest interval (0-0.05), but three events have ratios as great as or greater than those determined for wet events. One ratio of peak discharge rate to average 15-minute rainfall intensity ($Q_m$/R15) was greater than one. This is a plausible occurrence when a momentary runoff rate is compared with a 15-minute average rainfall rate.

Each of the dry set distributions for volume, maximum flow rate, ratio of runoff volume to rainfall volume, ratio of maximum flow rate to average 5- and 15-minute rainfall intensities had values as great as or greater than wet event distributions. These high values of the dry set are rather consistently associated with one of the five largest "dry" rainfall events. The rankings of the six largest events with respect to total average depth and average 5- and 15-minute intensities are shown in Table 3.

Two wet rainfall events appear in Table 3, but they are not associated with large values of wet runoff parameters (shown in Figures 4-11). The wet event of September 20, 1941, had high average 5- and 15-minute rainfall intensities but the total storm depth was relatively low at 0.73 inch.

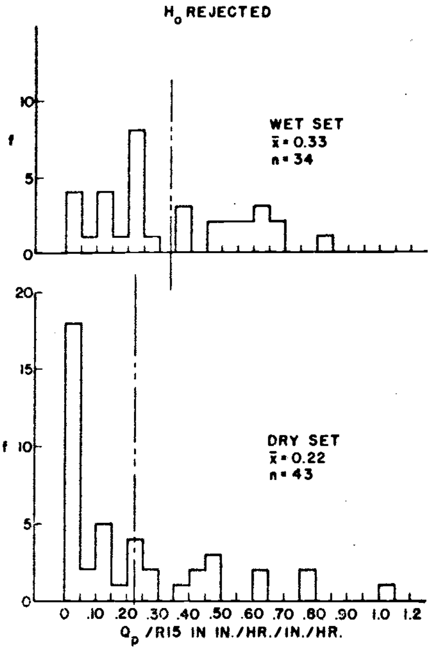


FIGURE 8. RATE RATIO DISTRIBUTIONS (TESTS B-16)

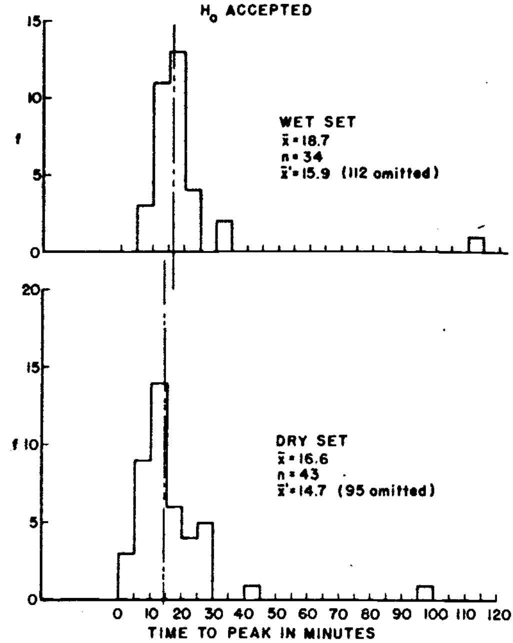


FIGURE 9. TIME TO PEAK DISTRIBUTIONS (TESTS B-17)

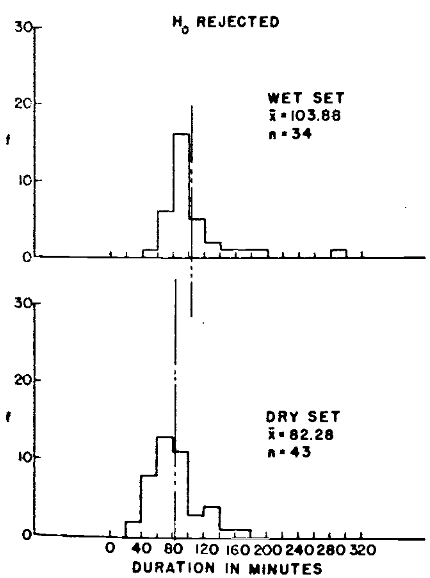


FIGURE 10. DURATION DISTRIBUTIONS (TESTS B-18)

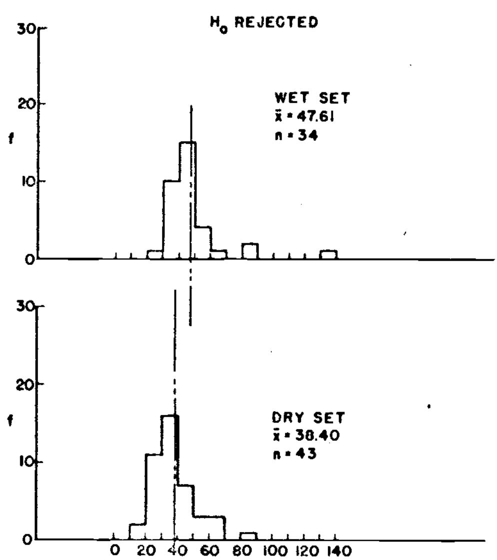


FIGURE 11. 1st MOMENT DISTRIBUTIONS (TESTS B-19)

Table 3. Ranking of 47.002 rainfall events with respect to average of two gage measurement of depths and intensities.

| Event Date day mo yr | Dry Set | Wet Set | Parameter | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | R5 in/hr | Rank | R15 in/hr | Rank | D in. | Rank |
| 20 09 41 | | X | 5.92 | 4 | 2.41 | 6 | | |
| 30 06 42 | X | | 6.03 | 2 | 3.19 | 4 | 1.12 | 5 |
| 28 06 43 | | X | | | | | 1.32 | 1 |
| 04 09 47 | X | | 4.18 | 6 | 3.21 | 3 | 1.20 | 2 |
| 24 08 57 | X | | 4.26 | 5 | 2.79 | 5 | 1.13 | 4 |
| 10 06 66 | X | | 6.01 | 3 | 4.51 | 1 | 1.18 | 3 |
| 22 07 69 | X | | 6.11 | 1 | 3.32 | 2 | .99 | 6 |

Also, the antecedent rainfall for this event was below average, with the 0.13 inch accumulated amount occurring at the limit of 24 hours (see Figure 3). The one wet rain that had the greatest depth (June 28, 1943) lasted 265 minutes, with maximum 5- and 15-minute intensities of 3.70 and 1.70 inches per hour.

Not the merest association could be made between large wet runoff events and large wet rainfall events as could be done for the dry set. The general situation of no association between high rainfall parameters and resulting runoff parameters for the wet set is indicative of an antecedent moisture influence. For the "dry" events there is one definite initial moisture state, but for the wet events the initial moisture ranged from nearly dry to very wet. Thus, the resulting runoff in the wet set is confounded by moisture state of the soil and the resulting largest volumes, peak flows, and ratios of volumes and rates were not associated with the largest rainfall events but rather a complex interaction between the input and the moisture state of the watershed system.

Comments on Definition of Wet Set.

The definition of a wet set as one in which rainfall equal to a greater than some selected depth falling within the preceding 24 hours is admittedly crude. Well-verified soil moisture accounting models and necessary physical data for semiarid rangelands are lacking; thus, as an expediency the antecedent rainfall definition for wet was used. It would have been preferable if these tests had been made on two sets in which the initial moisture state could have been determined more directly, or with a "good" model and the wet set defined using soil moisture at the beginning of the rainfall-runoff event equal to or greater than some selected level. The set of events would be altered by eliminating some in the wet set as being not wet enough and including some that had sufficient rainfall prior to 24 hours to contribute to the selected moisture state at the beginning of the event.

## CONCLUSIONS

A distribution-free statistical comparison between two sets of convective rainfall-runoff events, one considered to have no soil moisture at the beginning of rainfall and one considered to be moist by virtue of antecedent rainfall, demonstrated conclusively that antecedent rainfall influences runoff from a semiarid watershed. This is in contrast to previously published conclusions, which may be insufficiently supported or poorly drawn from available evidence. For the tested watershed, runoff volumes, ratios of runoff volume to rainfall depth, and ratios of peak flow rate to 5- and 15-minute rainfall rate were greater when the initial state was "moist."

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# OBJECTIVE AND SUBJECTIVE ANALYSIS OF TRANSITION PROBABILITIES OF MONTHLY FLOW ON AN EPHEMERAL STREAM

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Chester C. Kisiel[b]

## INTRODUCTION

In order to generate synthetic sequences of streamflows on aridland channels for subsequent evaluation of proposed water resource projects, it is necessary to study the structure of historical sequences of flow. Given the 58-record of monthly flows on Rillito Creek at Tucson, Arizona, the objective of the research was to determine its stochastic properties so as to develop a model of the historic flows. The stochastic properties of interest included (a) statistical moments (arithmetic mean, variance and skewness) over each of the twelve months over the period of record, (b) frequency of dryness (number of months with zero flows), (c) crosscorrelation between flows over adjacent months, and (d) state transition probabilities for flows in a specified range in one month given that flows were in a specified range in a previous month. This study was intent on a critical evaluation of the value of the lag-one autoregressive model (Fiering, 1967) for modeling of ephemeral flow. The findings of this previously unreported study led to consideration of event-based models (Kisiel, et al., 1971; Denny, et al., 1971).

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The complete time series of observed flows on any stream has many properties of interest both to scientists, engineers and managers. Our particular focus here is geared to ephemeral flow of semi-arid regions. Rillito flows have been studied in our research project for the purpose of developing methodology for engineering purposes and for scientific description. The strategy involves an interplay of data and models. Previous studies on the Rillito have focused on the annual flood maxima (Davis and Dvoranchik, 1971; Davis, et al., 1972) and on the individual flow events as an alternating wet-dry sequence (Baran, et al., 1971; Denny, et.al, 1971). In this paper, we focus on the monthly flow sequence.

## STOCHASTIC PROPERTIES OF THE MONTHLY FLOWS

The utility of the methods of time series analyses of streamflow has been amply demonstrated for channels with perennial flows (Fiering, 1967). However, when the time series contains zero flow over 53% of the months in the record (as is the case for Rillito Creek - see Table 1) serious questions arise about the interpretation of autocorrelation plots (correlograms) and variance spectra for such sequences. Crosscorrelations computed for streamflows in adjacent months (as shown in Table 2) do not reflect the storage capabilities of the basin because there is no base flow. The only explanation for such persistence in flows would be in the persistence of regional meteorological conditions. Even though this factor dominates flows in more temperate climates, it seems more natural to model aridland streamflows in terms of alternating sequences of wet and dry periods and to include in the model a provision for meteorological transitions.

Tables 1 and 2 summarize the results of a statistical study of total monthly streamflow on Rillito Creek for the period 1909 to 1966. A bimodal population of flow and no-flow periods is clearly indicated in the results. But the 53% of the

TABLE 1.--STATISTICS OF TOTAL MONTHLY STREAMFLOW[a]

| Month | Mean[b] | Variance | Standard Deviation | Coefficient of Variation | Skewness | Zero Flows | Percent Zero Flows |
|---|---|---|---|---|---|---|---|
| Oct. | 2.54 | 7.32 | 2.71 | 1.07 | 7.54 | 38 | 0.66 |
| Nov. | 8.20 | 188.3 | 13.72 | 1.67 | 7.53 | 41 | 0.71 |
| Dec. | 115.80 | 63,800. | 252.60 | 2.18 | 7.83 | 40 | 0.69 |
| Jan. | 54.32 | 17,080. | 130.70 | 2.41 | 7.54 | 31 | 0.53 |
| Feb. | 54.26 | 9,080. | 98.03 | 1.81 | 7.67 | 31 | 0.53 |
| Mar. | 31.62 | 1,910. | 43.71 | 1.38 | 7.54 | 25 | 0.43 |
| Apr. | 4.46 | 36.91 | 6.08 | 1.36 | 7.52 | 44 | 0.76 |
| May | 4.16 | 65.53 | 8.10 | 1.95 | 7.51 | 54 | 0.93 |
| June | 3.11 | 12.46 | 3.53 | 1.14 | 7.51 | 47 | 0.81 |
| July | 36.80 | 8,072. | 89.84 | 2.44 | 7.54 | 6 | 0.10 |
| Aug. | 45.25 | 2,573. | 50.73 | 1.12 | 7.53 | 4 | 0.07 |
| Sept. | 22.29 | 2,519. | 50.19 | 2.25 | 7.56 | 12 | 0.21 |

[a]Excluding zero flows.

[b]In acre-feet (AF)

[c]There were 373 months with zero flows over a total of 696.0 data points. The percentage of zero flows over this record is 0.53.

TABLE 2.--MONTHLY FLOWS ON RILLITO CREEK (1909-1966) CROSS CORRELATION ON MONTHS[a]

| | Oct. | Nov. | Dec. | Jan. | Feb. | Mar. | Apr. | May | June | July | Aug. | Sept. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Oct. | 1.00 | .25 | .39 | .27 | .33 | .28 | .37 | -.06 | -.13 | -.11 | -.24 | -.14 |
| Nov. | .25 | 1.00 | .31 | .20 | .56 | .30 | .47 | -.04 | -.09 | -.05 | -.17 | -.11 |
| Dec. | .39 | .31 | 1.00 | .40 | .88 | .55 | .62 | -.03 | -.08 | -.08 | -.14 | .04 |
| Jan. | .27 | .20 | .40 | 1.00 | .45 | .40 | .37 | -.04 | -.08 | -.08 | .00 | -.05 |
| Feb. | .33 | .56 | .88 | .45 | 1.00 | .53 | .69 | -.05 | -.10 | -.09 | -.10 | -.01 |
| Mar. | .28 | .30 | .55 | .40 | .53 | 1.00 | .46 | .40 | .16 | -.08 | -.12 | -.10 |
| Apr. | .37 | .47 | .62 | .37 | .69 | .46 | 1.00 | -.04 | -.10 | .21 | -.12 | -.09 |
| May | -.06 | -.04 | -.03 | -.04 | -.05 | .40 | -.04 | 1.00 | .52 | -.05 | -.11 | -.05 |
| June | -.13 | -.09 | -.08 | -.08 | -.10 | .16 | -.10 | .52 | 1.00 | .01 | -.02 | -.05 |
| July | -.11 | -.05 | -.08 | -.08 | -.09 | -.08 | .21 | -.05 | .01 | 1.00 | .50 | -.00 |
| Aug. | -.24 | -.17 | -.14 | -.10 | -.10 | -.12 | -.12 | -.11 | -.02 | .50 | 1.00 | .26 |
| Sept. | -.14 | -.11 | -.04 | -.05 | -.01 | -.10 | -.19 | -.05 | -.05 | -.00 | .26 | 1.00 |

[a]Confidence limits on the cross correlation coefficients were based on a two-tailed test at the five percent significance level:
Lower limit of confidence band is approximately -0.28
Upper limit of confidence band is approximately +0.28

months in which no flow occurred is deceptive because in many of the months the flows were recorded only a few days within a month. For example, in the month of December over the period of record, the mean flow was 26 cfs but 70% of the variance of the monthly flows came from one average value of 976 cfs. This condition repeats itself in many other streamflow records and provides a strong argument for stochastic models that preserve the timing of flows within the month.

The coefficients of variation noted in Table 1 are quite typical of aridland conditions. The skewness reflects the wide scatter in observed flows in each month. For these reasons, no attempt was made to fit probability distributions to each of the 12 sets of monthly flows.

Table 2 gives the crosscorrelation matrix for each month with respect to the remaining 11 months. Of significance in this table is the substantial positive crosscorrelation between the set of months from October to May. These strong seasonal linkages and the extreme variability of monthly flow totals are important features to be preserved in any proposed stochastic model.

Upon exclusion of the zero flows (373 months), the remaining set of months with flows (323) were taken as a group, irrespective of seasonal pattern to determine if a negative exponential probability distribution were a suitable hypothesis to describe the average monthly flows. By means of the chi-squared test it was inferred that the exponential distribution could be not accepted because of the large number of extreme events in the tails of the empirical frequency distribution. Thus, the designation of a month as a suitable time basis for massaging of hydrologic data is arbitrary and can lead to misleading results at least in ephemeral streams. As in many cases, the use to which the results are to be put will determine whether averages over large time intervals (month or year) are to be preferred to measures of variability and temporal structure over shorter time intervals.

Confidence bands on mean and variance of monthly flows

Because statistics, like the mean $\bar{x}$ and variance $s^2$, based on record length n, are estimates of their population values, $\mu$ and $\sigma^2$, respectively, it is desirable to localize or circumscribe the variability of these estimates. The variance of $\bar{x}$, (Var x), and of $s^2$, (Var $s^2$), must be determined by assuming an underlying population from which samples have been collected. On repetitive sampling of this population, other sets of 58 monthly flows could be obtained and new values of $\bar{x}$ and $s^2$ found. Because such sampling in nature is not possible, a theoretical or a computer simulation approach is necessary to find the sampling distributions of $\bar{x}$ and $s^2$. In what follows we assume a normal distribution $N(\mu,\sigma^2)$ for the population even though monthly flows are usually skewed and assume independence of a month's flow from year to year. The latter assumption is easy to justify in terms of zero serial correlation between monthly flows, whereas normality is difficult to justify except for reasons of theoretical convenience. At least, the resulting limits of confidence in estimates of $\bar{x}$ and $s^2$ from "short" records give a crude measure of parameter variability and serve as a cautionary warning when making economic analyses based on the single estimates from the historical record.

The two-sided confidence region for the mean $\mu$ may be constructed from (Lindgren, 1968):

$$\bar{x} - \frac{\sigma}{n^{1/2}} z_{1-\frac{\alpha}{2}} < \mu < \bar{x} + \frac{\sigma}{n^{1/2}} z_{1-\frac{\alpha}{2}} \qquad (1)$$

in which z is the standard normal variate, N(0,1), with zero mean and unit variance at a significance level of $\alpha$ (usually taken as 0.05). The two-sided confidence region for the variance $\sigma^2$ may be constructed from (Lindgren, 1968):

$$\frac{n s^2}{\chi^2_{\alpha}} > \sigma^2 > \frac{n s^2}{\chi^2_{1-\alpha}} \tag{2}$$

In inequality (1) no assumption of normality is necessary and because of the central limit theorem the Var $\bar{x}$ = Var $x/n$ for mutually independent random variables. But in inequality (2) for sample variances the "central limit theorem effect" does not hold. In both inequalities (1) and (2), the confidence regions should get tighter (smaller) around the true mean $\mu$ and variance $\sigma^2$, respectively, as n becomes large provided the assumptions hold approximately. The rate of convergence is slower, however, for the bounds on $\sigma^2$.

Concerning confidence limits for the skewness shown in Table 1, Kendall (1947) illustrates how the sampling variance of a parameter (moment in this case) depends on a moment that has twice the order of moment of interest. Hence, calculations of sampling variance of mean, variance, and skew depend, respectively, on the 2nd, 4th and 6th order moments. Because estimates of 6th order moments are notoriously weak for available sample sizes in hydrology, no effort was made to construct confidence regions for the skew. The available confidence regions for the skew are based on $N(\mu,\sigma^2)$ populations; for gamma (Pearson Type III), log-gamma, and other skew populations computer simulation experiments would be necessary to get the form of the sampling distribution for the sample moments.

For Rillito Creek monthly flows Table 3 gives the separate confidence regions for monthly means and variances. For example, in the month of December, the 95% confidence region on the mean is (-2.572 < 35.950 < 74.471) and on the variance it is (15,782 < 21, 949 < 33,298). The region is not tight and so not as much confidence can be placed in $\bar{x}$ and $s^2$ as would be desirable. It is well known that a future 58-year record (in which any design or plan would have to function) would

Table 3. Separate Confidence Regions on Monthly Means and Variances on Rillito Creek (Period of 1909-1966)

| | | | Confidence Limits (95%) - may be rounded off to 2-3 digits. | | | |
|---|---|---|---|---|---|---|
| | | | On $\mu$ Alone | | On $\sigma^2$ Alone | |
| MONTH | MEAN(cfs) | VARIANCE | LOWER BOUND | UPPER BOUND | LOWER BOUND | UPPER BOUND |
| October | .887 | 3.9 | .362 | 1.392 | 2.8 | 6.0 |
| November | 2.403 | 67.0 | .275 | 4.533 | 48.2 | 101.7 |
| December | 35.950 | 21,949.9 | -2.572 | 74.471 | 15,982.1 | 33,298.2 |
| January | 25.286 | 8,538.4 | 1.238 | 49.331 | 6,139.1 | 12,952.8 |
| February | 25.256 | 5,128.9 | 6.637 | 43.876 | 3,687.7 | 7,780.5 |
| March | 17.993 | 1,322.9 | 8.540 | 27.446 | 950.5 | 2,005.5 |
| April | 1.076 | 12.1 | .170 | 1.981 | 8.7 | 18.4 |
| May | .287 | 4.6 | -.270 | .843 | 3.3 | 7.0 |
| June | .590 | 3.7 | .090 | 1.091 | 2.7 | 5.6 |
| July | 32.997 | 7,349.9 | 10.706 | 55.287 | 5,277.5 | 11,149.8 |
| August | 42.127 | 2,526.5 | 29,059 | 55.186 | 1,816.6 | 3,832.9 |
| September | 17.681 | 2,071.5 | 5,849 | 29.514 | 1,489.4 | 3,142.5 |

not have the same statistical properties as the past record. This fact is an important justification for the emergence of stochastic hydrology. Note that the negative value of the lower bound on the mean is a consequence of skewness in the original data and emphasizes an inherent weakness about reasoning in terms of variances of skew populations.

Simultaneous confidence regions may be constructed by assuming independence of $\bar{x}$ and $s^2$. This is justified when the underlying population is $N(\mu,\sigma^2)$. In this case the joint probability distribution of $\bar{x}$ and $s^2$ is described by a normal-chi square distribution (in reality the normal-gamma distribution NG because the chi-square belongs to the gamma family). The NG distribution may give tighter bounds than those shown in Table 3 because of the additional prior information introduced on the joint distribution (provided the assumptions hold). Table 4 gives the simultaneous confidence regions for $\mu$ based on the lower and upper values of $\sigma^2$ given in Table 3. The mean discharge given in the second column of Table 4 may be inserted between the lower and upper bounds in each of the three cases to evaluate the spread for both independent and simultaneous confidence regions. The significance of the spread may be judged in terms of the cost of one acre-foot of water for agricultural purposes ($10/AF say) or for municipal purposes ($100/AF). Such figures may be used to assess the costs of errors of overestimation and underestimation of mean monthly flows.

STATE TRANSITION PROBABILITIES

Modeling the transition from one flow range to another flow range has been in terms of (a) use of the serial correlation or crosscorrelation of flows at adjacent time periods, or (b) use of the probability distribution that describes the transition from one state of flow to another state.

Table 4. Simultaneous Confidence Regions for Monthly Means Based on Variance Bounds shown in Table 3

| | | | | 95% Confidence bounds on $\mu$ and $\sigma^2$ are taken together; numbers may be rounded off to 2-3 digits. | | | |
|---|---|---|---|---|---|---|---|
| | | 95% Confidence bounds on $\mu$ alone | | Based on lower bound on $\sigma^2$ | | Based on upper bound on $\sigma^2$ | |
| Month | Mean* (cfs) | Lower** | Upper | Lower** | Upper | Lower** | Upper |
| October | .887 | .362 | 1.392 | .243 | 1.314 | .440 | 1.511 |
| November | 2.403 | .275 | 4.533 | -.217 | 4.204 | .602 | 5.023 |
| December | 35.950 | -2.572 | 74.471 | -11.470 | 68.580 | 3.320 | 83.370 |
| January | 25.286 | 1.238 | 49.331 | -4.254 | 45.646 | 4.926 | 54.826 |
| February | 25.256 | 6.637 | 43.876 | 2.376 | 41.016 | 9.496 | 48.136 |
| March | 17.993 | 8.540 | 27.446 | 6.373 | 16.003 | 9.983 | 19.613 |
| April | 1.076 | .170 | 1.981 | -.027 | 1.843 | .309 | 2.179 |
| May | .287 | -.270 | .843 | -.398 | .759 | -.185 | .972 |
| June | .590 | .090 | 1.091 | -.111 | 1.014 | .166 | 1.291 |
| July | 32.997 | 10.706 | 55.287 | 5.537 | 51.267 | 14.727 | 60.457 |
| August | 42.127 | 29.059 | 55.196 | 26.037 | 53.147 | 31.107 | 58.217 |
| September | 17.681 | 5.847 | 29.514 | 3.121 | 27.661 | 7.701 | 32.241 |

* The only justification for the number of significant figures in the mean monthly discharge rate is to permit more accurate conversion to acre-foot of total flow within the month.

**Negative lower confidence bounds arise because of the normal assumption made on the distribution of the means. In a sense, negative values suggest a major weakness of the notion of confidence bounds.

The lag-one autoregressive model (Fiering, 1967) :

$$x_i - \overline{x} = r_1 (x_{i-1} - \overline{x}) + \varepsilon_i . \tag{3}$$

exemplifies the first approach; in eq.(3) $x_i$ and $x_{i-1}$ are the flows in adjacent time periods i and i-1, $r_1$ is the lag-one serial correlation coefficient, and $\varepsilon_{i-1}$ is a random component that allows for preservation of the variance of $x_i$ or of both the variance and skewness ($g_1$). Then, either a N(0,1) distribution or a gamma distribution, G(0,1,g) is sampled to obtain $E_i$ values at each time i. Eq. (3) is not proposed as a model for ephemeral flows but is presented here and later only to illustrate the conceptual relationship between deterministic and stochastic concepts.

The second approach is more general in that one is willing to assume more about the overall properties of the stochastic process. The probability distribution of a state transition may be postulated outright on the basis of prior knowledge or may be determined from empirical analysis of actual data. The latter direction requires large sample sizes as will be shown shortly for monthly flows on Rillito Creek. In the former case, it is common in hydrology to assume a bivariate normal distribution for the flow $x_i$ or for some transform like the logarithm ($\log x_i$); the use of logarithms is, however, fraught with difficulties when the moment properties of the original $x_i$'s are compared with those moment properties derived from the inverse transforms of $\log x_i$ obtained from Eq. (3). This arises because the logarithmic transform does not induce a normal distribution for most hydrologic variables (except perhaps for annual flood extremes, mean annual flow, or precipitation extremes in some cases). It is for these reasons and others that we chose to pursue our analysis in the "natural" space of the flows rather than in the logarithmic space.

The transition from one state (flow) at time t, S(t), to another state at time (t+1), S(t+1), may be generally written as

$$S(t+1) = F\,[S(t),\, X(t)\,] \tag{4}$$

wherein F is a state transition function that may be deterministic or stochastic and X(t) is an input disturbance at time t. For example, in eq.(3), if $\varepsilon_i = 0$, then F is deterministic and the transition from $x_{i-1}$ to $x_i$ is given by the linear regression equation:

$$x_i - \bar{x} = r_1(x_{i-1} - \bar{x}) \tag{5}$$

wherein $r_1$ is simply the least squares estimate of the slope of a straight line. Eq.(5) is the discrete form of the classical hydrograph recession equation:

$$x(t) = x(o)\, \exp\,(-Kt) \tag{6}$$

in continuous time wherein x(o) is the initial flow rate for the recession and K is the recession or decay constant. Dividing eq.(6) by $x(t-1) = x(o)\exp(-K(t-1))$ we obtain:

$$x(t) = x(t-1)e^{-K} \tag{7}$$

from which we find $r_1 = e^{-K}$ or $\ln(1/r_1) = K$, thus permitting us to compute easily the recession constant K from the serial correlation coefficient $r_1$. In this manner, we see an interconnection between deterministic and stochastic concepts; the transition from x(t-1) to x(t) is deterministic at any one time instant t but the overall pattern is stochastic because x(t) varies stochastically.

An example of a probabilistic state transition F is given by a Markov chain:

$$P\,(\,x_i \mid x_{i-1}) = P\,(\,x_i \mid x_{i-1},\, x_{i-2},\, \ldots) \tag{8}$$

in which the probability $P(x_i|x_{i-1})$ tells us that the probability of being in state $x_i$ at time i depends only on the state at time i-1 and not on the infinite past. The generality of eq.(8) is not well understood because too often it is misrepresented as eq.(3). In fact, eq.(8) encompasses nonlinear state transitions in contrast to that given by eqs.(3) and (5). The Markov condition encompasses both linear and nonlinear differential equations that might be used as models for F [ S(t), X(t) ]. A further discussion of these issues is given by Denny, et al., (1971) and also by Kisiel and Duckstein (1971) who show how nonlinear behavior of the hydrograph affects the predictions given by eq.(3).

If prior information justifies use of a bivariate normal distribution, then eq.(8) may be written as a joint density function:

$$p(x_i, x_{i-1}) = \frac{1}{2\pi\ \sigma_{x_i}\ \sigma_{x_{i-1}}\ (1-r_1^{\ 2})^{1/2}} \exp\left(- \frac{z_{x_i}^{\ 2} - 2r_1\ z_{x_i}\ z_{x_{i-1}} + z_{x_{i-1}}^{\ 2}}{2(1-r_1^{\ 2})}\right) \tag{9}$$

in which

$$z_{x_i} = \frac{x_i - \mu_{x_i}}{\sigma_{x_i}} \tag{10}$$

$\sigma_{x_i}$ and $\sigma_{x_{i-1}}$ are the standard deviations of $x_i$ and $x_{i-1}$, respectively, and $\mu_{x_i}$ and $\mu_{x_{i-1}}$ are the means of $x_i$ and $x_{i-1}$, respectively. If the stochastic process $\{x_i;\ i=1,2,\ldots\}$ is stationary, then $\mu_{x_i} = \mu_{x_{i-1}}$ and $\sigma_{x_i} = \sigma_{x_{i-1}}$, and $r_1$ is constant over any pair of adjacent time intervals. Thus, we have given the necessary assumptions that justify computation of $\bar{x}$ and s as estimates $\mu_x$ and $s_x$ and of $r_1$ over a sample of size n. If eq.(9) is valid as a model for probabilistic state transitions, then either tables of correlated normal variates or a computer program may be used to simulate a process like that given by eq.(3).

Empirical evaluation of state transition probabilities

State transition probabilities for monthly flows on the Rillito were evaluated for transitions from a state of zero flow in one month to zero flow in the following month or to one of ten other bounded flow intervals. This was done by counting the number of times a monthly flow satisfied these conditions. The process was repeated for transitions from finite flow to zero or finite flow in the next month. The procedure was repeated for the 12 pairs of adjacent months and are given in Table 5.

From Table 5 we note that many of the cells in the 12 x 12 state transition matrix are empty. This sparsity is a function of record length n and size of class intervals (eleven in this case) used to capture the fineness of state transitions. Each cell gives the absolute frequency $f_{ij}$ with which the particular transition occurred in the record; thus, there were 31 transitions from zero flow to zero flow for the October-November pair. To convert this to an estimate of probability of transition from state i to state j, $P_{ij}$, use

$$\frac{f_{ij}}{\sum_{j=1}^{12} f_{ij}} = \frac{f_{ij}}{f_i} \tag{11}$$

in which $f_i$ is the total number of state transitions in row i. Note that $\sum_{j=1}^{12} P_{ij} = 1$ by definition because it is certain that a state transition has occurred. Stable estimates of $P_{ij}$ are possible only if sufficient data were available. However, inspection of the state transition tables suggests that many of the cells should have a finite value rather than zero. In the next section, we put forward a subjective procedure for filling in these cells.

Subjective estimation of state transition probabilities

Herein, a procedure is proposed that allows for prior knowledge of the investigator (including his biases). There is no claim that consistent results are possible from one person to another. Nonetheless, serious consideration of the approach seems warranted for two reasons: (a) inadequacy of "objective" methods as outlined earlier in this paper, and (b) common use of subjective guessing in actual hydraulic practice. Pending resolution of problems in the stochastic modeling of ephemeral flows, a structured subjective approach to data analysis offers the chance of consistent results from person to person. Each person's

Table 5. Frequency of specified state transitions from month i to month i+1 for monthly flows (in cfs) on Rillito Creek (for period of 1909-1966).

(1) October-November state transitions

| OCT State i \ NOV State j | 0 | 0-10 | 11-20 | 21-30 | 31-40 | 41-50 | 51-60 | 61-80 | 81-100 | 101-150 | 151-250 | 251-largest values |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 31 | 6 | - | - | 1 | - | - | - | - | - | - | - |
| 0-10 | 10 | 7 | 1 | 1 | - | 1 | - | - | - | - | - | - |
| 11-20 | - | - | - | - | - | - | - | - | - | - | - | - |
| 21-30 | - | - | - | - | - | - | - | - | - | - | - | - |
| 31-40 | - | - | - | - | - | - | - | - | - | - | - | - |
| 41-50 | - | - | - | - | - | - | - | - | - | - | - | - |
| 51-60 | - | - | - | - | - | - | - | - | - | - | - | - |
| 61-80 | - | - | - | - | - | - | - | - | - | - | - | - |
| 81-100 | - | - | - | - | - | - | - | - | - | - | - | - |
| 101-150 | - | - | - | - | - | - | - | - | - | - | - | - |
| 151-250 | - | - | - | - | - | - | - | - | - | - | - | - |
| 251-largest value | - | - | - | - | - | - | - | - | - | - | - | - |

Table 5. continued

(2) November-December state transitions

| NOV State i \ DEC State j | 0 | 0-10 | 11-20 | 21-30 | 31-40 | 41-50 | 51-60 | 61-80 | 81-100 | 101-150 | 151-250 | 251-largest values |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 34 | 4 | - | 1 | - | - | - | - | 1 | - | - | 1 |
| 0-10 | 6 | 3 | 1 | - | - | 1 | - | - | 1 | - | 1 | - |
| 11-20 | - | 1 | - | - | - | - | - | - | - | - | - | - |
| 21-30 | - | - | - | - | - | - | - | - | - | - | - | 1 |
| 31-40 | - | - | - | 1 | - | 1 | - | - | - | - | - | - |
| 41-50 | - | - | - | - | - | - | - | - | - | - | - | - |

(3) December-January state transitions

| DEC State i \ JAN State j | 0 | 0-10 | 11-20 | 21-30 | 31-40 | 41-50 | 51-60 | 61-80 | 81-100 | 101-150 | 151-250 | 251-largest value |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 29 | 7 | 1 | - | 1 | 1 | - | - | - | - | - | 1 |
| 0-10 | 2 | 3 | - | 2 | 1 | - | - | - | - | - | - | - |
| 11-20 | - | 1 | - | - | - | - | - | - | - | - | - | - |
| 21-30 | - | 1 | - | 1 | - | - | - | - | - | - | - | - |
| 31-40 | - | - | - | - | - | - | - | - | - | - | - | - |
| 41-50 | - | - | - | - | - | - | - | 1 | - | - | 1 | - |
| 51-60 | - | - | - | - | - | - | - | - | - | - | - | - |
| 61-80 | - | - | - | - | - | - | - | - | - | - | - | - |
| 81-100 | - | 2 | - | - | - | - | - | - | - | - | - | - |
| 101-150 | - | - | - | - | - | - | - | - | - | - | - | - |
| 151-250 | - | - | - | - | - | 1 | - | - | - | - | - | - |
| 251-largest value | - | 1 | - | - | - | - | - | - | - | - | - | 1 |

Table 5. continued

(4) January-February state transitions

| JAN State i \ FEB State j | 0 | 0-10 | 11-20 | 21-30 | 31-40 | 41-50 | 51-60 | 61-80 | 81-100 | 101-150 | 151-250 | 251-largest values |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 23 | 5 | - | - | - | - | 2 | - | 1 | - | - | - |
| 0-10 | 5 | 4 | 3 | 1 | - | - | - | - | 1 | - | - | - |
| 11-20 | - | 1 | - | - | - | - | - | - | - | - | 1 | - |
| 21-30 | - | 2 | - | - | - | - | - | 1 | - | - | - | - |
| 31-40 | 2 | - | - | - | - | - | - | - | - | - | - | - |
| 41-50 | - | 1 | - | - | - | - | - | 1 | - | - | - | - |
| 51-60 | - | - | - | - | - | - | - | - | - | - | - | - |
| 61-80 | - | - | - | - | - | - | - | - | - | - | 1 | - |
| 81-100 | - | - | - | - | - | - | - | - | - | - | - | - |
| 101-150 | - | - | - | - | - | - | - | - | - | - | - | - |
| 151-250 | 1 | - | - | - | - | - | - | - | - | - | - | - |
| 251-largest value | - | - | - | - | 1 | - | - | - | - | - | - | 1 |

Table 5. continued

(5) February-March state transitions

| FEB State i \ MAR State j | 0 | 0-10 | 11-20 | 21-30 | 31-40 | 41-50 | 51-60 | 61-80 | 81-100 | 101-150 | 151-250 | 251-largest values |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 21 | 5 | - | - | 2 | - | 1 | 1 | - | 1 | - | - |
| 0-10 | 1 | 9 | 1 | - | - | - | - | - | - | 2 | - | - |
| 11-20 | 2 | 1 | - | - | - | - | - | - | - | - | - | - |
| 21-30 | - | - | - | 1 | - | - | - | - | - | - | - | - |
| 31-40 | - | - | - | - | - | - | 1 | - | - | - | - | - |
| 41-50 | - | - | - | - | - | - | - | - | - | - | - | - |
| 51-60 | 1 | 1 | - | - | - | - | - | - | - | - | - | - |
| 61-80 | - | - | 1 | - | - | - | - | - | - | 1 | - | - |
| 81-100 | - | 1 | - | 1 | - | - | - | - | - | - | - | - |
| 101-150 | - | - | - | - | - | - | - | - | - | - | - | - |
| 151-250 | - | - | - | - | 2 | - | - | - | - | - | - | - |
| 251-largest value | - | - | - | - | - | - | - | - | - | - | 1 | - |

Table 5. continued

(6) March-April state transitions

| MAR State i \ APR State j | 0 | 0-10 | 11-20 | 21-30 | 31-40 | 41-50 | 51-60 | 61-80 | 81-100 | 101-150 | 151-250 | 251-largest values |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 23 | 2 | - | - | - | - | - | - | - | - | - | - |
| 0-10 | 11 | 5 | 1 | - | - | - | - | - | - | - | - | - |
| 11-20 | 2 | - | - | - | - | - | - | - | - | - | - | - |
| 21-30 | 2 | - | - | - | - | - | - | - | - | - | - | - |
| 31-40 | 2 | 2 | - | - | - | - | - | - | - | - | - | - |
| 41-50 | - | - | - | - | - | - | - | - | - | - | - | - |
| 51-60 | 2 | - | - | - | - | - | - | - | - | - | - | - |
| 61-80 | - | 1 | - | - | - | - | - | - | - | - | - | - |
| 81-100 | - | - | - | - | - | - | - | - | - | - | - | - |
| 101-150 | 3 | 1 | - | - | - | - | - | - | - | - | - | - |
| 151-250 | - | - | - | 1 | - | - | - | - | - | - | - | - |
| 251-largest value | - | - | - | - | - | - | - | - | - | - | - | - |

Table 5. continued

(7) April-May state transitions

| APR State i \ MAY State j | 0 | 0-10 | 11-20 | 21-30 | 31-40 | 41-50 | 51-60 | 61-80 | 81-100 | 101-150 | 151-250 | 251-largest values |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 41 | 2 | 1 | - | - | - | - | - | - | - | - | - |
| 0-10 | 11 | 1 | - | - | - | - | - | - | - | - | - | - |
| 11-20 | 1 | - | - | - | - | - | - | - | - | - | - | - |
| 21-30 | 1 | - | - | - | - | - | - | - | - | - | - | - |
| 31-40 | - | - | - | - | - | - | - | - | - | - | - | - |

(8) May-June state transitions

| MAY State i \ JUN State j | 0 | 0-10 | 11-20 | 21-30 | 31-40 | 41-50 | 51-60 | 61-80 | 81-100 | 101-150 | 151-250 | 251-largest values |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 44 | 10 | - | - | - | - | - | - | - | - | - | - |
| 0-10 | 3 | - | - | - | - | - | - | - | - | - | - | - |
| 11-20 | - | 1 | - | - | - | - | - | - | - | - | - | - |
| 21-30 | - | - | - | - | - | - | - | - | - | - | - | - |

Table 5. continued

(9) June-July state transitions

| JUNE State i \ JULY State j | 0 | 0-10 | 11-20 | 21-30 | 31-40 | 41-50 | 51-60 | 61-80 | 81-100 | 101-150 | 151-250 | 251-largest values |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 6 | 21 | 6 | 4 | 3 | 1 | 1 | 1 | 2 | - | - | 2 |
| 0-10 | - | 5 | 1 | - | 1 | 1 | 2 | - | - | 1 | - | - |
| 11-20 | - | - | - | - | - | - | - | - | - | - | - | - |

(10) July-August state transitions

| JUL State i \ AUG State j | 0 | 0-10 | 11-20 | 21-30 | 31-40 | 41-50 | 51-60 | 61-80 | 81-100 | 101-150 | 151-250 | 251-largest values |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 1 | 2 | - | 1 | 2 | - | - | - | - | - | - | - |
| 0-10 | 2 | 12 | 3 | - | 3 | 2 | - | 1 | - | 3 | - | - |
| 11-20 | - | - | 2 | 2 | 1 | - | 1 | - | - | 1 | - | - |
| 21-30 | 1 | - | - | - | 1 | - | - | 2 | 1 | - | - | - |
| 31-40 | - | - | - | - | 1 | - | - | - | - | 1 | - | - |
| 41-50 | - | - | - | - | - | 1 | - | 1 | 1 | 0 | - | - |
| 51-60 | - | 1 | - | 1 | - | - | 1 | - | - | - | - | - |
| 61-80 | - | - | - | - | - | - | - | - | - | 1 | - | - |
| 81-100 | - | - | - | - | - | 1 | - | - | - | - | 1 | - |
| 101-150 | - | 1 | - | - | - | - | - | - | - | - | - | - |
| 151-250 | - | - | - | - | - | - | - | - | - | - | - | - |
| 251-largest value | - | - | - | - | - | - | - | 1 | - | - | - | 1 |

Table 5. continued

(11) August-September state transitions

| AUG State i \ SEPT State j | 0 | 0-10 | 11-20 | 21-30 | 31-40 | 41-50 | 51-60 | 61-80 | 81-100 | 101-150 | 151-250 | 251-largest values |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 3 | - | 1 | - | - | - | - | - | - | - | - | - |
| 0-10 | 6 | 5 | 3 | 1 | - | - | - | 1 | - | - | - | - |
| 11-20 | - | 4 | - | - | - | - | - | - | - | 1 | - | - |
| 21-30 | - | 2 | 1 | - | - | 1 | - | - | - | - | - | - |
| 31-40 | 1 | 5 | 2 | - | - | - | - | - | - | - | - | - |
| 41-50 | - | 2 | 1 | - | - | 1 | - | - | - | - | - | - |
| 51-60 | - | 1 | 1 | - | - | - | - | - | - | - | - | - |
| 61-80 | - | 4 | - | - | - | - | 1 | - | - | - | - | - |
| 81-100 | 1 | 1 | - | - | - | - | - | - | - | - | - | - |
| 101-150 | 1 | 2 | 1 | - | - | - | - | 1 | - | - | - | 1 |
| 151-250 | - | - | - | - | - | - | - | - | 1 | - | - | - |
| 251-largest value | - | 1 | - | - | - | - | - | - | - | - | - | - |

Table 5. continued

(12)September-October state transitions

| SEPT State i \ OCT State j | 0 | 0-10 | 11-20 | 21-30 | 31-40 | 41-50 | 51-60 | 61-80 | 81-100 | 101-150 | 151-250 | 251-largest values |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 7 | 5 | - | - | - | - | - | - | - | - | - | - |
| 0-10 | 17 | 10 | - | - | - | - | - | - | - | - | - | - |
| 11-20 | 7 | 3 | - | - | - | - | - | - | - | - | - | - |
| 21-30 | - | 1 | - | - | - | - | - | - | - | - | - | - |
| 31-40 | 0 | - | - | - | - | - | - | - | - | - | - | - |
| 41-50 | 2 | - | - | - | - | - | - | - | - | - | - | - |
| 51-60 | 1 | - | - | - | - | - | - | - | - | - | - | - |
| 61-80 | 2 | - | - | - | - | - | - | - | - | - | - | - |
| 81-100 | 1 | - | - | - | - | - | - | - | - | - | - | - |
| 101-150 | 1 | - | - | - | - | - | - | - | - | - | - | - |
| 151-250 | - | - | - | - | - | - | - | - | - | - | - | - |
| 251-largest value | - | 1 | - | - | - | - | - | - | - | - | - | - |

insight can suggest modifications so as to strengthen the consistency of the procedure.

The objective is to construct a square matrix of subjective state transition probabilities so as to generate sequences of monthly flow that possess properties similar to the historical record. Only in the probabilistic (and statistical) sense are the synthetic and historic sequences presumed to be similar.

In the procedure, a freedom of choice exists for (a) size of m in the m x m square matrix, (b) size of flow interval for each cell of the matrix, (c) subjectiveness in filling in the empty cells of the matrix, and (d) generating function that determines variable magnitude of flow.

The following set of rules and guidelines are given for the sake of consistency:

(a) Based on the number of data points, determine the size m of the matrix to obtain meaningful probabilities. (We had 696 data points in 58 years and used 12 x 12 matrices since the range of monthly flows was 0 cfs to 976 cfs with sample means ranging from 0.287 cfs/month to 42.13 cfs/months. In this manner, we hoped to allow freedom to have sufficient variability in the synthetically generated flows.)

(b) Choose the flow intervals for the cells. They do not have to be evenly spaced; in semi-arid and arid regions, a cell should be allocated for a flow of 0 cfs.

(c) Compute the "historic" transition probability matrices between each pair of months by entering the number of flows that fill a cell given that the previous month's flow fell in a certain interval. From these numbers compute the transition probabilities for each cell of the matrix.

(Hundredths of cfs is precise enough for most cases. Note that the hundredths arise only because of averaging; no claim is made that flows are measured with such precision.) Make a note of the mean and maximum flow for each month; confidence regions as given in Tables 3 and 4 may help in the subjective analysis.

(d) Based on the mean, maximum flow, historic probabilities and number of occurrences, compute or fill in the matrices subjectively using the following rules and guidelines:

(1) Summation of transition probabilities across each row should equal one.

(2) Precision to the hundredths.

(3) Rows in the vicinity of the mean (where the majority of flows are expected) are most sensitive from a modeling viewpoint than rows where less flows are expected.

(4) If a row has $p_{ij}$ of 0.50 and 0.50 in two cells, it is probable that this row contained only 2 flows. Whereas a row with $p_{ij}$ of 0.10 in each of 10 cells probably was based on 10 flows. Thus, more subjective weight should be placed on rows where it appears that a large number of flows was used to compute $p_{ij}$; less weight should be placed in rows where few occurrences arise. Knowledge of $f_i$, the absolute frequency, for each row would help in this evaluation of the $p_{ij}$'s.

(5) When a zero $p_{ij}$ is encountered between finite $p_{ij}$'s, the empty cell is assigned a $p_{ij}$ that reasonably fits between the adjacent $p_{ij}$'s.

(6) Usually the first 2 or 3 rows of the matrix are of most significance since the majority of flows will probably fit in these intervals (see

Table 5). The simulation results would be more sensitive in this region than in higher-indexed rows.

(7) If $p_{ij}$ is the last probability in a row and $j \neq m$, then assign a finite p to $p_{i(j+1)}$ and zero to $i^{\underline{th}}$ row cells with column index greater than j+1. Thus, if maximum flow in a row is 40 cfs and it lies in cell #5 out of 10, then the last finite probability in that row should be assigned to cell #6 and cells #7 to 10 will have zero p. This rule does not apply if the last cell in the row has finite p. The rule allows generation of flows larger than actually recorded.

(8) A certain consistency should be maintained in the subjective matrices both in the months and between months. For example, the $p_{ij}$ in the extreme cells should be greater than or equal to the p in the previous cell because of possible dependence from month to month (see Table 2) and because a higher flow in a month is more probable if the preceding month's flow was high (see Table 1 for relative values of means for adjacent months).

To illustrate the above rules, consider the simple 6 x 6 matrix in Table 6. Little weight is assigned to the $p_{ij}$ in the lower 4 rows because a small number of occurrences entered the cells.

The last two authors of this paper used the above rules to fill in the cells shown in Table 5. A Student's t-test showed that the historic and simulated mean flows for the months were statistically similar but the Fisher's ratio test on variances showed that their simulated variances were "out of the ball park." The latter result was produced by the tendency to assign too high or a very small probability to transitions to high flows, that is, the senior authors had a tendency

Table 6. Illustration of development of subjective matrix (Maximum flow = 220 cfs; $\overline{x}$ = 25.0 cfs).

"Historic" matrix

| Month i \ Month j = i+1 | 0 | 1-20 | 21-50 | 51-100 | 101-200 | > 200 |
|---|---|---|---|---|---|---|
| 0 | .55 | .33 | .12 | - | - | - |
| 1-20 | .61 | .29 | .08 | .02 | - | - |
| 21-50 | .40 | - | .20 | - | .20 | .20 |
| 51-100 | - | .50 | - | - | - | .50 |
| 101-200 | - | - | - | - | - | - |
| > 200 | - | - | - | 1.0 | - | - |

Subjectively assigned matrix:

| | 1 | 2 | 3 | 4 | 5 | 6 |
|---|---|---|---|---|---|---|
| 1 | .55 | .30 | .13 | .02 | - | - |
| 2 | .60 | .30 | .05 | .03 | .02 | - |
| 3 | .50 | .30 | .10 | .05 | .03 | .02 |
| 4 | .50 | .30 | .10 | .05 | .03 | .02 |
| 5 | .50 | .30 | .10 | .05 | .03 | .02 |
| 6 | .50 | .30 | .10 | .05 | .03 | .02 |

to fill in all cells. This suggests that an upper limit should be placed on the guessing of probabilities at the extremities. Other results, consistent with the significant difference in variances, were the production of higher extremes than historically observed and decrease in number of zero flows. However, the cross-correlation matrix (Table 2) turned out quite well. The results suggest that the matrix might also be tested for finer breakdown of flow values.

Explanation of computer generation model

The subjective matrices are taken and converted to cumulative probability matrices across each row. An initial flow is taken, say 0, and the next flow is computed by random number generation as follows:

1) Go to row dictated by last flow generation
2) Get a pseudo-random number R
3) Compare that number to cumulative probabilities in the specified row to ascertain location of the cell in that row
4) Get another pseudo-random number R
5) Compute new flow by taking the lower bound of specified cell and adding some function of the generated number in 4) to the lower bound. Return to 1).

The function of the random number (R) used in this routine was $R^2$. This yields an exponential function. There is only intuition behind the choice of this function, but the model is not highly sensitive to any function so long as it is somewhat between this type of exponential and a uniform distribution. Approximate bounds could be stated as $R^3 \leq f(R) \leq R$.

Flow charts on the computer program are given in Appendix I. This represents the main generation procedure.

## SUMMARY AND CONCLUSIONS

A critique of statistical properties of monthly flows on an ephemeral stream has been given. An ad hoc subjective procedure is proposed for sequential generation of the monthly flows. Such a method seems justified for managerial purposes that are not concerned with the variability of flow within the month and with the large number of days within any month for which the flow is zero. It is our considered judgment that ephemeral flows should be modeled by starting with historical daily flows (at least). If this is done, then more meaningful monthly flow models (and then synthetic sequences) could be built. These would serve as a check on alternative monthly flow models (subjective matrix method proposed in this paper, lag-one autoregressive model, harmonic model, bivariate normal model, or bivariate log-normal model). Modern computers permit low cost analysis of hydrologic records at smaller time intervals like a day (or even an hour).

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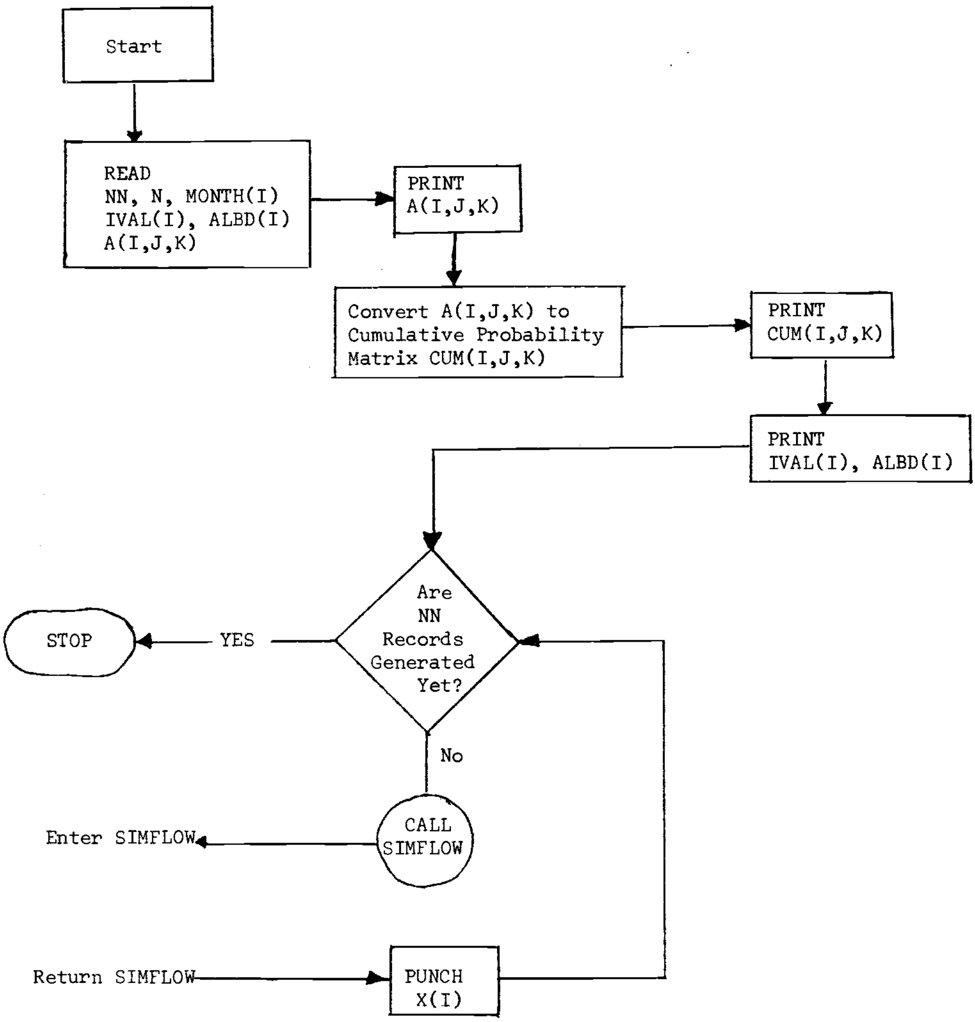
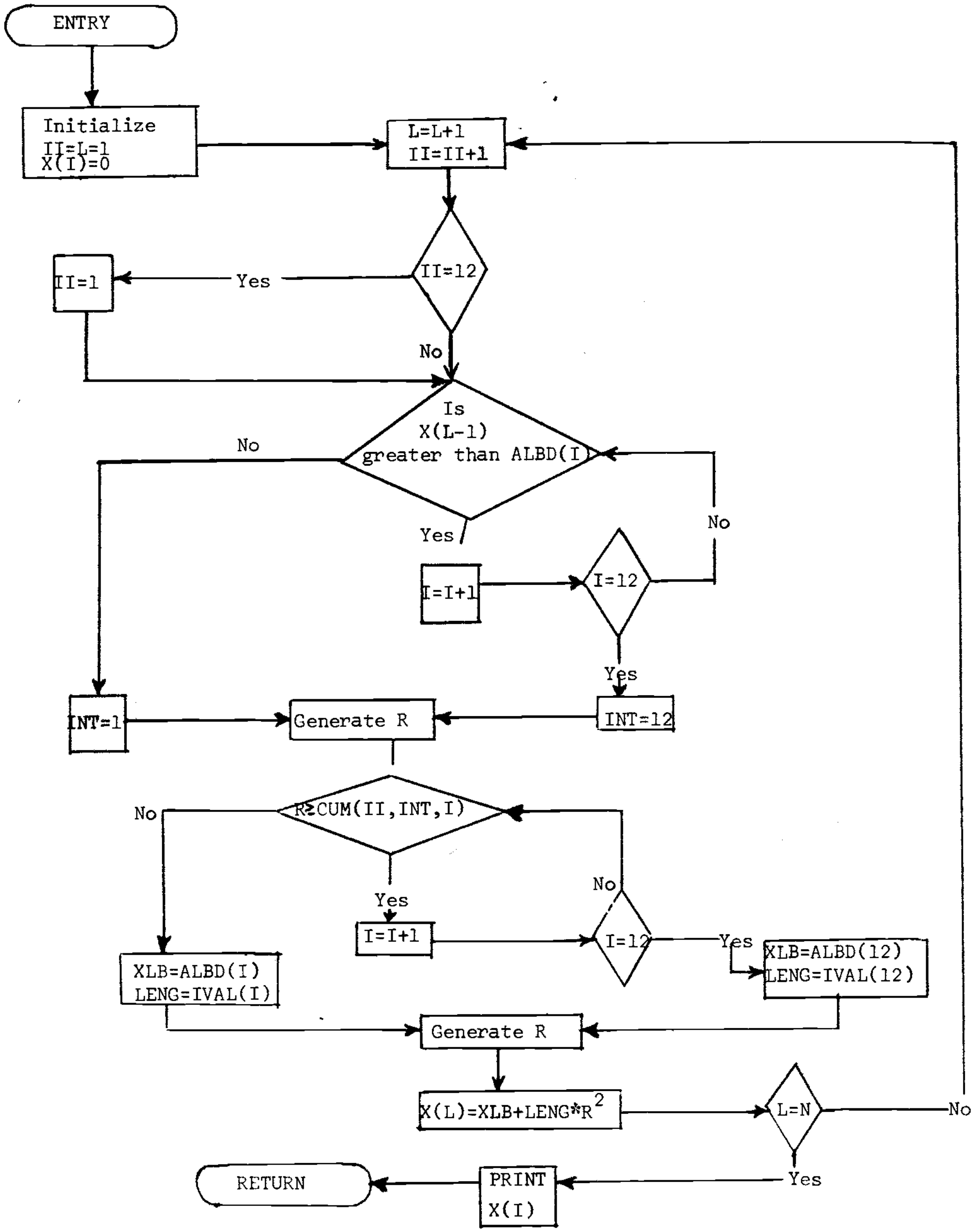
APPENDIX I

Flowchart for Streamflow Model



Appendix I (continued)

Subroutine SIMFLOW



Appendix I (continued)

Definitions of Variables Used in Flowchart

| | |
|---|---|
| NN | number of records to be generated |
| N | number of flows per record |
| MONTH (I) | array of abbreviations for the months |
| IVAL (I) | interval width |
| ALBD (I) | lower bound of the interval |
| A (I,J,K) | subjective matrix for month i, row j, column k |
| CUM (I,J,K) | cumulative subjective matrix for month i, row j, column k |
| X (I) | generated streamflows |
| II | month counter |
| L | flow counter |
| INT | row number of preceding flow |
| XLB | lower bound for next flow |
| LENG | width of interval |

# A PROPOSED MODEL FOR FLOOD ROUTING IN ABSTRACTING EPHEMERAL CHANNELS [1/]

Leonard J. Lane [2/]

## INTRODUCTION

In much of southern Arizona, almost all runoff from semiarid rangeland watersheds results from intense highly variable thunderstorm rainfall [Osborn and Reynolds (1963)]. Compounding the problems of predicting runoff resulting from such rainfall is the presence of broad alluvium-filled channels that abstract large quantities of runoff [Babcock and Cushing (1942), Keppel (1960), Keppel and Renard (1962), Allis, Dragoun, and Sharp (1964), Renard and Keppel (1966), Qashu and Buol (1967), Wallace and Renard (1967), Burkham (1970a, 1970b), and Lane, Diskin, and Renard (1971)]. These abstractions or transmission losses play an important role in diminishing streamflow, in supporting riparian vegetation, and in providing natural water recharge to local aquifers and the regional groundwater [Renard (1970)].

Ephemeral streams in arid or semiarid zones introduce an added difficulty in flood routing because of the large transmission losses [Keppel and Renard (1962)]. An example of the influence of transmission

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losses on hydrographs in ephemeral streams is shown in Figure 1. These hydrographs represent the inflow and outflow on a 4.1-mile reach of channel on the Walnut Gulch Experimental Watershed. Notice that the maximum discharge rate is reduced some 50% while the volume of flow is reduced about 35% in traveling the 4.1 miles in the absence of tributary inflow. While this is intended as a typical example, the reduction in volume is not always less than the reduction in peak discharge. In this study, flood movement and transmission losses are represented by a system using storage in the channel reach as a state variable which determines loss rates.

## FLOOD ROUTING PROCEDURES

The term flood routing refers to the determination of a flood wave at a downstream location when the flood wave at some upstream location is known. While there is an extensive body of literature on flood routing, a few primary sources are Gilcrest (1950); Linsley, Kohler, and Paulhus (1949, 1958); SCS Engineering Handbook (1957); and Chow (1959, 1964). In very irregular and rough natural channels, the changes in the flood wave or hydrograph can be significant. Chow (1959) differentiates between hydraulic and hydrologic flood routing as follows: "The hydraulic method of flood routing is distinguished from the hydrologic method by the fact that the hydraulic method is based on the solution of the basic differential equations for unsteady flow in open channels whereas the hydrologic method makes no direct use of these equations but approximates in some sense to their solutions." Throughout the remainder of this paper, the term flood routing will be restricted to the simplified hydrologic method of flood routing.

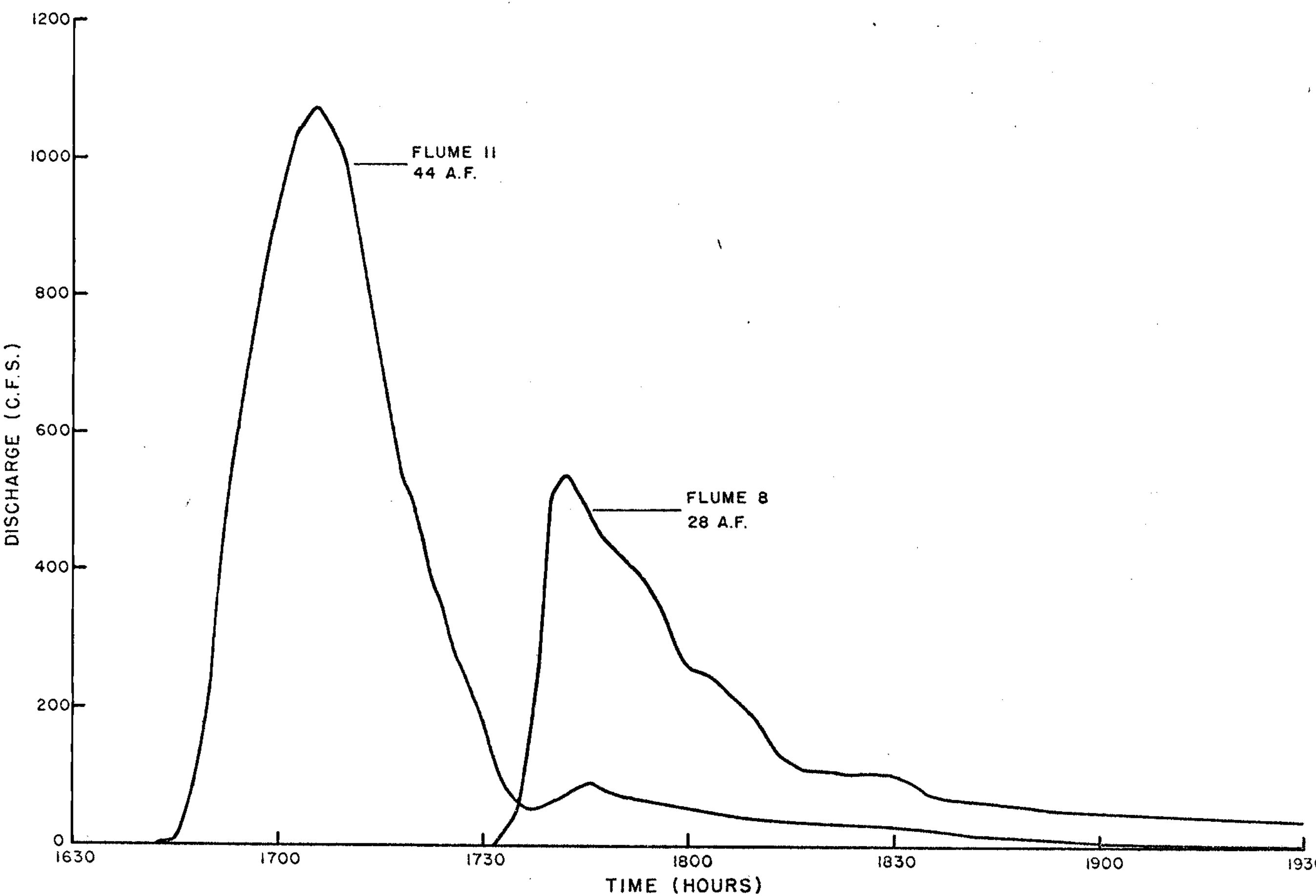


Figure 1. Hydrographs for reach 11-8, event of 300766.

In natural channels with little or no tributary inflow or outflow and no losses or abstractions, the inflow, outflow, and storage are related by the continuity equation

$$\dot{S} + Q = P \tag{1}$$

where:

$\dot{S}$ is the time derivative of storage,

$Q$ is the outflow from the reach, and

$P$ is the inflow to the channel reach.

In addition, P, Q, and S are related to the depth of flow, Chow (1959). If X is defined as a weighting factor, then the storage within a reach is given by

$$S = XS_i + (1-X)S_o \tag{2}$$

where $S_i$ and $S_o$ are storages determined by the depths at the upstream and downstream points of the reach. Since $S_i$ and $S_o$ are determined from the inflow and outflow, respectively,

$$S = K[XP^n + (1-X)Q^n] \tag{3}$$

where the variabiles are as defined above, and K and n are constants. In rectangular channels n = 0.6, and in natural channels n is assumed equal to unity [Chow (1959)]. The popular Muskingum method is a form of Equation 3 where n = 1.0 [McCarthy (1938)].

Kulandaiswamy, Krishnaswami, and Ramalingam (1967) list three storage equations commonly used in flood routing:

$$S = KQ \tag{4}$$

$$S = K\ [XP + (1 - X)Q] \tag{5}$$

$$S = KQ^n \tag{6}$$

Equation 4 is the so-called linear reservoir model, Equation 5 is the Muskingum method, and Equation 6, unlike the other two, can be nonlinear if n is not equal to 1.0. The above authors then proposed a more general storage equation derived by Kulandaiswamy (1964) relating the derivatives of the inflow and outflow to storage. In a brief but elegant paper, Diskin (1967) derived the general solution of the Muskingum flood routing equation using Laplace transforms. Other than specifying the parameters for applying the Muskingum method, which can be found in several handbooks, the theoretical development of the method was completed by Diskin (1967).

## GENERAL FORM OF THE MODEL

The proposed model is also a simplification of the known process and is a storage model. In this study we added transmission losses so that the continuity equation becomes

$$\dot{S} + L + Q = P \tag{7}$$

where:

$\dot{S}$ is the time derivative of the storage,

L is the transmission loss rate,

Q is outflow from the reach, and

P is the inflow to the reach.

All variables are functions of time.

If Equation 7 is rewritten with L and Q as functions of the storage, we have

$$\dot{S} + F(S) + G(S) = P \tag{8}$$

with

$$L = F(S) \tag{9}$$

and

$$Q = G(S) \tag{10}$$

where the variables are as defined previously. Equation 8 is a general representation of a storage routing model, and can be linear or nonlinear depending on the forms of F and G.

## Flood Routing in an Ephemeral Channel

In the usual situation where only the inflow is known and the losses, F(S), and outflow from the channel reach, G(S) are to be determined, the continuity equation is

$$\dot{S} + F(S) + G(S) = P \tag{8}$$

In this analysis it was assumed that

$$L(t) = F(S) = C_1 S^{b_1} \tag{11}$$

and

$$Q(t) = G(S) = C_2 S^{b_2} \tag{12}$$

Then the continuity equation becomes

$$\dot{S} + C_1 S^{b_1} + C_2 S^{b_2} = P \tag{13}$$

The above equation is proposed as a general storage equation for flood routing in ephemeral channels.

In addition to the differential equation, it is sometimes desirable to describe a conceptual model of the system. A general reservoir with abstractions or losses and a siphon outflow is proposed as the basic component of the model. The general model would then be a cascade of such components similar to the Nash cascade of linear reservoirs conceptual model of a watershed [Nash (1957)]. If we define $H_0$ as a critical value of depth (or storage) in the reservoir, then there are two separate cases or modes of operation: 1) $S < H_0$ so that there are losses but no outflow, and 2) $S \geq H_0$ so that there are both losses and outflow.

Numerical solutions to Equation 13 have been obtained using the Runge-Kutta method. Initial analyses are concentrating on determining the relative sensitivity of the solution to values of the four parameters. Solutions have been obtained for the case with arbitrary input. However, the next step is to determine the analytic solutions so that the components can be cascaded and the general solution obtained.

A Specific Case

In this case it is assumed that the inflow and outflow are known and further that the transmission losses are a linear function of the storage in the channel reach; that is

$$\dot{S} + C_1 S + Q = P \qquad (14)$$

With these assumptions, the problem becomes one of determining $C_1$, the coefficient relating storage and transmission losses. Rewriting Equation 14 and rearranging terms, both sides of the equation can be integrated so that

$$\int_0^t P(t)dt - \int_0^t Q(t)dt = C_1 \int_0^t S(t)dt + S(t) - S(0) \qquad (15)$$

where the variables are as defined previously and S(0) is the storage at time zero. In discrete form, assuming S(0) = 0, the integrations become summations and dt becomes $\Delta t$ so that,

$$S_N = \frac{2}{C_1\Delta t + 2}\left[\sum_{i=1}^{N} P_i\Delta t - \sum_{i=1}^{N} Q_i\Delta t - C_1 \sum_{i=1}^{N-1} S_i\Delta t - \frac{C_1\Delta t}{2} S_{N-1}\right] \tag{16}$$

which is an equation in terms of $S_N$, the storage at time $t = N\Delta t$. SUMP, SUMQ, SUMS, and SUML are defined as the total volumes of inflow, outflow, storage (integral of storage), and losses respectively. An initial approximation to the solution is:

$$\Delta S/\Delta t = P(t) - Q(t) - (\text{SUMP}-\text{SUMQ})/\text{TOT} \tag{17}$$

$$S_N = \sum_{i=1}^{N} P_i\ \Delta t - \sum_{i=1}^{N} Q_i\Delta t - N\Delta t(\text{SUMP}-\text{SUMQ})/\text{TOT} \tag{18}$$

$$C_1 = (\text{SUMP} - \text{SUMQ})/\text{SUMS} \tag{19}$$

$$L_N = C_1 S_N \tag{20}$$

where TOT is the total duration of flow at the point of interest. With this initial solution for S(t) and thus L(t), Equations 16, 18, 19, and 20 are solved by an iterative technique until the successive values of $C_1$ are arbitrarily close.

The analysis in this example is based on data derived from a 4.1-mile ephemeral channel reach on the Walnut Gulch Experimental Watershed in southeastern Arizona. The channel reach is steep with low banks and an alluvium bed from zero to 11 feet thick. The mean depth to conglomerate is about 6 feet. The hydrographs shown in Figure 1 are measured inflow and outflow to this reach reflecting the losses to the alluvium.

The iterative solution to Equation 16 as described above was obtained for 12 inflow-outflow hydrographs on the reach where there was little or no tributary inflow between measuring stations. Figure 2 is a plot of

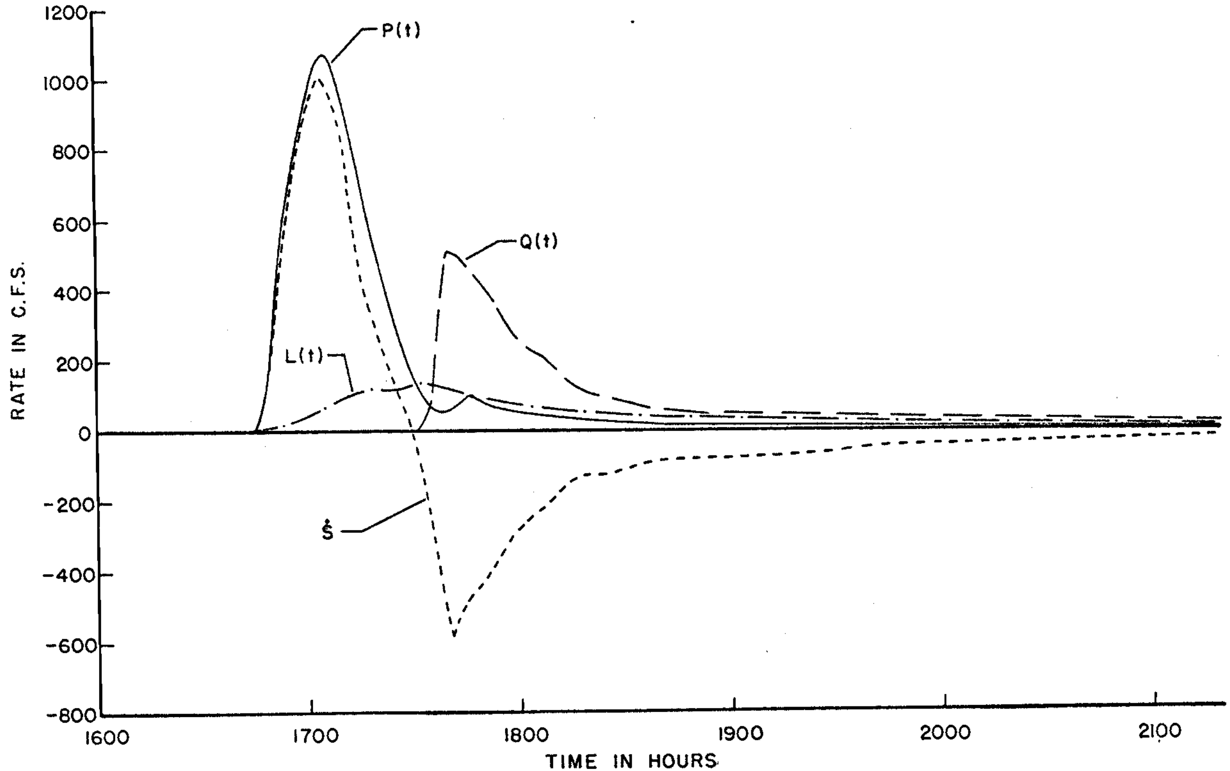


Figure 2. Solution to the differential equation assuming inflow and outflow are known, reach 11-8, event of 300766.

$P$, $Q$, $L$, and $\dot{S}$ for the hydrographs shown in Figure 1. Notice that $L$ is for the entire channel reach. For this event, as in all events, a final value of $C_1$ was obtained in 20 iterations or less. Table 1 shows hydrograph characteristics and the derived $C_1$ values for the 12 runoff events used in this study. The relation between $C_1$ and the peak discharge of the inflow hydrograph is shown in Figure 3. Although the relationship is not strong, $C_1$ seems to decreased with increasing inflow to the channel reach. The scatter about the regression line in Figure 3 is no doubt due to a variety of sources including the effects of antecedent moisture and measurement errors. If a prediction equation were desired, including these variables would undoubtedly improve the coefficient of determination. However, the decrease in $C_1$ with increasing inflow can be taken as a measure of the nonlinearity of the system for ephemeral channels.

## SUMMARY

A model for flood routing in abstracting ephemeral channels is proposed as a cascade of the general components. Initial analyses have yielded numerical solutions to the equations for the individual components. A linear form of a storage equation with the outflow known has been calibrated to 12 individual runoff events on a 4.1-mile reach of ephemeral channel. Wide variation in the parameters of this linear model with increasing inflow indicates that the assumption of a linear relation between transmission losses and storage is probably incorrect for ephemeral channels. Additional work is in progress to determine the analytical form of the solution for the individual components and for the cascade of components.

Table 1. Hydrograph characteristics and derived $C_1$ values for 12 runoff events on channel reach 11-8.

| Date of event | Peak Discharge Inflow | Outflow | Final Value of $C_1$ |
|---|---|---|---|
| | (cfs) | (cfs) | (sec-1) |
| 020863 | 210. | 14. | 0.0125 |
| 310764 | 97. | 2. | .0122 |
| 020864 | 720. | 460. | .0095 |
| 050864 | 360. | 140. | .0041 |
| 110964 | 2000. | 1900. | .0041 |
| 180865 | 26. | 7. | .0162 |
| 020965 | 97. | 76. | .0096 |
| 300766 | 1100. | 540. | .0054 |
| 130867 | 260. | 97. | .0183 |
| 250967 | 70. | 18. | .0278 |
| 020868 | 330. | 64. | .0153 |
| 050868 | 1100. | 430. | .0075 |

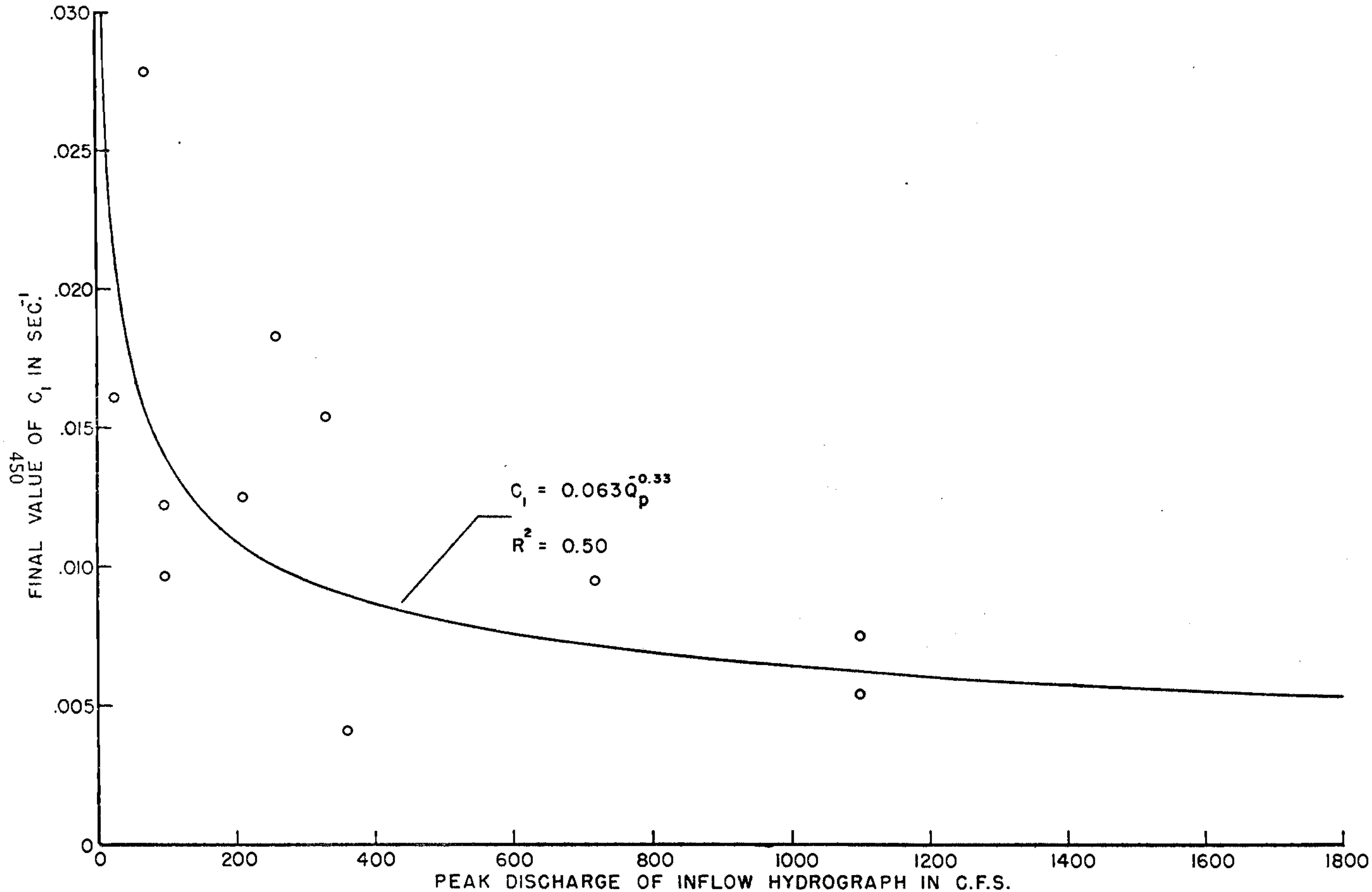


Figure 3. Variation in $C_1$ with increasing inflow to reach 11-8.

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# BED MATERIAL CHARACTERISTICS AND TRANSMISSION LOSSES IN AN EPHEMERAL STREAM [1/]

J. B. Murphey, L. J. Lane, and M. H. Diskin [2/]

## INTRODUCTION

Walnut Gulch is an ephemeral tributary to the San Pedro River, joining it near Fairbank, Arizona. The upper 58 square miles of the Walnut Gulch watershed are under intensive study by the Agricultural Research Service of the U.S. Department of Agriculture (Fig. 1). This bed material study deals with Hughes Draw, a 5.98-square-mile subarea of Walnut Gulch. The watershed has a youthful drainage system incised into a high foothills alluvial fan. Alluvial materials range from clays and silt to well-cemented boulder conglomerates. The mixed brush-grass vegetation is typical of southeastern Arizona and southwestern New Mexico rangeland. Elevation varies from 4000 feet at the outlet to about 6200 feet at the upper boundary in the Dragoon Mountains. Most of the streamflow occurs between July and early October and is derived from intense convective storms. An average of 6 to 13 such flows are recorded annually at the gaging sites. Most flows last less than 6 hours, and the channels are dry 99% of the time.

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[1/] Contribution of the Southwest Watershed Research Center, Agricultural Research Service, USDA, Soil and Water Conservation Research Division, in cooperation with the Arizona Agricultural Experiment Station, Tucson.

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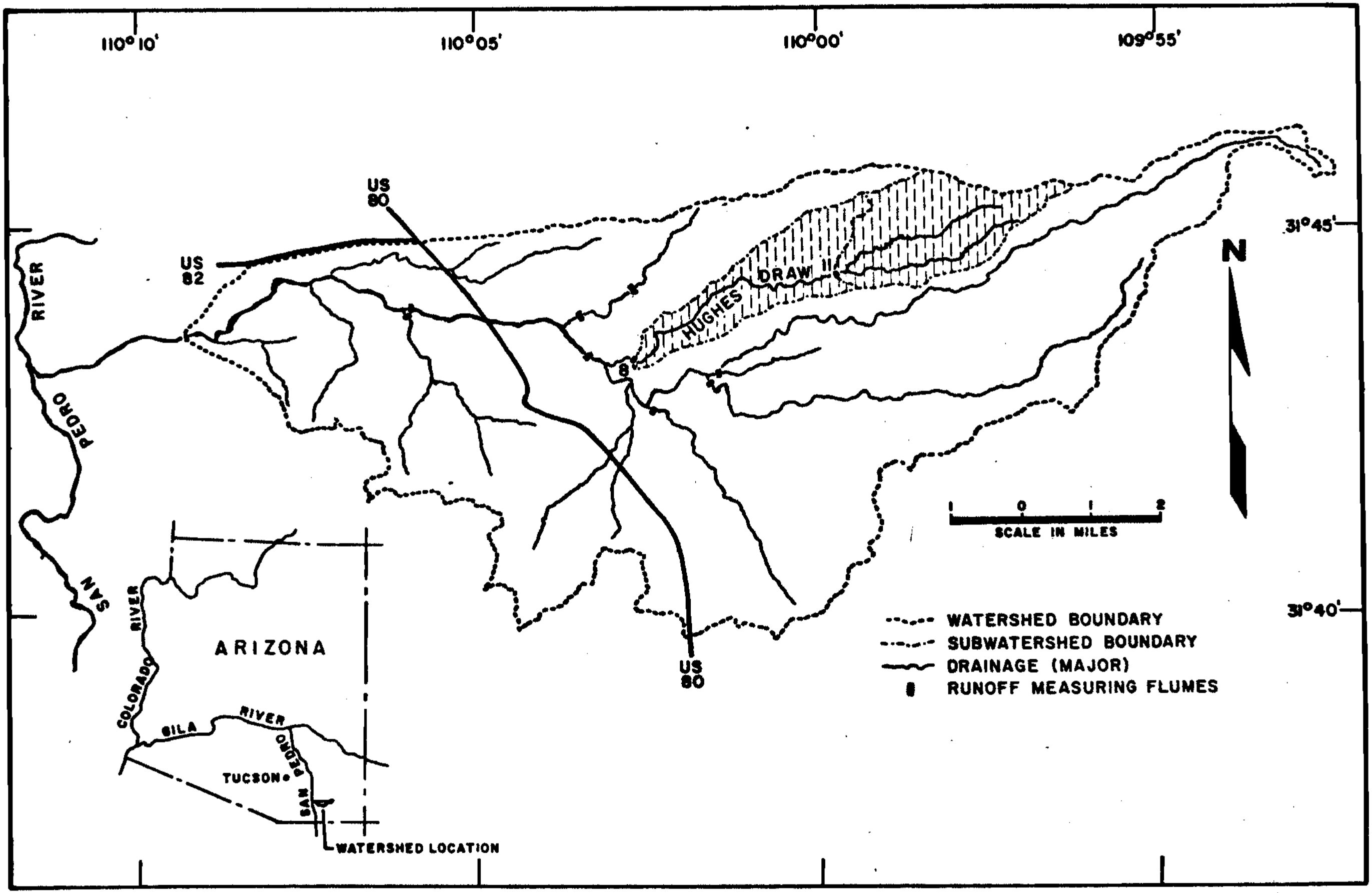


FIGURE I. WALNUT GULCH EXPERIMENTAL WATERSHED

The channel material is characterized (Wallace and Renard, 1967) as a dry alluvium which yields high infiltration rates (transmission losses) during flow events. The major static limitations to transmission losses are: (1) nature (type and configuration) and volume of alluvium beneath the channel; (2) available total porosity and specific yield of that alluvium. Antecedent moisture conditions before a flow event also limit transmission losses. Pre-existing moisture conditions may be considered virtually "static" except where two flows are separated by only a few hours. Hydraulic conductivity of the bed material and possible sealing effects of suspended sediment during flows are also important, but this paper is restricted to a description of the limiting factors imposed by the geology and geomorphology of the channel.

## DATA AND RESULTS

The subwatershed known as Hughes Draw is defined by Flume 11 at its upper end and Flume 8 at its lower end (Fig. 1). The area gaged at Flume 8 encompasses 5.98 square miles with a principal waterway about 8 miles long. Flume 11, approximately 4 miles upstream from Flume 8, gages the upper 3.18 square miles. The channel between the two flumes has only two significant tributaries and is the area on which the investigation was made.

The whole Walnut Gulch watershed is covered by a dense rain gage network of about 1 gage per 0.64 square mile. The Hughes Draw location was chosen because of the regular morphology and long narrow shape. It is therefore easy to select convective thunderstorms of limited areal extent that produce runoff at Flume 11 with little or no tributary inflow below the flume.

Transmission losses, measured as the difference in the hydrograph volumes at the two flumes, can therefore be related to the characteristics of the channel between the flumes.

Geology of the Area. Igneous, sedimentary, and some metamorphic rocks are exposed in outcrops on about one quarter of the surface area of the watershed. Their ages range from Pre-cambrian to Quaternary. Combined with rocks of the same formations which are exposed in the Dragoon Mountains to the east, they comprise the parent material of the Tertiary and Quaternary alluvial fill which covers the remainder of the watershed. Recent seismic surveys (Libby, Wallace, and Spangler, 1970) have shown this alluvium to be greater than 3000 feet deep in the northern portions of the watershed. This alluvium consists of disconnected layers and lenses of sands, gravels, and conglomerate. Some layers are unconsolidated. Others are highly cemented with $CaCO_3$ in the form of caliche. This caliche conglomerate is essentially impermeable when compared to the channel alluvium. There appear to be a number of lithologically similar, but stratigraphically separated, caliche conglomerate horizons. Some of these cemented layers form steep cliffs due to faulting, thus affecting local drainage patterns.

The geology of Subwatershed 63.008, the watershed contributing to Hughes Draw, is comprised solely of Quaternary and Tertiary alluvium with the exception of a small olivine basalt plug near the outlet. Two conglomerate series are beneath the recent alluvium of the Tombstone Pediment* in the subwatershed studied.

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* The term "pediment" as used here is obviously not compatible with the classic definition of the term, as bedrock is nowhere near the surface. "Tombstone Pediment" is the stratigraphic name given by Gilluly (1956) to the thick veneer of coarse gravels which overlie the basin alluvium.

The younger conglomerate crops out along the upper third of the 4.13-mile channel reach. Although separated from the pediment surface by only a slight disconformity, it appears considerably older than that surface. The older Tertiary conglomerate lies unconformably below the younger one and crops out along the lower third of the channel reach. Along the middle third of the reach, the upper conglomerate is interrupted by an outlier (or facies zone) of thinly-bedded red silty sandstone. Conglomerates beneath and surrounding the red sandstone persist in lenses to depths of over 1,200 feet. However, the discontinuous nature of the conglomerates makes the alluvium fairly permeable (on a long-term basis), and there are probably locations along the channel where water seeping into the channel bed can percolate to the regional water table some 425 feet below.

Geomorphology of the Channel. Geometric surface properties of the reach were examined to determine the effect of the geology on the channel regimes and on the flows. A detailed slope course survey was performed, and 46 cross-sections were measured. The slope and cross-section data revealed that the surface drainage pattern could be divided into three zones roughly conforming to the geologic outcrops mentioned above (Fig. 2). It was also apparent that while the channel slopes (a mean slope of 1.14%) showed little sensitivity to any geologic change, the variations in channel cross-sections were highly correlated to changes in regime and bedrock (caliche conglomerate) outcrop patterns.

The drainage pattern in the upper third of the reach is a slightly entrenched meander with younger conglomerate exposed on the outsides of the bends (Fig. 2). The channel follows either an old drainage channel or

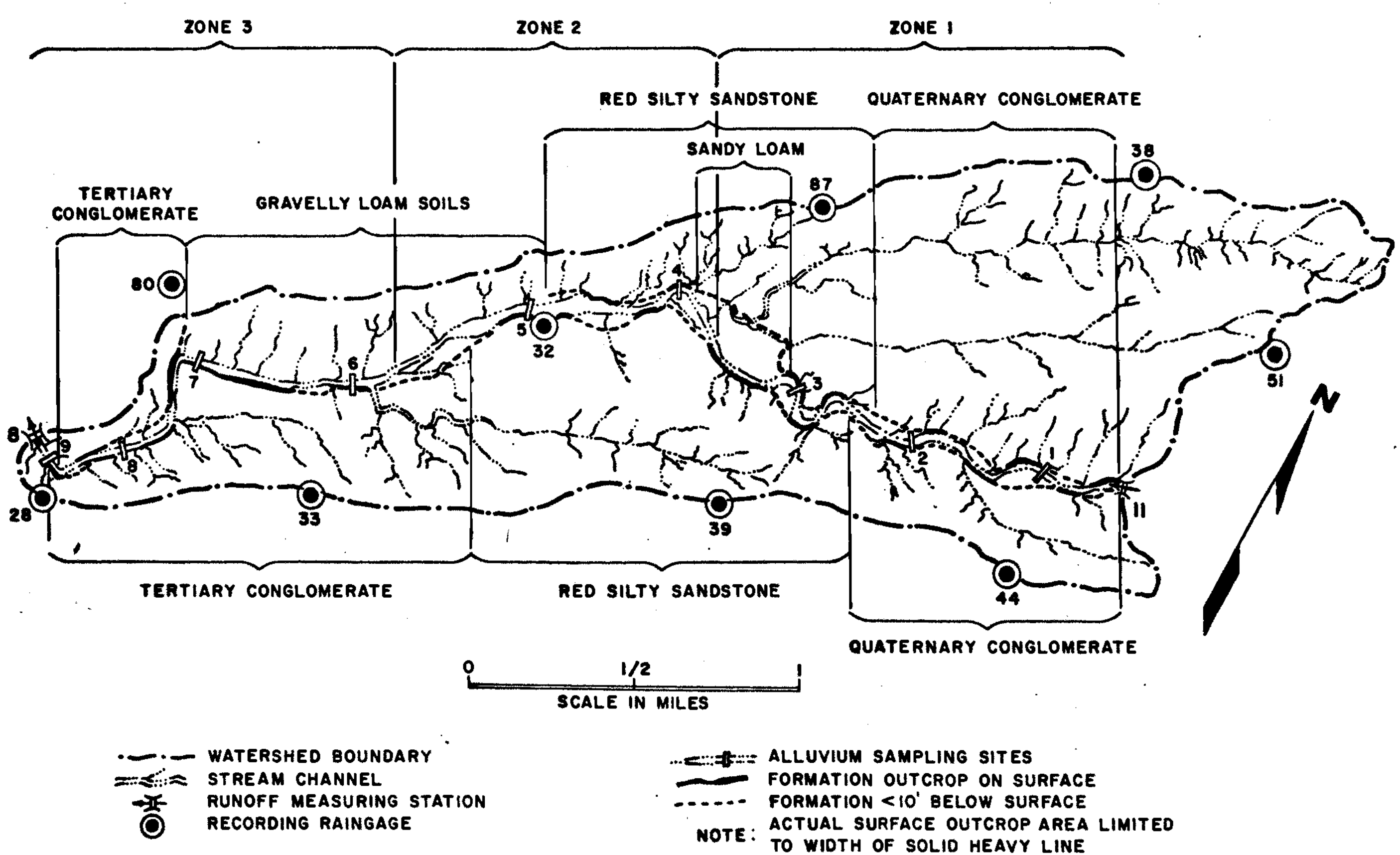


FIGURE 2. CHANNEL SURFACE GEOLOGY, SOILS, AND SAMPLING SITES.

alternates between the more easily eroded grout zones of two parallel or en echelon fault systems having trends striking N85°W and N45°E. The average channel width at low flows was about 38 feet in the upper section. Cross-sectional surveys indicate that during large runoff events, the wetted perimeter or inundated channel width increases by no more than 50%.

The middle third of channel traversing the red silty sandstone is very different from the other portions. Because of the high degree of sandstone erodibility, this reach is a series of low, multiply-braided channels and sand bars (Fig. 2). The 33-foot low-flow wetted perimeter in this section could increase to over 200 feet during the larger floods. When this occurs, the enlarged wetted perimeter produces very high transmission loss rates.

The channel in the lower one third of the reach traverses the old Tertiary conglomerate and has a narrow constricted channel usually bounded on one side by steep conglomerate walls (Fig. 2). The reaches are straight, and the direction changes are abrupt, suggesting entrenched, fault-controlled drainage. There are four major trends of channel orientation or faulting in the last channel mile above Flume 8. The average channel width in this lower section is only 36 feet, and the maximum increase in width at flood stage would not be more than 50%.

The relationship between the geology and the channel regime would therefore seem to be very important as the channel regime directly controls the width of the cross-section of the channel and this, in turn, controls the magnitude of the wetted perimeter. Therefore, channel geology and morphology are highly correlated with the potential transmission losses.

Volume, Porosity, and Specific Yield of Alluvium. A sampling program was initiated after the geometric configuration of the channel was determined to quantify the "static" limiting parameters of the bed material. Total volumes of alluvium beneath the wetted perimeter at various discharges were determined and the porosity of the samples (disturbed and re-compacted) were measured in the laboratory. When combined with a particle size analysis, specific yield was computed using the method described by Eckis (1934).

A previous investigation (Qashu and Buol, 1967) of the alluvium in an ephemeral channel downstream from the study area revealed the existence of a layer of higher clay content which could have a limiting effect on transmission losses. The results, however, were based upon samples taken from one pit. In the present study it was hypothesized that such clay lenses would have to be extensive to affect transmission losses, and therefore, a more complete and representative sample was obtained.

Nine pits were dug in the channel at roughly half-mile intervals to determine alluvium characteristics and depths (Fig. 2). These sample pits were dug to either bedrock (caliche conglomerate) or to the seven foot limit of the backhoe used. Bed material in excess of this depth was probed with steel rods until they struck an unyielding horizon. Large boulders buried in recent alluvium could be differentiated from the conglomerate by characteristic differences in their reaction to impact. The probe tips were pointed and slotted so that the material would be forced through the slot while the rod was being driven. The material last probed by the rod tip remains in the slot upon withdrawal. In all cases, caliche was the indicated bedrock material. The bedrock depth determinations were supplemented with surface observations at the 46 cross-sections to determine the configuration of the "impermeable" sublayer (Fig. 3).

Using the dimensions obtained from surface surveys, cross-sections, and depth determinations, the alluvium quantity available for infiltration for discharges less than about 50 cubic feet per second (cfs) was calculated to be approximately 106 acre-feet. For discharges between 50 and 140 cfs, the alluvium volume available for infiltration during flows would increase to about 111 acre-feet.

Above 140 cfs, overbank flow begins,and the total volume of alluvium beneath the wetted area increases rapidly. Except for the sand bars in the braided section, this volume of alluvium is probably less significant with regard to infiltration rates because duration of out-of-bank flow is short, and depths of inundation are shallow. In addition, the bank alluvium permeability is much less (Holtan, et al., 1968) than that of the channel material.

Forty-seven samples were collected from the 9 sample pits in the channel with the number per pit varying from 2 to 7. Measurements were made of each sample's moisture content, porosity, and particle-size distribution.

The porosity of individual samples varied from 30% to about 45%, with a mean of 36%. The corresponding mean specific yield, estimated from porosity versus specific yield curves (Eckis, 1934) was 28%. The median particle diameter for individual samples varied between 0.31 mm and 20 mm, with a mean value of 3.2 mm. The composition was roughly 53% gravel (greater than 2 mm), 42% sand (< 2mm and > .063 mm) and 5% silt and clay. The bank alluvium adjacent to the upper 5 sampling sites is predominantly

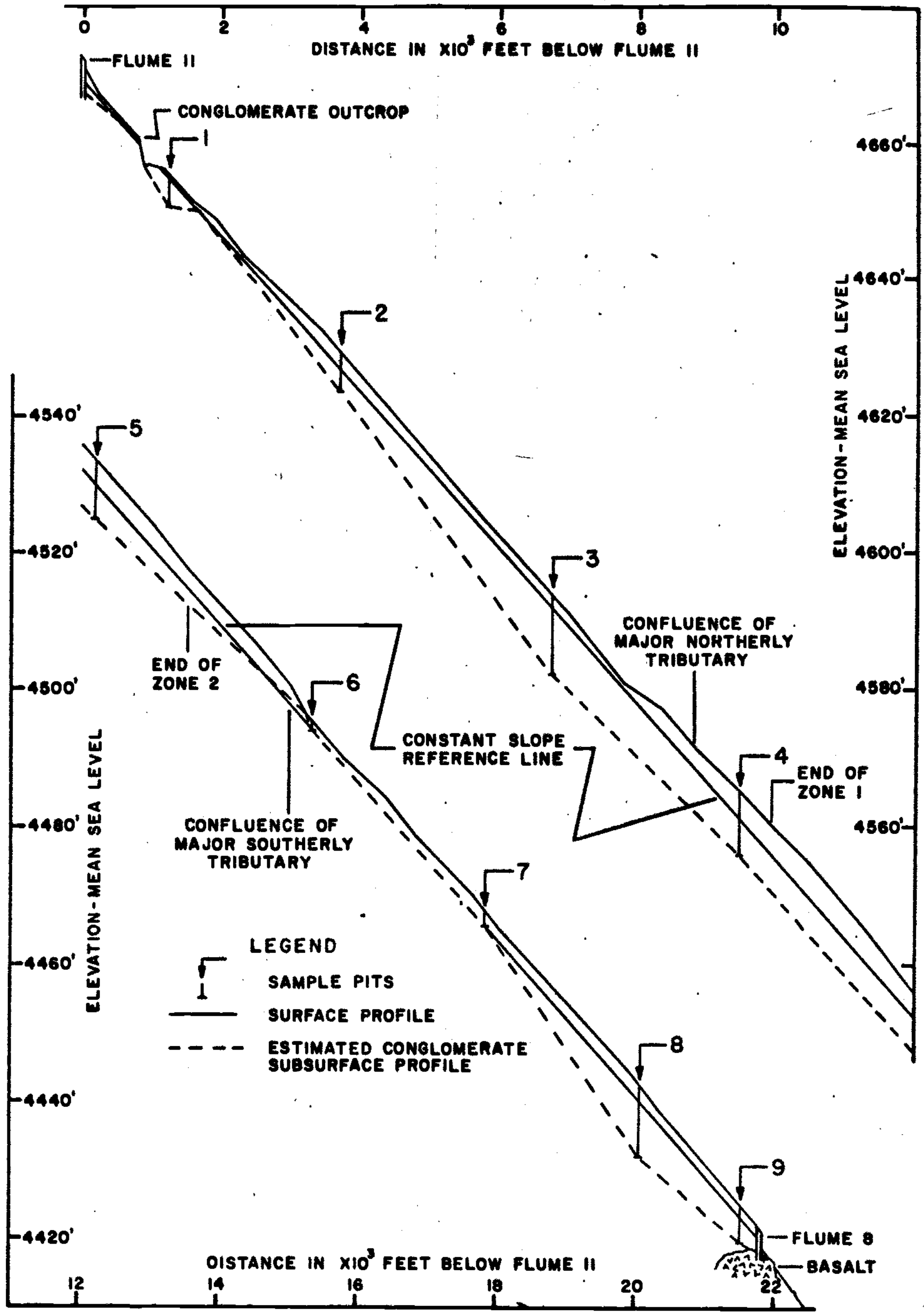


FIGURE 3. HUGHES DRAW-CHANNEL SURFACE AND SUBSURFACE SLOPE COURSE PROFILES.

Hathaway-Bernardino gravelly loams and for the lower 4 sites is exclusively Rillito-Laveen gravelly loams (Gelderman, 1970). Holtan, et al. (1968), found mean total porosities in both soils to be about 43%, but the mean specific yield values derived from their data were 17.7% in the upper stretch and 20.5% in the lower stretch. Analysis indicated that the specific yield for the channel material in the upper portion of the reach is 57% higher than that of the bank alluvium, and in the lower portion is about 31% higher than the bank alluvium. These findings, combined with the greater relative time required for moisture content changes in loams than in sands, reinforce the statement that infiltration losses to overbank-flow-inundated bank material represent a small percentage of the quantity of water lost to the channel alluvium. Tables 1 and 2 show the characteristics of the bed material samples with respect to sampling sites and depth of sampling, respectively.

Sample porosity in the channel was found to be positively correlated to the percent silt and clay of the sample and to the depth and negatively correlated to percent gravel. However, percent silt and clay was positively correlated to the depth of the sample, and percent gravel in the sample was negatively correlated to the depth. No linear relation was found between the depth at which the sample was taken and the median particle size. No clay lenses capable of limiting infiltration were found.

Analyses of variance of porosity indicated no significant variation with sampling site, but the variation with sampling depth was significant at the 5% level. This suggests the finer material from the surface layers is washed out or down (by eluviation) during the passage of small flood

TABLE 1.--Summary of results for bed material survey -- channel reach between Flumes 11 and 8, August 1969.

| Sampling site no. | Depth to conglomerate | Number of samples | Mean porosity | Mean estimated specific yield | Median particle size |
|---|---|---|---|---|---|
| | (ft) | | (%) | (%) | (mm) |
| 1 | 4.9 | 5 | 36 | 25 | 1.7 |
| 2 | 5.5 | 4 | 37 | 28 | 3.5 |
| 3 | 11.4* | 6 | 38 | 26 | 5.8 |
| 4 | 9.5* | 6 | 36 | 30 | 1.7 |
| 5 | 8.7* | 6 | 36 | 30 | 2.2 |
| 6 | 1.3 | 2 | 32 | 26 | 11.8 |
| 7 | 1.8 | 4 | 33 | 27 | 4.9 |
| 8 | 10.5* | 7 | 38 | 28 | 1.8 |
| 9 | 5.7 | 7 | 38 | 26 | 2.2 |

* Note: Samples were taken only from the top seven feet.

TABLE 2.--Characteristics of bed material in the ephemeral channel reach between Flumes 11 and 8, August 1969.

| Mean sample depth | Number of samples | Median particle size | Percent gravel | Percent sand | Percent silt and clay | Mean porosity | Mean estimated specific yield |
|---|---|---|---|---|---|---|---|
| (ft) | | (mm) | (>2 mm) | | (<.063) | (%) | (%) |
| 0.2 | 9 | 2.4 | 50.3 | 46.1 | 3.6 | 34 | 28 |
| 0.6 | 9 | 4.1 | 52.6 | 43.7 | 3.7 | 35 | 26 |
| 1.4 | 7 | 2.9 | 44.9 | 50.0 | 5.1 | 37 | 27 |
| 2.6 | 8 | 4.1 | 64.9 | 30.0 | 5.1 | 37 | 27 |
| 4.0 | 7 | 3.1 | 47.8 | 42.3 | 7.9 | 40 | 27 |
| 5.5 | 7 | 2.9 | 56.1 | 37.5 | 6.4 | 38 | 31 |
| Mean of all samples | | 3.2 | 53.1 | 41.6 | 5.3 | 37 | 28 |

waves. During larger flows the upper few inches of the bed are probably disturbed, lifted, and moved along with the flowing water. The short duration of high discharge minimizes the movement for the heavier particles, while the smaller and lighter particles are carried off.

Data from the various sampling sites were regrouped to form sampling site subsets separated by more than 0.5 mile to test the sampling interval needed. Partitioning of the data in this way indicated the results were significantly changed by the coarser sampling scheme. Such a coarser sampling scheme would not yield the same results as were obtained with the half-mile sampling interval. Unfortunately, whether or not the half-mile interval was small enough could not be determined.

Transmission Losses. With approximately 106 acre-feet of alluvial material having a mean specific yield of 28%, the maximum water-absorbing capacity of the channel reach is about 29 acre-feet or 7 acre-feet per mile. The actual transmission losses in the channel reach studied were found (Lane, Diskin, Renard, 1971) to be highly correlated to the volume of the inflow hydrograph at Flume No. 11. They found the relationship between transmission losses, L, (in acre-feet) and inflow volume, V, (in acre-feet) for storms with no rainfall between Flumes 11 and 8 was

$$L = 0.36\ V + 2.29 \qquad (1)$$

with coefficient of determination, $R^2 = 0.94$, and standard error of estimate = 1.5 acre-feet. This relationship holds true provided the inflow volume is at least 3.6 acre-feet. For smaller inflow volumes all runoff is usually lost in the 4.13-mile reach.

The maximum transmission loss recorded in the reach was 82 acre-feet for a flood wave of 128 acre-feet having a 1960 cfs peak discharge. This flow, however, covered an extensive portion of the flood plain and all of the braided zone of the channel. The second and third highest floods in the channel in 7 years of record were 44 and 40 acre-feet with corresponding peak flows of 1074 and 725 cfs. These flows remained within the stages for which available volumes of alluvium were computed. The transmission losses for the two flows were 16 and 17.5 acre-feet, respectively, or about 4 acre-feet per mile of channel. These two losses each represent some 60% of the computed capacity of 29 acre-feet. The presence of a significant quantity of pre-existing moisture in the reach material would of course reduce the transmission loss capacity of the channel by an amount proportional to the volume present.

## SUMMARY AND CONCLUSIONS

The bed material investigations were carried out in the Walnut Gulch Experimental Watershed in a portion of an ephemeral channel draining the 5.98 square miles of Hughes Draw. Channel slope appears insensitive to changes in the geologic material beneath it or to changes in the flow regime. The channel cross-section, on the other hand, appears highly sensitive to changes in both geology and flow regime. The geology and flow characteristics control the width and configuration of the channel which, in turn, governs the magnitude of the channel wetted perimeter. Both ultimately control the magnitude of potential transmission losses to the bed material in the channel.

The bed material in this channel reach had a 3.2 mm mean particle size. The composition was roughly 53% gravel (greater than 2 mm.), 42% sand (< 2 mm and > .063 mm.) and 5% silt and clay. The 5% clay fraction present was not found arranged in lenses that would limit infiltration.

The thickness of the unconsolidated alluvium in the channel varied from practically zero to more than 11 feet with a mean of 6.5 feet. A caliche conglomerate underlies the channel and persists in randomly spaced layers to great depths. The conglomerate's overall effect is that of an impermeable seal which lies at the bottom of and limits the volume of highly porous and permeable channel alluvium in the reach. Transmission losses in this alluvium are not trapped there but, due to the lateral discontinuous nature of the conglomerates, eventually may percolate to the water table 425' below. Surface extractions such as evaporation and transpiration of course take their toll of this potential recharge during the time the percolating water is still within the root zone.

The total volume of unconsolidated alluvium in the 4.13 miles of channel was estimated at 106 acre-feet. The material mean porosity was estimated at 37% with a 28% mean specific yield. The maximum water absorbing capacity is about 7 acre-feet per mile of channel. This capacity was exceeded only by the largest flow event recorded in the reach which had a considerable amount of out-of-bank flow. The second and third largest events had losses measured at 60% of this maximum capacity.

Porosity of bed material increases with depth. It is believed the sifting processes which occur during runoff events tend to sort the surface layer. The transmission loss factors (infiltration, porosity, etc.) are primarily controlled by the texture and amount of material underneath the channel. The total alluvium in the valley is important only during periods of long-duration overbank flow. An ephemeral stream such as that studied here is characterized by short-duration flows with limited overbank flow.

Actual transmission losses were found to be highly correlated to the volume of the inflow hydrograph. This should be expected because the magnitude of the inflow hydrograph directly controls infiltration by controlling the width, depth, and time of inundation of the channel reach, and because the potential loss volume exceeds the alluvium storage volume for most flows. Exceptions to this rule would occur only with flows large enough and long enough to exceed the capacity of the channel alluvium, or where antecedent moisture content is still high enough from a previous flow to limit the potential absorption capacity of the alluvium to a volume smaller than the limit imposed by the inflow hydrograph itself.

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# WATER DISPOSITION IN EPHEMERAL STREAM CHANNELS

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## INTRODUCTION

Questions have been raised concerning the contribution of flows from small watersheds to ground water recharge. Empirical methods have been developed to estimate infiltration and recharge for ephemeral stream channels but knowledge about the infiltration characteristics of small ephemeral stream channels is lacking.

Water disposition in the stream channel depends on the infiltration characteristics of the channel and the evapotranspiration characteristics of the channel vegetation. The amount of water infiltrating during a flow event is a function of the initial moisture content which is determined from the evapotranspiration rate and the time since the last flow event. Consequently, the study of transmission losses in ephemeral stream channel can be divided into two areas. The first area is the study of the infiltration parameters, and the second area is the study of the evapotranspiration parameters.

The generalized objectives of the study was to develop an infiltration equation for estimating transmission losses during a flow event in an ephemeral stream channel near Tucson. Involved in the study was the investigation of the effect of soil texture and water content on the infiltration function, and a method for determining soil moisture disposition after a flow event.

## THE STUDY AREA

The study was conducted at the Santa Rita experimental range located 30 miles south of Tucson, Arizona (Figure 1). The investigation site is located at T18S, R14E Sec. 14 SE 1/4 of the Santa Rita Quadrant in an area which is operated by the Rocky Mountain Forest and Range Experiment Station as an outdoor laboratory. Stream channels in the area form a massive dendritic pattern, but the research site consists of 1/4 mile of channel without any major tributaries. The area of the watershed contributing to the selected channel is about 5 square miles.

## VEGETATION

The riparian vegetation cover on the research area is slightly different from the surrounding vegetation because of the localized increased soil water storage in the sandy channel bed. The major species along the banks of the ephemeral channel in the study area are: Palo Verde (Cercidium microphyllum), Desert Hackberry (Celtis pallida), Cholla (Opuntia fulgida), Mormon Tea (Ephedra trifurca), Mesquite (Prosopis juliflora var. velutina).

## SOILS

The channel reach where the research was conducted consists of a sandy bed 95% sand varying in depth from 3 feet, where the channel in narrow, to about 8 feet on the wider reaches. The width varies from about 8 feet to 35 feet. Some reaches have many little islands which are flooded only during high flows. Since the stream channel under investigation is high on an alluvial terrace the sediment load of the flood waters would be expected to be low.

## INSTRUMENTATION

Installation of equipment at the Santa Rita Experiment Range was completed in July 1970. A climatic station to collect meteorologic data consisted of a

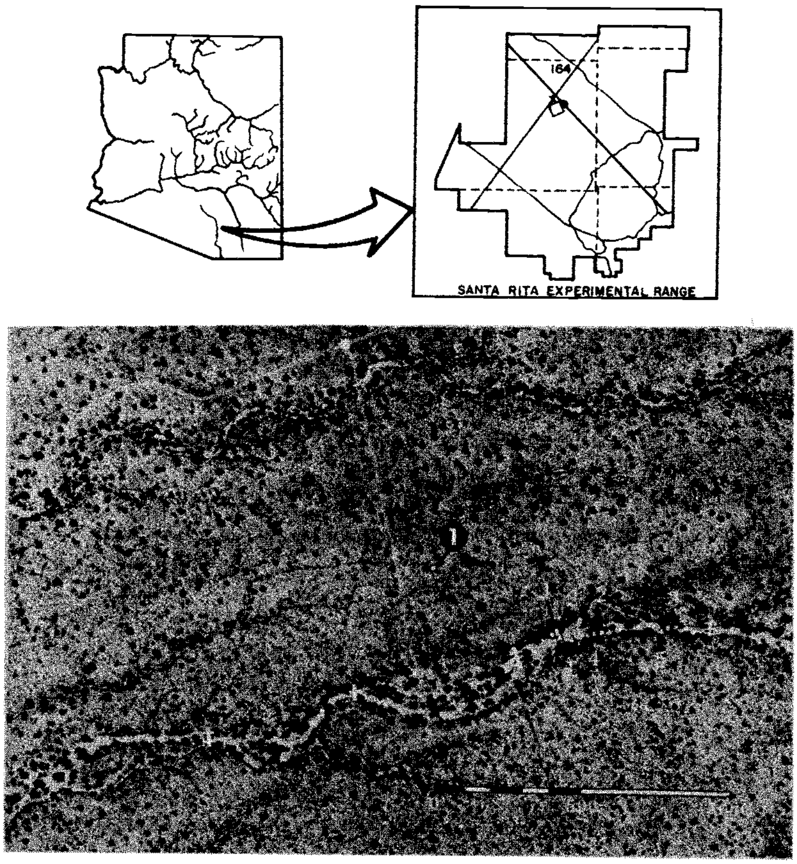


Figure 1. A Map of the Study Area

Legend

‡ Neutron Probe Access Tube Transect (each box represents one tube)
□ Infiltrometer Area
o Ring Infiltrometer (Number corresponds with infiltrometer)
Δ Locations of Soil Samples
● Climate Station
× Flume

hydrothermograph installed in a shelter, a universal recording rain gauge, and a Glass "A" U.S. Weather Bureau evaporation pan.

Evapotranspiration was determined in the ephemeral channel by a water balance method using soil water content measurements determined by the neutron scattering technique described by Stone, Kirkham and Road (1955), Van Bavel (1958), Gardner and Kirkham (1952).

Infiltration measurements were made at the site using an infiltrometer (Figure 2) designed by the author to simulate a flow event in the channel. The infiltrometer consists of water tanks and flow system to supply water at a rate greater than the infiltration rate. Excess water is collected in two tanks on the outlet of the infiltrometer area. Water stage recorders on the inlet and outlet tanks have ten-turn precision resisters connected to a float which change resistance value as the depth in the tank changes. The resistance value is recorded using a Hewlett-Packard Data Acquisition System.

The infiltration simulator was calibrated for accurate measurement of inflow and outflow of water over the meter area and a computer program was written to derive the infiltration function.

A buffer soil zone, around the infiltrometer, was supplied with water from another supply tank. The size of the area of infiltration in the simulator is adjustable and can be changed, but because of a limited water supply, a one-meter area was selected. Inifltration measurements were conducted at three areas in the channel (Figure 1). Repeated infiltration measurements were made at different locationsin the channel. Neutron Access tubes were installed at each location where the simulator was operated and water content of the soil was measured before and after each run.

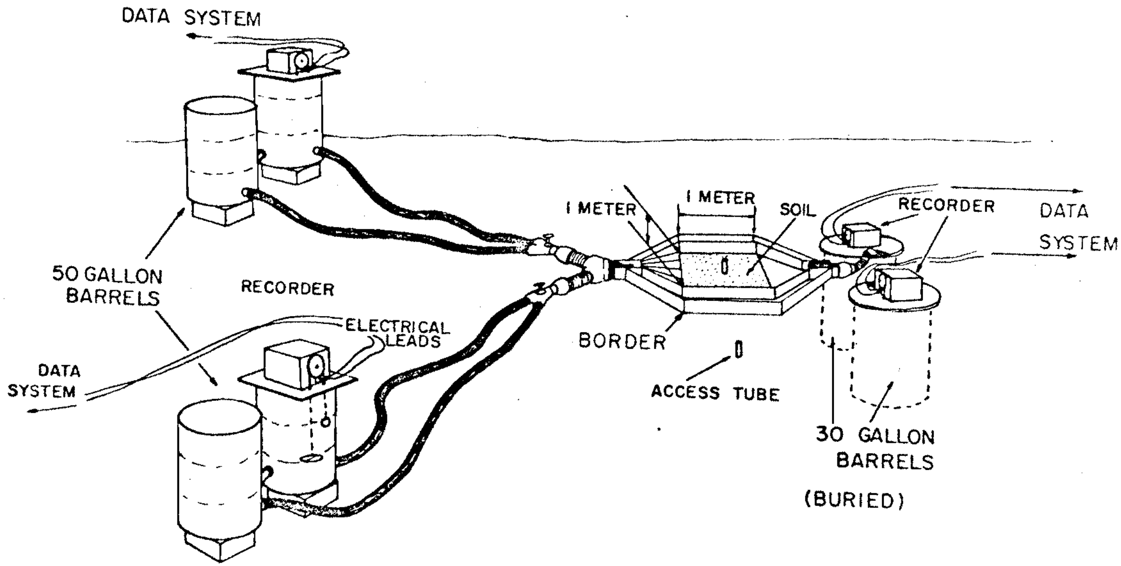


Figure 2. Infiltration Simulator

INFILTRATION

The infiltration function has been described mathematically by both physically based models and empirical equations. Philip (1957a-f) derived an equation based on the assumption that infiltration is a process of diffusion. The following assumptions apply for the derivation of that equation:

1. The soil is homogeneous.
2. Darcy's law applies for unsaturated flow (Pressure head of soil water less than zero).
3. The system is isothermal.
4. Biological, chemical and hysteresis forces are negligible.
5. The only force retarding flow is viscosity.

The equation used in this study which was developed by Philip (1957a-b) is

$$Q = At^{1/2} + Bt$$

where Q is the accumulative infiltration, T is Time, and A and B are constants which are normally determined empirically rather than by numerical integration and, consequently, the equation is considered an empirical equation.

A computer program was written to calculate accumulative infiltration for individual infiltration areas. The coefficients A and B in equation (1) were estimated using a least square fitting technique (Table 1). Recorded infiltration data did not always fit a smooth curve. The variability was attributed to experimental errors evolving from changes in head and thus storage of water over the square-meter area. However, Philip's equation still represented the infiltration curves for the different runs (correlation coefficients), comparing field data with computed infiltration using Philip's equation, range from 0.87 to 1.00 with most of the values around 0.98 (See Table 1).

TABLE I

INFILTRATION ANALYSIS

| Date, Location[1] of Sample | | Average Soil Moisture % in Top 2.5 ft. before | after | Philip's Equation Constants A cm/min | B cm/min | Correlation Coefficient | Duration of Measurements Minutes |
|---|---|---|---|---|---|---|---|
| 4/25 | 01 | 6.7 | -- | .29 | 1.73 | .99 | 35 |
| 6/25 | 11 | 7.6 | 27.8 | 1.62 | 1.47 | .96 | 15 |
| 6/29 | 11 | 8.4 | 23.6 | 1.31 | 2.04 | 1.00 | 40 |
| 7/1 | 21 | 4.2 | 24.0 | 4.16 | 2.71 | .99 | 35 |
| 7/9 | 21 | 6.3 | 24.1 | .79 | 2.32 | 1.00 | 50 |
| 5/2 | 02 | 3.1 | 29.5 | .012 | .20 | 1.00 | 50 |
| 5/9 | 02 | 7.0 | 28.5 | .31 | .05 | .97 | 15 |
| 5/26 | 02 | 4.0 | 19.7 | 2.25 | .03 | .87 | 52 |
| 7/10 | 03 | 4.6 | 21.5 | 1.98 | 2.35 | .98 | 38 |
| 7/16 | 03 | 5.4 | 19.9 | 2.80 | 1.06 | .99 | 40 |
| 7/23 | 13 | 10.0 | -- | .82 | 1.13 | .99 | 45 |
| 7/28 | 13 | 5.6 | 24.7 | .85 | .35 | .98 | 30 |
| | | | | Double Ring Infiltrometer | | | |
| 8/26 | 3.1 | 8.0 | -- | 2.42 | 1.44 | .99 | 30 |
| 8/27 | 12 | 9.0 | 24.1 | 2.61 | 1.70 | .99 | 20 |
| 9/14 | 22 | 7.0 | 23.8 | 4.65 | 1.63 | 1.00 | 25 |
| 9/26 | 23 | 4.3 | 24.6 | 1.08 | 1.61 | 1.00 | 25 |
| 9/26 | 33 | 4.3 | 24.3 | 3.57 | 1.53 | 1.00 | 27 |
| 9/26 | 43 | 4.3 | -- | .82 | 1.09 | 1.00 | 13 |

1. The first number is the Location Number. The second number is the Area Number given on Figure 1.

The constants A and B in equation (1), the correlation coefficient, duration of run and the average soil moisture in the top 2.5 feet of the channel are tabulated in Table 1 for each infiltration run made.

The constant B represents the saturated hydraulic conductivity of a homogeneous soil. Depending upon the initial moisture content, constant (B) appeared to be reached in 15 minutes after the start of an infiltration run. The runs were of durations from 20 to 55 minutes. The mean value of B for each area in the channel was determined and then assuming a normal population distribution, a statistical test was made to determine if the values represented the same or different populations. Location 2 was from a different population.

The coefficient A in equation (1) reflects the absorptive characteristics of soil and is a function of the initial moisture content. The coefficient A for the different runs using the infiltration simulator were plotted against the average soil moisture content in the top 2.5 feet of the channel (Figures 3).

Infiltration measurements were made with a double ring infiltrometer (Table 1) at the three locations. The average B value for the infiltration simulator (location 1 and 3) and the double ring infiltrometer is 1.59 cm/min. This is the value that should be used as representative of the channel soil saturated hydraulic conductivity.

## EVAPOTRANSPIRATION

Water losses from emphemeral channels in the desert are due to two major processes: evaporation from the soil surface, and transpiration by riparian vegetation. The sum of the two losses is referred to as evapotranspiration. There are many methods for estimating potential evapotranspiration. The determining factors for choosing a method are: (1) the available data, (2) the degree

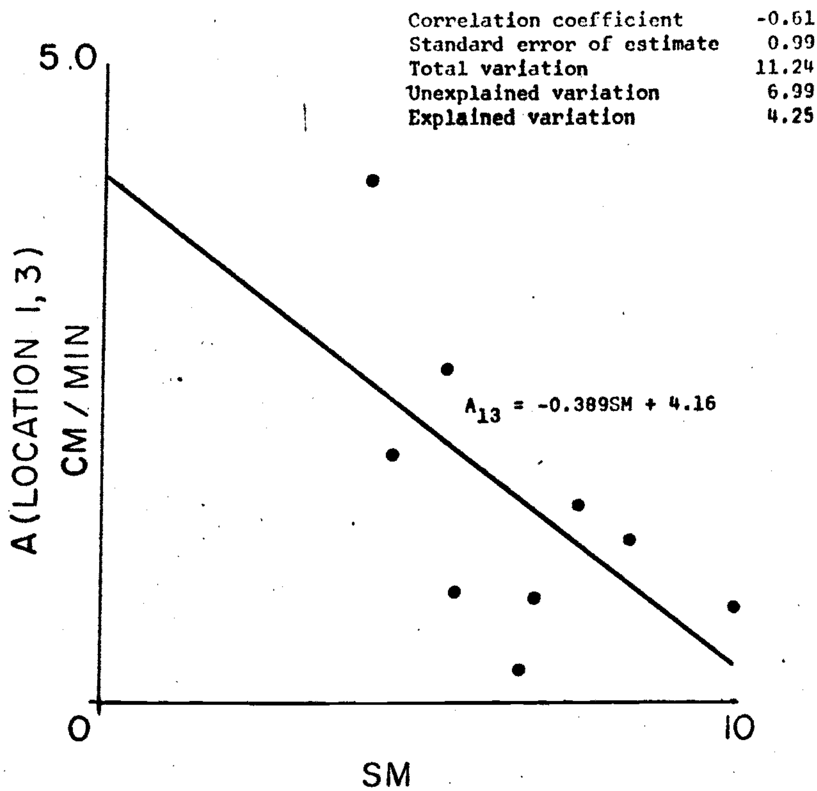


Figure 3. Linear Correlation to Soil Moisture of Coefficient "A" in Philip's Equation for Sample Sites 1 and 2

Subscripts refer to sampling location.
SM denotes soil moisture content in percent.

of accuracy required, and (3) the length of period for which PET is required.

Thornthwaite's (1948) method for estimating ET is probably the most widely used empirical method. The only data required to apply Thornthwaite's formula is mean daily temperature data. The equation can be solved using a nomogram and tables supplied by Thornthwaite. This method has a temperature limitation. It can only be applied for mean daily temperatures equal to or less than 38.0 degree Centrigrade, a mean temperature that can be exceeded in desert areas.

Thorthwaite and Mather (1955) stated that in arid areas where some energy is used in heating the air, use of Thornwaite's method will result in some error.

U.S. Weather Bureau Glass A evaporation pans have been used also as an index of evapotranspiration. Pruitt (1964) using weighing lysimeter data showed that evapotranspiration from grass areas is proportional to pan evaporation. However, Pruitt (1960) determined that the pan exposure made a big difference when comparing pan evaporation to computed evaporation. A pan located in a dry land environment showed 25-35 percent higher evaporation rates than a pan located in an irrigated field.

When soil moisture becomes a limiting factor evapotranspiration rate decreases. There seem to be two main views on how soil moisture status affects evapotranspiration. Veihmeyer and Hendrickson (1950) contend that for both annuals, perennials and trees, the rate of moisture extraction is not influenced by the amount of water present in the soil when the soil moisture is above the permanent wilting percentage. Thornthwaite and Mather (1955) stated that as the soil dries the transpiration rate diminishes over all the range at a rate proportional to the ratio of available water to that present at field capacity. A complete discussion and comments for both points of view by noted experts is presented in a paper by Veihmeyer and Hendrickson (1955). As moisture decreases below field capacity the physiology of the plant, the soil texture and depth, and the root system of the plant seem to play the determining roles in controlling evapotranspiration rate.

Evapotranspiration for a given duration was calculated as the loss in total soil moisture storage for the period between measurements plus any rainfall. Monthly calculations of ET were determined by the extrapolating ET values when no end of month readings were taken. (Table 2)

For each measured time period, a ratio of evapotranspiration to pan evaporation and the ratio of total soil moisture at the end of the time interval to the maximum total soil moisture that the channel could hold (SMFC) were determined (Table 2). These variables were plotted with the ratio of evapotranspiration to pan evaporation being the dependent variable (See Figure 4).

Thornthwaite's method was used to determine potential evapotranspiration (Table 3) and a linear regression coefficient was determined using the calculated ET/PET (Figure 5).

These results support Thornthwaite's and Mather's contention that transpiration rates diminish linearly as soil moisture diminishes.

The relationship of evapotranspiration to soil moisture applies only during the active growing season and thus the linear regression coefficients determined for the different methods of calculating the ET/PET and SM ratios apply only to the period April through October. During the dormant period ET/PET ratios are very low and do not correlate with soil moisture availability. Therefore, ET/PET coefficients should be used during this period independent of soil moisture content.

## TABLE 2

### EVAPOTRANSPIRATION PARAMETERS

| Period Ending 1970 | No of Days | Total[1] Soil Moisture cm | Change in Soil Moisture cm | Precipi- tation cm | Evaporation from soil surface ET. Measured cm | Estimated cm | Accum ET cm | Pan Evap cm | Accum Pan Evap cm | ET/EP | SM/SMFC |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 7-27 | 3 | 42.8 | | | | | 0.0 | 0.0 | 0.0 | | |
| 7-31 | 4 | 38.4 | 4.4 | 0.0 | 4.4 | | 4.4 | 3.6 | 3.6 | 1.22 | 0.90 |
| 8-05 | 5 | 36.6 | 1.8 | 2.0 | 3.8 | | 8.2 | 5.5 | 9.1 | 0.69 | 0.85 |
| 8-10 | 5 | 33.6 | 3.0 | 0.2 | 3.2 | | 11.4 | 4.2 | 13.3 | 0.76 | 0.78 |
| 8-17 | 7 | 32.0 | 1.6 | 1.1 | 2.7 | | 14.1 | 5.2 | 18.5 | 0.52 | 0.75 |
| 8-21 | 4 | 29.8 | 2.2 | 0.1 | 2.3 | | 16.4 | 3.1 | 21.6 | 0.73 | 0.70 |
| 8-26 | 5 | 27.3 | 2.5 | 0.0 | 2.5 | | 18.9 | 6.2 | 27.8 | 0.40 | 0.64 |
| 9-02 | 7 | 25.5 | 1.8 | 0.0 | 1.8 | | 20.7 | 7.2 | 35.0 | 0.25 | 0.60 |
| 9-07 | 5 | 40.0 | +14.5 | 6.0 | | 1.3 | 22.0 | | | | |
| 9-16 | 12 | 32.4 | 7.6 | 0.0 | 7.6 | | 29.6 | 8.8 | 43.8 | 0.86 | 0.76 |
| 10-3 | 14 | 27.0 | 5.3 | 0.4 | 5.7 | | 35.3 | 10.8 | 54.6 | 0.53 | 0.63 |
| 11-28 | 56 | 18.9 | 8.1 | 0.4 | 8.5 | | 43.8 | 33.5 | 88.1 | 0.25 | 0.44 |
| 1971 | | | | | | | | | | | |
| 1-15 | 48 | 18.3 | 0.6 | 1.5 | 2.1 | | 45.9 | 16.3 | 104.4 | 0.13 | 0.43 |
| 3-21 | 95 | 17.9 | 0.4 | 2.4 | 2.8 | | 48.7 | 44.5 | 148.9 | 0.06 | 0.42 |
| 4-17 | 27 | 18.4 | +0.5 | 0.8 | 0.3 | | 49.0 | 28.1 | 177.0 | 0.009 | 0.43 |
| 6-03 | 47 | 12.7 | 5.7 | 0.0 | 5.7 | | 54.7 | 74.4 | 251.4 | 0.08 | 0.30 |
| 6-24 | 21 | 12.3 | 0.4 | 0.0 | 0.4 | | 55.1 | 54.8 | 306.2 | 0.006 | 0.29 |
| 7-08 | 14 | 14.0 | +1.7 | 1.0 | | 0.2 | 55.3 | 36.4 | 342.6 | 0.006 | 0.33 |
| 7-21 | 13 | 13.4 | 0.6 | 0.8 | 1.4 | | 56.7 | 24.8 | 367.4 | 0.06 | 0.31 |
| 7-24 | 3 | 16.3 | +2.9 | 1.8 | | 0.3 | 57.0 | 3.8 | 371.2 | 0.08 | 0.33 |
| 7-29 | 5 | 33.0 | +16.7 | 1.7 | | | | 6.7 | 377.9 | | |

1. Depth of soil profile 8.5 ft.

TABLE 3

POTENTIAL EVAPORATION PARAMETERS

| Month 1971 | Measured ET cm | Thornthwaite Potential ET cm | ET/PET | SM/SMFC |
|---|---|---|---|---|
| Jan | 1.14 | 1.98 | 0.57 | 0.43 |
| Feb | 0.82 | 1.57 | 0.51 | 0.42 |
| Mar | 0.70 | 4.39 | 0.15 | 0.42 |
| Apr | 1.73 | 5.28 | 0.33 | 0.39 |
| May | 3.60 | 8.79 | 0.42 | 0.30 |
| Jun | 0.80 | 15.49 | 0.05 | 0.31 |
| Jul | 3.29* | 17.88 | 0.18 | 0.31 |
| 1970 | | | | |
| Aug | 15.82 | 17.88 | 0.88 | 0.61 |
| Sep | 13.71 | 11.51 | 1.19 | 0.66 |
| Oct | 5.40 | 5.48 | 0.99 | 0.54 |
| Nov | 4.32 | 3.38 | 1.28 | 0.44 |
| Dec | 1.39 | 1.45 | 0.96 | 0.43 |

*Date ended 7/21

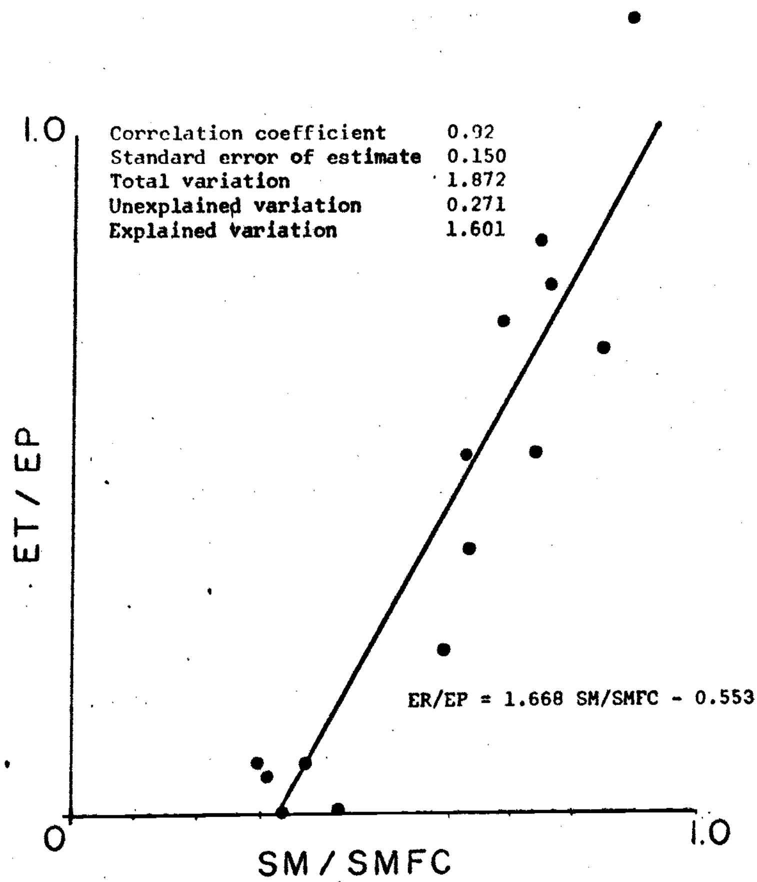


Figure 4. Linear Correlation of ET/EP and SM/SMFC

| | |
|---|---|
| Correlation coefficient | 0.94 |
| Standard error of estimate | 0.162 |
| Total variation | 1.159 |
| Unexplained variation | 0.131 |
| Explained variation | 1.027 |

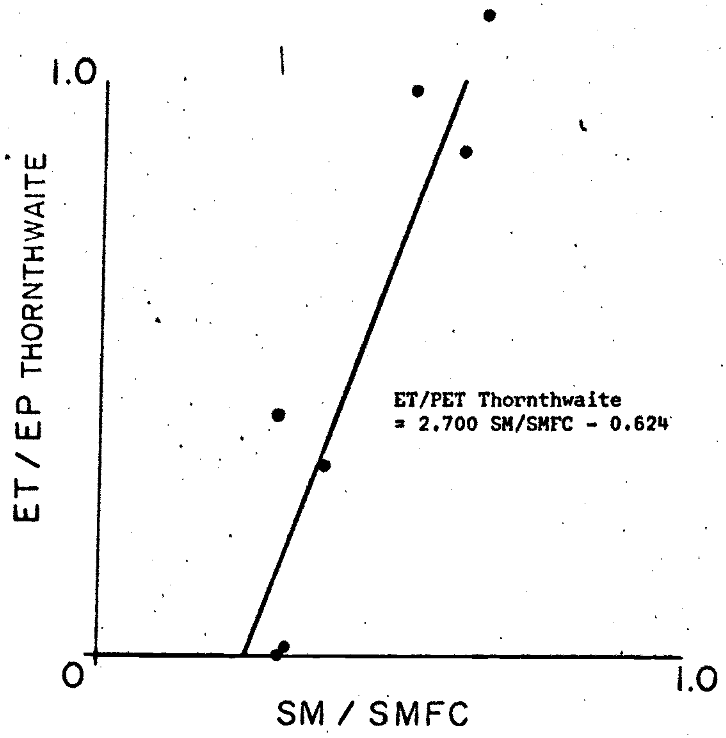


Figure 5. Linear Correlation of ET/PET Thornthwaite and SM/SMFC

For the time period August 1970 to July 1971, under the existing moisture conditions (Table 2) the consumptive use of riparian vegetation was 57.0 cm. Because this period represented a dry year the consumptive use in a normal year of rainfall may be larger.

Measured ET for each month of the year was less than the measured and computed pan evaporation and was less than the Potential Evaporation calculated by Thornthwaite's method except during the months of September and November.

The measured Evapotranspiration of riparian vegetation was low from January to March. In April Et rate increased as the plants commenced growth and continued to increase until water became limiting and the channel moisture storage was depleted usually by the end of May or June. When the summer rains commenced in July and August, ET rates increased and approached the Potential rates. ET rate then decreased in September through December as the plants became less active due to low temperatures and most of the mesquite lost their foliage.

RESULTS

In field studies more data and data covering repeated measurements in time would give greater confidence to conclusions reached. In this study an attempt was made to reduce the time by simulating flow events and develop some prediction models. The following conclusions were reached:

1. Philip's equation

$$Q = At^{1/2} + Bt$$

when applied under the assumptions in its derivations is an excellent model to mathematically represent infiltration in a homogeneous sandy ephemeral channel. The correlation coefficient comparing the variables, accumulated infiltration measured in the field with an infiltrometer range from 0.87 to 1.00.

2. Initial moisture content seems to affect the initial infiltration rate. In Philip's equation this rate is determined mainly by the absorptivity coefficient "A" and this coefficient appears to be linearly related (correlation coefficient -0.61) to the soil moisture in the top 75 cm of the stream channel bed.

3. The infiltration simulator and the ring infiltrometer both gave measurements of the B coefficient in Philip's equation close to the average value of 1.59 cm/min. This coefficient approaches the saturated hydraulic conductivity for infiltration measurements of long duration. Some of the scatter for measurement of the B coefficient using the infiltration simulator can be explained by the fact that some of the runs were of short duration and the coefficient B is then not the saturated hydraulic conductivity but depends on the initial moisture content. The higher the initial moisture content relative to the saturated moisture content, the closer the coefficient B will be to the saturated hydraulic conductivity for infiltration runs of short duration. Using this average B coefficient in Philip's equation to compute accumulative infiltration and comparing the answer to the measured infiltration for short flow events, the computed value falls with $\pm$ 30%. Although the B coefficient is high compared to values obtained by other investigators who measured saturated hydraulic conductivity (average value of 0.25 cm/min) the difference may be attributed to the fact that the channel under investigation was 95 to 98 percent sand and no valve effect occurred from sediment load in the channel water as other

investigators measuring hydraulic conductivity have observed in large ephemeral channels.

4. Infiltration rates are affected by the soil structure of the stream channel bed. All locations in the channel had similar textural analysis of the soil profile but the average final infiltration rate measured at location 2 (0.09 cm/min) was 18 times lower than the measured value at the other locations. This difference can be attributed to the difference in the structure of the soil profile at location 2 as compared to the other locations. The soil material at location 2 was compacted and held together to such a degree that taking soil samples was extremely difficult.

5. Consumptive use measurements of the riparian vegetation and measured climate data support Thornwaite's and Mather's (1945) contention that transpiration rates diminish linearly proportional to the ratio of available water to that present at field capacity. This concept applies only to active growing plants. Assuming an active growing season from April to October the linear correlation coefficient comparing the variables measured ET/Ep and SM/SMFC, was 0.92. Substituting Thornthwaite's computed potential ET for Ep resulted in a 0.94 linear correlation coefficient. Given climate data and an initial moisture content of a ephemeral channel a depletion water budget of the channel can be determined using one of the linear regression equations comparing transpiration rates to soil moisture. Knowing the initial soil moisture and the duration of flow, Philip's equation can be used to determine the channel recharge or transmission loss.

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