Longenbaugh, R
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IRRIGATION WATER SUPPLY MANAGEMENT

Larry F. Land

Civil Engineering Department
Engineering Research Center
Colorado State University
Fort Collins, Colorado

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ABSTRACT

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The major objective of this study was to illustrate a method for comparing allocation patterns on the basis of water-use efficiency and economic soundness. Two supporting objectives include presenting three methods for calculating return flow and a technique for evaluating an allocation pattern.

To make the study on irrigation water supply management, a hypothetical basin was used. The basin consisted of several unit-areas which were arranged in the shape of the basin. For a reference level of water application, two irrigation patterns, one using only ground water and the other using only surface water, were assigned to a unit-area. Various levels of water application were obtained by adjusting the reference level with coefficients. A combination of ground water and surface water sources was obtained by combining ground water with surface water application. From these various levels of water application and the two methods of distributing the water, several allocation patterns were devised for the limited amount of irrigation water.

Water-use efficiency and economic returns were used in comparing the allocation patterns. The efficiency was determined from the volume of water consumed by crops and the volume delivered for irrigation. For economic comparison,

it was assumed that the water was applied at a depth of three feet for each year and yielded a constant return for each unit of water. The water cost was also assumed to be a constant cost. The other farm costs were assumed to be the same for each irrigated acre. With these values the net return and the benefit-cost ratio were calculated for each allocation pattern.

The results indicated that considerable return flow was lost when only surface water was used in the lowest part of the reach. However, if only ground water was used, the cost in obtaining the water was higher. The most reasonable allocation pattern seemed to be one that balanced, to some degree, ground water and surface water applications.

Larry F. Land
Agricultural Engineering Department
Colorado State University
Fort Collins, Colorado
March, 1967

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LIST OF SYMBOLS

Symbol	Dimension	Definition
A, B, C, E	-	Coefficients used in Mathematical Model
d'	L	Thickness between stream and bedrock
D	L	Saturated thickness
dx, dy	L	Differentials and dimensions of finite grid blocks
F	L^2T^{-1}	Rate of flow per unit width at distance x
f_i	$	Marginal value products obtained from the i^{th} diversion
H	L	Elevation of water table above a datum
h'	L	Drainable depth
H'	L	Initial drainable depth
i, j	-	Identification of grid blocks, unit-areas, and diversions
I	LT^{-1}	Infiltration rate
K	LT^{-1}	Hydraulic conductivity
L	L	Twice the distance between the stream and the vertical impermeable boundary
MVP	$	Marginal value products
n	-	Consecutive odd integers used in specifying terms in a series
PR	-	Fraction of the applied water remaining between the stream and the vertical impermeable boundary
q	L^3T^{-1}	Net volumetric rate of extraction or application

LIST OF SYMBOLS (continued)

Symbol	Dimension	Definition
q_1	L^2T^1	Flow per unit length of line source or irrigated strip
R_i	-	Fraction of diverted water from the i^{th} diversion that returns to the stream
s	L	Drawdown from initial water table
S_i	L^3	Amount of stream water diverted in the i^{th} diversion
t	T	Time
TMVP	$	Total marginal value products
V	L^2	Total saturated volume of aquifer per unit width caused by the applied water
x	L	Horizontal distance perpendicular to the line source
x_1	L	Distance from stream to the line source
x_2, x_3	L	Distance from stream to edges of irrigated strip
x, y	L	Length and width dimension in mathematical model block
α	L^2T^{-1}	$\frac{KD}{\phi}$
ϕ	-	Storage coefficient
π	- L^3	3.1416 and the total feasible productive use

Chapter I

INTRODUCTION

The historical development of water resources for irrigation in the western United States has been irregular and without a definite plan. As a result inefficiencies have developed under the doctrine of prior appropriation. Any deviation from this doctrine to increase the water use efficiencies will cause some social and legal problems. However, this law is possibly being violated at the present time by the use of some irrigation wells. Any changes in the present arrangement must result in a plan that would cause as little controversy as possible and still improve the present operation.

With this type of problem in mind, a plan to optimize the allocation of water application pattern that uses the most efficient distribution method or methods is essential. In this thesis the term <u>application pattern</u> will refer to the location of the irrigated area and the associated depths of water applied. <u>Distribution methods</u> will refer to the methods that farmers use in obtaining irrigation water (i.e., pumping or canal systems). An <u>allocation pattern</u> is used to denote a particular basin-wide application pattern that uses a particular distribution method or combination of methods.

Objectives of the Study

The study in this thesis involves manipulating the patterns of application and the distribution method in order to evaluate various allocation patterns within a hypothetical basin. The primary objective is to illustrate a method of comparing allocation patterns on the basis of water-use efficiency and economic soundness. Two supporting objectives include the presentation of three methods for calculating return flow and a technique for evaluating an allocation pattern. Finally, the technique of comparing allocation patterns is evaluated for adequacy under field conditions.

The Hypothetical Basin

In this study a hypothetical basin composed of unit-areas is utilized. The unit-area is a small rectangular area that provides a detailed description of a portion of the basin. The hypothetical basin model consists of several unit-areas arranged in the shape of the basin. The characteristics of the unit-areas models can be varied with coefficients so that many basin-wide patterns of application and methods of distributing the irrigation water can be studied.

The Economic Limitations

For the purpose and scope of this paper, the economic evaluation is subject to some limiting assumptions. These assumptions include a constant operating and fixed cost on the farms. This includes a constant cost in obtaining the irrigation water. The returns are also considered to be constant for each acre-foot of applied irrigation water. These simplifications can be made only for a hypothetical case or for illustration purposes. For the economic analysis to have any value in a field case, the variables that affect the costs and benefits must be taken into account.

Economics of Irrigation Water Supply Management

According to Mitchell (33), management of irrigation water supplies is a program to deliberately pump and recharge a ground water basin and to divert surface water from some stream or other surface supply. These practices are performed in such a fashion that undesirable effects of overdraft and water logging are either prevented or corrected. Hartman (28) extends the concept to include a plan that maximizes the net benefits of a basin.

Hufschmidt and Maass (29) state that the optimum program is reached when any change of a variable causes the net benefits to decline. In the usual case, the optimum condition is a point where the marginal cost equals the marginal benefits.

Ciriacy-Wantrup (8,9) writes that the utilization of natural resources is intended to produce useful goods and services. An increase of real income over time is fundamental to the theory of welfare economics and is the goal to which a private enterprise applies its allocation of natural resources. The competitive market economy, which governs private enterprise, could be expected to produce an efficient allocation of natural resources if the basic assumptions are met. These assumptions are (1) that producers rationally pursue profit maximization under conditions of decreasing returns, (2) that consumers have rational preferences under conditions of decreasing utility, (3) that the actions of both producers and consumers are independent, and (4) that the market operates under perfect competition.

In reality, perhaps none of the assumptions for the competitive model are met perfectly in the case of water resources. The interdependence of water occurrence and its use particularly violates the assumptions of physical independence. In explanation, the same water may be used more than once and its quality, quantity, timing and location of flow alter successive uses. Thus, water users may be subjected to economic effects external to their individual planning and works. Such imperfections of the market system are the basis for the development of public regulation to deal with the market imperfections in an organized manner.

Hufschmidt and Maass (29) use a production function to relate inputs to outputs while seeking an optimal design. Specifically, a production function means the maximum quantity of an output or combination of outputs obtainable with given levels of inputs. To determine the production function of a river is an almost impossible task. However, there are three basic approaches to such a study. These approaches are (1) "working around the edges," a method which selects, by engineering judgment and experience, a limited number of combinations for detailed investigation; (2) simple mathematical models, which skeletonize the problem so that much of the analysis is handled by standard mathematical and statistical methods; and (3) simulation of the system on high speed digital computers.

Chun, Mitchell, and Mido (7) describe an investigation procedure to obtain the most desirable plan for utilizing the available water. This procedure is represented by Figure 1, a flow diagram showing the individual steps and their order of occurrence in the investigation. In this coordinated study, the number of the future facilities and their operational costs are obtained for several alternative plans. For the entire study various combinations of primary canals, surface storage facilities, and ground water facilities are examined to determine the combination most economically meeting the water demand. Finally, the costs of all alternatives are compared for selection of an optimum plan of operation. They assume that all the different plans satisfy the objectives to the same degree; therefore, the benefits are the same for all plans.

Integration of Surface Water and Ground Water

According to Hartman (28) the variables that must be controlled for a successful integration include pumping, direct river diversions for irrigation, diversions for recharge, diversions to and from surface storage and the number of acres. These variables are to be controlled in such a

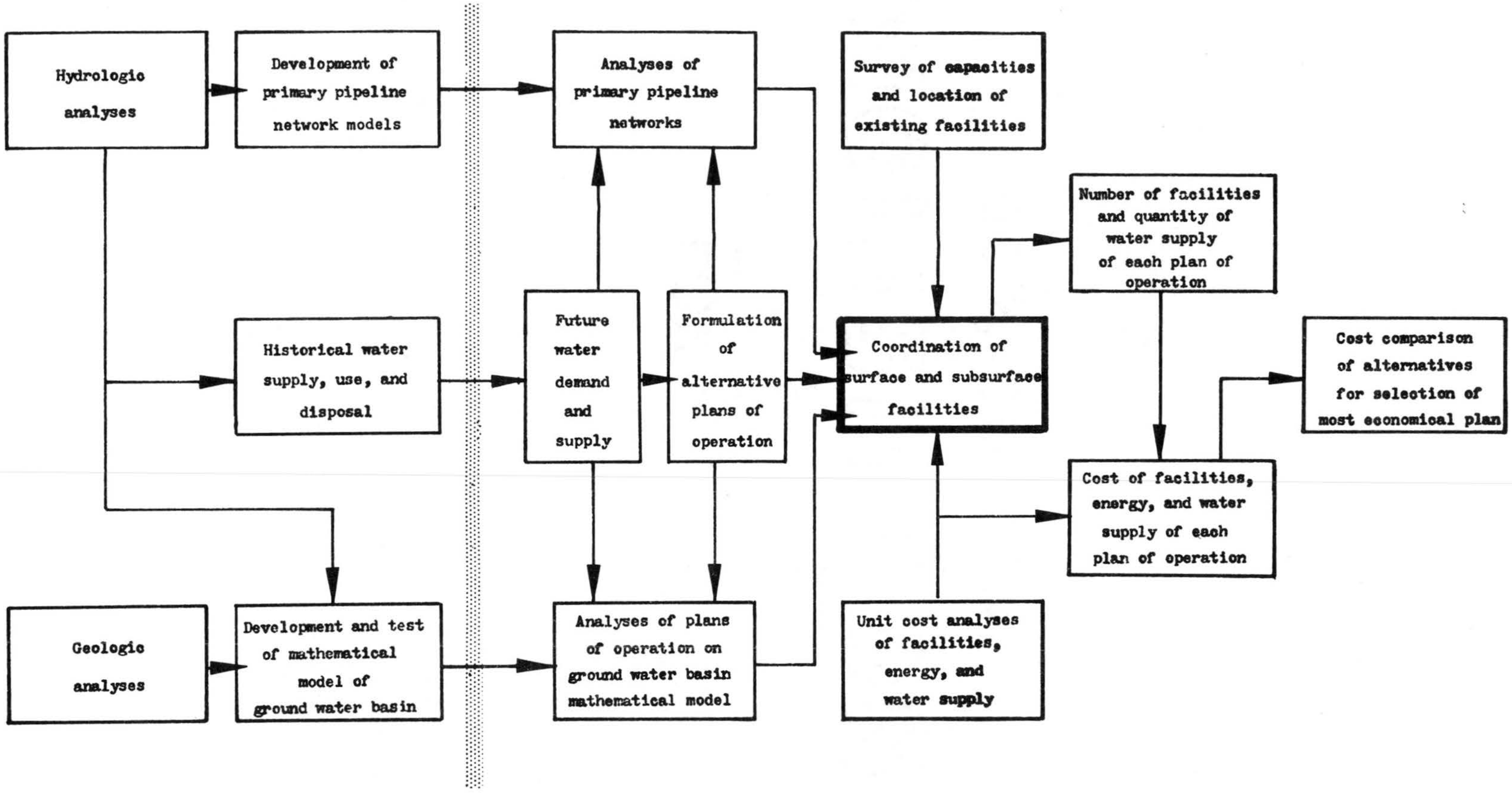


Figure 1. Simplified Flow Chart of Investigation of Planned Utilization of Ground Water Basins (from Chun, Mitchell, and Mido (7))

manner that the operation is consistent with long run irrigation development.

According to Fowler (14) the utilization of the best features of surface water and ground water is necessary for the development of the optimum coordinated operation. Where surface storage facilities are limited or subject to large evaporation losses and the stream flow is irregular, the ground water may be used. Where the ground water system is limited by a low transmissibility between points of recharge and demand, the ground water can be supplemented by canals and surface storage facilities. To make the selection of the distribution system, storage and flow equations analogous to both underground and surface conditions are derived. These equations relate (1) the underground storage to surface storage; (2) the rate of recharge of the ground water reservoirs to the flow into surface reservoirs; (3) the pumping capacity to discharge capacity from surface reservoirs; (4) transmission characteristics of aquifers to locations and delivery capacities of surface distribution facilities; and (5) piezometric surface or water table of the aquifer to the pressure head or hydraulic grade line in the surface distribution system. According to Fowler, these relations permit a controlled integration of the capabilities of the ground water system with those of the surface water delivery facilities to meet the varying water demands.

<u>Safe Yield</u>--According to Thomas (37), the safe yield of a ground water reservoir is generally considered to be

the yield at which the long term water use will be less than or equal to the amount artificially and naturally recharged. This definition makes allowance for depletion of the ground water supply during years of less than average recharge. In such a program, excess surface flows are salvaged for use in years of low flows. Thus, a ground water reservoir is used for cyclic storage and operated in conjunction with surface reservoirs to provide a firm supply of water.

Taylor (35) believes that a practice of continually drawing down surface water reservoirs throughout the dormant season by conveyance to recharge locations may serve to take advantage of the economics of "overhead cost" or full capacity operation. A lower cost per unit of water delivered results from the operation of a given conveyance system at as near full capacity as possible. In this case the water users must use a system of wells in conjunction with a surface delivery system.

Return Flow--According to Bittinger and Eshett (2), usually only part of the diverted surface water is consumptively used. The residual water is returned to the streams by direct overland flow or percolates into an aquifer that may return this water to the stream. Thus, an initial flow may be used and reused progressively along the course of the stream. Each time the initial flow is used it is diminished in quantity and often diminished in quality which affects utility, but the total use can be greater than the original flow.

According to Hartman and Seastone (26), the traditional static equilibrium test of the optimal allocation of resources compares the marginal value products, which is the return for the last input unit, resulting from different uses. This concept cannot be applied straight forward since an income from an existing allocation of a given flow may possibly be increased by reallocation where the use of the return flow is greater. Reallocation refers to any change in the present allocation pattern. Hartman and Seastone (24) illustrate this point with the following example. Assume there are n stream diversions on a stretch of the river. If the fraction returned to the stream is designated by R_i from the i^{th} diversion and S_1 is denoted as the amount diverted from the farthest upstream diversion, then the total feasible productive use π of S_1 is

$$\pi = S_1 + R_1S_1 + R_2(R_1S_1) + \ldots + R_{n-1}(R_{n-2} \ldots R_1S_1). \quad (1)$$

A schematic illustration of this principle is shown in Figure 2.

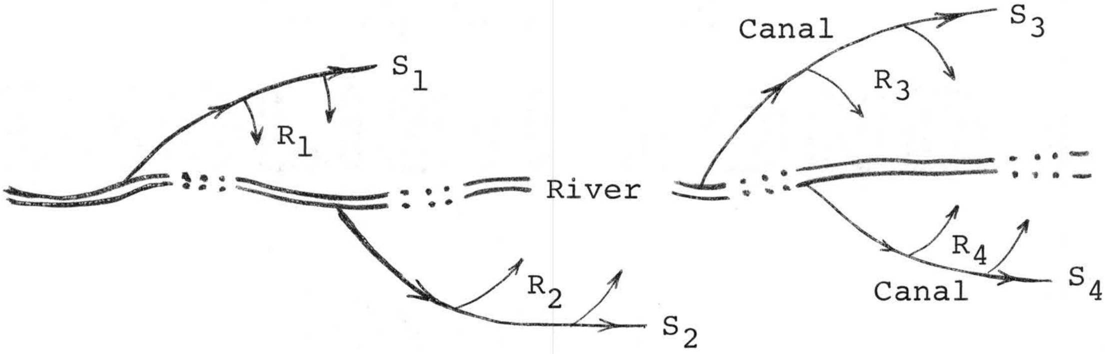


Figure 2. Schematic Sketch of Diversions and Return Flows in a River Basin

If f_i is designated as the marginal value products MVP of a unit of water for the i^{th} diversion, the total marginal value products TMVP of a unit of diverted water for the first diversion is

$$TMVP = f_1 + f_2(R_1) + f_3(R_1R_2) + \ldots + f_n(R_1 \ldots R_{n-1}). \quad (2)$$

If only f_1 is considered to find the optimal allocation, the return flow used cannot influence the allocation as it should. Consequently, the TMVP should be used to find the optimal allocation.

Water Quality--According to Kaufman (31), irrigation practices are deteriorating the ground water quality in many important river basins. Generally, about two-thirds of the irrigation water is used consumptively while only a very small portion of the salts are consumed. The remainder of the irrigation water, enriched in salts by the evapotranspiration process, and leaching of fertilizer, percolates to the water table and into the ground water. Subsequent users of the water are faced with an even greater salinity problem and must apply increasing quantities of water in order to maintain the essential salt balance. If a salinity problem develops, the management program must be altered to supply the increased demand for water. The benefits from return flow reuse may be nullified if the return flow is too salty for irrigation purposes.

Chapter II

TECHNIQUE OF INVESTIGATION

The technique used to develop several allocation patterns consisted of four basic parts. The first step was the development of a hypothetical basin model. The second step was the assignment of two reference or index application patterns for unit-areas making up the basin model. One application pattern was for water from surface facilities and the other was for pumped water. The third step consisted of varying the levels of water application on the unit-areas. The last step involved a method for developing an allocation pattern. Finally, several allocation patterns, called cases, were evaluated on the basis of water-use efficiency and economic soundness.

In this study an _application pattern_ was considered to denote the layout of the irrigated area and the associated depths of water application. The _distribution methods_ refer to the means of supplying the water (i.e., by surface facilities or wells). An _allocation pattern_ denotes a particular application pattern that uses a particular method or combination of methods for distributing the water.

The properties of the unit-areas making up the hypothetical basin were selected to approximate part of the South Platte River Basin in Colorado.

The values selected included an initial saturated thickness of 100 feet, a hydraulic conductivity of 0.0060 ft/sec, a storage coefficient of 0.170, and a basin width of 12 miles with the river in the center and a length of 160 miles (22).

The Hypothetical Basin Model

A unit-area is used to simulate a rectangular area ten miles long by six miles wide. The area is bounded by a vertical impermeable boundary on the two ends and along one side. The fourth side represents the stream. In the x-direction, parallel to the river, the unit-area is divided into ten strips of equal width; while the y-direction, normal to the river, is divided into eight strips. Six of these eight strips are approximately one mile wide and the other two are fifty feet wide. These eight strips represent, starting from the stream, three one-mile wide strips of farm land, a fifty-foot wide irrigation canal, three one-mile wide farming strips and finally another fifty-foot wide canal. The result is two canals and 60 grids of 640 acres in size. A schematic diagram of the unit area is shown in Figures 3a and 3b.

The canals shown in Figure 3b are primary canals and have an assumed seepage rate of one cfs/mile. They are assumed to operate for five months each year. The secondary distribution canals are assumed to be lined; therefore, they contribute no seepage.

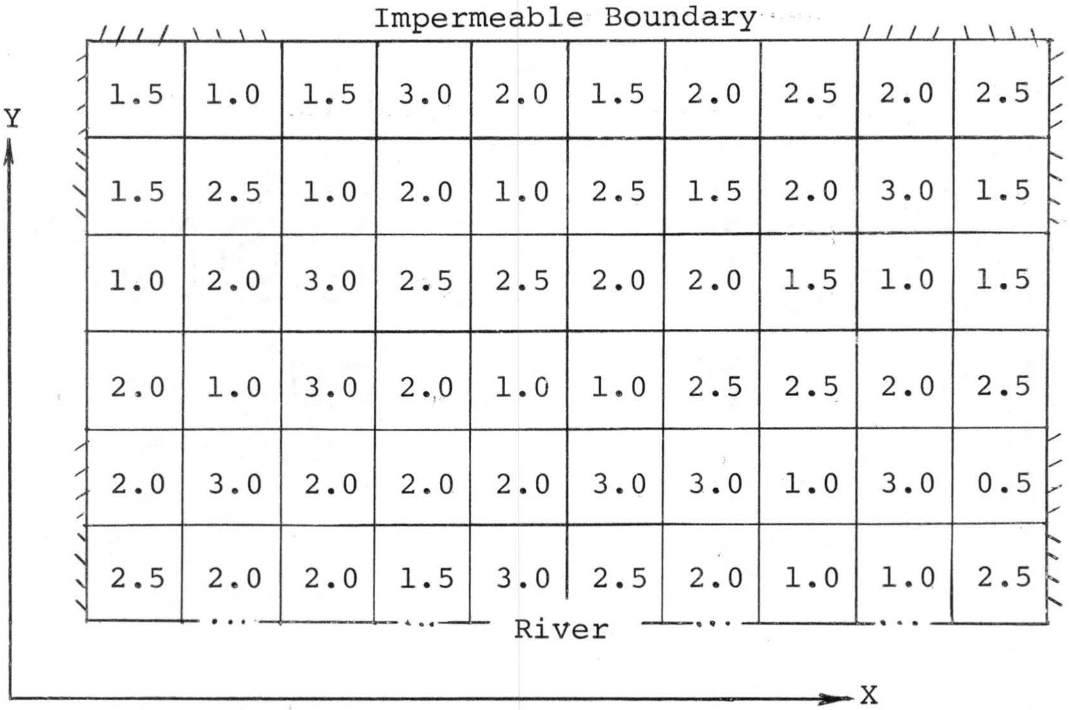


Figure 3a. Pumped Water Application Pattern for the Unit Area

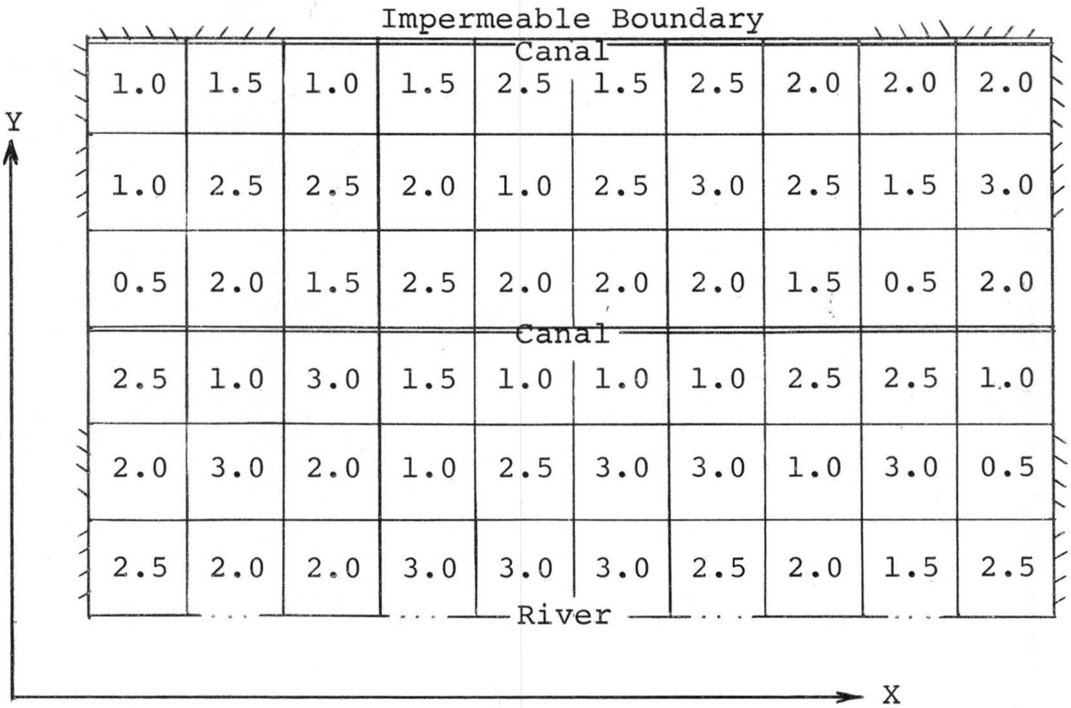


Figure 3b. Surface Water Application Pattern for the Unit-Area

The hypothetical basin is composed of sixteen pairs of unit-areas. These unit-areas are arranged to simulate a river through the center of the basin. Figure 4 illustrates this hypothetical basin and the individual unit-areas comprising the basin model. Each pair of unit-areas, where the

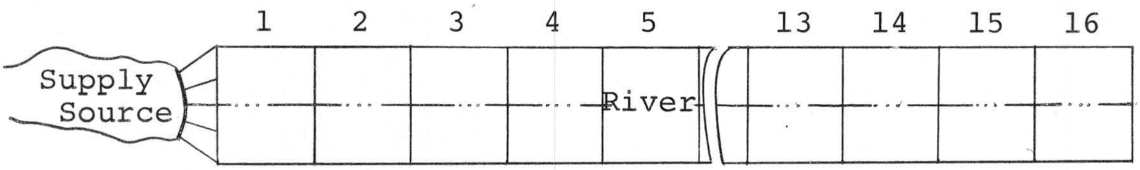


Figure 4. Basin Model

stream is a common boundary, have the same level of pumping and/or surface water application. The volume of either surface water or ground water application varies considerably from one end of the basin model to the other, depending upon the water allocation pattern.

In the development of a unit-area application pattern that could very roughly approximate a field case, various amounts of water were made available for the 640 acre grids. These amounts of water range from 320 ac-ft through 1920 ac-ft in 320 ac-ft increments for each year. Each grid in the unit-area is assigned two levels of application. One level represents the amount of pumped water, Figure 3a, and the other is for surface water, Figure 3b. In each case, 75,200 ac-ft of water are available for the unit-area. For later discussion these two levels of application are referred to as the reference or index application patterns of the unit.

For an economic evaluation, the same volume of water that is assigned to the grid is applied at a depth of three feet to a fraction of the grid. It is applied as needed during the growing season. For return flow calculations, the water is applied uniformly over the entire grid and over the entire year.

To develop a flexible hypothetical basin that allows a comparison of various allocation patterns, various amounts of water were made available for use in the unit-areas. Each unit-area may receive 0%, 20%, 40%, 60%, 80%, or 100% of the index or reference levels. These percentage levels of application on the unit-area reduce the amount applied in each section by the same percentage. Table 1 refers to the amount of water available for the various unit-area application levels. When a combination of distribution methods is used, the total applied water is to the sum of the two application levels for each method.

TABLE 1

AMOUNT OF APPLIED IRRIGATION WATER AT VARIOUS UNIT-AREA APPLICATION LEVELS

Multiplier and Reference Number	Percent of Application to Reference Model	Total Available Water per Unit-Area (Ac-Ft)	Irrigated Acres in Unit-Area
1.0	100	75,200	25,060
0.8	80	60,160	20,048
0.6	60	45,120	15,036
0.4	40	30,080	10,024
0.2	20	15,040	5,012
0.0	0	0	0

Operation of the Hypothetical Model

A continuous supply of water is assumed to be available at the head of the hypothetical basin. This supply is limited to a given quantity which is assumed to be uniform each year. The supply is located far enough upstream from the basin that the water can be diverted by gravity to the head of the basin's canals. However, the water can be released to the streams and/or canals with no water losses occurring until the water reaches the basin.

To predict the availability of irrigation water within the basin, the return flow is assumed to re-enter the stream within the same unit-area in which it is applied as irrigation water. It is also assumed that the return flow derived from the i^{th} unit-area is available for diversion by a well or canal in any unit-area below the unit-area in question. The amount of return flow depends upon the deep percolation of the surface water which is estimated to be 35 percent of the gross amount of water applied in this study (22). Similarly, it is estimated that the pumps deplete the aquifer by 65 percent of the volume pumped (22). The other factors that influence the return flow include canal seepage, recharge from precipitation, and the consumptive use of phreatophytes. Canal seepage is assumed to occur at the rate of one cfs/mile for five months of each year (22). The recharge from precipitation is assumed to be one-half inch per year (22) while the phreatophytes consume an estimated 960 acre-feet each year per river mile (3). These values are assumed to roughly approximate the South Platte River Basin. They are used for illustration purposes; therefore, only a limited amount of realism is intended.

Technique for Evaluating Allocation Patterns

In the development of a technique for evaluating a selected allocation pattern, the principle of a water balance is used. The first step in the technique is to divide the basin into unit-areas of relative uniform characteristics. In the hypothetical cases, the unit-areas are the same size and of the same characteristics. The second step is to sketch or plot a selected basin-wide application pattern and to select the distribution method or methods for each unit-area. The third step is to determine the annual amount of available irrigation water at the head of the basin and to assign the water to the head of the stream and/or the canals. The decision as to the amount of water allocated to the stream and/or canals is based on the distribution method or methods within each unit-area. The fourth step includes determining the water balance in the stream and canals at the downstream end of unit-area No. 1. The stream balance at the downstream end of the i^{th} unit-area is

$$\begin{aligned} \text{Stream Balance (i)} = {} & \text{Stream Balance (i-1)} \\ & + \text{Precipitation Return (i)} \\ & + \text{Canal Return (i)} \\ & + \text{Surface Water Return (i)} \\ & - \text{Pump Depletion (i)} \\ & - \text{Canal Diversions (i)} \\ & - \text{Phreatophyte Use (i)} \qquad (3) \end{aligned}$$

while the canal balance for the same unit-area is determined from

$$\begin{aligned} \text{Canal Balance (i)} = {} & \text{Canal Balance (i-1)} \\ & + \text{Stream Diversion (i)} \\ & - \text{Canal Seepage (i)} \\ & - \text{Surface Water Diverted (i)}. \end{aligned} \quad (4)$$

The canal balance can be calculated directly from the data provided with the selected allocation pattern and previously determined data. The stream balance requires calculating the return flow for the surface water, the pumped water, and the canal seepage. The fifth step is to proceed to the end of the adjacent and downstream unit-area and to apply equations 3 and 4 again. The last step is to repeat the fifth step until all the available water is used or the end of the basin is reached.

In the cases studied the technique just presented was applied for only the 15th year due to time limitations. The reason for selecting the 15th year was to allow the system to approximate equilibrium. However, in most applications of this technique the balances should be calculated each year or at some other interval to properly evaluate the performance of an allocation pattern.

To illustrate an application of the technique just presented, consider an allocation pattern that uses a combination of surface and ground water in most of the unit-areas, a heavy water use in the lower part of the basin, and a mild fluctuation about the mean level of water use in the other part of the basin. This is the case presented in Case No. 5 of Chapter IV.

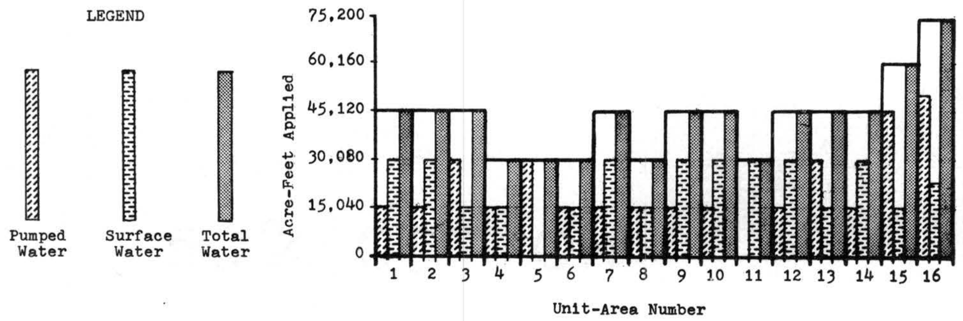
The water supply for the basin is based upon a constant volume of water available for each year. The supply is distributed according to the preselected allocation pattern. In this case there was 500,000 ac-ft assigned to the canals and an equal amount to the stream. The next step was to calculate the canal and stream balance at the end of the first unit-area. This was accomplished by tabulating the known data (i.e., canal losses, precipitation recharge, phreatophyte use, and applied surface and pumped water), calculating the return flow attributed to the 15th year, and applying equations 3 and 4. This procedure is continued until the end of the basin is reached. The procedure was adjusted to account for the diversions and extra canal seepage in unit-area Nos. 8 and 9. Table 2 is used to tabulate the results and known data. The bar graph presented in the table was developed for only one side of the basin while the table is for the entire basin. As stated earlier the activities on one side of the basin are identical to the other side.

Methods of Comparing Allocation Patterns

Because of the interrelation of the distribution method or methods and the application pattern, a complete evaluation was only possible on a basin-wide, allocation pattern. To make a direct, basin-wide comparison of several

TABLE 2

SAMPLE CALCULATION OF RUN FOR CASE NO. 5



Unit-Area Number	Canal Losses (ac-ft)	Precipitation Recharge (ac-ft)	Phreatophyte Use (ac-ft)	Return Flow (ac-ft)	Diversions (ac-ft)	Applied Water Surface (ac-ft)	Applied Water Pumped (ac-ft)	Stream Balance (ac-ft)	Canal Balance (ac-ft)
								500,000	**500,000**
1	11,920	3,200	9,600	21,056 -19,552		60,160	30,080	507,024	**427,920**
2	11,920	3,200	9,600	21,056 -19,552		60,160	30,080	514,048	355,840
3	11,920	3,200	9,600	21,056 -39,104		30,080	60,160	490,992	313,840
4	11,920	3,200	9,600	10,528 -19,552		30,080	30,080	487,488	271,840
5	11,920	3,200	9,600	0 -39,104		0	60,160	453,904	259,920
6	11,920	3,200	9,600	10,520 -19,552		30,080	30,080	454,400	217,920
7	11,920	3,200	9,600	21,056 -19,552		60,160	30,080	457,424	145,840
8	17,880	3,200	9,600	10,528 -19,522	211,920	30,080	30,080	247,960	103,840
9	23,840	3,200	9,600	21,056 -19,552	205,960	60,160	30,080	62,984	29,760
10	11,920	3,200	9,600	21,056 -19,552		60,160	30,080	70,008	357,680
11	11,920	3,200	9,600	21,056 0		60,160	0	84,584	285,600
12	11,920	3,200	9,600	21,056 -19,552		60,160	30,080	103,608	213,520
13	11,920	3,200	9,600	10,528 -39,104		30,080	60,160	80,552	171,520
14	11,920	3,200	9,600	21,056 -19,552		60,160	30,080	87,576	99,440
15	11,920	3,200	9,600	10,528 -58,656		30,080	90,240	44,968	59,440
16	11,920	3,200	9,600	16,632 -67,120		43,520	103,000	0	0

allocation patterns, several cases were devised. The comparisons were based upon physical efficiency of water-use and economic efficiency.

Water-Use Efficiency--The efficiency of an allocation pattern was determined by comparing the total amount of productive consumptive use in the basin to the total amount of available irrigation water. The consumptive use was assumed to be 65 percent of the water applied. The remaining 35 percent returns to the aquifer and can be captured and used later to increase the basin-wide efficiency.

Economic Efficiency--The allocation patterns were also analyzed in monetary terms in order to find the most profitable alternative allocation pattern. This economic comparison was accomplished by determining the cost and benefits in each case. For simplicity the costs of pumping and delivery of surface water were estimated from previous studies (1,5). Studies by Hartman and others (25,27) provided data to determine the average farm expense per acre and the average return from the application of a given volume of water. These constant figures are far from the actual case in a field situation because of the many variables that affect cost and benefits. The constant values are justified for the hypothetical case and are used only for illustration purposes.

Chapter III

THEORETICAL DEVELOPMENT FOR PREDICTING RETURN FLOW

To calculate the return flows for equation 3, three methods of calculating return flow are presented. They include a mathematical model that uses a digital computer for a solution and two analytical methods. These methods are (1) a stream depletion method that calculates the return flow from an irrigated strip parallel to the river and (2) a steady, uniform infiltration method that assumes a steady and uniform irrigation practice over the entire unit-area of the basin.

Mathematical Model

According to Eshett and Longenbaugh (13), Irby and Lamoreaux (30) and others, the basic equation describing unconfined flow in porous media is a non-linear, second-order partial differential equation having no general solution. However, an approximate solution for particular boundary conditions can be obtained by employing a method of numerical finite-difference solutions.

The general partial differential equation describing two dimensional transient, saturated flow in porous media can be written as

$$\frac{\partial}{\partial x}\left(KD\right)\frac{\partial H}{\partial x} + \frac{\partial}{\partial y}\left(KD\right)\frac{\partial H}{\partial y} = \Phi\frac{\partial H}{\partial t} + \frac{q}{dxdy} \quad , \qquad (5)$$

where

D = saturated thickness of aquifer

K = hydraulic conductivity

q = net volumetric rate of extraction

Φ = storage coefficient of aquifer

dx, dy = dimensions of cell

H = elevation of water table above a datum, and

t = time.

(Note: The symbols used by individual authors have been changed to a common notation.)

Equation 5 may be approximated by

$$\frac{KD\Delta H}{\Delta x(\overline{\Delta x})}_{inflow} - \frac{KD\Delta H}{\Delta x(\overline{\Delta x})}_{outflow} + \frac{KD\Delta H}{\Delta y(\overline{\Delta y})}_{inflow} -$$

$$\frac{KD\Delta H}{\Delta y(\overline{\Delta y})}_{outflow} = \Phi\,\frac{\Delta H}{\Delta t} + \frac{q}{\Delta x\Delta y} \qquad (6)$$

where $\overline{\Delta x}$ and $\overline{\Delta y}$ are the mean lengths of adjacent finite areas. By applying a central finite-difference scheme to a system of finite grids such as shown in Figure 5, equation 6 may be written as

$$\left[\frac{(KD)_{i,j+1/2}\left[H_{i,j+1} - H_{i,j}\right]}{\Delta x_{i,j}\left(\dfrac{\Delta x_{i,j} + \Delta x_{i,j+1}}{2}\right)} - \cdots + \cdots - \right.$$

$$\left.\frac{(KD)_{i-1/2,j}\left[H_{i,j} - H_{i-1,j}\right]}{\Delta y_{i,j}\left(\dfrac{\Delta y_{i,j} + \Delta y_{i-1,j}}{2}\right)}\right] = \Phi\left[\frac{(H_{i,j})_{t+\Delta t} - (H_{i,j})_{t}}{(t + \Delta t) - (t)}\right]$$

$$+ \frac{q}{\Delta x_{i,j}\,\Delta y_{i,j}} \quad . \qquad (7)$$

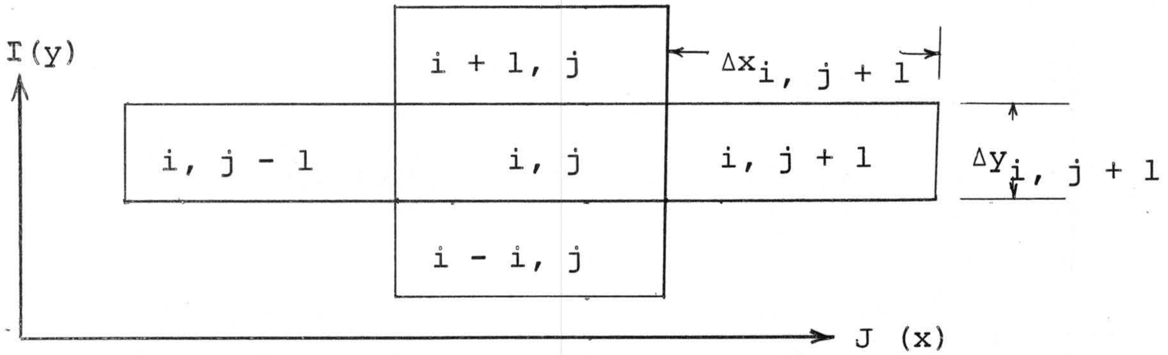


Figure 5. System of Finite Grids for Mathematical Model

The subscripts i and j in equation 7 are used to identify grid blocks. By defining coefficients A, B, C, and E to have the form

$$A = \frac{2(KD)_{i,j+1/2}}{\Delta x_{i,j}(\Delta x_{i,j} + \Delta x_{i,j+1})} \quad . \qquad (8)$$

Equation 7 may be simplified and rearranged, with all unknown parameters on the left hand side of the equation as

$$\Big[AH_{i,j-1} + BH_{i-1,j} + (-A - B - C - E - \Phi/\Delta t)\,H_{i,j} + CH_{i,j+1} + EH_{i+1,j}\Big]_{t+\Delta t} = \Big[-H_{i,j}\,(\Phi/\Delta t) + \frac{q}{(\Delta x_{i,j})(\Delta y_{i,j})}\Big]_t \quad . \qquad (9)$$

The coefficients A, B, C, and E are computed from potentials obtained in the previous time steps. Such an explicit computation linearizes equation 5 and eliminates the need for an iterative solution. Equation 9 is written for each of <u>n</u> grid blocks of the aquifer; thus, a solution of the aquifer system requires the simultaneous solution of <u>n</u>

equations with _n_ unknowns. A Gaussian Elimination Procedure is used to solve these _n_ equations. The computer program used for this portion of the study is listed in Appendix I.

The mathematical model requires dividing the area into finite grids and applying the necessary data uniformly to each grid. The method first calculates the water table elevations at the center of each grid, for each time step. Using these water table elevations and Darcy's Law, the amount of water exchanged between the adjacent aquifer grids and the river is calculated for each time step. This exchange of water represents the return flow. A positive value indicates water is flowing from the aquifer to the river; a negative sign represents water flowing from the river to the aquifer.

The mathematical model provides a detailed simulation of the area, but the result is a lengthy solution requiring a high-speed digital computer. However, the results should be the best of the three methods studied because of the detail of data and fewer simplifying assumptions.

Stream Depletion Method

A method of estimating the water exchange between a stream and a ground water aquifer was adapted from Glover's (19,21) method of analysis. First, Glover developed a stream depletion equation for a pumping well drawing from an aquifer connected to a stream. He later showed that the same equation held for a line source or sink. This equation is modified to calculate either depletion or return flow from an irrigated strip parallel to the stream.

According to Glover (15,17,20,21) in order to describe the water table curvature due to the operation of a line source or sink in an infinite aquifer, the differential equation

$$\alpha \left(\frac{\partial^2 s}{\partial x^2} \right) = \frac{\partial s}{\partial t} \tag{10}$$

and the boundary conditions

$s = 0$ when $t = 0$ for $x > 0$ and

$-KD\left(\frac{\partial s}{\partial x}\right) = \frac{q_1}{2}$, when $x = 0$ for $t > 0$ must be satisfied.

A solution that satisfies these conditions is

$$s = \frac{q_1 x}{2\pi KD} \sqrt{\pi} \int_{\frac{x}{\sqrt{4\alpha t}}}^{\infty} \frac{e^{-u^2}}{u^2}\, du \quad , \tag{11}$$

where

s = drawdown from initial water table

x = horizontal distance perpendicular to line source or sink

q_1 = flow per unit length of line source, and

α = KD/Φ.

The rate of flow per unit width, symbolized by F , at a distance x from the line source or sink is described by Glover to be

$$F = -KD\left(\frac{\partial s}{\partial x}\right) \quad . \tag{12}$$

Taking the partial derivative of s with respect to x in equation 11 gives

$$\frac{\partial s}{\partial x} = \frac{q_1 \sqrt{\pi}}{2\pi KD}\left[\frac{-\sqrt{4\alpha t}\; e^{-\left(x^2/4\alpha t\right)}}{x} + \int_{\frac{x}{\sqrt{4\alpha t}}}^{\infty} \frac{e^{-u^2}}{u^2}\, du\right] \quad . \qquad (13)$$

Substituting equation 13 into equation 12 and simplifying results in

$$F = \frac{q_1}{2}\left[\, 1 - \frac{2}{\sqrt{\pi}} \int_0^{\frac{x}{\sqrt{4\alpha t}}} e^{-u^2}\, du\right] \quad . \qquad (14)$$

Equation 14 was obtained by assuming an infinite aquifer. The case under consideration in this study is a stream parallel to the line sink. To simulate this condition, an image of the real sink is projected across the stream as an imaginary source of the same magnitude. Utilizing this image theory, the total flow from the stream from the real line sink is found by summing the flows of the real sink and the imaginary line source across the stream. Since the distances from the real line sink and imaginary line source are the same, the flow from the stream is given by

$$F = q_1\left[\, 1 - \frac{2}{\sqrt{\pi}} \int_0^{\frac{x_1}{\sqrt{4\alpha t}}} e^{-u^2}\, du\right] \quad , \qquad (15)$$

where x_1 is the distance from the stream to the line source or sink. A sketch of this arrangement is shown in Figure 6.

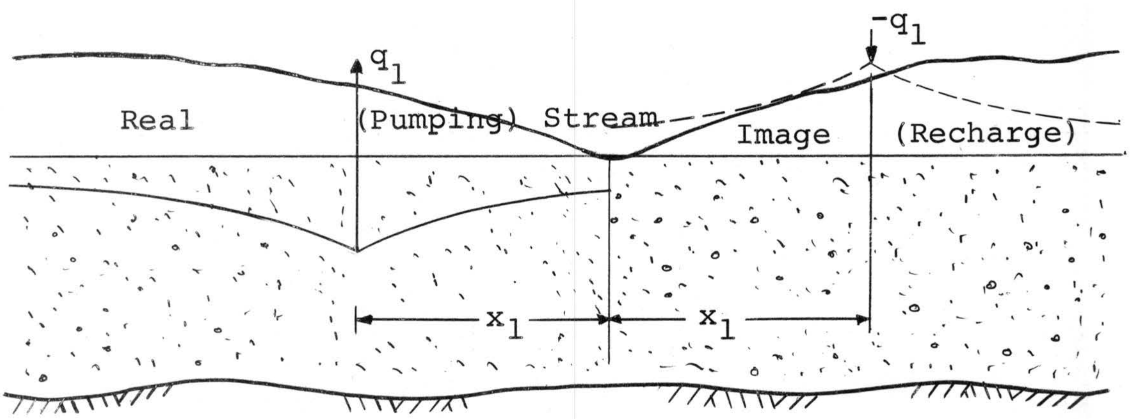


Figure 6. Sketch of River Cross Section, Line Source, and Image.

To obtain the volumetric stream depletion, equation 15 is multiplied by dt and then integrated with respect to time as shown in equation 16 and 17.

$$\int_0^t F\,dt = q_1 \int_0^t dt - q_1 \int_0^t \left[\frac{2}{\sqrt{\pi}} \int_0^{\frac{x_1}{\sqrt{4\alpha t}}} e^{-u^2}\,du \right] dt \;, \quad (16)$$

which results in

$$\frac{\int_0^t F\,dt}{q_1 t} = 1 - \frac{2}{\sqrt{\pi}} \int_0^{\frac{x_1}{\sqrt{4\alpha t}}} e^{-u^2}\,du - \left(\frac{2x_1^{\,2}}{\pi(4\alpha t)} \right) \left(\sqrt{\pi} \int_{\frac{x_1}{\sqrt{4\alpha t}}}^{\infty} \frac{e^{-u^2}}{u^2}\,du \right) . \quad (17)$$

Equation 17 was developed for a line source or sink; however, it also holds for a pumping well. The reader is referred to references by Glover (19,20) for a more detailed development of this equation.

Equation 17 gives the ratio of the volume of flow into a constant head, such as a stream, to the volume of water supplied by the line source or sink as a function of time. Figure 7 relates this ratio in dimensionless form with the scaled distance $x/\sqrt{4\alpha t}$.

To find $\int_0^t F\,dt/q_1 t$ for an irrigated strip parallel to the stream, Figure 7 is graphically integrated between the values $x_2/\sqrt{4\alpha t}$ and $x_3/\sqrt{4\alpha t}$, where x_2 and x_3 are the distances from the stream to the respective edges of the irrigated strip, and the result divided by $(x_2/\sqrt{4\alpha t} - x_3/\sqrt{4\alpha t})$. This graphical integration is accomplished by using Simpson's Rule. A schematic diagram of this procedure is shown in Figures 8a and 8b. Referring to Figure 8b, $\int_0^t F\,dt/q_1 t$ for the irrigated strip is equal to the area FCDE divided by the area ABDE. An impermeable boundary is accounted for by using the same procedure on imaged irrigated strips.

The stream depletion method assumes a uniform hydraulic conductivity, an initially level water table, level bedrock, uniform specific yield over the unit-area, and uniform and steady application or withdrawal of water over the entire strip as well as an infinitely large aquifer bounded by a stream on one side.

To calculate the return flow by the stream depletion method, the area is divided into strips that are the length

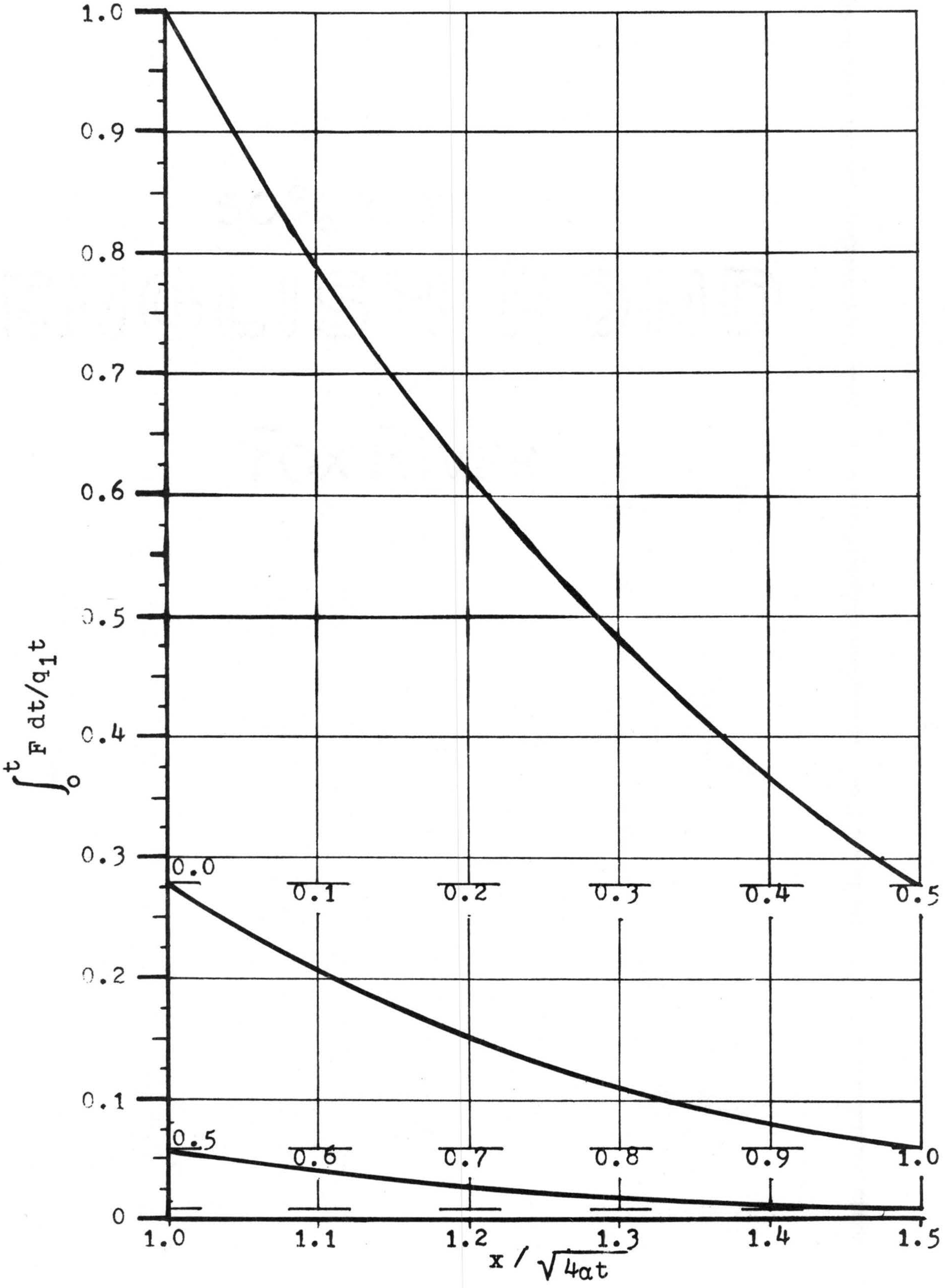


Figure 7. Relationship of Volumetric Depletion of a Line Sources from a Stream versus $x/\sqrt{4\alpha t}$

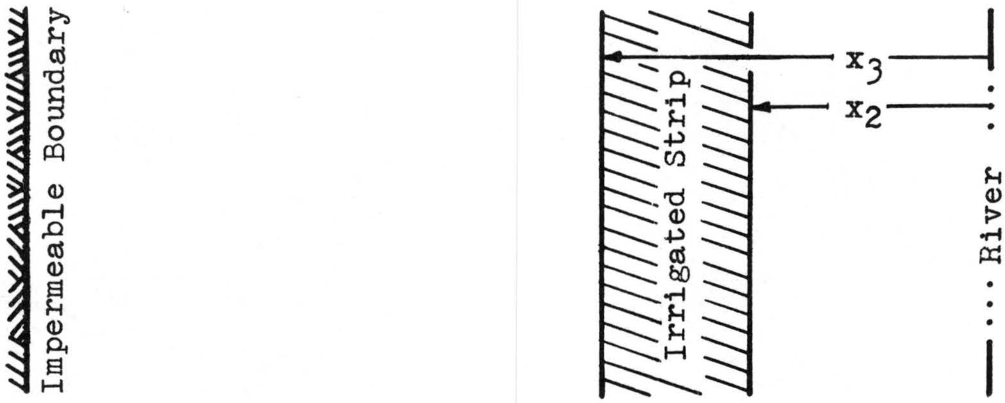


Figure 8a. Plan View of Irrigated Strip, River and Impermeable Boundary

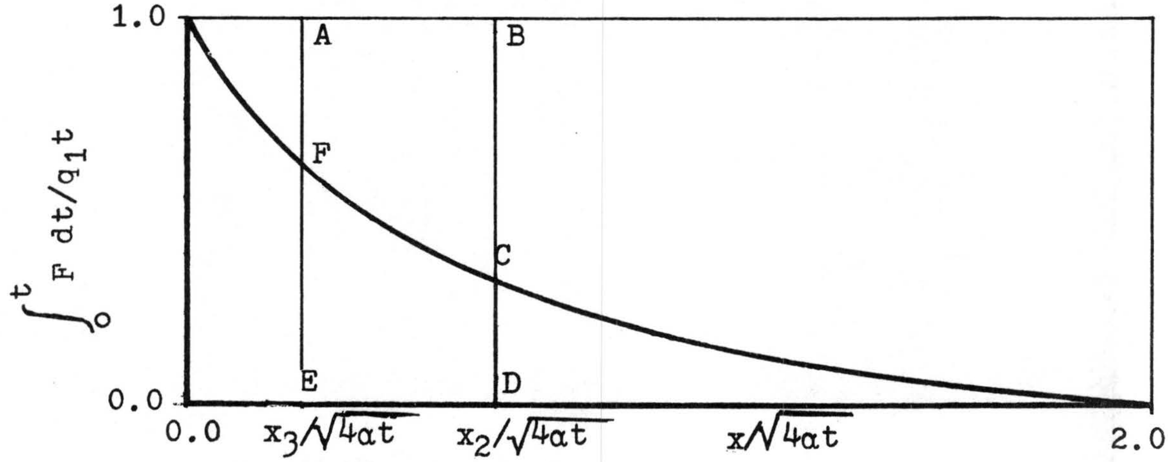


Figure 8b. Procedure to Use Figure 5 to find $\int_0^t F\,dt/q_1t$ for an Irrigated Strip

of the unit-area and parallel to the stream. Starting with a level water table at $t = 0$, the return flow was calculated, using Figure 7, for each strip, including the images, at time t . The return flows contributed by each strip are summed algebraically to determine the total return flow. If pumping is the source of water in the irrigated strip, the discharge in Equation 17 is negative; if the source is surface water, the discharge is positive.

The stream depletion method has assumptions that prohibits it from being applied rigorously to many field cases. However, the use of strips provide some detail in the application patterns. The calculations are somewhat laborious, but they can be performed with a desk calculator and applicable curves. The method would be most useful to persons or organizations of limited finances and to preliminary investigations. Because this method simulates the application patterns to some detail and meets the boundary and initial conditions of the hypothetical basin, it was used for return flow calculations in the allocation patterns presented later.

Steady, Uniform Infiltration Method

Since a stream-aquifer system behaves like a drainage system, drainage equations can be used to predict the water table elevations and the return flow. The basic transient drainage equation derived by Glover (15,16,21) was used in this portion of the study.

Equation 19, which follows, satisfies the differential equation

$$\alpha\left(\frac{\partial^2 h'}{\partial x^2}\right) = \frac{\partial h'}{\partial t} \qquad (18)$$

and the following boundary conditions.

$h' = 0$ when $x = 0$ for $t > 0$, and

$h' = H'$ when $x > 0$ when $t = 0$.

Glover's equation is

$$h' = H' \frac{4}{\pi} \sum_{n=1,3,5,\ldots}^{n=\infty} \left[\left(\frac{\mathrm{Sin}\left(\frac{n\pi x}{L}\right)}{n} \right) \left(e^{-n^2\pi^2\alpha t/L^2} \right) \right], \qquad (19)$$

where d' = thickness between stream and bedrock

h' = drainable depth

H' = initial drainable depth

n = consecutive odd integers used in specifying terms of a series

x = perpendicular distance from stream

L = twice the distance between the stream and the vertical impermeable boundary

α = Kd'/ϕ .

A cross-section of the stream-aquifer system and the notation used is shown in Figure 9.

Equation 19 describes the water table profile that results after a uniform, instantaneous application of water on an initially level water table as a function of time. The

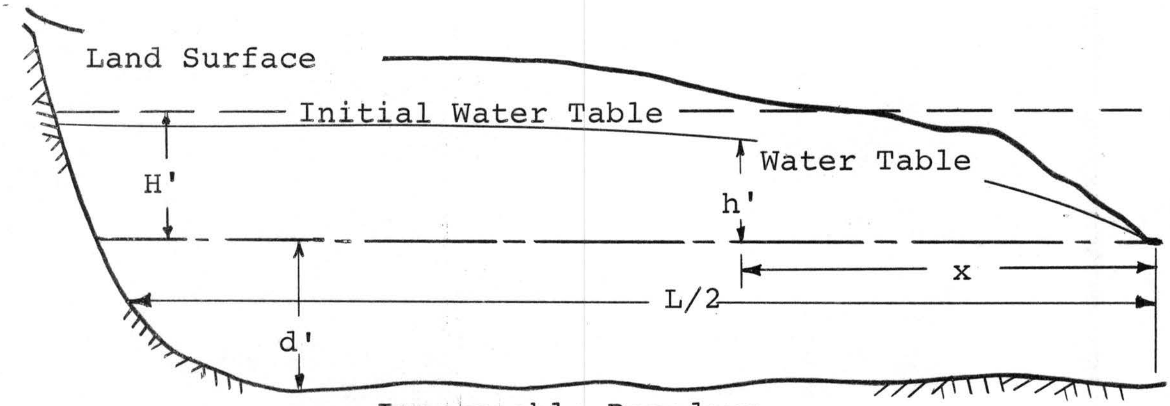


Figure 9. Cross-Section of Stream-Aquifer System

other assumptions include homogeneous aquifer, level bedrock, and a vertical impermeable boundary parallel to the stream.

To avoid the assumption of instantaneous application, Maasland, according to Glover (21), modified equation 19 to assume steady, uniform infiltration. Maasland accomplished this modification by expressing the initial drainable depth from an instantaneous application as a function of time, or

$$H' = \frac{It}{\phi} \tag{20}$$

where I equals the steady infiltration rate. Substituting equation 20 into equation 19 and integrating with respect to time between the limits $t = 0$ and $t = t$, Maasland obtained

$$h' = \frac{4L}{\pi^3 \alpha} \left(\frac{I}{\phi}\right) \sum_{n=1,3,5,\ldots}^{n=\infty} \left[\left(\frac{\operatorname{Sin} \frac{n\pi x}{L}}{n^3} \right) \left(1 - e^{-\alpha n^2 \pi^2 t / L^2} \right) \right] . \tag{21}$$

The return flow volume is the difference between the volume of water applied from time zero to time t and the volume of water remaining at time t. The volume of saturated aquifer remaining above the initial water table at time t was found by integrating equation 21 with respect to x between the limits of 0 and L/2. The resulting equation, expressing the saturated volume per unit width, is

$$\int_0^{L/2} h'dx = \frac{4L^3}{\pi^4 \alpha}\left(\frac{I}{\phi}\right)\sum_{n=1,3,5,\ldots}^{n=\infty}\left[\left(-\frac{\cos\frac{n\pi x}{L}}{n^4}\right)\left(1 - e^{-\alpha n^2 \pi^2 t/L^2}\right)\right]_0^{L/2}, \tag{22}$$

which reduces to

$$\int_0^{L/2} h'dx = \frac{4L^3}{\pi^4 \alpha}\left(\frac{I}{\phi}\right)\sum_{n=1,3,5,\ldots}^{n=\infty}\left[\frac{1 - e^{-\alpha n^2 \pi^2 t/L^2}}{n^4}\right] \tag{23}$$

The total saturated volume of aquifer per unit width due to the volume of water applied V is

$$V = \frac{It}{\phi}(L/2) \tag{24}$$

The part or fraction of the applied water still remaining between the stream and the impermeable boundary at time t, symbolized by PR, is found by dividing equation 23 by equation 24. After simplification, the result is

$$PR = \frac{8}{\pi^4}\left(\frac{L}{\alpha t}\right)\sum_{n=1,3,5,\ldots}^{n=\infty}\left[\frac{1 - e^{-\alpha n^2 \pi^2 t/L^2}}{n^4}\right] \quad . \tag{25}$$

The return flow at time t is that portion of the applied water removed by the stream (1 - PR) multiplied by the volume of water applied. The solution of equation 25 is shown graphically in Figure 10.

This method has a limited application in calculating return flows for most field situations because of the assumptions. The steady, uniform infiltration rate is the most limiting assumption. However, the method will provide quick estimates with a minimum of effort.

Comparison of Methods for Calculating Return Flows

For a comparison of the three methods used to calculate return flow, Table 3 is presented. The values in the table were calculated for the reference unit-areas in the preceding chapter. These values were calculated each year for a 15 year period. The steady infiltration method doesn't warrant significant digits for the last three digits while the stream depletion method doesn't warrant significant digits in the last two digits. This is due to the curves and the assumptions used. For the mathematical model, a close examination of its results and the steady state values (i.e., 48,880 ac-ft for pumping, 32,280 ac-ft for surface water plus canal seepage, and 26,320 ac-ft for surface water) indicates that the return flow values do not approach a steady state asymptotically. A further analysis of the mathematical model's results showed that the return flow values were in error by approximately 2.5 percent.

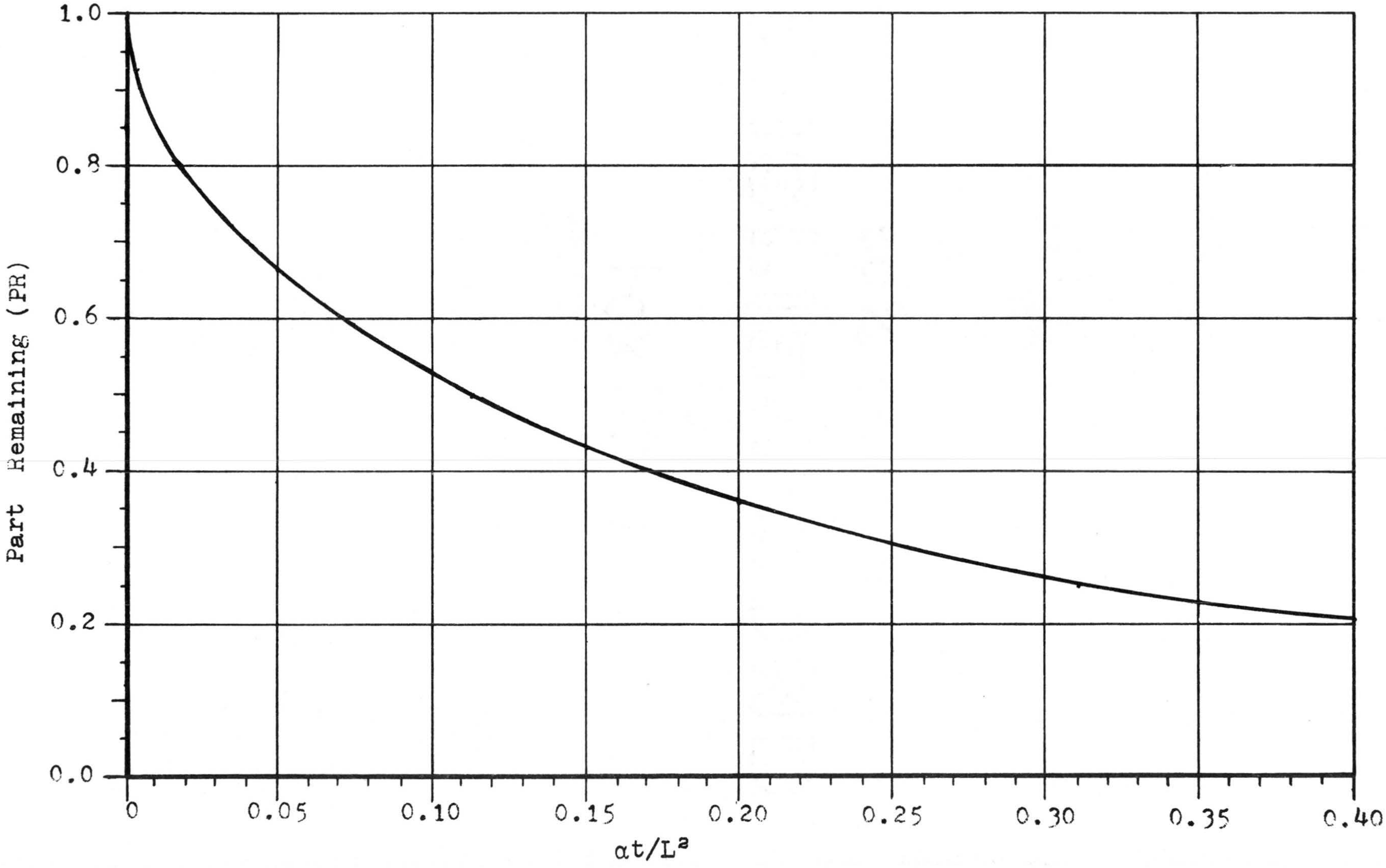


Figure 10. Part of Applied Water Remaining in the Aquifer due to Steady, Uniform Infiltration

TABLE 3

ANNUAL RETURN FLOW (ACRE-FEET)

YEAR (end)	SURFACE WATER ONLY			SURFACE WATER + CANAL WATER			PUMPED WATER		
	Mathematical Model	Stream Depletion Equation	Steady Infiltration Method	Mathematical Model	Stream Depletion Equation	Steady Infiltration Method	Mathematical Model	Stream Depletion Equation	Steady Infiltration Method
1	6,954	7,400	7,000	7,435	7,800	8,000	11,724	12,700	12,000
2	12,569	12,600	12,000	14,312	14,300	15,000	21,495	22,800	22,000
3	16,052	15,900	15,000	18,902	18,700	19,000	27,194	29,100	28,000
4	18,680	18,500	18,000	22,399	22,000	22,000	31,339	34,000	34,000
5	20,683	20,200	20,000	25,065	24,300	25,000	34,487	37,200	37,000
6	22,201	21,700	21,000	27,074	26,100	26,000	36,925	40,100	40,000
7	23,344	22,700	23,000	28,579	27,500	28,000	38,843	41,900	42,000
8	24,202	23,600	23,000	29,697	26,000	29,000	40,370	43,600	43,000
9	24,841	24,200	24,000	30,524	29,500	30,000	41,596	44,800	45,000
10	25,319	24,800	25,000	31,136	30,200	30,000	42,590	45,900	46,000
11	25,671	25,100	25,000	31,584	30,700	31,000	43,400	46,600	46,000
12	25,937	25,400	25,000	31,915	31,000	31,000	44,066	47,300	47,000
13	26,131	25,500	25,000	32,158	31,100	31,000	44,614	47,400	47,000
14	26,275	25,900	26,000	32,335	31,700	32,000	45,069	48,100	48,000
15	26,383	26,100	26,000	32,465	32,000	32,000	45,447	48,200	49,000

Chapter IV

RESULTS AND DISCUSSION

Return Flow Relationships

Initially, the return flow was calculated for the reference level of water application in the unit-areas. To find the return flow for the other levels of application, the reference number was multiplied by the return flow calculated from the reference level. The return flow was calculated for each year in a fifteen year period in order to obtain a relation of the return flow to time. The calculations were made for surface water applications, surface water applications plus canal seepage, and pumped water applications. These computations were made by each of the three methods that were discussed; however, the results of only the stream depletion method were used in the comparison of the different allocation patterns. Return flows determined by the stream depletion equation are shown in Figure 11 for the reference level of water application. The maximum rate of return flow from the unit-area (i.e., after steady-state is reached) is indicated by a dashed line. This illustration indicates that a steady state is approached approximately fifteen years after initiating the irrigation practice above a level water table.

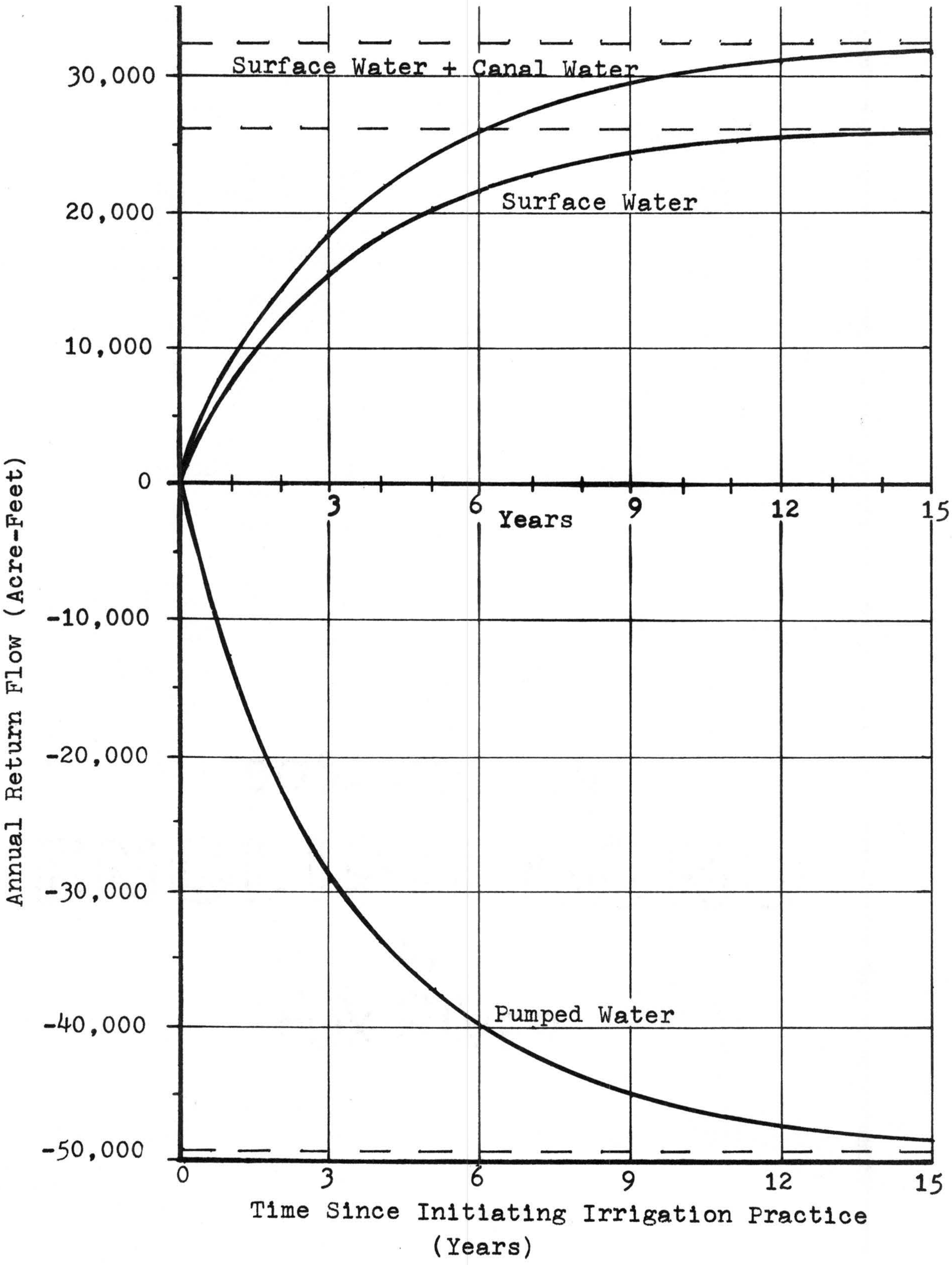


Figure 11. Return Flow versus Time for the Reference Level of Water Application

Evaluation of Allocation Patterns

Several allocation patterns were selected for evaluation and comparison on the basis of water use efficiency and economic soundness. For each allocation pattern, a water supply of 1,000,000 acre-feet was available to the basin each year. This water was distributed to the unit-areas according to the preselected allocation pattern.

The comparative economic returns for the alternative allocation patterns studied were determined by assuming that water was applied at the rate of three acre-feet per acre per year (i.e., in sufficient depth to completely supply the crop's needs) and yielded an average return of $30 per acre-foot of water applied (25,27). The pumping lift was assumed to average fifty feet over the entire basin which resulted in a cost of $2.50 per acre-foot pumped (5). The average cost of delivering surface water was estimated at $2.00 per acre-foot diverted at the headgate (1,5). The average farm cost less water cost was estimated at $40 per acre irrigated (27). For this study it was assumed that the main factors that influence the economics and efficiency are limited to the degree of consumptive use by phreatophytes, recharge from precipitation, and the use of return flow.

The allocation patterns, appearing in the following cases, are simulated by bar graphs. A set of bars indicates the annual pumping volume, annual surface water application volume, and the total volume applied in each year for each unit-area.

The results presented with each basin-wide allocation pattern include: (1) volume of surface water applied, (2) volume of applied pumped water, (3) total volume of water applied, (4) volume of water consumed, (5) efficiency, (6) irrigated acreage, (7) pumping cost, (8) surface water cost, (9) total water cost, (10) cost per acre-foot applied, (11) farm cost less water cost, (12) total cost, (13) total returns, (14) net returns, (15) benefit-cost ratio. These values were calculated for the 15th year of operation after initiating the irrigation practice. In addition to the above results, the annual positive, negative, and net return flow volumes are given at three year intervals for the entire basin. The negative return flow, stream to aquifer, is due to pumping. The positive return flow, aquifer to stream, is due to surface water, precipitation and canal seepage.

<u>Case No. 1</u>--The first case studied involved an allocation pattern that used pumping in the upper half of the basin and surface water application in the lower half of the basin. The application volumes were approximately the same for pumping and surface water application. A visual description of this allocation pattern is shown in Figure 12. The values on the graph are for only one side of the basin.

Annual results for the 15th year of operation follow.

Volume of Annual Inflow	=	1,000,000 ac-ft
Volume of Surface Water Applied	=	626,000 ac-ft
Volume of Pump Water Applied	=	602,000 ac-ft
Total Volume of Water Applied	=	1,228,000 ac-ft

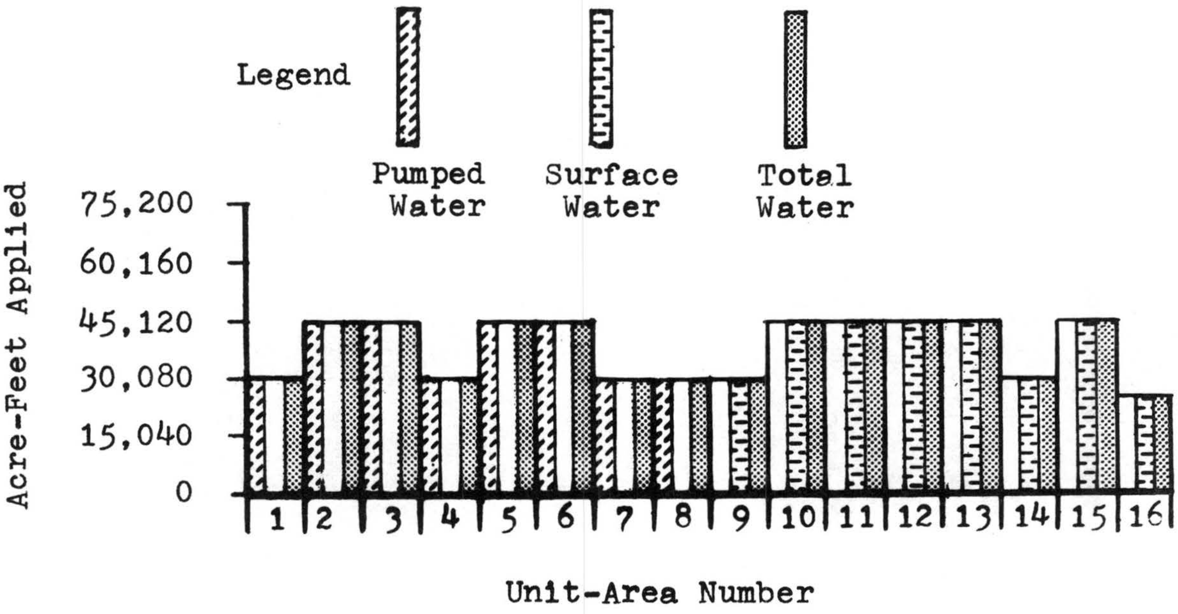


Figure 12. Allocation Pattern for Case No. 1

TABLE 4

ANNUAL RETURN FLOW FOR CASE NO. 1

Year	Positive Return Flow (ac-ft)	Negative Return Flow (ac-ft)	Net Return Flow (ac-ft)
3	148,000	232,000	- 84,000
6	208,000	320,000	-112,000
9	234,000	358,000	-124,000
12	248,000	378,000	-130,000
15	256,000	392,000	-136,000

Total Volume Consumed	=	798,000 ac-ft
Efficiency	=	79.8 percent
Irrigated Area	=	410,000 acres
Pumping Cost	=	\$ 1,504,000
Surface Water Cost	=	\$ 1,512,000
Total Water Cost	=	\$ 3,016,000
Cost per Acre-Foot Applied	=	\$ 2.46
Farm Cost Less Water Cost	=	\$16,400,000
Total Costs	=	\$19,416,000
Total Returns	=	\$36,800,000
Net Returns	=	\$17,384,000
Benefit-Cost Ratio	=	1.89:1

The allocation pattern represented by this case allows some water to leave the basin. This water, 100,000 acre-feet, represents the return flow below the last diversion after fifteen years of operation. Even during the early years of operation some water is lost because the wells did not draw their entire supply from the river. However, part of this water was used to supply the last canal diversion. Normally, this last diversion depends upon the return flow from the irrigated areas served by the earlier diversions. Table 4 illustrates the development and magnitude of the return flows as the basin approaches equilibrium. The table shows that the return flow builds rapidly at first and finally slows and approaches a steady state asymptotically.

Case No. 2--This allocation pattern essentially reverses the patterns of pumping and surface water applications of the previous case. However, a transition between the pumps and surface water application is introduced instead of using an abrupt change. Again, the application volumes are approximately the same for the two methods of distributing water. The layout of this case is shown in Figure 13.

Annual results for the 15th year of operation follow.

Volume of Annual Inflow	=	1,000,000 ac-ft
Volume of Surface Water Applied	=	722,000 ac-ft
Volume of Pumped Water Applied	=	658,000 ac-ft
Total Volume of Water Applied	=	1,380,000 ac-ft
Total Volume Consumed	=	898,000 ac-ft
Efficiency	=	89.8 percent
Irrigated Area	=	460,000 acres
Pumping Cost	=	$ 1,646,000
Surface Water Cost	=	$ 1,730,000
Total Water Cost	=	$ 3,376,000
Cost per Acre-Foot Applied	=	$ 2.45
Farm Cost Less Water Cost	=	$18,400,000
Total Cost	=	$21,776,000
Total Returns	=	$41,400,000
Net Returns	=	$19,624,000
Benefit-Cost Ratio	=	1.90:1

The allocation pattern, expressed in this case, permitted all of the available water supply to be consumed within the basin after equilibrium had been established. In the meantime, the negative return flow developed faster than the positive return flow as indicated in Table 5.

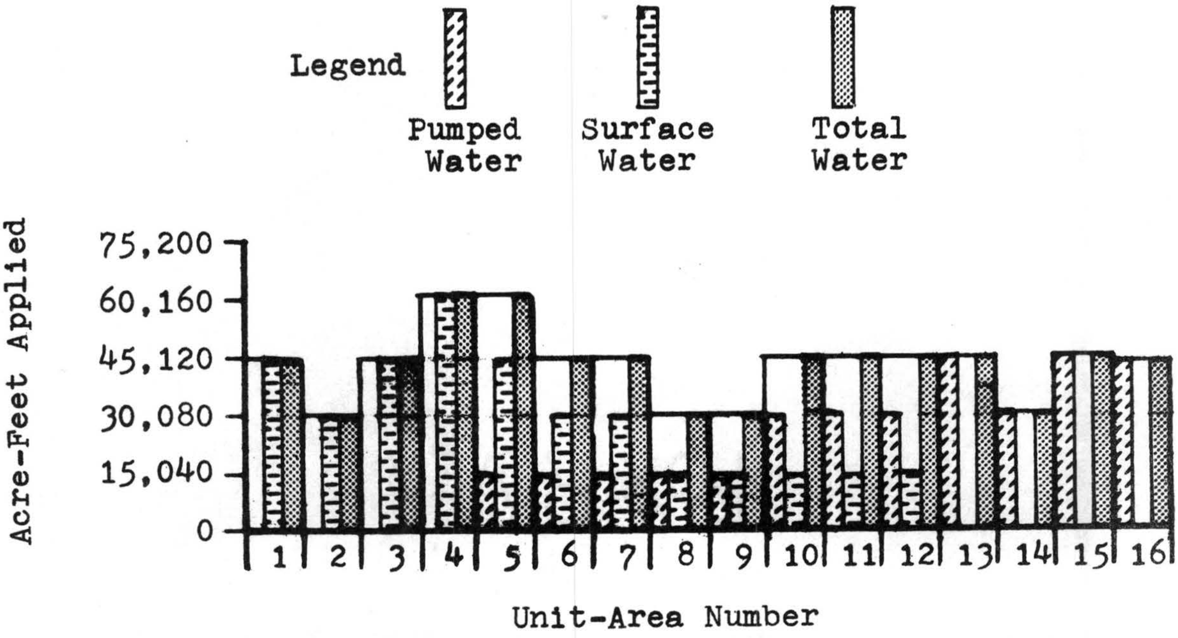


Figure 13. Allocation Pattern for Case No. 2

TABLE 5

ANNUAL RETURN FLOW FOR CASE NO. 2

Year	Positive Return Flow (ac-ft)	Negative Return Flow (ac-ft)	Net Return Flow (ac-ft)
3	228,000	254,000	-26,000
6	320,000	350,000	-30,000
9	362,000	392,000	-30,000
12	380,000	412,000	-32,000
15	396,000	428,000	-32,000

Case No. 3--This case was devised to determine the effect of an allocation pattern in which only surface water was used. The total available irrigation water supply was released to the canals of the upper reach while the lower diversions depend upon return flow from the upper diversions for their water supply. A diagram of this allocation pattern is given in Figure 14.

Annual results for the 15th year of operation follow.

Volume of Annual Inflow	=	$ 1,000,000 ac-ft
Volume of Surface Water Applied	=	1,226,000 ac-ft
Volume of Pumped Water Applied	=	0 ac-ft
Total Volume of Water Applied	=	1,226,000 ac-ft
Total Volume Consumed	=	796,000 ac-ft
Efficiency	=	79.6 percent
Irrigated Area	=	409,000 acres
Pumping Cost	=	$ 0
Surface Water Cost	=	$ 2,940,000
Total Cost	=	$ 2,940,000
Cost per Acre-Foot Applied	=	$ 2.40
Farm Cost Less Water Cost	=	$16,360,000
Total Cost	=	$19,300,000
Total Returns	=	$36,780,000
Net Returns	=	$17,480,000
Benefit-Cost Ratio	=	1.90:1

To operate this allocation pattern over the entire basin, equilibrium must be realized in the upper areas before the lower reaches can be completely irrigated. Because any

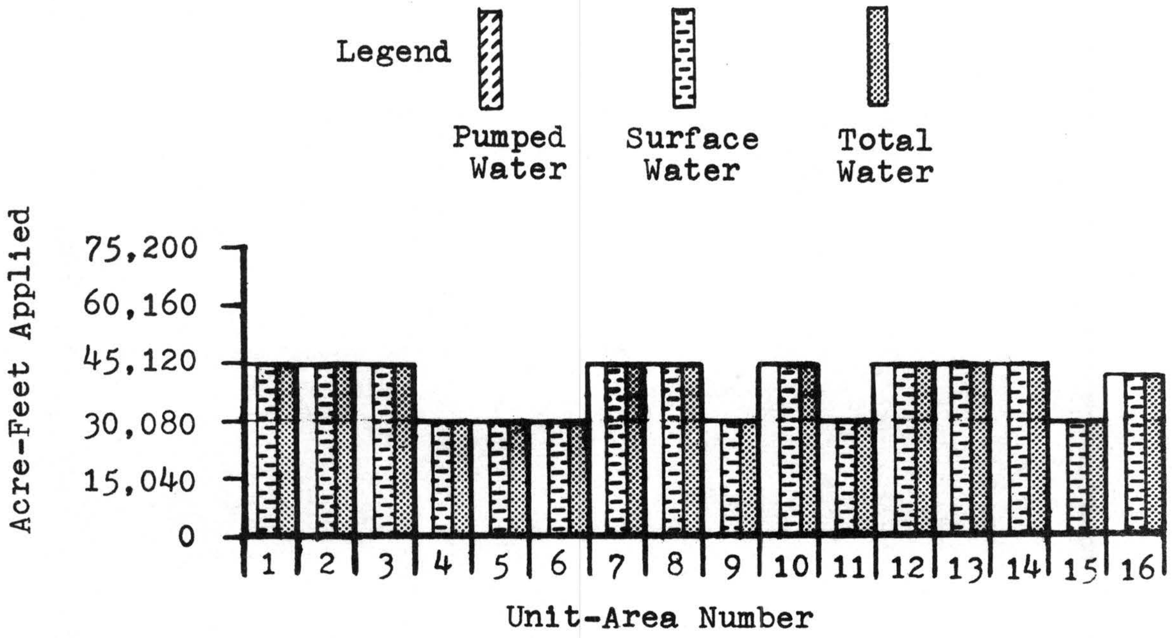


Figure 14. Allocation Pattern for Case No. 3

TABLE 6

ANNUAL RETURN FLOW FOR CASE NO. 3

Year	Positive Return Flow (ac-ft)	Negative Return Flow (ac-ft)	Net Return Flow (ac-ft)
3	390,000	0	390,000
6	546,000	0	546,000
9	614,000	0	614,000
12	648,000	0	648,000
15	674,000	0	674,000

of the return flow occurring in the upper reaches is needed in the lower reaches, no water is lost until the area below the last diversion is irrigated. After this last section has been irrigated for fifteen years, the lost return flow is 102,000 acre-feet per year.

Case No. 4--Case No. 4 was devised so that an evaluation could be made of the effects on a basin that used only ground water supplies. The application pattern is approximately the same as the previous case; however, pumping is used instead of surface water application. The plan required the pumps to divert their supply of water from the stream by developing a water-table gradient sloping away from the stream. Figure 15 graphically represents this allocation pattern.

Annual results for the 15th year of operation follow.

Volume of Annual Inflow	=	1,000,000 ac-ft
Volume of Surface Water Applied	=	0 ac-ft
Volume of Pumped Water Applied	=	1,380,000 ac-ft
Total Volume of Water Applied	=	1,380,000 ac-ft
Total Volume Consumed	=	898,000 ac-ft
Efficiency	=	89.8 percent
Irrigated Area	=	460,000 acres
Pumping Cost	=	$ 3,450,000
Surface Water Cost	=	$ 0
Total Water Cost	=	$ 3,450,000
Cost per Acre-Foot Applied	=	$ 2.50
Farm Cost Less Water Cost	=	$18,400,000

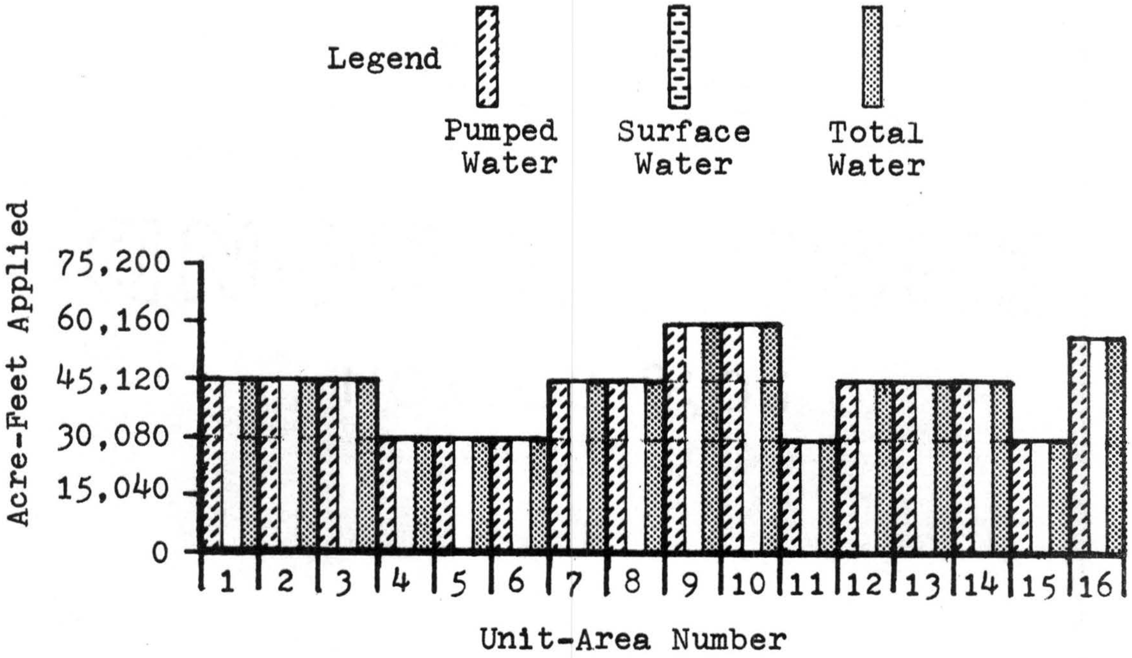


Figure 15. Allocation Pattern for Case No. 4

TABLE 7

ANNUAL RETURN FLOW FOR CASE NO. 4

Year	Positive Return Flow (ac-ft)	Negative Return Flow (ac-ft)	Net Return Flow (ac-ft)
3	0	-534,000	-534,000
6	0	-736,000	-736,000
9	0	-824,000	-824,000
12	0	-868,000	-868,000
15	0	-898,000	-898,000

Total Cost	=	$21,850,000
Total Returns	=	$41,400,000
Net Returns	=	$19,550,000
Benefit-Cost Ratio	=	1.89:1

An all pumping hypothetical basin required fifteen years to reach steady state. In the meantime, the water table gradients were developing. Before equilibrium was reached a considerable volume of water had left the basin. For any year, the volume lost can be determined by subtracting the return flow for that year from the steady-state return flow or the return flow of the fifteenth year since initiation of the irrigation practice.

Case No. 5--Case No. 5 is an allocation pattern using a combination of ground water and surface water application throughout the reach. The application pattern was approximately the same as the two previous allocation patterns. The source of water for the wells is derived from the stream and/or the deep percolation of surface water and canal seepage. Figure 16 represents the allocation pattern in a graphical manner.

Annual results for the 15th year of operation follow.

Volume of Annual Inflow	=	1,000,000 ac-ft
Volume of Surface Water Applied	=	706,000 ac-ft
Volume of Pumped Water Applied	=	674,000 ac-ft
Total Volume of Water Applied	=	1,380,000 ac-ft
Total Volume Consumed	=	898,000 ac-ft
Efficiency	=	89.8 percent

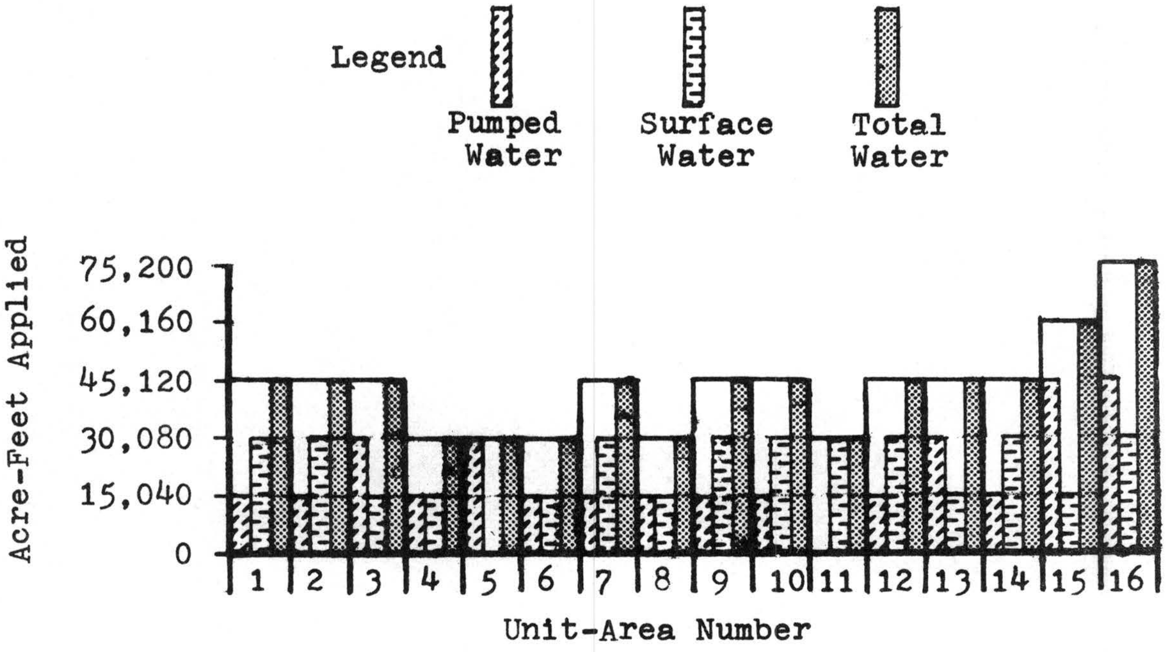


Figure 16. Allocation Pattern for Case No. 5

TABLE 8

ANNUAL RETURN FLOW FOR CASE NO. 5

Year	Positive Return Flow (ac-ft)	Negative Return Flow (ac-ft)	Net Return Flow (ac-ft)
3	264,000	260,000	+ 4,000
6	370,000	358,000	+ 6,000
9	416,000	400,000	+ 8,000
12	438,000	422,000	+ 8,000
15	456,000	436,000	+ 10,000

Irrigated Area	=	460,000 acres
Pumping Cost	=	$ 1,348,000
Surface Water Cost	=	$ 1,832,000
Total Water Cost	=	$ 3,180,000
Farm Cost Less Water Cost	=	$18,400,000
Total Cost	=	$21,580,000
Cost per Acre-Foot Applied	=	$ 2.30
Total Returns	=	$41,400,000
Net Returns	=	$19,820,000
Benefit-Cost Ratio	=	1.92:1

The important characteristic of this allocation pattern was that the pumps consume approximately all the deep percolation from the surface water applications and the canal seepage. The net return flow was positive, but it was only a small amount. The phreatophytes consumed this water as well as some water that was discharged to the stream from the annual 1,000,000 acre-foot supply. This allocation pattern consumed all of the available irrigation water supply within the basin.

Case No. 6--Case No. 6 has an allocation pattern that uses a combination of the two distribution methods. The application pattern restricted the water use to the upper eleven unit-areas of the hypothetical basin (i.e., a larger percentage of the acreage in the upper areas was irrigated). The purpose of this plan was to reduce the total consumptive use of the phreatophytes. A diagram illustrating this case is given in Figure 17.

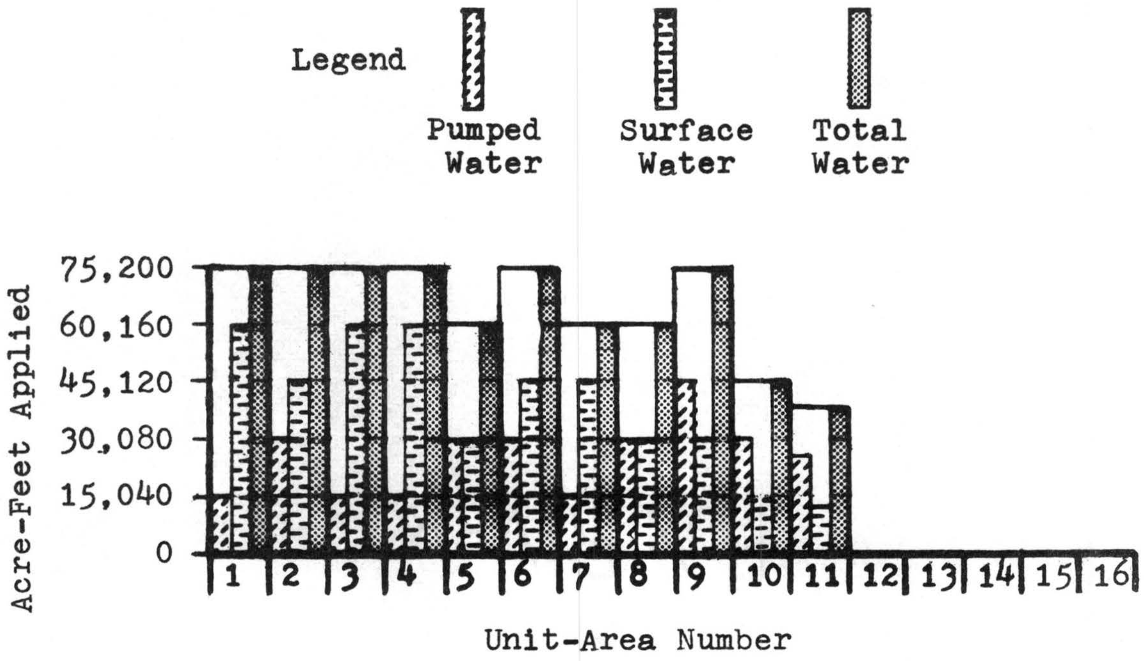


Figure 17. Allocation Pattern for Case No. 6

TABLE 9

ANNUAL RETURN FLOW FOR CASE NO. 6

Year	Positive Return Flow (ac-ft)	Negative Return Flow (ac-ft)	Net Return Flow (ac-ft)
3	252,000	216,000	+36,000
6	354,000	298,000	+56,000
9	398,000	334,000	+64,000
12	418,000	350,000	+68,000
15	436,000	364,000	+72,000

Annual results for the 15th year of operation follow.

Volume of Inflow	=	1,000,000 ac-ft
Volume of Surface Water Applied	=	868,000 ac-ft
Volume of pumped Water Applied	=	560,000 ac-ft
Total Volume of Water Applied	=	1,428,000 ac-ft
Total Volume Consumed	=	928,000 ac-ft
Efficiency	=	92.8 percent
Irrigated Area	=	476,000 acres
Pumping Cost	=	$ 1,400,000
Surface Water Cost	=	$ 2,000,000
Total Water Cost	=	$ 3,400,000
Cost per Acre-Foot Applied	=	$ 2.38
Farm Cost Less Water Cost	=	$19,040,000
Total Cost	=	$22,440,000
Total Returns	=	$42,800,000
Net Returns	=	$20,360,000
Benefit-Cost Ratio	=	1.91:1

In this case, all of the available irrigation water is diverted to the canals at the head of the basin, then it is distributed only to the farms within the first 110 miles. Within the same area, the irrigation wells are used to capture some of the deep percolated water and the return flow. The remainder of this water was consumed by the phreatophytes. The positive net return flow, as indicated in Table 9, was completely used by the phreatophytes. Every year, all of the available water was consumed before it had a chance to leave the basin. The water table was relatively stable because the pumping nearly balances the deep percolation from the surface water applications and the canal seepage.

Comparison of Allocation Patterns

Generally, the comparison of different projects is best described by the benefit-cost ratio, which is determined by dividing total benefit by total cost. The net return of the different projects is also used for comparison, but this method is usually less satisfactory because it favors large projects. However, for the comparison of alternative projects in the same field and the same size, the net return method compares the projects on an unbiased basis. If the benefit-cost ratio is used in the selection process, the region may receive something less than the maximum net return from the fixed supply of water resources. Both methods are used in this study. For simplicity, the analysis is based upon a constant return for each acre-foot of water applied and a constant cost in obtaining the water. This assumption is far from the actual conditions; however, the constant depth of application in a hypothetical case permits the restricting assumption.

Table 10 presents a comparison of the selected allocation patterns. From the standpoint of economics and efficiency, Table 10 indicates that the most favorable allocation pattern is Case No. 6. The next most favorable patterns are Case No. 2, Case No. 4 and Case No. 5. These allocation

TABLE 10[1]

COMPARISON OF CASES PRESENTED

	Case No. 1	Case No. 2	Case No. 3	Case No. 4	Case No. 5	Case No. 6
Total Volume Applied (millions) ac-ft	1.228	1.380	1.226	1.380	1.380	1.428
Efficiency (percent)	79.8	89.8	79.8	89.8	89.8	92.8
Total Return (millions) $	36.80	41.40	39.78	41.40	41.40	42.80
Total Cost (millions) $	19.42	21.78	19.30	21.85	21.58	22.44
Net Return (millions) $	17.38	19.62	17.48	19.55	19.82	20.36
Benefit-Cost Ratio	1.89:1	1.90:1	1.90:1	1.89:1	1.92:1	1.91:1

[1]The values in this table are made by using some very restricting assumptions. The most important ones are the constant cost and benefits for a given quantity of water. If the costs and benefits had been different the results also may have been quite different. Therefore, when field conditions are being studied the best possible economic data are necessary to prevent unrealistic results.

patterns produce approximately the same results from the standpoint of economics and efficiency. The least favorable allocation patterns are Case No. 1 and Case No. 3 because considerable amounts of return flow leave the basin. In all of the other cases, the entire return flow was consumed within the basin. The superiority of Case No. 6 is due to the least amount of water lost to phreatophytes. If the results tabulated in Table 10 are studied closely, some of the results may indicate that there isn't a significant difference in the several cases. However, caution is necessary before drawing definite conclusions. The reason is the assumption and simplifications used in the economic analysis. This approach prevented many variables from influencing the results as they properly should. If a serious economic analysis is undertaken the variables such as sunk cost, opportunity cost, management effects, operation capacity, etc. must be evaluated. If the data are not too detailed the error may be accounted for by limiting the significant digits of the results.

In summary, three methods for calculating return flow were presented; then a technique was devised to physically evaluate an allocation pattern. Finally several allocation patterns were evaluated and compared on the basis of water-use efficiency and economic soundness.

Applying the Modeling Technique to Field Conditions

The first step in making a water use efficiency or an economics study on a river basin is to define the purpose and scope of the project. With the purpose and financial limitations in mind, a selection of a method for calculating return flow should be made. If the project has a limited purpose, limited finances, or a preliminary investigation, the steady infiltration method and the stream depletion method are two suggested methods for calculating return flow. These two methods provide reliable results with a minimum effort. However, the results become less reliable as the necessary assumptions, outlined in Chapter III, are violated. If the project's purposes require a detailed analysis, the mathematical model provides an accurate simulation of the basin and the most reliable results.

Because of the tremendous complexity of the geometry of most river basins, there are many physical irregularities that can affect the calculations and water management plans. The assumption of a level bedrock and an initially level water table is often violated. This assumption permits the wells to derive water from the stream as well as to intercept water flowing toward the stream. However, if the bedrock and water table slopes toward the stream only the wells near the stream derive water from it while the wells farther from the stream intercept water flowing toward the stream and thus also reduce the stream's flow.

Another common complexity is the heterogeneous properties of the basin and its irregular shape. For the steady infiltration method and the stream depletion method, the problem can be reduced by dividing the basin into unit-areas of unequal sizes and properties. The mathematical model accounts for the problem by dividing the basin into grids that may have varying properties.

Another problem would be the natural and artificial inflows and outflows. These inflows and outflows could be taken into account by the stream balance calculations if they occur in the stream or as additions and withdrawals from the aquifer if they occur in the general farm area.

A problem or a potential problem in many farming areas is salinity control of the irrigation water. The problem may develop if only return flow is used for irrigation water. Possibly, a practice of "mixing" water of low salt content with water of a high salt content could be used to maintain a tolerable level of water quality.

As stated previously, the economic analysis used in the hypothetical basin is extremely simplified with assumptions. The assumptions of a basin-wide constant costs and benefits are never found in a field case. The variations are significant and should be taken into account. As a result, some of the variables are listed. A few of the main cost variables for a smoothly operating management plan include (1) the management efficiency of each farm, (2) the values of the crops grown, (3) the level of water use, (4) fertility of the land, (5) operating costs, (6) sunk costs, and (7) capital costs. However if a decision is made to shift from one management plan to another, there are many other costs that must be considered. Some of these costs include (1) an opportunity cost, (2) a development cost, (3) a social cost, and (4) a capital cost. On the benefit side of the problem, benefits must be determined for the present plan and also for the proposed plan. Direct benefits are defined to be the value of the products and services that result from a plan. In some cases the indirect benefits are also determined.

Another caution to the planner would be to always give special consideration to the preferences of the people affected by the management plans. This is necessary because an optimum basin-wide plan will rarely be the best plan in view of the majority of the farmers. In this case the plan should be adjusted to satisfy most of the people affected by the plan.

Finally, the related social and legal institutions must also be evaluated to prevent unnecessary hardships and difficulties. These factors must be taken into account in the final selection of a management plan as well as in the selection of several overall plans to be evaluated.

Chapter V

CONCLUSIONS AND RECOMMENDATIONS

All three methods that were presented to predict return flow can be applied to a unit-area model; however, the reliability and operational difficulties of application vary considerably. The mathematical model is by far the most difficult to solve, but much detail (i.e., the bedrock slope, initial water table slope, aquifer properties, irrigation patterns, and varying rates of water application) can be simulated which in turn produces quite reliable results.

The stream-depletion equation requires assumptions of a homogeneous aquifer, a level bedrock, an initially level water table and a steady application rate. The water is applied in strips that are parallel to the stream. The result is a solution that is easier to solve but less reliable or less directly applicable to actual problems than the mathematical model.

The steady-infiltration method has the same assumptions as the previous method except that the irrigation water is assumed to be applied uniformly over the entire unit-area. This additional idealization reduces the difficulty of calculation and the reliability of results below those of the stream-depletion equation for general analysis. It provides some confidence and a reasonably simple solution. However, the purpose and scope of the project should determine the method.

In the development of an allocation pattern, the basin is divided into unit-areas that are assumed to represent the detail of the basin. The unit-areas can be varied in size and properties to help simulate a field case. The purpose of this simulation is to provide a method for developing and evaluating an allocation pattern. The method or technique is presented in Chapter II. Such a technique provides a flexible tool in studying the reactions of a wide range of allocation patterns. After an allocation pattern has been developed, the data resulting from the development process can be used in the economic and water-use efficiency calculations. The result of this technique can be laborious; however, it can be useful for manual calculations or possibly adapted to a computer solution.

The first recommendation that is suggested is a comparison, under field conditions, of the three methods used to predict return flow. A range of field conditions would be desirable so that the performance of each method could be determined for the variety of conditions. The second recommendation suggested is a study to adapt a varying rate of deep percolation to the stream-depletion and steady infiltration methods. Possibly, the theory of superposition could be applied.

Prior to making a major management decision, the author recommends that several allocation patterns be analyzed for the real basin. This most likely would require devising several unit-area models of varying size and properties. Much better economic data is necessary for any serious comparison of the allocation patterns. Also, the results should be determined at the end of each year instead of waiting for steady-state conditions to exist. The comparisons must include the physical, legal, and social limitations. Finally, a publication by the White House Committee (36) is suggested as a reference for a study of this nature. It gives a very complete, yet general, approach to the evaluation of a regional development project. Other reports that may be of some benefit have been published by the Ford Foundation and the White House--Interior Panel.

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APPENDIX I

COMPUTER PROGRAM

FORTRAN IV PROGRAM TO CALCULATE RETURN FLOWS AND WATER TABLE ELEVATIONS

```
      DIMENSION H(200), Z(200), HT(200), CR(200), CMATRX(130,20),
     1DELX(200), DELY(200), DELQ(200),CDELQ(10), YAW(200), YPT(200),
     2YPW(200), YCW(200), FMAW(180), FMPT(180), FMCW(180), FMPW(180),
     3FK(200)
      READ (5,1) I, J,JI, JL
    1 FORMAT (4I5)
      ICH = 1
      IJ = I*J
      DO 2 K = 1, IJ
    2 READ (5,3) DELY(K), DELX(K), YAW(K), YPT(K), YPW(K), YCW(K), FK(K)
    3 FORMAT (7F8.1)
    6 FORMAT (4F7.4)
    4 DO 99 K =1, 12
    5 READ(5,6) FMAW(K), FMPT(K), FMPW(K), FMCW(K)
      DO 99 LL = 1, 15
      L = 12*LL + K
      FMAW(L) = FMAW(K)
      FMPT(L) = FMPT(K)
      FMPW(L) = FMPW(K)
   99 FMCW(L) = FMCW(K)
    7 READ (5,8) (Z(K), K = 1,IJ)
    8 FORMAT (10F8.1)
      READ (5,8) (H(K), K = 1, IJ)
   50 ICT = ICH
    9 READ (5,10) CAW, CPT, CPW, CCW, PHI, ICH
   10 FORMAT (5F7.3,I3)
   11 GO TO (12, 27, 30, 35)ICT
   12 READ (5,13) T, DELT, DTCOEF, DTCOE2
   13 FORMAT (4F6.1)
   14 WRITE (6,15) T, DELT
   15 FORMAT (1H1, 12HTOTAL TIME = F6.1, 36H DAYS    CALCULATING TIME IN
     1TERVAL = F6.1, 5H DAYS)
      WRITE (6,16) I, J, JI, JL
   16 FORMAT (1H-, 7HWIDTH = I3, 26H GRIDS,           LENGTH = I3,
     129H GRIDS,            RIVER REACH I3, 3H TO I3, 6H BLOCK)
      WRITE (6,17)
   17 FORMAT (1H1, 37X, 58HSURFACE WATER APPLICATION MAP    (AC-FT PER Y
     1EAR PER GRID))
      WRITE (6,18)
   18 FORMAT (1H0,7X,117H    K+9        K+8        K+7        K 6        K+5
     1         K+4        K+3        K+2        K+1          K
     2    K)
      DO 19 K = 1, IJ, I
   19 WRITE (6,20) YAW(K+9), YAW(K+8), YAW(K+7), YAW(K+6), YAW(K+5),
     1YAW(K+4), YAW(K+3), YAW(K+2), YAW(K+1), YAW(K), K
   20 FORMAT (1H0, 7X, 10F10.1, 15X, I3)
      WRITE (6,21)
   21 FORMAT (1H1, 57X, 36HPUMPING    (AC-FT PER YEAR PER GRID))
```

```
      WRITE (6,18)
      DO 22 K = 1, IJ, I
   22 WRITE (6,20) YPW(K+9), YPW(K+8), YPW(K+7), YPW(K+6), YPW(K+5), YPW
     1(K+4), YPW(K+3), YPW(K+2), YPW(K+1), YPW(K), K
      WRITE (6,23)
   23 FORMAT (1H1, 48X, 35HDELTA-Y MAP, SPACING IN Y-DIRECTION)
      WRITE (6,18)
      DO 24 K = 1, IJ, I
   24 WRITE (6,20) DELY(K+9), DELY(K+8), DELY(K+7), DELY(K+6), DELY(K+5)
     1, DELY(K+4), DELY(K+3), DELY(K+2), DELY(K+1), DELY(K), K
      WRITE (6,25)
   25 FORMAT (1H1, 48X, 35HDELTA-X MAP, SPACING IN X-DIRECTION)
      WRITE (6,18)
      DO 26 K = 1, IJ, I
   26 WRITE (6,20) DELX(K+9), DELX(K+8), DELX(K+7), DELX(K+6), DELX(K+5)
     1, DELX(K+4), DELX(K+3), DELX(K+2), DELX(K+1), DELX(K), K
   27 WRITE (6,51)
   51 FORMAT (1H1, 45X, 21HMONTHLY DISTRIBUTIONS)
      WRITE (6,52)
   52 FORMAT (1H-, 25X, 59HMONTH            PPT          APP.W.        PUM.W
     1.        CNL.W.)
      DO 28 K = 1, 12
   28 WRITE (6,29) K, FMPT(K), FMAW(K), FMPW(K), FMCW(K)
   29 FORMAT (1H0, 26X, I2, 2X, 4F13.4)
   30 WRITE (6,31)
   31 FORMAT (1H1,44X,45HINITIAL WATER TABLE ELEVATIONS ARE AS FOLLOWS)
      WRITE (6,18)
      DO 32 K = 1, IJ, I
   32 WRITE (6,20) H(K+9), H(K+8), H(K+7), H(K+6), H(K+5), H(K+4), H(K+3
     1), H(K+2), H(K+1), H(K), K
      WRITE (6,33)
   33 FORMAT (1H1, 55X, 21HBEDROCK ELEVATION MAP)
      WRITE (6,18)
      DO 34 K = 1, IJ, I
   34 WRITE (6,20) Z(K+9), Z(K+8), Z(K+7), Z(K+6), Z(K+5), Z(K+4), Z
     1(K+3), Z(K+2), Z(K+1), Z(K), K
   35 WRITE (6,36) CPT, CCW
   36 FORMAT (1H1, 20X, 13HCOEF OF PPT =F6.3, 15X,21HCOEF OF CANAL WATER
     1 =F6.3)
      WRITE (6,37) CAW, CPW
   37 FORMAT (1H0, 20X, 23HCOEF OF APPLIED WATER = F6.3, 5X,
     122HCOEF OF PUMPED WATER = F6.3)
      WRITE (6,38)
   38 FORMAT (1H-, 20X, 42HHYDRAULIC CONDUCTIVITY = 0.0060 FT PER SEC)
      WRITE (6,39)
   39 FORMAT (1H0, 20X, 22HSPECIFIC YIELD = 0.170)
  100 CNT = 0.0
      PCNT = 1.0
      PCNT2 = 1.0
      DO 103 K = JI, JL
  103 CDELQ(K) = 0.0
      CSDELQ = 0.0
      DO 101 K = 1, IJ
  101 HT(K) = H(K)
      GO TO 500
  200 CNT = 1.0
```

```
      IM1 = I - 1
      IM2 = I - 2
      JM2 = J - 2
      LLIM = 2*IM2 + 1
      KLIM = IJ -2*I - 2*JM2
  201 DO 202 L=1, LLIM
      DO 202 K=1, KLIM
  202 CMATRX(K,L) = 0.0
      DO 304 L=1, KLIM
      CR(L) = 0.0
      LCOMP = (I-3+L)/IM2
      LA = L + 2*LCOMP - 1
      LI1 = LA + I
      IF (H(LI1)) 304, 203, 203
  203 LB = LI1 - 1
      LC = LI1 + 1
      LD = LI1 + I
      CMATRX(L,1) = (FK(LA)*(ABS (HT(LA)) - Z(LA)) + FK(LI1)*(HT(LI1) -
     1 Z(LI1)))/(DELX(LI1)*(DELX(LI1) + DELX(LA)))
      CMATRX(L,IM2) = (FK(LB)*(ABS (HT(LB)) - Z(LB)) + FK(LI1)*(HT(LI1)
     1 - Z(LI1)))/(DELY(LI1)*(DELY(LI1) + DELY(LB)))
      CMATRX(L,I) = (FK(LC)*(ABS (HT(LC)) - Z(LC)) + FK(LI1)*(HT(LI1)
     1 - Z(LI1)))/(DELY(LI1)*(DELY(LI1) + DELY(LC)))
      CMATRX(L,LLIM) = (FK(LD)*(ABS (HT(LD)) - Z(LD)) + FK(LI1)*(HT(LI1)
     1 - Z(LI1)))/(DELX(LI1)*(DELX(LI1)  + DELX(LD)))
      IF (HT(LI1) - Z(LI1) -1.0) 204, 204, 210
  204 IF (HT(LI1)-ABS (HT(LA))) 251, 251, 250
  250 CMATRX(L,1) = 0.0
  251 IF (HT(LI1)-ABS (HT(LB))) 206, 206, 205
  205 CMATRX(L,IM2) = 0.0
  206 IF (HT(LI1)-ABS (HT(LC))) 208, 208, 207
  207 CMATRX(L,I) = 0.0
  208 IF (HT(LI1)-ABS (HT(LD))) 210, 210, 209
  209 CMATRX(L,LLIM) = 0.0
  210 IF (H(LA)) 211, 214, 214
  211 IF (H(LA)+9.0) 212, 213, 212
  212 CR(L) = CMATRX(L,1)*HT(LA)
      CMATRX(L,IM1) = -CMATRX(L,1)
  213 CMATRX(L,1) = 0.0
  214 IF (H(LB)) 215, 218, 218
  215 IF (H(LB)+9.0) 216, 217, 216
  216 CR(L) = CR(L) + CMATRX(L,IM2)*HT(LB)
      CMATRX(L,IM1) = CMATRX(L,IM1)  - CMATRX(L,IM2)
  217 CMATRX(L,IM2) = 0.0
  218 IF (H(LC)) 219, 222, 222
  219 IF (H(LC)+9.0) 220, 221, 220
  220 CR(L) = CR(L) + CMATRX(L,I)*HT(LC)
      CMATRX(L,IM1) = CMATRX(L,IM1) - CMATRX(L,I)
  221 CMATRX(L,I) = 0.0
  222 IF (H(LD)) 223, 226, 226
  223 IF (H(LD)+9.0) 224, 225, 224
  224 CR(L) = CR(L) + CMATRX(L,LLIM)*HT(LD)
      CMATRX(L,IM1) = CMATRX(L,IM1) - CMATRX(L,LLIM)
  225 CMATRX(L,LLIM) = 0.0
  226 CMATRX(L,IM1) = CMATRX(L,IM1) -(CMATRX(L,1) + CMATRX(L,IM2) +
     1 CMATRX(L,I) + CMATRX(L,LLIM) + (PHI/DELT))
```

```
300 QT = 0.0
301 NT = 1.0 + (CNT-1.0)*DELT/30.0
302 QT = (-CPT*FMPT(NT)*YPT(LI1) - CAW*FMAW(NT)*YAW(LI1) -  CCW*FMCW(N
   1T)*YCW(LI1) + CPW*FMPW(NT)*YPW(LI1))/30.0
303 CR(L) = CR(L) - HT(LI1)*PHI/DELT + (43560.0*QT)/(DELX(LI1)*
   1DELY(LI1))
304 CONTINUE
400 CALL BSOLVE(CMATRX, KLIM, LLIM, CR)
401 DO 407 L = 1, KLIM
    LCOMP = (I-3+L) / IM2
    LI1 = L + 2*LCOMP - 1 + I
    IF (CR(L)) 403, 402, 403
402 HT(LI1) = H(LI1)
    GO TO 404
403 HT(LI1) = CR(L)
404 IF (ABS(HT(LI1))-Z(LI1)) 405, 407, 407
405 IF (HT(LI1)) 407, 406, 406
406 HT(LI1) = Z(LI1)
407 CONTINUE
500 DO 501 K = JI, JL
501 DELQ(K) = 0.0
    SDELQ = 0.0
502 DO 503 K = JI, JL
    KA = K*I - 1
    KRI = K*I
    DELQ(K) = (FK(KA)*(HT(KA) - Z(KA))*DELY(KA) + FK(KRI)*(ABS (H(KRI)
   1) - Z(KRI))*DELY(KRI))*DELX(KA)*(HT(KA) -ABS (H(KRI)))*2.0*DELT/
   2((DELY(KA) + DELY(KRI))**2*43560.0)
    CDELQ(K) = CDELQ(K) + DELQ(K)
503 SDELQ = SDELQ + DELQ(K)
    CSDELQ = CSDELQ + SDELQ
504 TIME = CNT*DELT
    WRITE (6,700) TIME, SDELQ
700 FORMAT (1H0, 41HTHE RETURN FLOW FOR A 10 DAY PERIOD AFTER F8.1,3X,
   17HDAYS IS F10.2, 5X, 5HAC-FT)
    PTIME = PCNT * DTCOEF
    IF (TIME - PTIME) 600, 750, 750
750 PCNT = PCNT + 1.0
    WRITE (6,701) PTIME, CSDELQ
701 FORMAT (1H0, 5X, 32HTHE ACCUMULATIVE RETURN FLOW FOR F8.1, 5X,
   17HDAYS IS F15.2, 5X, 5HAC-FT)
    PWT = PCNT2 * DTCOE2
    IF (TIME - PWT) 600, 702, 702
702 PCNT2 = PCNT2 + 1.0
    WRITE (6,703) TIME
703 FORMAT (1H0, 29HTHE WATER TABLE ELEVATIONS AT F8.1, 5X, 4HDAYS)
    WRITE (6,18)
    DO 704 K = 1, IJ, I
704 WRITE (6,20) HT(K+9), HT(K+8), HT(K+7), HT(K+6), HT(K+5), HT(K+4),
   1HT(K+3), HT(K+2), HT(K+1), HT(K), K
600 IF (CNT-1.0) 200, 601, 601
601 CNT = CNT + 1.0
    IF (CNT*DELT-T) 201, 201, 602
602 CONTINUE
    GO TO (50, 4, 7, 50) ICH
    END
```

```
      SUBROUTINE BSOLVE(C,N,M,V)
      DIMENSION C(130,20), V(130)
      LR = (M-1) /2
      DO 114 L = 1, LR
      IM = LR - L + 1
      DO 114 I = 1, IM
      DO 110 J = 2, M
  110 C(L,J-1) = C(L,J)
      KN = N - L
      KM = M - I
      C(L,M) = 0.0
  114 C(KN+1,KM+1) = 0.
      LR = LR + 1
      IM = N - 1
      DO 231 I = 1, IM
      NPIV = I
      LS = I + 1
      DO 211 L = LS, LR
      IF (ABS(C(L,1))-ABS(C(NPIV,1))) 211, 211, 210
  210 NPIV = L
  211 CONTINUE
      IF (NPIV - I) 220, 220, 213
  213 DO 216 J = 1, M
      T = C(I,J)
      C(I,J) = C(NPIV,J)
  216 C(NPIV,J) = T
      T = V(I)
      V(I) = V(NPIV)
      V(NPIV) = T
  220 V(I) = V(I) / C(I,1)
      DO 222 J = 2, M
  222 C(I,J) = C(I,J) / C(I,1)
      DO 228 L = LS, LR
      T = C(L,1)
      V(L) = V(L) - T*V(I)
      DO 227 J = 2, M
  227 C(L,J-1) = C(L,J) - T*C(I,J)
  228 C(L,M) = 0.
      IF (LR - N) 230, 231, 231
  230 LR = LR + 1
  231 CONTINUE
      V(N) = V(N) / C(N,1)
      JM = 2
      DO 311 I = 1, IM
      L = N - I
      DO 308 J = 2, JM
      KM = L + J
  308 V(L) = V(L) - C(L,J)*V(KM-1)
      IF (JM - M) 310, 311, 311
  310 JM = JM + 1
  311 CONTINUE
      RETURN
      END
```

SYMBOLS USED IN MATHEMATICAL MODEL

Symbol	Description
FK	Hydraulic Conductivity of Aquifer
H	Initial Water Table Elevation
HT	Working H , which is a Function of Time
PHI	Storage Coefficient
DELX	Length Dimension of Grid (X - Direction)
DELY	Width Dimension of Grid (Y - Direction)
I	Number of Grids in Model Width
IJ	Number of Grids in Model
J	Number of Grids in Model Length
JI	J at which River Gain Computations are to Begin
JL	J at which River Gain Computations are to End
CDELQ	Accumulation of DELQ for Individual River Grids for the Particular DELT
CSDELQ	Accumulation of SDELQ from T = 0.0
DELQ	River Gain from each Adjacent Grid during the Particular DELT
SDELQ	Sum of DELQ for the River Reach under Consideration for the Particular DELT
QT	Net Water Additions to Grid for Particular DELT
DELT	Time Increment for Computation Purposes

SYMBOLS USED IN MATHEMATICAL MODEL (continued)

Symbol	Description
DTCOEF	Time Increment for Print-out of CSDELQ
DTCOE2	Time Increment for Print-out of HT
T	Total Time of Run
CAW	Fraction of Applied Surface Water reaching Water Table
CCW	Fraction of Canal Seepage reaching Water Table
CPT	Fraction of Precipitation reaching Water Table
CPW	Fraction of Pumped Water reaching Water Table
FMAW	Fraction of Annual Applied Surface Water that is Applied in each Month
FMCW	Fraction of Annual Canal Seepage that is Applied each Month
FMPT	Fraction of Annual Precipitation that is Applied in each Month
FMPW	Fraction of Annual Pumped Water that is Applied in each Month
YAW	Annual Application of Surface Water to each Grid
YCW	Annual Loss of Canal Water to each Grid
YPT	Annual Precipitation to each Grid
YPW	Annual Application of Pumped Water to each Grid

SYMBOLS USED IN MATHEMATICAL MODEL (continued)

Symbol		Description
CNT		Counter for the Particular DELT at which Computations are being made
NT		Selector of proper Monthly Data for Computations
PCNT		Counter for Print-out of CSDELQ
PCNT2		Counter for Print-out of HT
TIME	=	CNT * DELT
PTIME	=	PCNT * DTCOEF
PWT	=	PCNT2 * DTCOE2
ICH		Code Numbers for Multiple Runs
ICT		Code Number for Multiple Runs
CMATRX		Coefficient of Matrix
CR		Right-Hand Side of CMATRX Vector

NOTE: IM1, IM2, JM2, KLIM, LLIM, LA, LB, LC, LD, LCOMP -- Grid Identification for Computation Purposes