DEVELOPMENT OF A DIGITAL GROUNDWATER MODEL WITH APPLICATION TO AQUIFERS IN IDAHO

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ABSTRACT

Investigations of water resources systems involving groundwater simulation are generally data-scarce projects. Insufficient geohydrologic data severely inhibits simulation of historic water table behavior.

A finite difference model is expanded to include iterative procedures to adjust the initial estimates of geohydrological parameters (transmissibility, leakance factor, head difference in leaky aquifer system and storage coefficient) to match historic aquifer behavior. The model includes a mass balance calculation routine, a routine to simulate open surface drains and a routine to calculate flow across hydraulically connected boundaries, such as a lake or stream.

The parameter calibration routine as well as all other routines were tested and successfully applied to the gravel aquifer of the Snake River fan in eastern Idaho and the basalt aquifer of the Snake River plain in southern Idaho. The operational model successfully simulated the historic water table behavior in the aquifers and provides a reliable tool for studying effects of water management changes on aquifer behavior for research, planning or administrative purposes. The dissertation contains 52 references.

BIOGRAPHICAL SKETCH OF THE AUTHOR

The author was born in Beverwijk, The Netherlands, on February 3, 1947. He attended the Pius X College for five years. In September, 1964, he entered the University of Technology of Delft majoring in Civil Engineering from where he received his Candidate's degree in February, 1970. In the summer of 1970, he came to the United States to do graduate studies at the University of Idaho. He received his M.S. Degree in Civil Engineering in May, 1972. Since then he has been a Ph.D. student and research associate for the Water Resources Research Institute of the University of Idaho.

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CHAPTER I

INTRODUCTION

General

Because of the increasing water resource development in Idaho as well as in other semi-arid states there is a need for techniques to study and solve regional water management problems. In May 1970, a research study was begun in the Rigby-Ririe area of the Upper Snake River Basin in Idaho to develop alternative solutions to groundwater problems being experienced in that area.

Early in the study it was evident that to evaluate the response of groundwater systems to time and spatially variant inputs such as irrigation a general simulation model was required. The availability of large digital computers and new finite difference techniques for solving the flow equations made a digital model most feasible. The mathematical model developed is a finite difference digital model and, like models of Bredehoeft and Pinder (1970), it is based on the alternating direction implicit method as introduced by Peaceman and Rachford et al. (1953) and calculates hydraulic head values on a grid point basis.

With the model an attempt was made to simulate the historical seasonal water table changes as they occurred in the Rigby-Ririe area. Simulation of historical water table behavior may be achieved if extensive amounts of geohydrological data of the area are available, however water resources systems involving groundwater simulation are as a rule data-scarce projects. Simulation of the historical water table changes of the Rigby-Ririe area was attempted by adjusting the largely unknown geohydrological parameters on a trial and error basis. A reasonable simulation was impossible with this method and as with any other general model (digital and analog) an automatic calculation procedure enabling the calibration of a model to an area despite minimal geological data was required.

Investigation of two aquifers in Idaho for which geohydrological data are limited is outlined in this dissertation. The Snake River Fan aquifer near the cities of Rigby and Ririe was investigated to arrive at solutions to alleviate the high water table problem in the shallow gravel aquifer. The Snake Plain aquifer, a large regional water table aquifer extending over some 9,000 square miles, is being modeled with the objective to more closely determine the aquifer characteristics and develop an operational simulation model. Investigations of this type are essential to optimum future planning of agricultural, domestic and industrial groundwater resource development on the plain.

Objectives

Problems of high groundwater tables and drainage exist in many areas of Idaho as well as other areas of the northwest. There is an increased need to develop techniques for planning and solving groundwater management problems. Mathematical models facilitate aquifer response predictions and are applicable to many aquifers in Idaho. The prime objective of this research was to develop a generalized digital groundwater model which can be applied to aquifers in Idaho.

To reach this objective research efforts were directed towards the following areas:

1. Development of a calibration program used conjunctively with an existing basic model which adjusts aquifer parameter values and in turn will provide a correct simulation of aquifer response to measured historical inputs.
2. Evaluation of the model on actual field situations. The model was applied to the Snake River Fan in eastern Idaho to investigate the validity of the calibration routine and to simulate the historical seasonal water table changes. Ultimately management decisions were proposed that, in effect, will alleviate the water table problems as they occur in the Rigby-Ririe area.
3. Updating and refining the model, the calibration routine and to include modeling techniques for simulating specific management procedures.
4. Application of the updated model to the regional groundwater table of the Snake Plain aquifer.

CHAPTER II

LITERATURE REVIEW

Pertinent articles as they apply to the development of the theory of groundwater motion and the study of groundwater basins include publications by M. King Hubbert (1940), Jacob (1950), Todd (1959), De Wiest (1964), J. Toth (1962, 1963), Tyson and Weber (1964), Weber, Peters and Frankel (1968), Vemuri and Dracup (1967), Vemuri and Karplus (1969), W. C. Walton (1962), W. C. Walton and J. C. Neill (1961), R. N. DeVries (1968), P. C. Trescott, G. F. Pinder and J. F. Jones (1970), G. F. Pinder (1969), Dabiri Green and J. Winslow (1970), and Bredehoeft and Pinder (1970). These publications were reviewed in detail in De Sonneville (1972).

The groundwater basin has been accepted as a unit for hydrologic study and computer models have been developed of non-homogeneous groundwater flow systems. Lack of adequate data on geohydrological parameters such as permeability and storage coefficient often limit the application of a model to the aquifer. Theis (1935) introduced a graphical matching technique to determine the aquifer constants transmissibility, T, and storage coefficient, S, from field observations of draw down. In a paper by Yeh and Touxe (1971) a new technique, quasilinearization was used to identify these parameters. Quasilinearization is a solution technique applicable to a system of differential equations. It involves solving a series of linear initial value problems so that the sequence of solutions converges to the solution of the original problem. Quasilinearization is here applied to the governing equation of radial flow to a well in an extensive homogeneous aquifer. Using this method the observations are converted into estimates of K and T such that the sum of squares of deviations between the observations and theoretical values are minimized. In another paper by Yeh and Touxe (1971) aquifer diffusivity identification was solved using the same technique while the fluctuations of the aquifer head in response to a flood wave were used as observations. Both papers represent analytical procedures to solve the problem of identifying aquifer transmissibility, T, and storage coefficient S, but are limited in application to homogeneous systems.

Parameter identification in non-homogeneous aquifers was presented in a paper by Y. Emsellem and G. deMarsily (1971). It combines the mathematical solution of the inverse problem (computing estimated hydraulic parameters from known piezometric head and flow rates of wells) with the available physical information of the desired parameter solution to arrive at the most continuous distribution of T. Continuous in this context does not represent the mathematical meaning; the distribution should be a 'soft' distribution meaning that for any point with a given T, the T-values surrounding the point should not fluctuate seriously, although continuous derivatives of T do not have to exist.

The mathematical solution of the inverse problem can be shown in the simplified example of steady state flow.

$$\frac{\partial}{\partial x}\left(T\,\frac{\partial \phi}{\partial x}\right) + \frac{\partial}{\partial y}\left(T\,\frac{\partial \phi}{\partial y}\right) + q = 0$$

When T is unknown and ϕ known the equation represents a first order partial differential equation to be solved for T. This problem has been treated by Stallman (1962, 1963), Nelson (1961, 1962, 1964, 1968), Nelson and McCollum (1969), and Emsellem and Prudhomme (1967). Nelson derived a complete energy dissipation theory for steady state flow systems in terms of the kinematic lines and the appropriate line integrals. Generally the theory can be summarized as follows:

It is assumed that a saturated flow region, R, exists where the permeability distribution is to be determined and that the potential distribution ϕ (x,y,z,t_n) is known sufficiently accurately throughout R at one time $t = t_n$. Also it is assumed that a permeability condition exists of known values on one surface $h^o(x,y,z) = 0$, in R, such that the surface h^o is pierced once and only once by every instantaneous streamline in R at $t=t_n$. (This condition provides one value of permeability K_o on every instantaneous streamline in R).

With this the unique permeability distribution is given by using the known potential distribution, ϕ, the instantaneous streamlines, which are generated in R at one time t_n by repetitive solution of the set of equations

$$\frac{dx}{ds} = - \frac{\partial\phi}{\partial x}$$

$$\frac{dy}{ds} = - \frac{\partial\phi}{\partial y}$$

$$\frac{dz}{ds} = - \frac{\partial\phi}{\partial z}$$

using the starting coordinates from the streamlines from h^o.

The permeability along each streamline is determined using the known K_o and by evaluation of

$$\ln \frac{K}{K_o} = - \int_{\phi_o}^{\phi} \frac{\nabla^2 \phi}{|\nabla\phi|^2} d\phi$$

along each streamline in R, generated by the solution of the set of equations.

This method yields very satisfactory results if indeed permeability values are known on surface h^o. Frequently however the permeability values on such surface are not known or the reliability of the measured permeability is doubtful.

Another way to solve the inverse problem is presented by Jacquard (1963) and Jahns (1966). A gradient method is utilized in which the comparison between calculated and historical head is used to adjust the initially estimated parameters.

Of above methods to solve the parameter identification some, which combine the mathematical foundation with physical information, are mathematically cleaner than others. Part of this paper is concerned with the development of approximative methods that in most cases will yield satisfactory results.

CHAPTER III

SYSTEM SIMULATION

The model which served as a starting point for this research was the product of the research study on water use in the Rigby-Ririe area, De Sonneville (1972).

Model Description

The mathematical model developed is a finite difference digital model and, like models of Bredehoeft and Pinder (1970), it is based on the alternating direction implicit method as introduced by Peaceman and Rachford (1953) and Douglas and Rachford (1956), and calculates hydraulic head values on a gridpoint basis.

The two-dimensional model accommodates non-homogeneous, confined and unconfined, leaky and non-leaky aquifers. All boundary conditions normally encountered can be handled, such as constant head boundaries, impermeable boundaries, and boundaries formed by lakes and streams in which the water level changes in time. A procedure for treatment of a flow boundary through which flow is variable and a function of the 'upgradient' flow regime was developed. An option for simulating an open drain is included in which the drain functions as a constant head any time when the water table around the drain is higher than the specified average water level in the drain.

Leakage from or to an underlying or overlying water-bearing formation is dependent on the hydraulic head in the aquifer and is generated in the model program. It is assumed that the head in the adjacent formation is constant during the simulation period.

Inputs or outputs not dependent on the hydraulic head include precipitation, irrigation, evapotranspiration, well discharges or recharges, constant leakage if present, inputs or outputs due to change of average water content of the soil profile above the water table and canal seepage (De Sonneville, 1970).

Canal seepage is dependent on canal water levels and can be calculated in the model program as such. Calculation procedures were developed that assume unsaturated flow beneath the canal, however, data on unsaturated vertical hydraulic conductivity are generally lacking and because the canal operating procedures for most study areas result in nearly constant wetted canal perimeters, seepage was assumed to be constant as measured in the field.

Many aquifers are overlain by several irrigation districts. Application of water on these different irrigation districts as well as the geology of the aquifer vary substantially so that the maximum amplitude of the water table rise during the season may vary from 5 to 50 feet, occurring at different times at each node point. To obtain a reasonable simulation it was considered necessary to approximate the input for each node at each timestep as accurately as possible.

Data on climate, soils, crop distribution, irrigation diversions and distribution losses are utilized in a separate input program to calculate a source term which serves as input to the main program. The main program is general enough to be applied to any aquifer. The separate input program allows greater flexibility in evaluating inputs because it can be tailored to the specific characteristics of an aquifer without changing the main program. The alteration of an input routine that is incorporated directly into the model program many times jeopardizes the operation of the model program.

The differential equation governing the nonsteady flow in an elastic non-homogeneous porous medium can be written as

$$\frac{\partial}{\partial x_i}\left(K_{i,j}\frac{\partial h}{\partial x_j}\right) = \frac{1}{b_{(i,j,t)}} S\frac{\partial h}{\partial t} + W_{(i,j,t)} \tag{1}$$

$K_{i,j}$ is hydraulic conductivity tensor (L/T)

h is hydraulic head (L)

S is the storage coefficient (dimensionless)

b is the depth of aquifer (L),

W is the volume flux per unit area (L/T).

If the coordinate axes are aligned with the principal directions of the conductivity tensor and with $T_{(x,y)} = K_{(x,y,t)}$ the finite difference approximations to equation (1) can be written as

$$T_{xx\ i+1/2,j} \frac{\left(h_{i+1,j,k} - h_{i,j,k}\right)}{(\Delta x)^2} + T_{xx\ i-1/2,j} \frac{\left(h_{i-1,j,k} - h_{i,j,k}\right)}{(\Delta x)^2} \qquad (2)$$

$$+ T_{yy\ i,j+1/2} \frac{\left(h_{i,j+1,k} - h_{i,j,k}\right)}{(\Delta y)^2} + T_{yy\ i,j-1/2} \frac{\left(h_{i,j-1,k} - h_{i,j,k}\right)}{(\Delta y)^2}$$

$$= S_{i,j} \frac{\left(h_{i,j,k} - h_{i,j,k-1}\right)}{\Delta t} + \frac{Q_{i,j,k}}{\Delta x \Delta y \Delta t} - \frac{Kv_{i,j}}{2\ B_{i,j}} \left(2Hc_{i,j} - h_{i,j,k} - h_{i,j,k-1}\right)$$

where

i	is the index in the x - dimension.
j	is the index in the y - dimension.
k	is the timestep.
Kv	is the vertical hydraulic conductivity of the restricting layer (L/T).
B	is the thickness of the restricting layer (L).
$\frac{Kv}{B}$	the leakance of the restricting layer separating the aquifers (1/T).
Hc	is the hydraulic head of the underlying or overlying aquifer.
$Q_{i,j,k}$	is the input term (L^3), in cubic feet for every node point at every timestep.

A more thorough explanation of the mathematical theory can be found in De Sonneville (1972). Equation (2) implies an implicit method of solution. Since an implicit solution for large grid systems requires a considerable amount of computation time, the alternating direction implicit method is preferable because it results in a system of equations with a tridiagonal coefficient matrix for which a simple algorithm exists.

Essentially the principle is to employ two difference equations which are used in turn over successive time steps, each of duration $\Delta t/2$. The first difference equation is implicit only in the x-direction and solves row by row for intermediate values of $h_{i,j}$ at $t = k+1/2$ which are used in the second equation, implicit in the y-direction solving now column by column, leading to the solution of $h_{i,j,k+1}$ at the end of the whole time interval Δt. Equation (2) for a row calculation in the alternating direction implicit method with coefficients A,B,C and D substituted for all known values yields (De Sonneville, 1972).

$$Ah_{i-1,j,k+1/2} + Bh_{i,j,k+1/2} + Ch_{i+1,j,k+1/2} = D \tag{3}$$

The hydraulic head is calculated in a system of tridiagonal equations similar to equation (3) in which the boundary equations have only two unknowns.

Input Program

Data on irrigation districts such as irrigation diversions, return flow and canal seepage, and data on climate, crop distribution, tributary valley underflows, surface flows, river gains and losses are used to calculate the external input to every node point for every half timestep. A summary and explanation of the control variables and input data to the input program and the format under which they are entered in the program are included in Appendix A. The total input per node is composed of the following terms.

Q (I,J) = - WATER - SEEPAG + OUT - RAIN + PUMP - FLOWIN + SINKIN (4)

WATER = total input from surface water irrigation diversions, irrigation canal seepage excluded.

SEEPAG = total input from irrigation canal seepage.

RAIN = total input from effective precipitation, that is precipitation that eventually recharges the ground water.

OUT = total output from consumptive use from surface water and ground water irrigated areas.

PUMP = total input or output by artificial recharge or pumpage from wells or well fields.

FLOWIN = total input from ground water flow of tributary valleys.

SINKIN = total input or output from reach gains or losses of perched streams or hydraulically connected streams.

Calculation of the WATER term

Dependent on the size of the mesh in the grid system the nodal area may encompass several irrigation districts, each with its own diversion. Each irrigation district is given a serial number. The nodal area may also contain surface water irrigated areas which are not organized into specific districts. For these areas no irrigation diversion records are known and have to be treated in an approximate manner. There may also be non-organized groundwater irrigated areas.

Computer space considerations limit the number of different irrigation districts per node (surface water and/or groundwater irrigation districts) to four while the non-organized surface water areas are lumped together as well as the non-organized groundwater irrigated areas. The WATER term is calculated in two parts:

Contribution from organized districts.

In this case the net irrigation diversion is calculated as the difference between total diversion and return flow per district. For each district two irrigation application rates are calculated.

For irrigation district K

$$APPNET(K) = (CH(K) * (TOTDIV(K)-RETFLO(K)) - SUMS(K))/TOTAC(K) \qquad (5)$$

and,

$$APNONE(K) = CH(K) * (TOTDIV(K) - RETFLO(K)/TOTAC(K) \qquad (6)$$

where

APPNET(K) = irrigation rate excluding the irrigation district canal seepage.

APNONE(K) = irrigation rate including the irrigation district canal seepage.

CH(K) = a management multiplier; in case of historical diversions CH(K) = 1.00

TOTDIV(K) = total irrigation diversion.

RETFLO(K) = total return flow.

SUMS(K) = total district canal seepage.

TOTAC(K) = total irrigated district acreage.

For the entire aquifer an overall average irrigation rate is computed as follows

$$APAV = \sum_{K}(CH(K) * (TOTDIV(K) - RETFLO(K))/\sum_{K} TOTAC(K) \tag{7}$$

The total contribution from organized districts for node point (I,J) is obtained by multiplying the irrigated areas of each district in that node by their respective irrigation rates.

$$WATER\ 1 = \sum_{N=1,4} SURFAR(I,J,N) * APPNET(NIR(I,J,N)) \tag{8}$$

in which

SURFAR(I,J,N) = surface water irrigated acreage in node point (I,J) for the Nth district in this node.

NIR(I,J,N)= K = the irrigation district number of the Nth district in node point (I,J).

Contribution from non-organized surface water irrigated areas.

The total non-organized areas are lumped together under the term SUREST(I,J). For these areas no surface water diversions or irrigation canal seepage is recorded. The contribution of this area is calculated in two ways.

a. If organized districts are located in node point (I,J) the average application rate over these districts (APUN) is calculated and applied to the non-organized acreage

$$APUN = \sum_{N=1,4}(SURFAR(I,J,N) * APNONE(NIR(I,J,N)))/\sum_{N=1,4} SURFAR(I,J,N) \tag{9}$$

The total contribution from non-organized areas is then the product of the area and the average application rate.

WATER 2 = APUN * SUREST (I,J)

b. If no organized districts are present in node point (I,J) the contribution is approximated using the overall irrigation rate for the aquifer. The contribution becomes

WATER 2 = APAV * SUREST (I,J)

The total contribution for node (I,J) from organized and non-organized irrigated lands is

WATER = WATER 1 + WATER 2 (10)

Calculation of the SEEPAG term

For the organized districts records are available of the irrigation diversions. For some districts information is available that allows an approximation of the seepage losses in the irrigation canals. In that case the total district seepage is subtracted from the net irrigation diversion and treated as a separate input. Functional relationships between canal seepage rate, depth to water table and depth of water in the canal require knowledge of the vertical hydraulic conductivity in the aquifer and would be difficult to determine. Usually information is not available on the vertical hydraulic conductivities and average seepage rates from field tests can more readily be obtained.

The wetted area of canals is a function of the operation of the canals. To record all water stages of each irrigation canal is a practical impossibility and for most areas the normal operating procedure is to maintain water levels as near maximum as possible. This allows the wetted area of canals to be considered constant in time. The seepage for node (I,J) can be calculated as follows:

SEEP(I,J) = ARWET(I,J) * FACTOR(I,J) * DELT/2.0 (11)

in which

ARWET(I,J) = total wetted area of all canals in node (I,J)

FACTOR(I,J)= average seepage rate for node (I,J)

DELT/2 = duration of the half timestep.

If more than one district is located in one node the seepage for every district in that node is proportioned according to the number of irrigated acres of each district in that node in relation to the total irrigated acreage. The total seepage for irrigation district K is then obtained by summing up these portions over all the nodes in the aquifer or,

$$\text{SUMS (K)} = \sum_{I,J} (\text{SURFAR(I,J,K)/ACRES (I,J)}) * \text{SEEP(I,J)} \quad . \qquad (12)$$

in which

ACRES (I,J) = total recorded surface water irrigated acreage of the organized districts in node I,J.

The term SUM (K) is used in the calculation of the WATER term, and the total contribution from seepage for node (I,J) can be expressed as

$$\text{SEEPAG} = \text{SEEP(I,J)}$$

Calculation of the OUT term

The OUT term represents the total consumptive use of the surface water and groundwater irrigated areas. The total aquifer is divided into climatic regions for which the crop distribution is computed as well as the average consumptive use. The total irrigated acreage for node (I,J) is

$$\text{GROUND} = \sum_{N=1,4}\text{SURFAR(I,J,N)} + \sum_{N=1,4}\text{GRWAR(I,J,N)} + \text{SUREST(I,J)} + \text{GRREST(I,J)} \qquad (13)$$

in which

SURFAR(I,J,N) = surface water irrigated area of the Nth organized district in node (I,J)

GRWAR(I,J,N) = groundwater irrigated area of the Nth organized district in node (I,J)

SUREST(I,J) = total surface water irrigated area of non-organized areas
GRREST(I,J) = total groundwater irrigated area of non-organized areas

The nodes with irrigated lands, located in climatic region No. L are defined by the array NODEL(I,J), or, NODEL(I,J) = L. Presence of irrigation is denoted by the array NREG(I,J); if nodes have no irrigation NREG(I,J) = 0. For nodes located in climatic region No. L with irrigated lands NREG(I,J) = NODEL(I,J) = L. For nodes for which NREG(I,J) > 0 the total contribution from consumptive use is

$$\text{OUT} = \text{GROUND USE(L)}$$

in which

USE(L) = consumptive use per surface unit for climatic region L
L = NREG(I,J)

The groundwater irrigated areas are treated as areas for which the net output from the area is represented by the consumptive use, thereby assuming that the water pumped for irrigation in excess of consumptive use returns to the groundwater aquifer at approximately the same place and time it was withdrawn. Surface water runoff from groundwater (pumped) irrigated areas is usually negligible since sprinkler irrigation is predominant. This treatment of the groundwater irrigated areas makes the determination of total pumpage from power company records unnecessary.

Calculation of the RAIN term

The RAIN term represents the total contribution from precipitation for node (I,J). For the operation of the groundwater model only that portion of the precipitation that reaches the groundwater table is important. This portion is called the effective precipitation. The percentage of the precipitation that is effective is a function of the vegetation, geology, soils and depth to water. The aquifer is divided into regions, each with its specific percentage of effective precipitation. Each area is given an identification number. For every node point this number is read in under the array NEFF(I,J). If NEFF(I,J) = L, node point (I,J) is located in percentage region No. L. The actual percentage of the precipitation that is effective for region L is stored under RECH(L) or, the actual percentage of the precipitation that is effective for node (I,J) is RECH(NEFF(I,J)). The RAIN term is computed in parts dependent on whether the precipitation falls on non-irrigated lands or irrigated lands.

Precipitation on non-irrigated lands.

For non-irrigated lands the total contribution (RA1) is obtained by multiplying the non-irrigated part of the nodal area by the precipitation of the climatic region of the node (I,J) and the percentage of effectiveness.

$$RA1 = (DELX * DELY - GROUND) * PRECIP(NODEL(I,J)) * RECH(NEFF(I,J)) \quad (14)$$

in which

DELX * DELY = nodal area

GROUND = total surface water or groundwater irrigated area

PRECIP(NODEL(I,J)) = total precipitation for that half timestep for climatic region No. 'NREG(I,J)'

RECH(NEFF(I,J)) = percentage of effectiveness of precipitation region No. 'NEFF(I,J)'

Precipitation on irrigated lands.

The total contribution from the precipitation on irrigated lands is calculated in several ways. During the irrigation season, the precipitation is considered to be 100% effective which, subtracted from the consumptive use results in the irrigation requirement as net output from the aquifer for that half timestep. During the non-irrigation ('=winter') season, the precipitation on irrigated lands is multiplied by the percentage of effective precipitation of the non-irrigated lands of that node. If weekly or monthly half timesteps are used the season can generally be divided into timesteps which fall either in the irrigation season or the winter season. If the half timestep covers a year a different situation exists. Therefore, dependent on the length of the half timestep the precipitation on the irrigated lands is treated as follows:

a. Half timestep equal to year.

Two precipitation terms are read in: The total precipitation for every climatic region over the half timestep and the total precipitation that falls in the irrigation season for every climatic region of that half timestep. The total precipitation then consists of two parts in which the summer precipitation is 100% effective and the remaining precipitation partly effective.

$$\text{RA2a} = \text{SUMPRE(NREG(I,J))} * \text{GROUND} + \text{(PRECIP(NREG(I,J)} - \text{SUMPRE(NREG(I,J))} * \text{(DELX} * \text{DELY} - \text{GROUND)} * \text{RECh(NEFF(I,J))} \quad (15)$$

in which

SUMPRE(NREG(I,J)) = total precipitation of the irrigation season for climatic region No. 'NREG(I,J)'

PRECIP(NREG(I,J)) = total precipitation over the full half timestep for climatic region No. 'NREG(I,J)'

RECH(NEFF(I,J)) = the percentage of effectiveness of precipitation in region No. 'NEFF(I,J)'

b. Half timestep less than a year-in irrigation season.

One precipitation term is read in representing the total precipitation for that half timestep. The total contribution from irrigated lands is

$$\text{RA2b} = \text{PRECIP(NREG(I,J))} * \text{GROUND} \quad (16)$$

c. Half timesteps less than a year-in the winter season

For this situation crop consumptive use equals zero and the contribution of irrigated lands is

$$\text{RA2c} = \text{PRECIP(NREG(I,J))} * \text{GROUND} * \text{RECH(NEFF(I,J))} \qquad (17)$$

The total contribution of non-irrigated lands and irrigated lands is one of the following combinations as described above.

$$\text{RAIN} = \text{RA1} + \text{RA2a} \quad \text{or,}$$
$$\text{RAIN} = \text{RA1} + \text{RA2b} \quad \text{or,}$$
$$\text{RAIN} = \text{RA1} + \text{RA2c}$$

Calculation of the PUMP term

The pumping term represents the contribution due to artificial recharge or discharge from wells other than irrigation wells. The total pumped volume is read in for every node for every half timestep as PU(I,J). The contribution from this term is

$$\text{PUMP} = \text{PU(I,J)} \qquad (18)$$

Calculation of the FLOWIN term

The FLOWIN term represents the total contribution from tributary valley under flow for node (I,J). In some cases where a large aquifer is to be modeled, the mountainous area surrounding the aquifer serves as an impermeable boundary. This impermeable boundary is interrupted at places where the model boundary intersects the mouth of secondary valleys or intervening valleys bordering on the main aquifer. The groundwater underflow reaching the main aquifer from these valleys has to be accounted for in the calculation of the input. In case intervening valleys are present, each valley is given an identification number. All node points inside the aquifer boundaries over which the groundwater underflow of a valley is to be distributed have this number which is denoted by the array NFLOW(I,J). For all node points of valley No. K, NFLOW(I,J) = K. For all node points with no valley underflow NFLOW(I,J) = 0. The total number of nodes for valley No. K over which the underflow is to be distributed is denoted NUM(K). UNDFLO(K) represents the total groundwater underflow in a half timestep for valley No. K. With these terms defined,

$$\mathrm{FLOW(K) = UNDFLO(K)/NUM(K)}$$

in which

FLOW(K) = the underflow per node for valley No. K

The total contribution for node (I,J) from groundwater underflow is

$$\mathrm{FLOWIN = FLOW(NFLOW(I,J))} \tag{19}$$

Calculation of the SINKIN term

If node (I,J) is part of a gaining or losing stream or lake in the aquifer the FLOWIN term represents the gain or loss for node point (I,J). Every reach for which reach gains or losses are recorded is given an identification number. All node points which are part of a reach over which the reach gain or loss is to be distributed have this number which is read in under NREACH(I,J). For all node points of reach No. K, NREACH (I,J) = K. For node points not located on streams NREACH(I,J) = 0. The total number of nodes over which the reach gain or loss is to be distributed is denoted by NO(K). REACH (K) represents the total reach gain or loss in a half timestep for reach No. K. Similarly, with these terms defined,

$$\mathrm{SINK(K) = REACH(K)/NO(K)}$$

in which

SINK(K) = the reach gain or loss per node for reach No. K.

Node (I,J) for which NREACH(I,J) > 0 have a reach gain or loss. The total contribution for node (I,J) from reach gains or losses is

$$\mathrm{SINKIN = SINK(NREACH(I,J))} \tag{20}$$

The streams or lakes for which reach gains or losses are recorded may be streams that are perched above the groundwater table to be simulated in the groundwater model. The unsaturated flow condition resulting from this situation between the perched body and the hydraulic head in the aquifer is impractical to calculate since generally the data necessary for such a calculation is lacking. For these streams it is therefore essential that the historic reach gains or losses are entered in the calculation of the external source term. The other situation involves the reach gains or losses of streams which have a hydraulic connection with the aquifer. In that case the node points of these streams constitute a constant head boundary in the model program. For these nodes the hydraulic head is known and therefore not calculated in the model program. In other words, with the hydraulically connected constant head nodes knowledge about the magnitude of the historic reach gains is not a prerequisite for a proper calculation of the hydraulic heads in the aquifer, other than to serve as a comparison check with the reach gains or losses calculated for these nodes via the INOUT subroutine (see page 59).

In the INPUT program, for every half timestep the sum of Q(I,J) over all node points inside the model boundaries is calculated with the exclusion of the reach gain or loss contributions from the hydraulically connected streams, as well as the total input over all half timesteps of the simulation.

The INOUT subroutine in the model program calculates the sum total of the reach gains and losses for hydraulically connected streams if the streams are inside and not part of the aquifer boundary. If the hydraulically connected streams are part of the aquifer boundary the subroutine calculates the total flow leaving or entering the aquifer. This is done for each half timestep. From this the total flow leaving or entering the aquifer system via the hydraulically connected streams over the entire simulation period is calculated. This total provides a check of the mass balance computation of the aquifer in case the calculated head values at the end of the simulation are the same as the starting head values. In that case the storage accretion is zero and the total flow calculated in the INOUT subroutine should be of opposite sign but of equal magnitude as the total net input, as calculated in the input program in the way described above.

Total Input

The amount of data required for the calculation of the input Q(I,J) of node (I,J) is quite large and there may be some areas in the aquifer for which the data concerning the irrigation district diversions, district total acreage or climatic data is incomplete so that the respective terms that make up the total input for node (I,J) cannot be properly calculated. For those areas only an estimate of the total input to the aquifer can be made. This estimate could be entered as a total lump sum distributed over the nodes of the area lacking adequate data. In other words, the area can be treated the same way as the groundwater underflow of the intervening valleys is treated. In order to distinguish the areas in the aquifer which lack adequate data and to treat them as described above an array INPUT(I,J) is read in which has the following functions:

1. INPUT(I,J) = 0

   If INPUT(I,J) = 0, adequate data is available and the input for (I,J) is calculated from data on irrigation diversions, acreage, water use, precipitation, seepage, well withdrawals or recharge, groundwater underflow and reach gains or losses if present.

2. INPUT(I,J) = 1

   If INPUT(I,J) = 1, no adequate data are available for nodes in this area to calculate the input term properly. In this case the area is entered in the program as an artificial valley with as groundwater underflow the estimated total input of the area to the aquifer. The nodes over which the input is to be distributed are in this case denoted by two arrays. For these nodes INPUT(I,J) = 1 and NFLOW(I,J) = K, representing the serial number of this artificial valley. The actual calculation of the input is as follows. For every node point regardless of the value of INPUT(I,J), regardless of whether the data for node (I,J) are incomplete, the terms WATER, SEEPAG, OUT, RAIN, PUMP, FLOWIN AND SINKIN are calculated. It must be realized that for the areas with incomplete data the first five terms of above set are erroneous. The term FLOWIN is calculated properly as the nodes in the data scarce areas are also distinguished by the values for

NFLOW(I,J) accompanied by correct values for FLOW(I,J), the estimated input. If the data are adequate to calculate the reach gains and losses for this area the term SINKIN is also calculated. If not, the value for NREACH(I,J) for the stream nodes must be set to 0. This makes the term SINKIN automatically zero.

After the calculation of all these terms the total input for node (I,J) is calculated in two ways.

If INPUT(I,J) = 0

$$Q(I,J) = -WATER-SEEPAG+OUT-RAIN+PUMP-FLOWIN+SINKIN \tag{21}$$

If INPUT(I,J) = 1

$$Q(I,J) = -FLOWIN+SINKIN \tag{22}$$

Calibration Program

Application of the model to the Rigby-Ririe area showed that simulation of historical seasonal water table changes could not be realized because of the scarce amount of hydrogeological data available. Changing the hydrogeological parameters on a trial and error basis did not give satisfactory results. A fair simulation of historical water table trends is necessary since only then can water table changes as a result of alternative management solutions be predicted with sufficient reliability. Generally, the data for any study area are composed of inputs associated with water management which are calculated in the input program, and data related to the geohydrological properties of the aquifer such as the hydraulic conductivity, storage coefficient, the aquifer bottom elevation, the leakance factor of the restricting layer, the initial head difference in case of a multi-aquifer system and the historical initial water table values. In many cases only historical water table elevations are known. Information about the other geohydrological parameters is often scarce.

The calibration routine adjusts the other parameters in an automated way to achieve simulation of historical water table elevations. Four parameters are considered for change; conductivity, leakance factor, initial head difference and storage coefficients. The aquifer parameters are changed based on differences between calculated and historical water table elevations at selected timesteps. Some aquifers show a seasonal rise of the groundwater table as a result of irrigation practices, of which the yearly amplitude is nearly constant; other aquifers show a general rise or decline of the groundwater table resulting from increasing development of the resources in the aquifer, and there are aquifers that show a water table behavior which is a combination of the above two. For the calibration purposes a simulation interval is chosen in which the water table behavicr shows a definite maximum and minimum level.

Calibration is based on comparison of water table levels at several timesteps; the maximum, possibly an intermediate stage and the minimum at the end of the interval. At each selected timestep for every node point the deviations from historical water table elevations are calculated as well as the sum of squares of the deviations over the entire aquifer. Parameter values are changed according to the magnitude of the deviations at one of the selected timesteps. This timestep will then render the best fit. For areas with a high water table problem best fit priority may be given to the timestep with the maximum water table elevation. In the calibration program the first parameter is adjusted as follows.

$$\text{Par}_{\text{new}} = \text{Par}_{\text{old}} + \text{Par}_{\text{old}} \left(\frac{\text{node point deviation}}{\text{maximum deviation in the aquifer}} \right) \quad (23)$$

With the new parameter values the simulation is repeated for the same interval until four runs have been made; after that the routine selects that set of values that resulted in the minimum overall sums of squares over all timesteps that were selected for comparison with the historical head values.

The routine then changes the parameter values back to the original starting values and a second parameter is adjusted similarly in four simulation runs. The remaining two parameters are adjusted in the same manner. At the termination of the first calibration these 4 x 4 simulation runs result in a set of data cards that for each parameter gives the least overall sum of squares of deviations over the calibration timesteps.

The initial data set of these parameters is replaced by the result of the first calibration and a second calibration is made. This procedure is followed until no decrease in total sum of squares is observed. This calibration routine in this initial stage of development was applied to the Rigby-Ririe study area.

CHAPTER IV

APPLICATION OF THE MODEL TO THE RIGBY-RIRIE AREA

Evaluation of the model and calibration procedure was accomplished by application to the alluvial aquifer in the Rigby-Ririe area of eastern Idaho. Investigations of the effects of water management alternatives on the configuration of the water table were performed in order to select management decisions that will alleviate the high water table problems in the area.

Study Area

The water management study area as shown in Figure 1 is located in Jefferson and Bonneville counties of Idaho and comprises approximately 100,500 acres. This area is an old alluvial fan of the Snake River and is served by an irrigation system developed in the late 1800's by private and cooperative groups. The Great Feeder Canal which is an old channel of the Snake River runs east and west through the area and delivers water to some 20 smaller canals, each one operated by a separate and independent canal company or irrigation district.

Data Collection

Geology

Available well logs from the Department of Water Administration and local residents indicate that the gravel aquifer is extensive over the fan and is underlain by the basalts of the Snake River Plain. However, very few of the domestic wells for which logs are available are over 100 feet deep so that the depth of gravels is not discernible over the entire fan.

Water balance computations together with some evidence of the existence of interspersing clay layers indicate that an important percentage of the total water diversions leaves the area via leakage to the regional

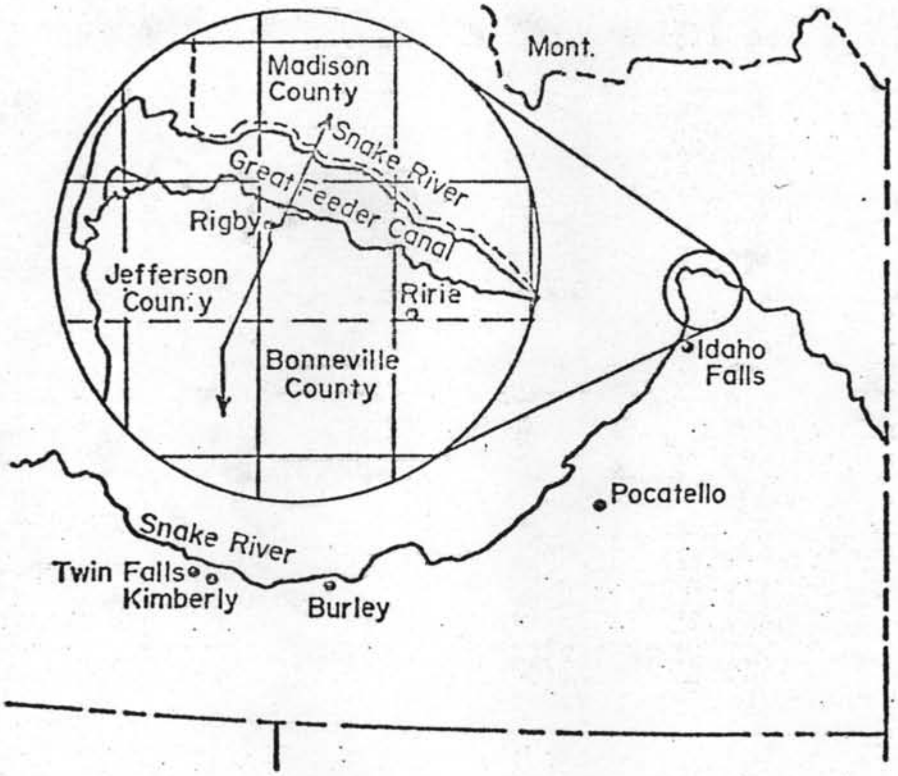

Figure 1. Location of the Study Area

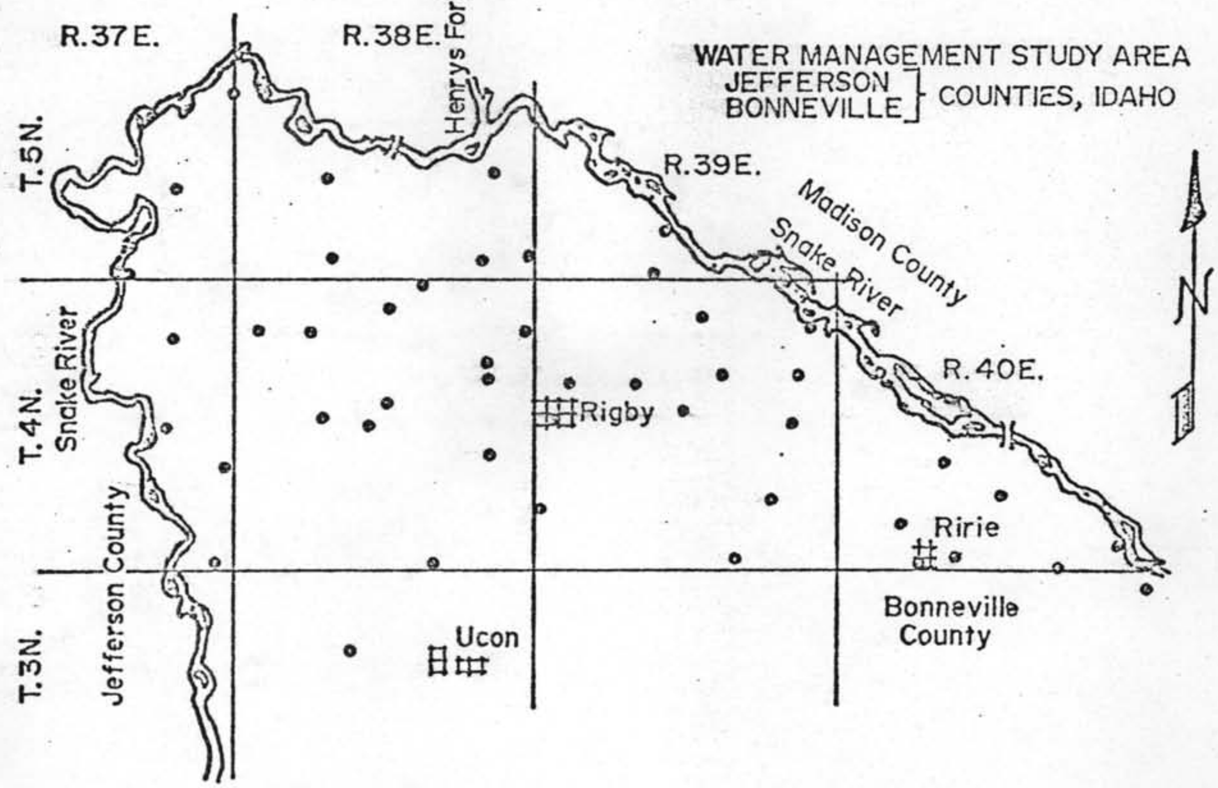

Figure 2. Location of Wells and Well Points

groundwater table of the Snake Plain (Brockway and De Sonneville, 1973). The location and the magnitude of this leakage is unknown. From the available geological information only an estimate can be made of the values of the geohydrological parameters such as hydraulic conductivity, storage coefficient, leakance factor, and head differences of a leaky aquifer system.

A resistivity study was made by a consulting firm, Group Seven, Inc., to assist in estimating the depth of gravel and approximate locations of clay lenses. Results indicated that the assumption of clay layers over most of the aquifer were valid.

Groundwater Table Elevations

A network of some 40 wells in the area was used to monitor changes in the water table throughout the study. Figure 2 shows the locations of wells and well points measured in the network. Water table elevation contours were interpreted from these well recordings at three selected times in 1972-1973 for use in calibration of the simulation model. Figure 3 shows the historical water table contours for August 30, 1972.

In the vicinity of Rigby the water table rises as much as 40 feet from the beginning of irrigation in May to August. Maximum water table elevations occur in August and associated problems are prevalent during August and September. Figure 4 shows the depth to the water table on August 30, 1972 as computed from the water table contours. The area north and west of Rigby as indicated in the figure had depths to water of five feet or less during July, August and September of 1972. The area around the city of Ririe is a local groundwater mound with depth to water of 10 feet and less.

Surface Water Diversions

Irrigation diversions and irrigated acres for the major canals in the study area for the May 1 - September 30 period are recorded in the reports of Water District 1. These measurements are taken by District 1 and U. S. Geological Survey personnel. During the 1972 season measurements

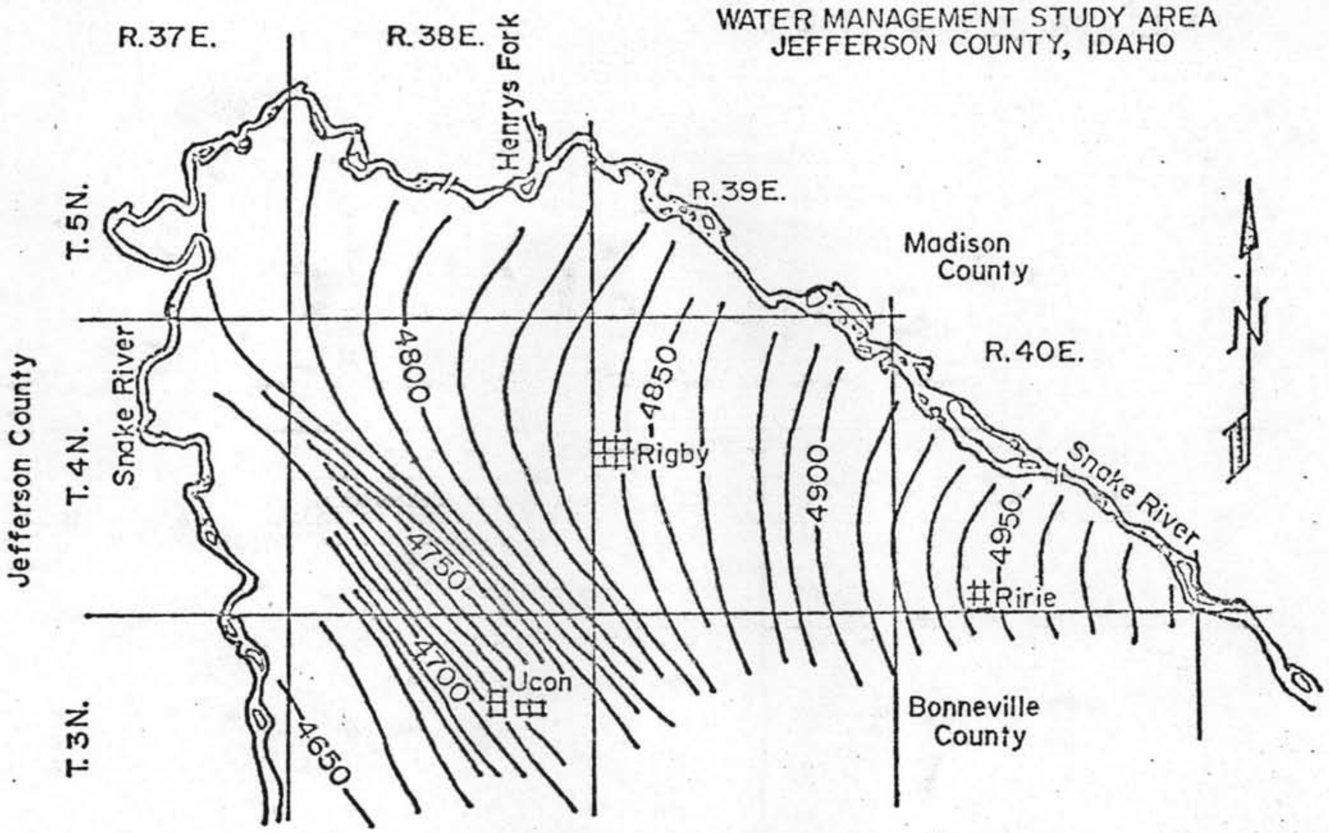

Figure 3. Groundwater Levels, August 30, 1972 - 10 Ft. Contour Interval

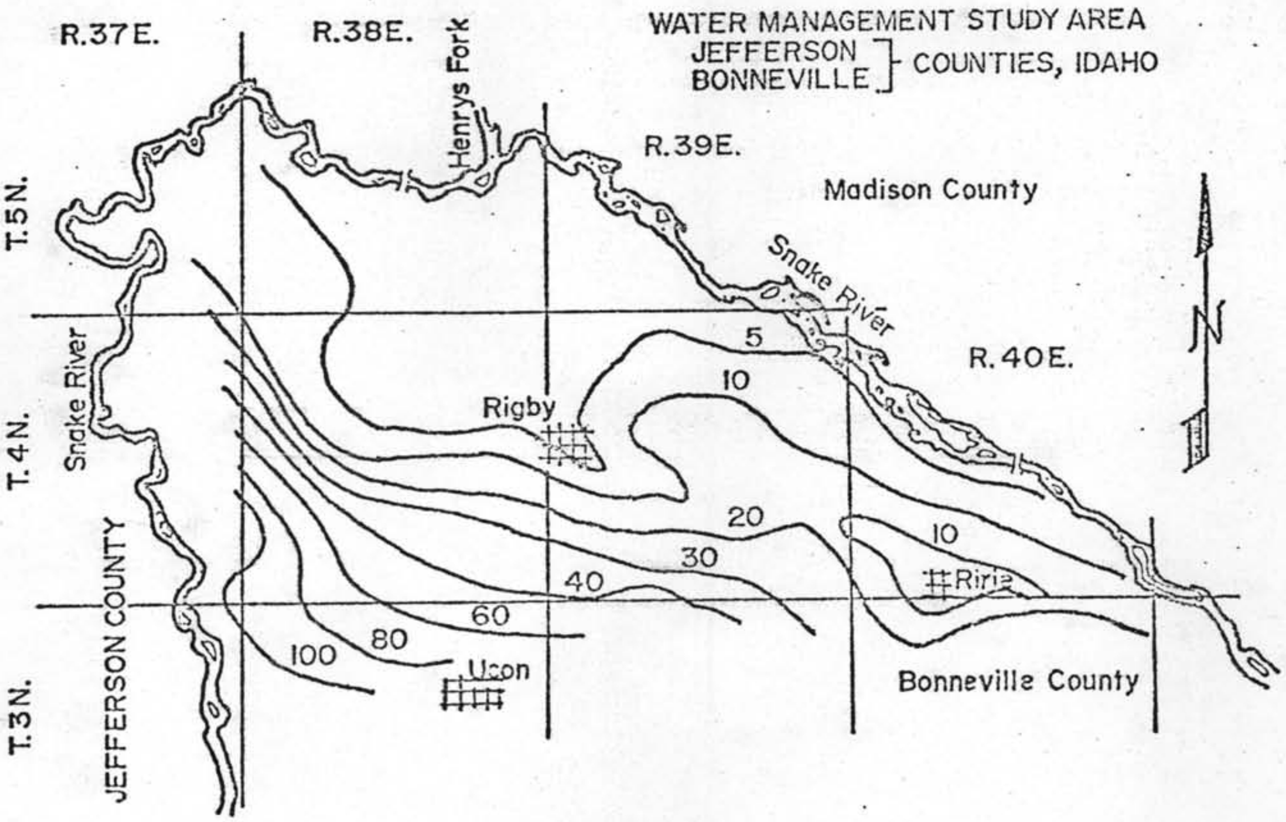

Figure 4. Map of Study Area Showing the Depth to Water Table August 30, 1972 in Feet.

were extended by the University of Idaho and ARS personnel past the normal September 30 cutoff date through November 30 or until all canals had been shut down for the winter. Return flows to the Snake River at the Burgess Canal, Long Island Slough and Great Feeder were measured throughout the season. Water transported out of the study area to the north by spring flow was measured as well as water transported out of the area to the south in the Anderson, Farmers Friend and Harrison canals.

Figure 5 shows the three year average of the published May-September diversions per acre from the Snake River for canals serving about 84,000 acres of the Snake River Fan. An increasing trend can be observed with the recorded 1972 May-September diversions approaching 13.1 ac ft/ac.

The seasonal distribution of total diversion and outflows for 1971 and 1972 is shown in Figure 6. The total canal diversions include all canals with service areas totally or partially included in the study area. Outflows include return flow to the Snake River and canal flows out of the study area. The net diversion is calculated as the measured diversion at the canal head gates minus all surface wastes. The distribution of diversions for the 1972 operating season is shown in Table 1.

The total diversion for May-November 1972 for the 82,250 irrigated acres, as measured from aerial photos, was 17.0 ac ft/ac of which 11.6% was diverted after September 30.

Canal Seepage Losses

The main canals of the system have a total water surface area of 717 acres or slightly less than 1% of the irrigated area. Seepage tests were made at 20 locations in late summer and fall of 1970 and at 16 locations in early spring of 1971. With an estimated accuracy of measurement between 5% and 10% seepage measurements averaged 3.50 ft/day. This seepage rate applied to all main canals in the system amounts of 501,550 ac ft over a 200 day season or 33% of the gross diversion.

The Great Feeder canal which has not been accounted for in the above calculation has a wetted area of 312 acres or 31.5% of the total wetted area of canals. During the latter part of the irrigation season the Great Feeder is a gaining stream and acts like a drain in the western

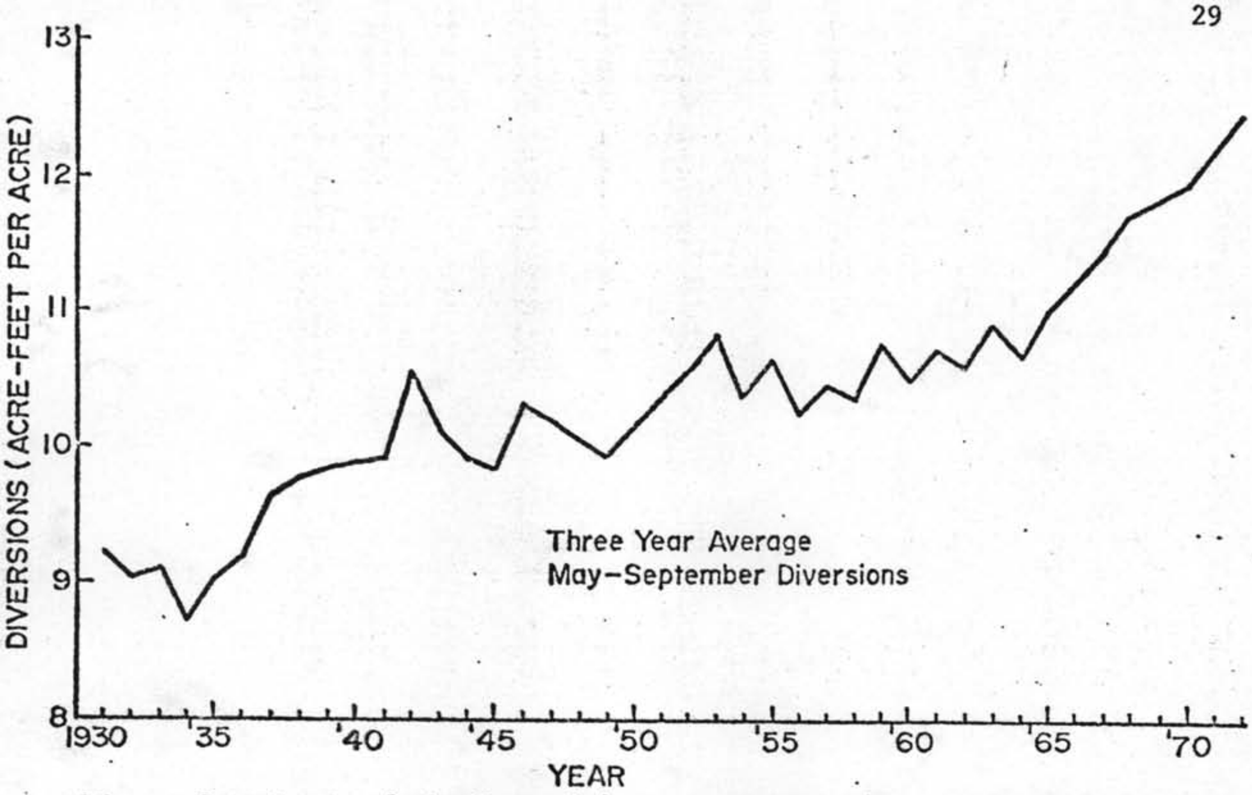

Figure 5. Graph of the Recorded May-September Diversions per Acre from the Snake River for All Canals of the Snake River Fan (Annual Irrigation Diversion).

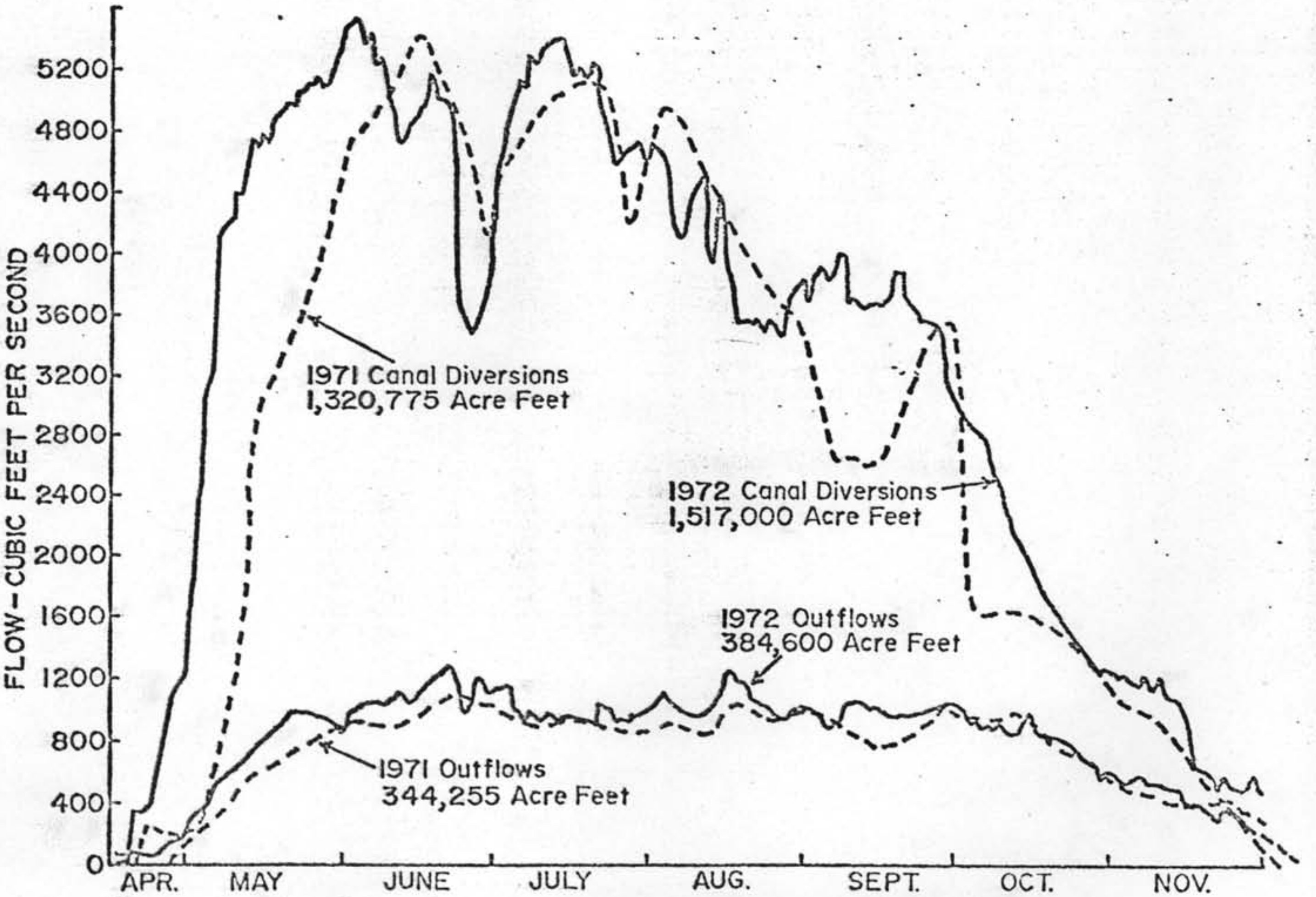

Figure 6. Total Canal Diversions and Outflows from Study Area.

part of the aquifer so that the effective seepage rate over the total length is less than the average for the area. With a seepage rate of 1 ft/day the yearly seepage of the Great Feeder adds 113,800 ac ft to the aquifer.

Table 1. Water Management Study - Jefferson County, Diversion Distribution

Irrigated Area - 82,250 acres		1972
Diversion*	1,517,000 ac ft	
Outflow**	384,600 ac ft	
Net Diversion	1,132,400 ac ft	13.8 ac ft/ac
Transmission Loss	501,500 ac ft	6.1 ac ft/ac
Net Application	630,900 ac ft	7.7 ac ft/ac
Evapotranspiration	164,500 ac ft	2.0 ac ft/ac
Deep Percolation	466,400 ac ft	5.7 ac ft/ac

*Total diversion includes all canals diverting from the Snake River which irrigate or are used to transport water through the study area except the Eagle Rock canal.

**Outflow includes water transported to lands south of the study area by the Anderson, Farmers Friend, and south branch of the Harrison Canal.

Snake River Losses

Recognizing that losses in the Snake River as it flows over the fan can contribute to the groundwater table rise, an attempt was made to evaluate these losses. Stearns (1938) reported an average loss of 288 cfs in the Heise-Lorenzo reach. Current meter measurements made three times in 1970 indicated an average loss of 408 cfs. Based on 408 cfs for a 200 day season, losses from the Snake River account for 163,200 ac ft of water added to the aquifer.

The Snake River loss represents 21% of the 778,500 acre feet of water added to the aquifer by seepage from irrigation canals, the Great Feeder, and the Snake River but represents only 13% of the total input of 1,244,900 ac ft added to the aquifer over the irrigation season.

Evapotranspiration

Evapotranspiration for the 1972 season was calculated using the Penman combination equation with crop coefficients and measured crop distribution. Differences in crop distribution throughout the area were not significant. The total evapotranspiration for the season was 2.0 ac ft/ac or 164,500 ac ft for the study area. The winter evapotranspiration was assumed to be negligible.

Boundaries for Snake River Fan

Figure 7 is a map of the study area with a one mile square grid imposed. The Snake River serves as a boundary with a variable water level adjusted according to the river management, and the southeast part of the aquifer is bounded by a mountainous area which serves as an impermeable boundary.

In the southwest corner of the study area the gravel aquifer connects with the deep groundwater table of the Snake River Plain basalts. The gradient is steep and generally in a southwest direction. The boundary is composed of two types: (1) an artificial no-flow boundary which parallels the direction of flow and (2) the flow boundary which terminates the southeast portion of the study area.

Period of Simulation

The start of simulation was chosen as May 1, 1972 since this date represents the low point of the recession curve of the water table before the water levels rise again as a result of irrigation diversions. Diversions take place until at least November 25 or later so the irrigation season in the model was extended to December 11. From December 11 to May 1, the winter season, no irrigation takes place and the evapotranspiration is considered negligible. Historical water level data show that the seasonal rise of the groundwater table is a repetitive cycle of which the yearly amplitude is nearly constant so that selection of any particular year for calibration is immaterial. The 12 month period May 1,

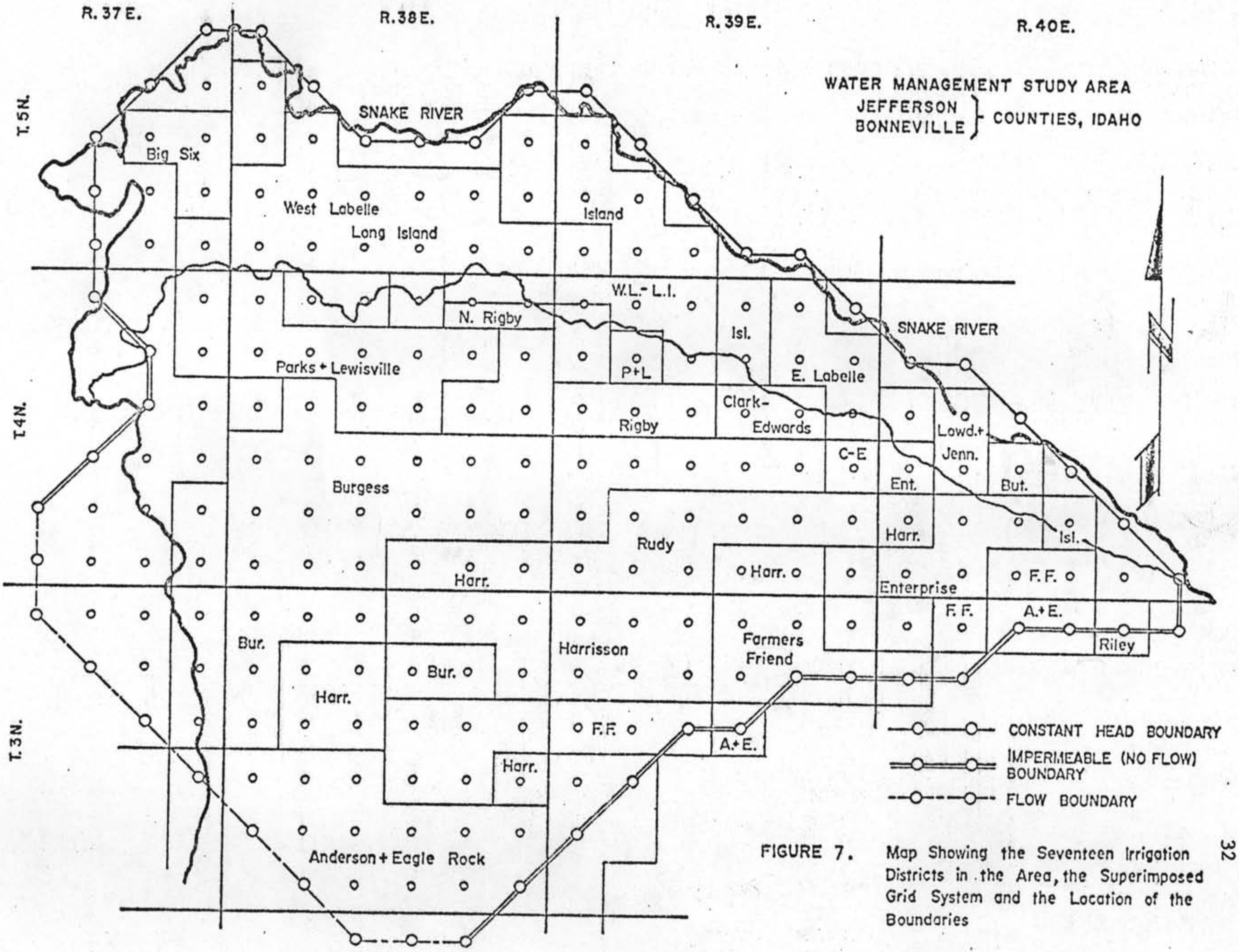

FIGURE 7. Map Showing the Seventeen Irrigation Districts in the Area, the Superimposed Grid System and the Location of the Boundaries

1972 to May 1, 1973 was chosen for the calibration.

Application of Calibration Routine

Aquifer water levels at three timesteps which would be most indicative of the cyclic behavior of the water table were chosen for comparison; the maximum, August 30; an intermediate stage, October 30; and the minimum at the end of the yearly cycle, May 1. Figure 8 shows the aquifer response for the Clark well near Rigby. In this area with a high water table problem best fit priority is given to the maximum water table, occurring approximately August 30.

Calibration of the model yielded very satisfactory results. After four calibration runs a decrease of sum of squares of deviations was observed. An example of the results of the calibration is given in Figure 9 which is a microfilm plot showing the deviations for four calibrations of the leakage factor at a particular node point. Deviations of calculated from measured head values at four timesteps are plotted. The total sum of squares of the deviations was calculated for the second, third and fourth timestep. The groups of numbers (1,2,3,4) on Figure 9 represents the deviations from measured values as a result of runs with successive parameter values.

For the last calibration run on the Snake River fan aquifer the final sum of squares of deviations between calculated and historical head values resulted in standard deviation of 1.25 feet for the second timestep (August 30), 3.2 feet for the third timestep (October 30), and 4.4 feet for the fourth timestep (April 30, 1973). Since the maximum rise of the groundwater table in the aquifer varies from 20 to 50 feet except near the Snake River, this result is satisfactory.

Figure 10 shows the historical and simulated well hydrograph after final calibration for the year 1972 of the Clark well located in the study area. This close simulation of the historical water table is representative of the simulation achieved over the entire area and demonstrates that the model accurately simulates historic water table fluctuations. It indicates the applicability of the model for evaluating management decisions.

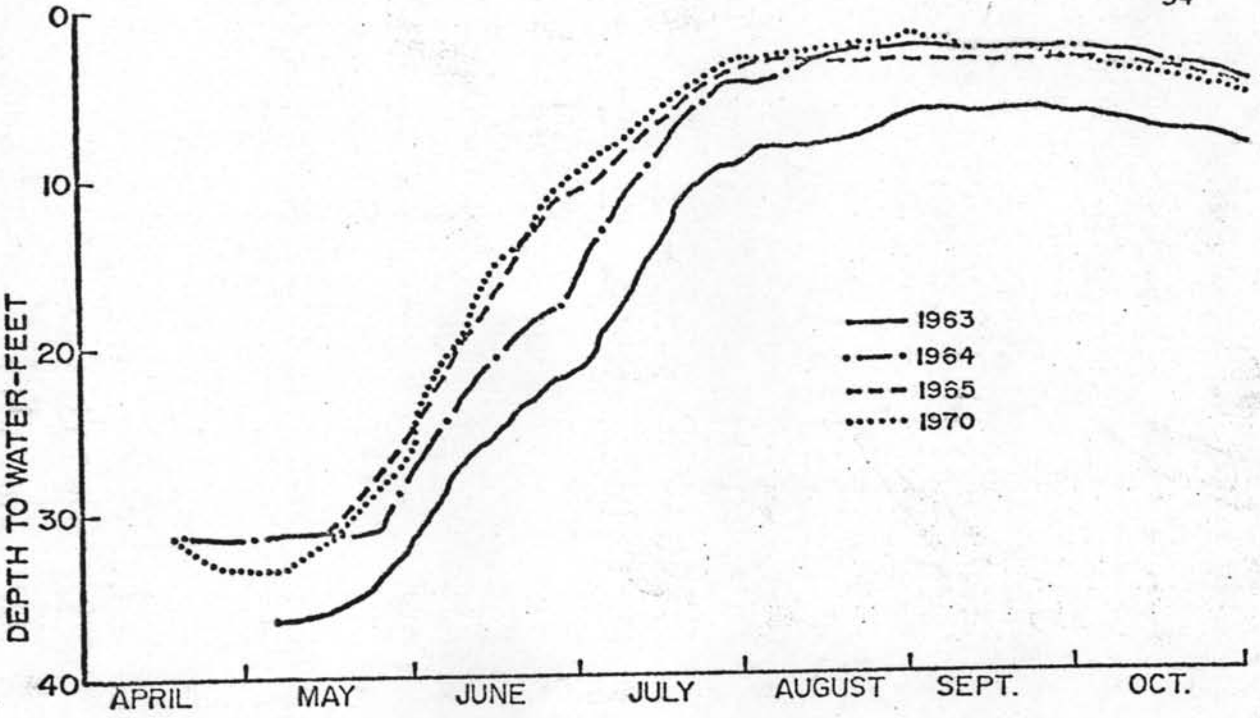

Figure 8. The Hydrograph of the Clark Well Northwest of Rigby for Several Seasons, Beginning with 1963.

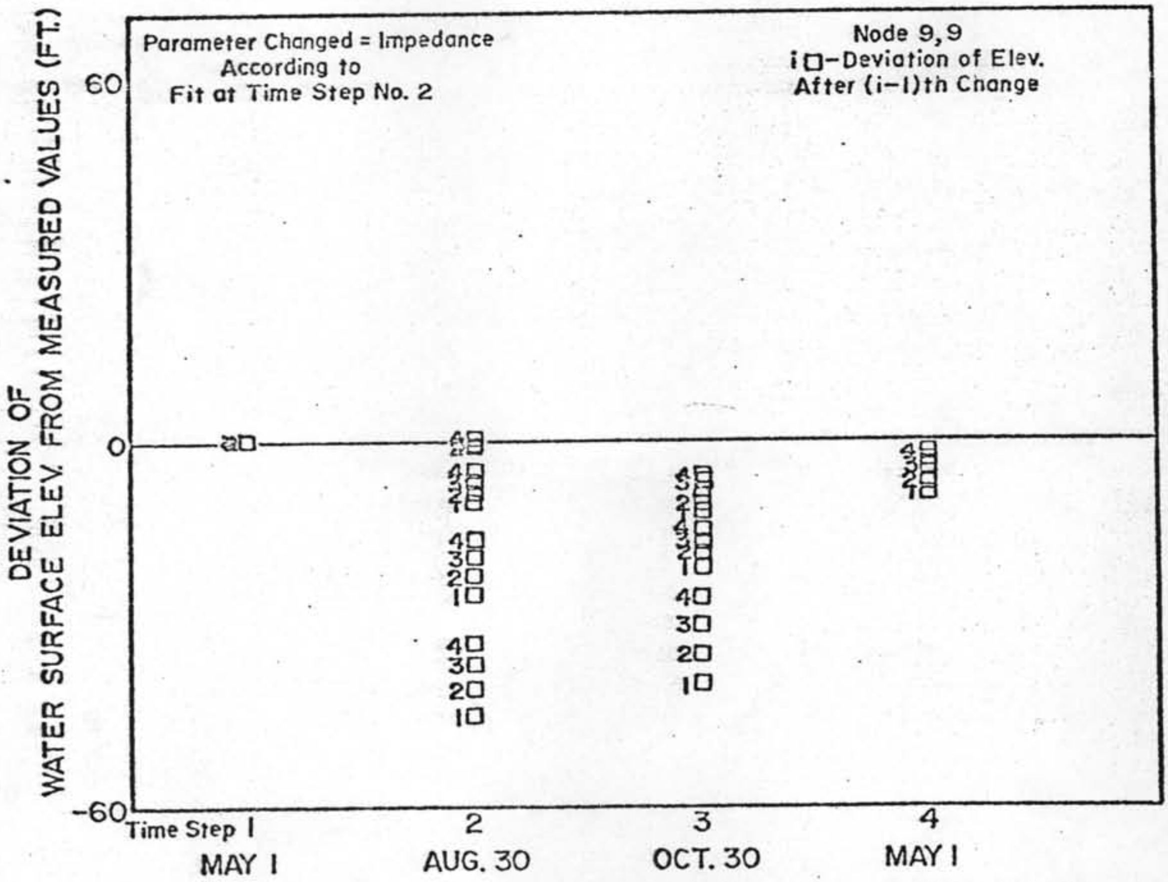

Figure 9. Deviations of Calculated from Measured Head Values at Four Time Steps During Four Calibrations.

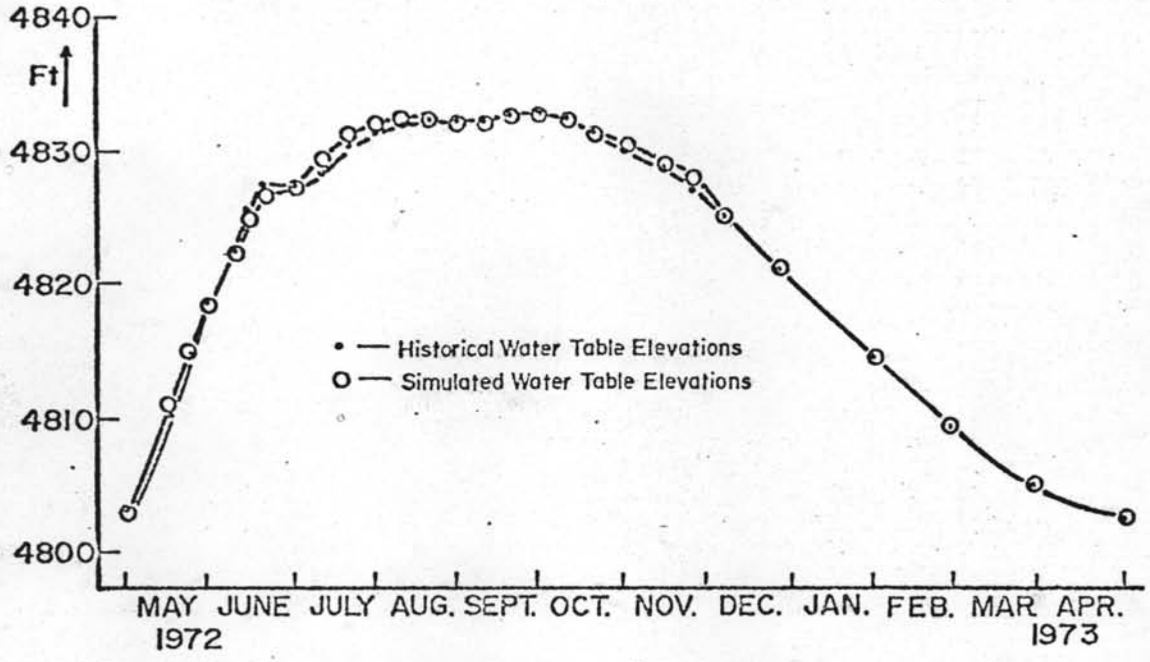

Figure 10. Comparison of Historic Well Hydrograph and Simulated Water Table Elevations at Node Point (9,9) - Clark Well.

Response to Water Management Changes

For every management alternative, the data for the input program were changed accordingly. Inputs to the groundwater model are the final calibrated values of the geohydrological parameters, the initial head values of May 1, 1972 and the magnetic tape which contains the external source term for every node point for each half timestep.

The model, with these inputs, calculates new head values for all timesteps in the yearly cycle and prints out the deviations of the management calculated water table elevations from the 1972 water table elevations at three selected timesteps for every node point.

The deviations of calculated from historical head values at the three selected timesteps are transferred to a subroutine that generates a contour plot of the deviations on microfilm. The selection of specific reasonable management alternatives was made utilizing information about the study area and the suggestions of the local people.

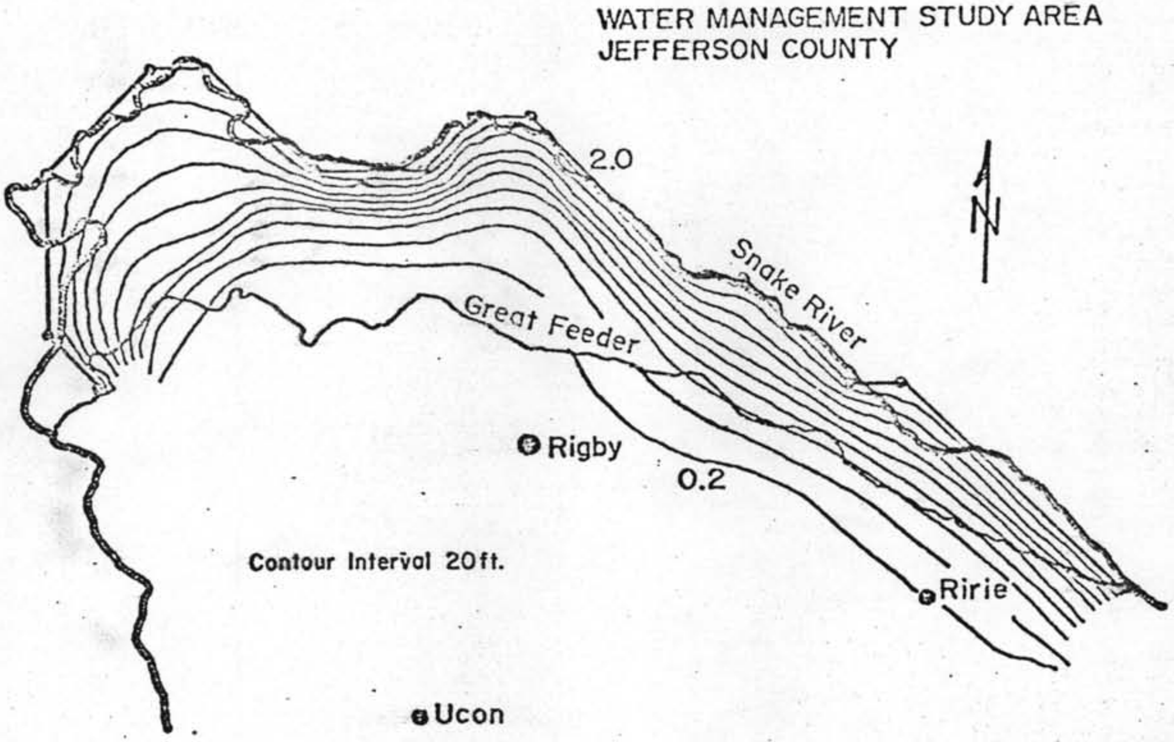

Figure 11. Contours of Reduction from the Maximum Water Table Condition, August 1972, With 2 Ft. Reduction of River Stage - Alternative 1.

<u>Alternative 1</u> - Two-foot reduction of Snake River Water Levels

The survey questionnaire of local residents indicated that the possible cause of the water table problem was the high water level of the Snake River. This suggestion was investigated by modeling the Snake River at a level two feet lower than the 1972 actual level at all points of the river during the total yearly cycle.

The model run shows a lowering of the maximum water table of about one foot at one mile distance from the river and about 0.5 foot at two miles distance from the river. A contour plot of deviations from historical maximum water table elevations is shown in Figure 11. Except for an influence strip parallel to the Snake River, the calculated maximum water table equals the historical maximum water table.

Alternative 2 - Lining of all Canals

The dense network of irrigation canals constructed in coarse gravels amount to 717 acres of canal with an average seepage rate of 3.5 ft/day. Seepage amounts to 501,550 ac ft or 6.1 ac ft/ac over the area and is a major contributing factor to the high water table problem.

Alternative 2 involves the lining of canals of all irrigation districts in the Snake River Fan, to determine the relative contribution of canal seepage to the water table rise in the aquifer. A considerable lowering of the maximum water table over the entire area as compared to the historical maximum occurs. Three to four foot decreases occur in the area north of the Great Feeder Canal. In the vicinity of Rigby the water table is 10-15 feet lower and near Ririe about 12 feet lower. Figure 12 is a contour plot of deviations from historical maximum water tables and shows clearly the overall decline due to the lining of canals. Where the water table of the study area connects with the regional water table of the Snake Plain aquifer (the southwest boundary of the model) the water table is 7 feet lower. Figure 13 is a contour plot of deviations from the minimum historical water table at the end of the simulation before the start of the new irrigation season. The calculated water table is lower than the minimum historical water table. The elimination of seepage reduces the water applied to the area by 44% or 501,500 ac ft. Because of this large reduction there is less recharge to the water table and the water levels at the end of the simulation follow part of a recession curve lower than the historical recession curve before rising again as a result of the irrigation in the next season. This lower minimum water table influences the maximum water table of the next year and is investigated in management alternatives 11 and 13.

Alternative 3 - Lining of Canals near Ririe

The area around the city of Ririe has a high water table problem, partly caused by the seepage of a dense network of irrigation canals that originate from the Snake River. To achieve local relief of the high water table at Ririe a solution may be the lining of all canals near the

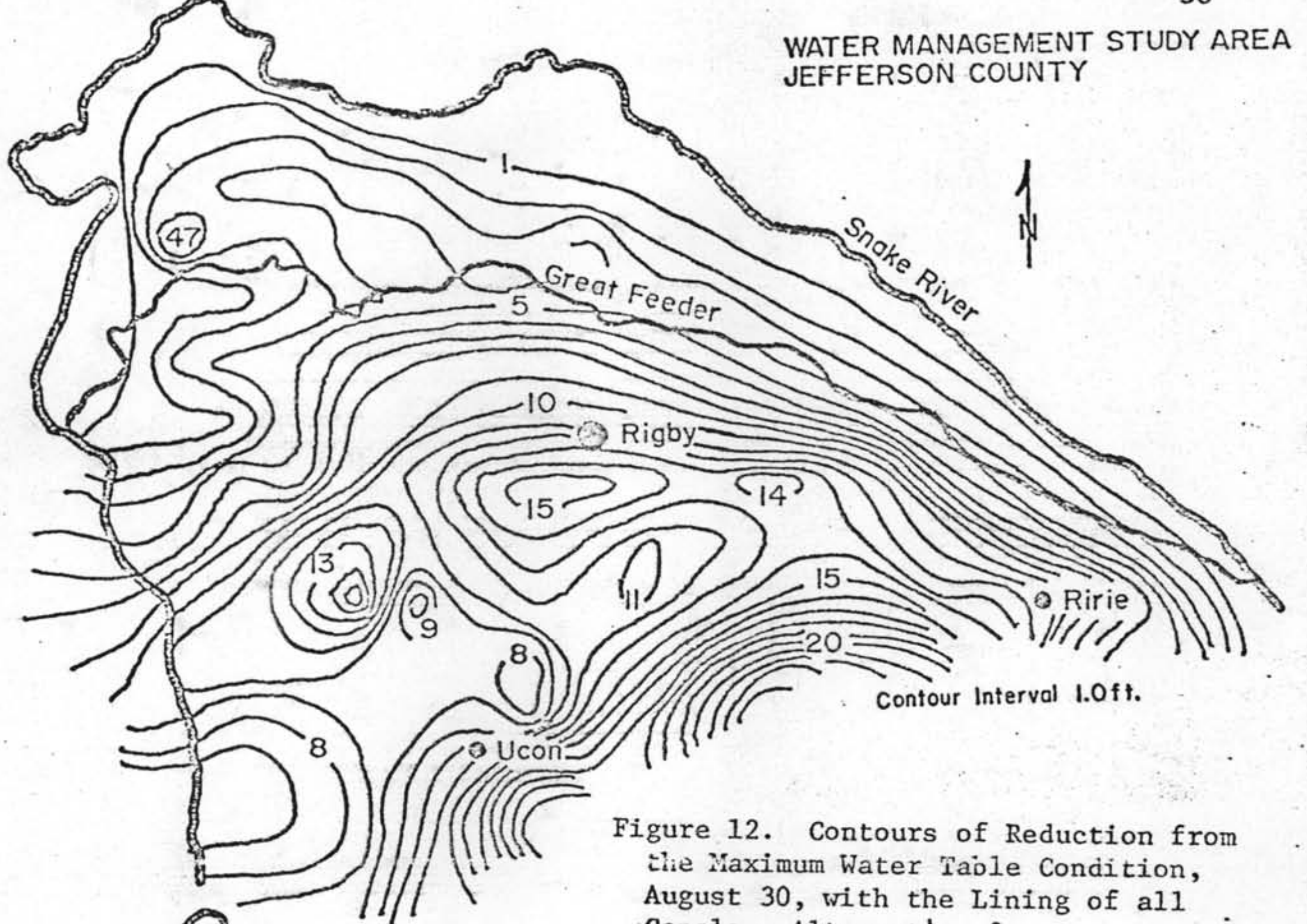

Figure 12. Contours of Reduction from the Maximum Water Table Condition, August 30, with the Lining of all Canals - Alternative 2.

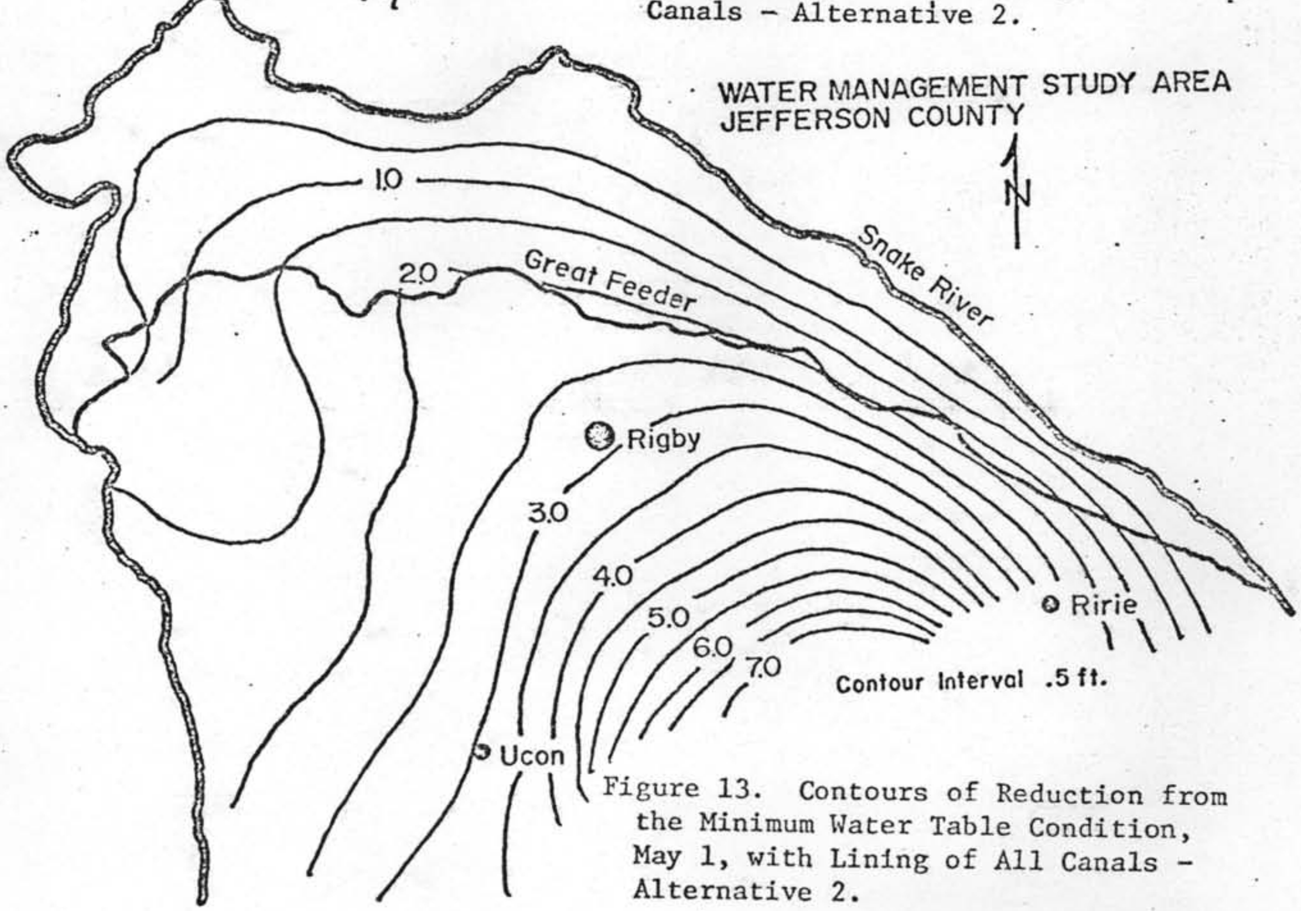

Figure 13. Contours of Reduction from the Minimum Water Table Condition, May 1, with Lining of All Canals - Alternative 2.

city; these are the canals in the sections 32 through 36 of T 4N, R 40E and sections 1, 2, 5 of T 3N, R 40E.

This alternative results in a calculated water table that is 10 feet lower than the historical maximum at the location of Ririe while the influence of the canal lining stretches out about three miles in any direction. Figure 14 represents a contour plot of deviations from historical maximum water table elevations under such a management alternative. From the absence of contour lines of deviations in most of the area it can be seen that the maximum water table elsewhere in the study area is not affected by this management alternative.

Alternative 4 - Nine, 10 cfs wells near Rigby

The Lewisville-Rigby area is the primary area where the water table rises to within a few feet of the land surface causing problems to the residents. To achieve local relief several suggestions were made. One way to take excess water out of the system is to introduce a series of relief wells located on a straight line running west to east one mile north of Rigby. For this alternative 9 simulated wells pumping 10 cfs each are located respectively in sections 12 of T 4N, R 37E, Sections 7-12 of T 4N, R 38E, and Sections 7 and 8 of T 4N, R 39E. For a 200 day season, starting May 1, the total water removed amounts to 36,000 ac ft.

In the immediate vicinity of the wells the maximum water table is effectively lowered between 5 and 7 feet. At one mile distance from the wells the water table is approximately 2.7 feet lower while no appreciable decline is observed in the area more than three miles away from the wells. Figure 15 is a contour plot of deviations from the historical maximum water table elevations. This alternative results in a 2.3 foot lowering of the maximum water table in the city of Rigby.

Alternatives 5 and 6 - Twenty and forty cfs wells near Rigby

Every year in the middle of the irrigation season a gravel pit, situated 1 mile west of Rigby is filled by the rising water table. To relieve the high water table problem in the city of Rigby a 5 cfs capacity

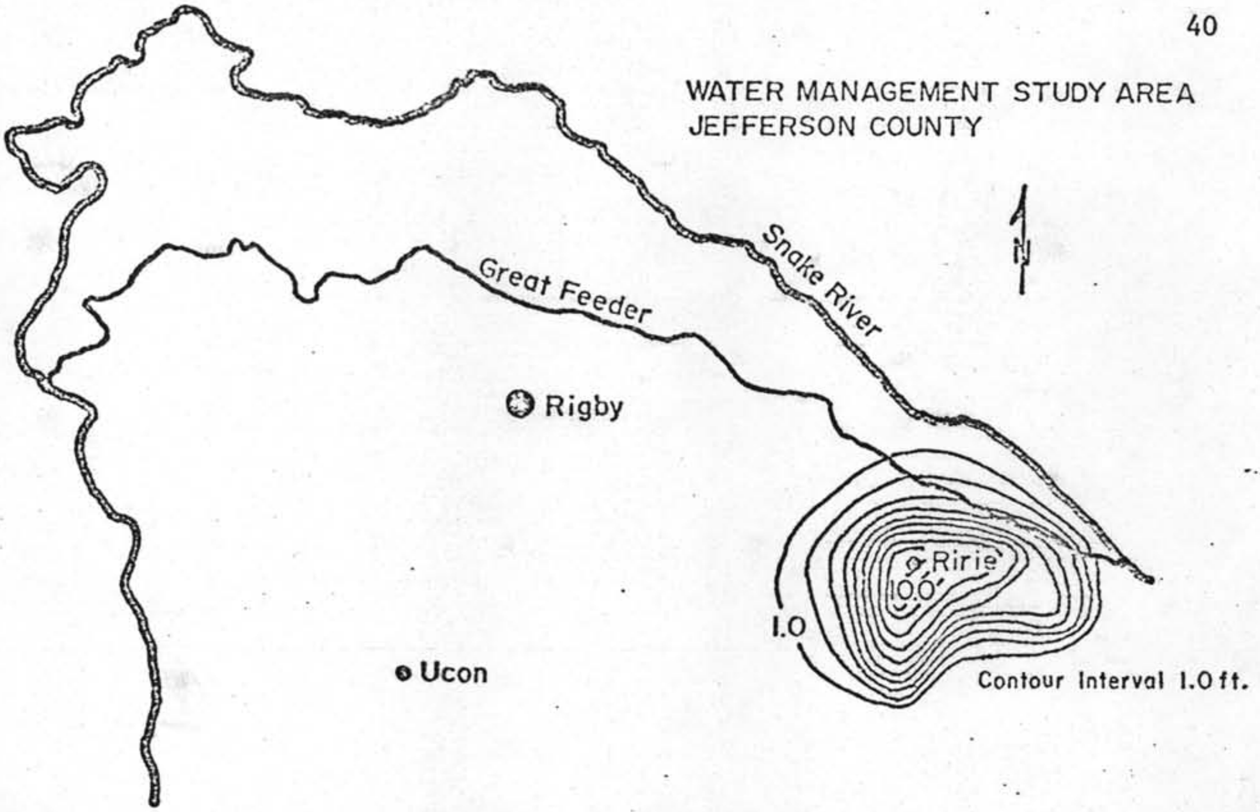

Figure 14. Reduction in Historical Maximum Water Table Levels Near Ririe, August 30 Conditions, With Lining of Canals Near Ririe - Alternative 3.

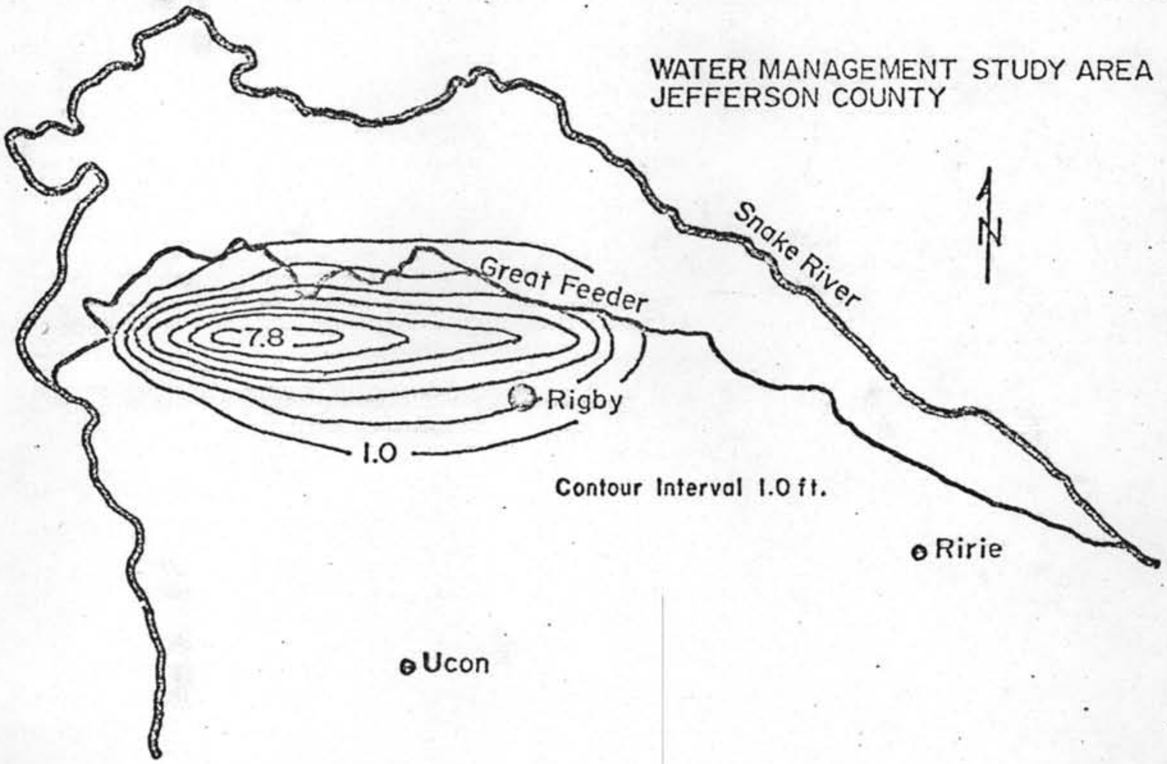

Figure 15. Reduction in Historical Maximum Water Table Levels Near Rigby, August 30 Conditions, with Nine 10 Cfs Wells East-West of Rigby - Alternative 4.

pump was installed at the site of the gravel pit, however with this capacity no noticeable lowering of the water table in the gravel pit could be achieved. In order to determine the required capacities sufficient to obtain a noticeable drawdown, two alternatives were run involving the introduction of a 20 cfs and 40 cfs drainage well respectively.

The model run with the 20 cfs well, pumped for 200 days (8,000 ac ft) shows an effective lowering of the water table of 7 feet below the historical water table in the immediate vicinity of the well. One mile away from the well (Rigby) the water table is only 2 feet lower than the historical maximum. A 40 cfs well,pumped for 200 days (16,000 ac ft) lowers the water table effectively 14 feet below the historical maximum water table in the immediate vicinity of the well and 4 feet at the center of Rigby. These two runs confirm the additivity of the computed well drawdowns. Figure 16 is a contour plot of deviations from the historical maximum water table for the 40 cfs capacity well.

Alternative 7 - Four 10 cfs wells northeast of Rigby

Another possible method of lowering the water table at Rigby was considered which involves the installation of four drainage wells of 10 cfs capacity each located north and east of Rigby in the center of sections 7, 8, 17 and 20 of T 4N, R 39E. The drainage wells were pumped for 200 days amounting to 16,000 ac ft.

Compared to the historical maximum water table, the water table at the well sites is effectively lowered 5 feet. In Rigby the decrease is 3 feet. A contour plot of deviations from historical maximum water table elevations is shown in Figure 17. Except for the area around the city of Rigby the maximum water table elsewhere is not affected by this management alternative.

Alternative 8 - Lining the main stem of Burgess Canal

The Burgess canal is a major irrigation canal that has its course through a large part of the area with high water table problems. Opinions of local residents resulted in the idea that the seepage from the

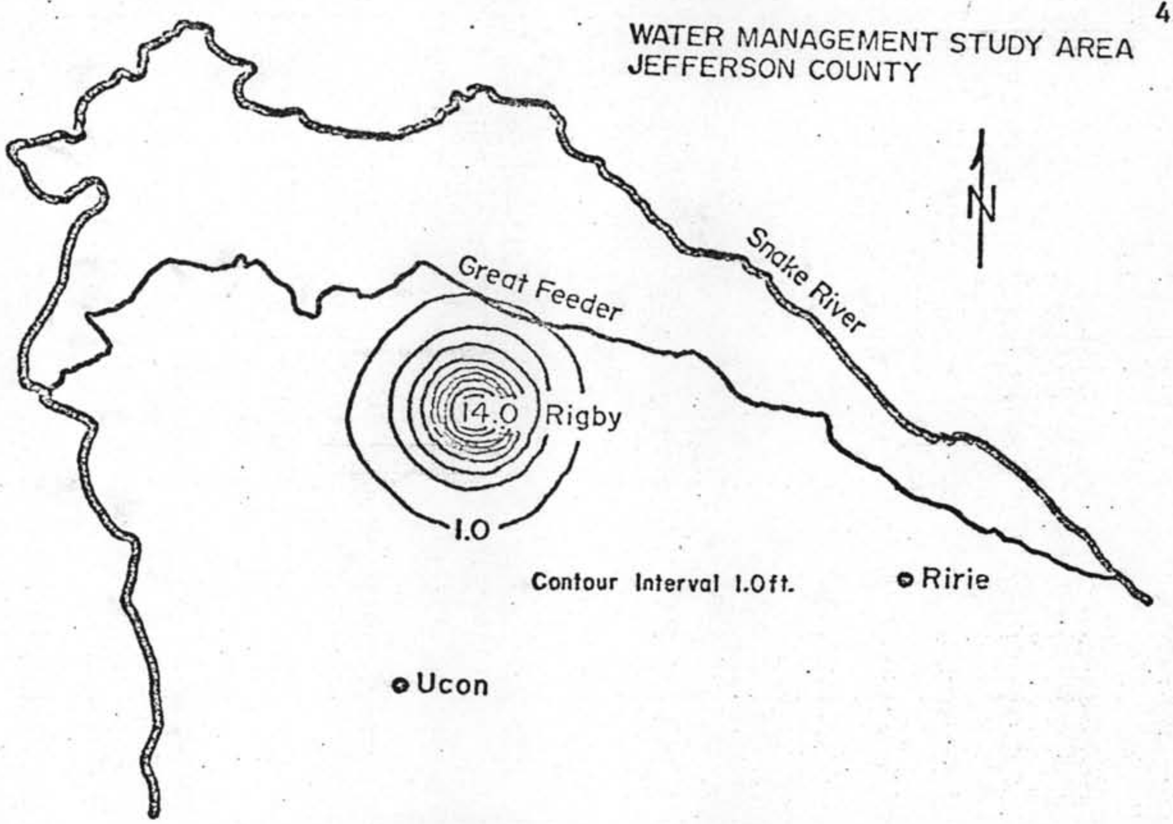

Figure 16. Reduction in Historical Maximum Water Table Levels Near Rigby, August 30 Conditions, with 40 Cfs Well Near Rigby - Alternative 6.

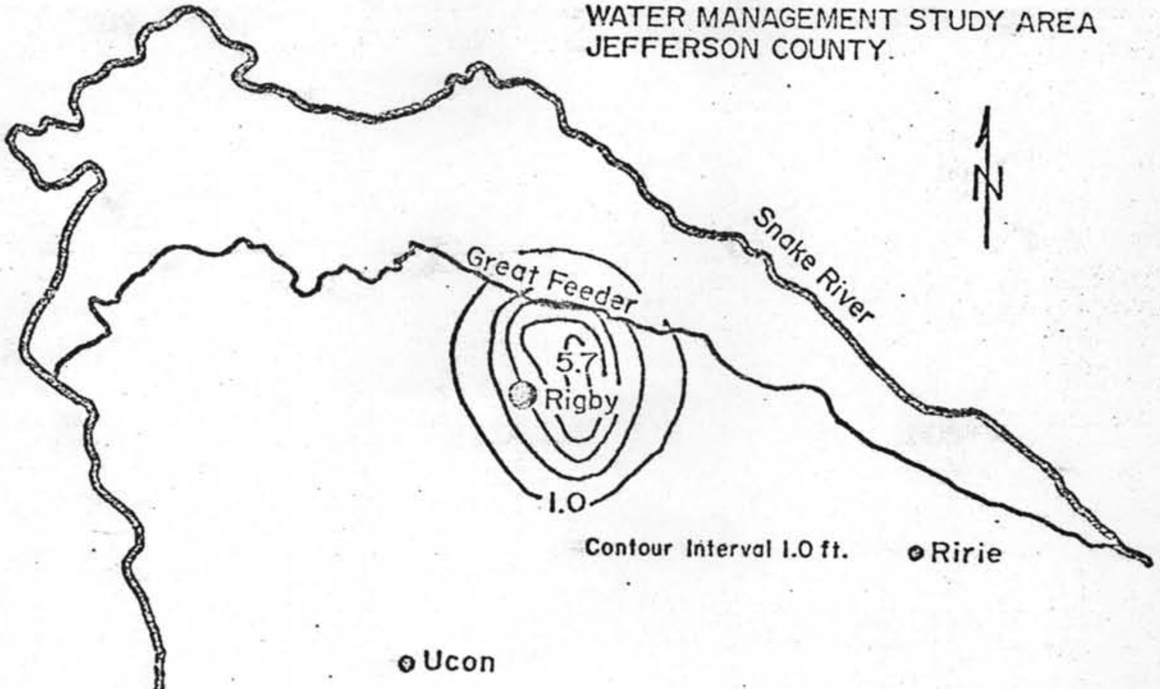

Figure 17. Reduction in Historical Maximum Water Table Levels Near Rigby, August 30 Conditions, with Four 10 Cfs Wells NE of Rigby - Alternative 7.

Burgess Canal is causing the water table problems. This became clear when local government officials contemplated suing the Burgess Canal Company for causing the high water table problem.

This alternative suggests the lining of the main stem of the Burgess Canal while other canals remain unchanged. The lining of the Burgess represents an 11% reduction of total seepage from irrigation canals in the study area or 55,000 ac ft.

In the immediate location of the Burgess Canal, the water table maximum decreases 8 feet and in the area immediately surrounding the canal decreases range between 2 and 4.5 feet. At the city of Rigby, less than one mile north of the Burgess canal the maximum water table elevation was decreased by 4.5 feet. Elsewhere no lowering of the maximum water table is observed. Figure 18 is a contour plot of deviations from historical maximum water table elevations resulting from lining the main stem of the Burgess Canal.

Alternative 9 - Surface drain near Rigby

Another method of alleviating the high water table problem in the Rigby-Lewisville area may be the introduction of an open surface drain. Drains have been proposed for the area in the past and one land reclamation project using open drains has been constructed. Since this proposal is regarded by many as one of the feasible solutions, the model was run with a drain installed at a level approximately 4.2 feet below the maximum water table elevation. The drain extends from Rigby 4 miles in a westerly direction. In this stage of the research calculation procedures required that the drain is installed in the most efficient way so that the rising water table will intersect the drain over the whole length at approximately the same time. The drain operates as a constant head any time that the average water table elevation in the area is equal to or greater than the average stipulated elevation of the drain.

Result of this management alternative show that at the location of the drain the maximum water table is reduced by 4.2 feet. At one mile distance from the drain the average decrease is 2.0 feet and at Rigby the drain results in a 3.1 foot lowering of the maximum water table. At a

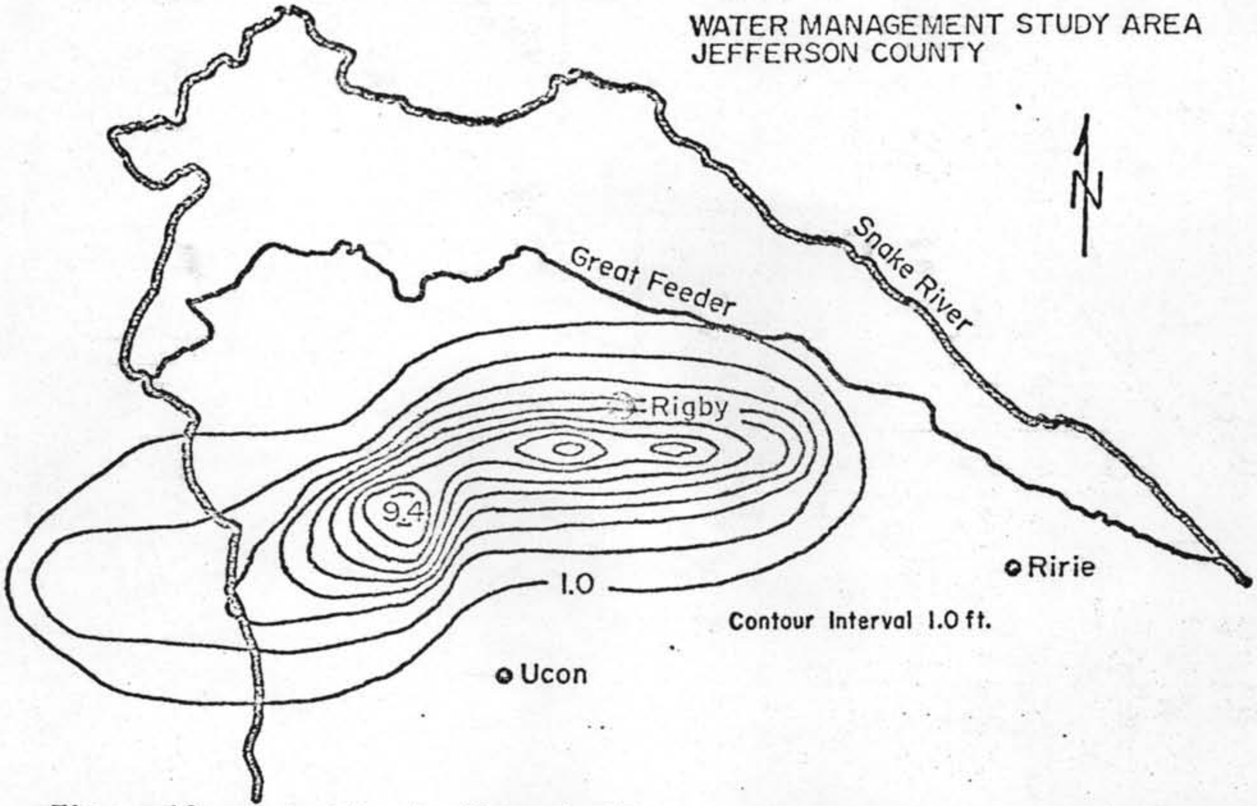

Figure 18. Reduction in Historical Maximum Water Table Levels in the Area, August 30 Conditions, with Lining Main Stem of Burgess - Alternative 8.

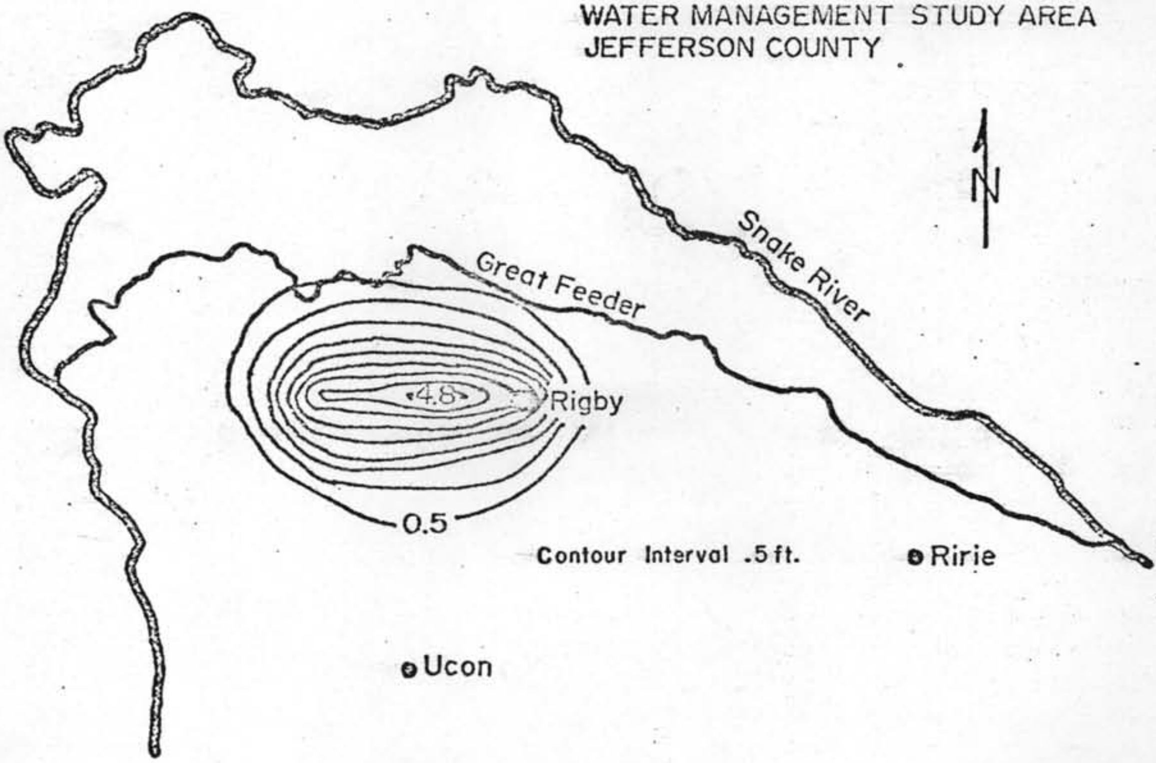

Figure 19. Reduction in Historical Maximum Water Table Levels near Rigby, August 30 Conditions, with Drain East-West of Rigby - Alternative 9.

distance of 3 miles from the drain no decline of the water table is observed. Deviations from historical maximum water table elevations as a result of the drain are shown in Figure 19. Figure 20 compares the hydrograph of a well at the drain site for the historical 1972 season with the water table levels calculated at the same location with the drain installed.

Alternative 10 - Thirty percent reduction in net diversions

The high water table problem is primarily caused by the excessive amount of water applied to the study area. Nearly all water applied originates as irrigation diversions from some 20 irrigation districts. The total net diversion (gross diversion minus return flow) amounted to an average of 13.8 ac ft/ac in 1972. Of that amount 6.1 ac ft/ac was seepage from the irrigation canals. The net irrigation application is 7.7 ac ft/ac. This alternative involves a 30% reduction of the net diversions for all irrigation districts. Assuming that the seepage (6.1 ac ft/ac) from the irrigation canals remains the same, the 30% reduction of the net diversion causes a 52% reduction of net irrigation application from 7.7 to 3.6 ac ft/ac or 589,000 ac ft.

A decisive decline of the maximum water table elevation over the entire area is observed. The water table north of the Great Feeder is 3 to 7 feet lower than the historical maximum. Around the city of Rigby the water table is 10-13 feet lower and near Ririe about 8 feet lower. Where the water table connects with the regional water table of the Snake Plain a 9 to 12 foot decrease is observed. Figure 21 is a contour plot of deviations from historical maximum water table elevations for August 30 conditions. Figure 22 is a contour plot of deviations from the historical minimum water table as it occurs at the end of the simulation before the start of the new irrigation season (May 1). The calculated minimum water table is lower because with a 30% reduction in net diversions there is less recharge to the groundwater table. As was the case in management alternative 2, water levels at the end of the simulation follow a recession curve at a lower level than the historical recession curve before rising again as a result of the irrigation in the next season.

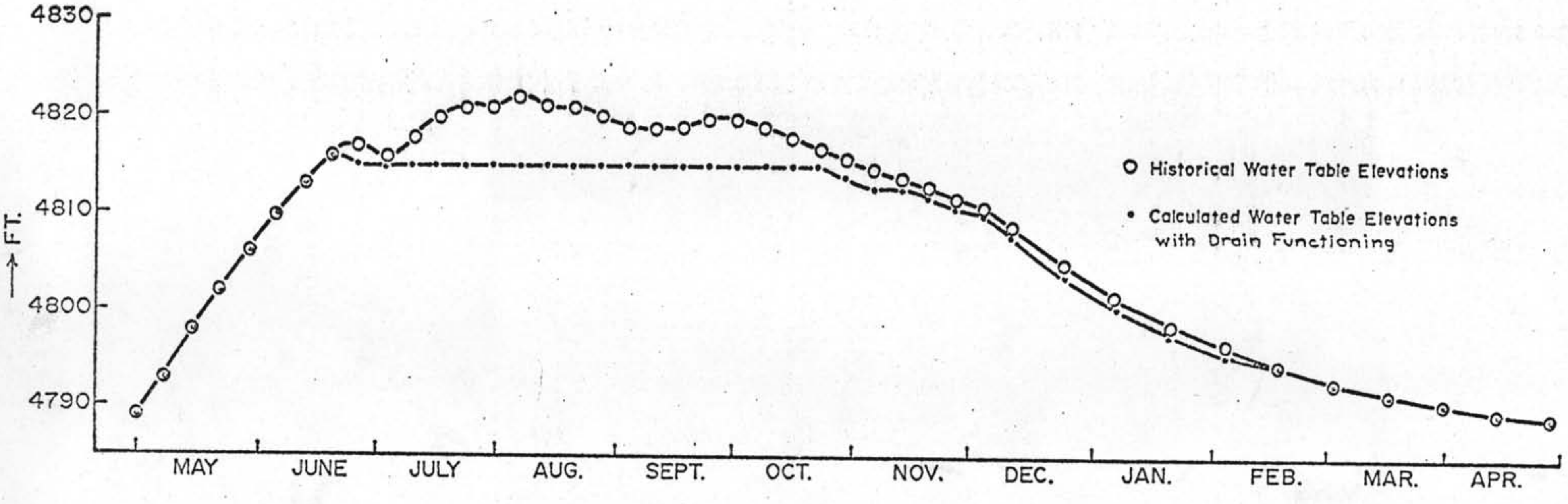

Figure 20. Water Levels at Surface Drain Site - Alternative 9.

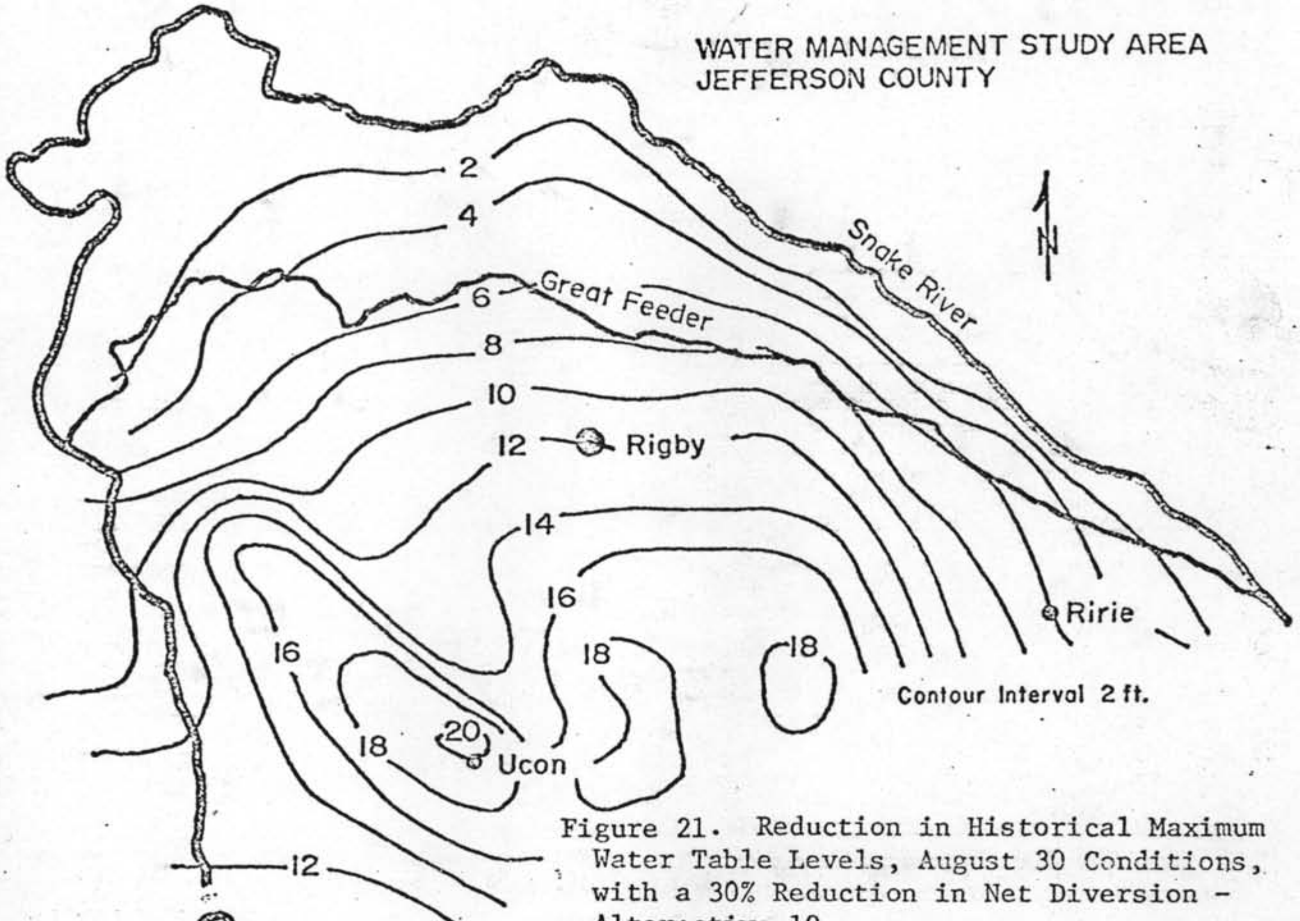

Figure 21. Reduction in Historical Maximum Water Table Levels, August 30 Conditions, with a 30% Reduction in Net Diversion - Alternative 10.

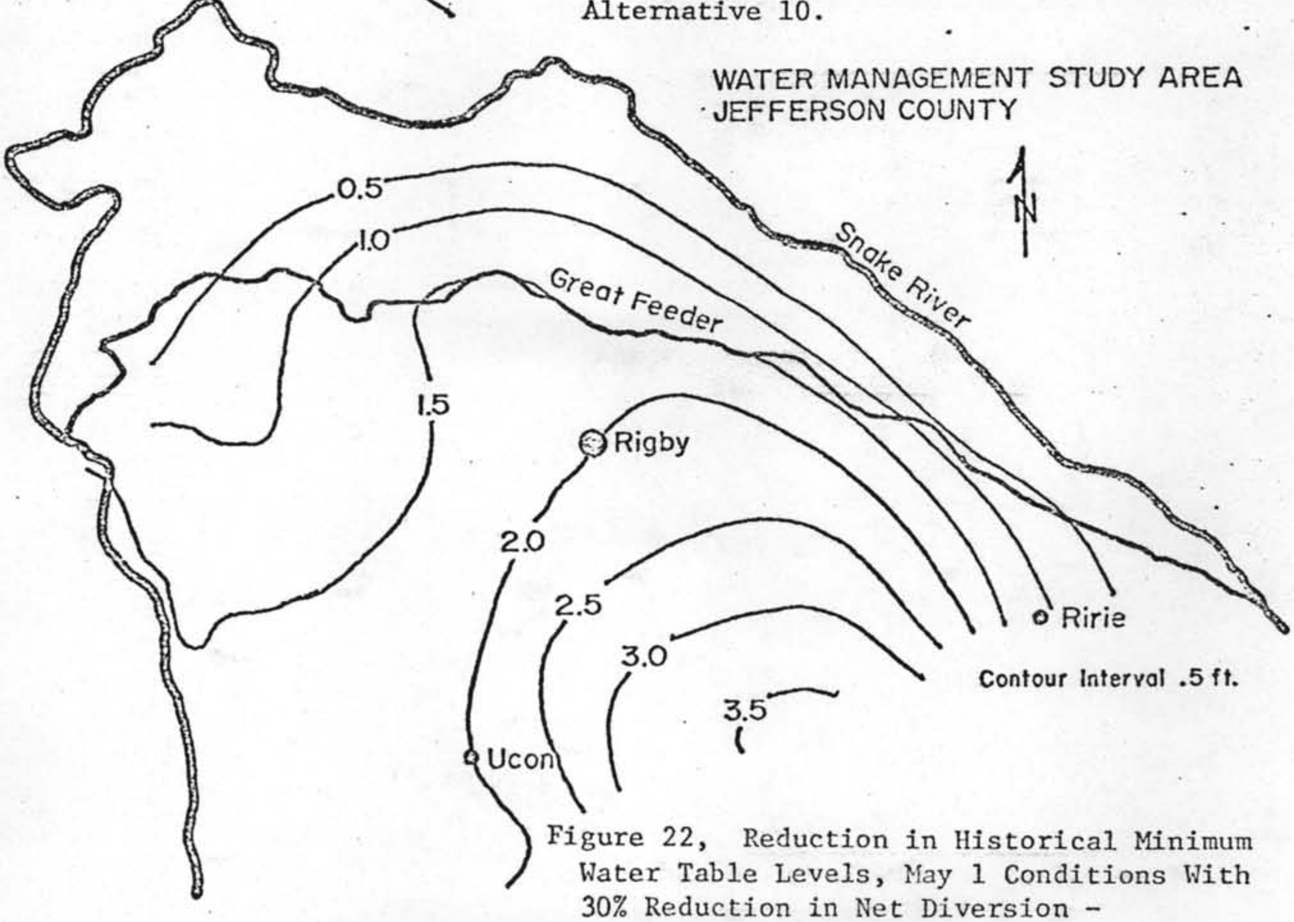

Figure 22, Reduction in Historical Minimum Water Table Levels, May 1 Conditions With 30% Reduction in Net Diversion - Alternative 10.

Alternative 11 - Twenty percent reduction in net diversion

Whether a smaller reduction of net diversion would also yield satisfactory results was investigated in this management alternative which involves a 20% reduction in the net diversion, again assuming that the seepage losses remain the same. This results in a 35% reduction of net irrigation application from 7.7 to 5.2 ac ft/ac, or 400,000 ac ft.

A considerable decline of the maximum water table over the entire area is evident. The area north of the Great Feeder averages a 2 to 5 foot decrease from the maximum water table. Around the city of Rigby the maximum water table is 8 - 9 feet lower and near Ririe about 6 feet. Where the water table connects with the regional water table of the Snake Plain, a lowering of the maximum water table between 6 and 8 feet occurs. Figure 23 is a contour plot of deviations from maximum historical water table elevations, and shows clearly the overall decline of the maximum water table. Figure 24 represents deviations from historical minimum water table and shows lower water table elevations as a result of the lower recession curve. The model was run for 5 consecutive years with a 20% reduction in net diversion to determine the effect of lower water tables at the beginning of each season on the maximum and minimum water table elevations. After 5 years, the maximum water table declined to an equilibrium value which is less than one foot below the value at the end of one year and then remains essentially at the same level, in equilibrium with the reduced input to the aquifer.

Alternative 12 - Sprinkler Irrigation

Considering the general soil type and topography condition in the study area, the most efficient type of irrigation for the Snake River Fan is sprinkler irrigation. A simulation run was made in which the entire area is irrigated with sprinklers with a 70% efficiency factor. A closed delivery system was assumed so that farm conveyance losses were eliminated. The average evapotranspiration was taken as 2.0 feet. With 70% efficiency, only 0.85 ac ft/ac (including precipitation) is added to the groundwater aquifer via deep percolation instead of the present 11.8 ac ft/ac.

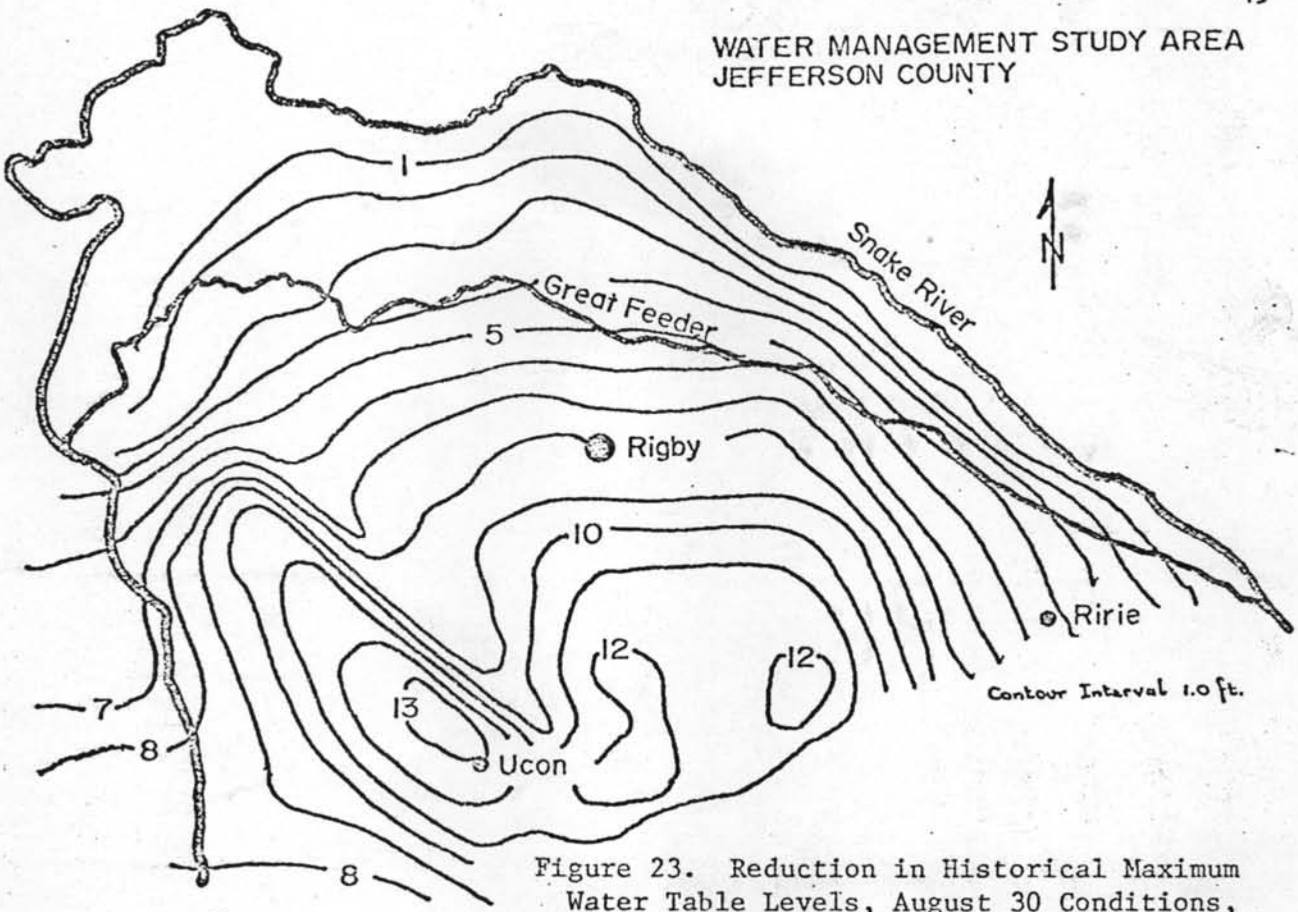

Figure 23. Reduction in Historical Maximum Water Table Levels, August 30 Conditions, With a 20% Reduction in Net Diversion - Alternative 11.

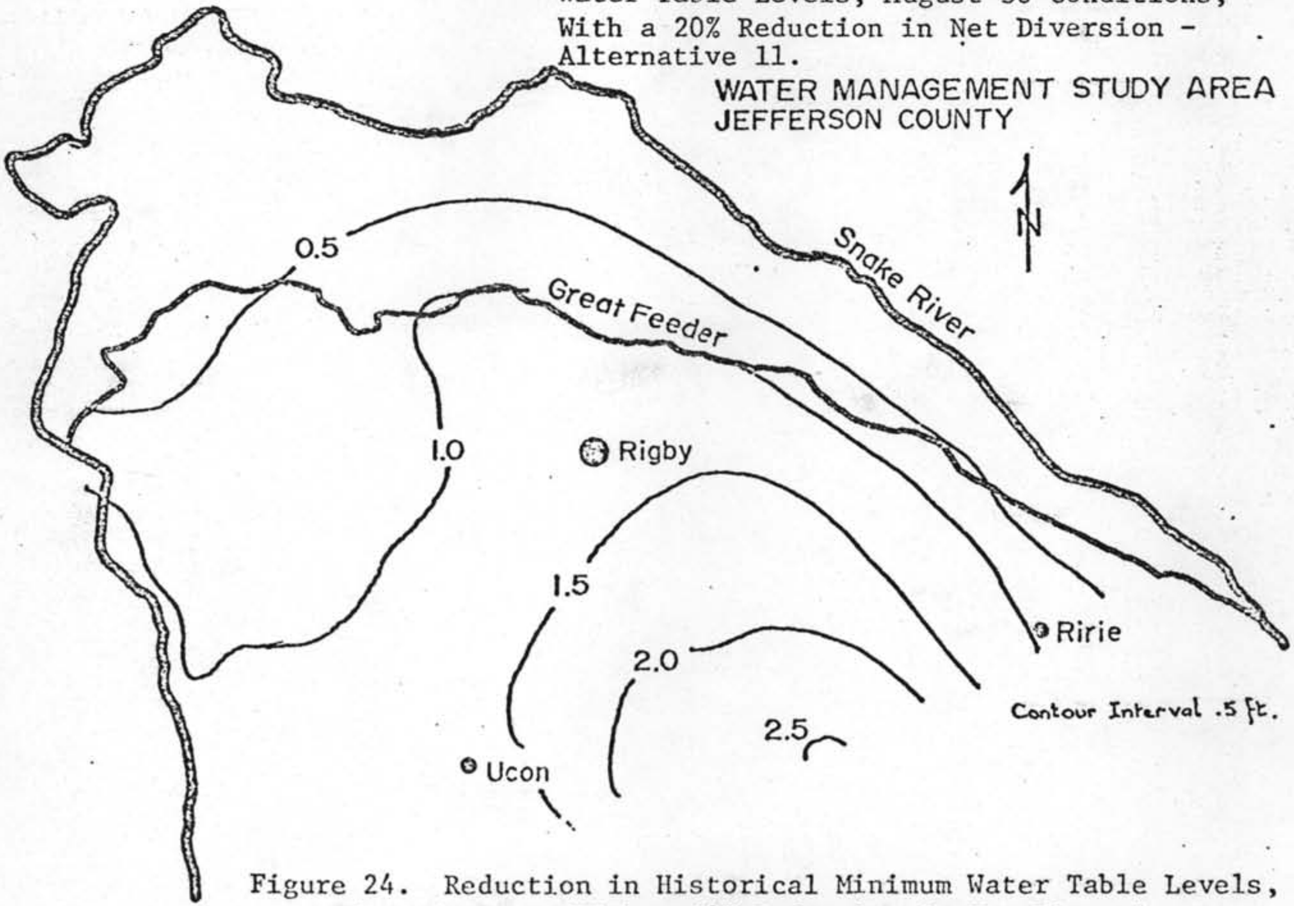

Figure 24. Reduction in Historical Minimum Water Table Levels, May 1 Conditions, With a 20% Reduction in Net Diversion - Alternative 11.

Historical water table rises of 35 to 40 feet were reduced to 3 and 5 feet, respectively. Where the water table connects with the regional water table of the Snake Plain, the maximum water table is 20 feet lower than the historical maximum.

The minimum water table at the end of a one-year simulation is between 3 and 5 feet lower than the historical minimum water table. These levels approach the minimum value of the groundwater recession curve since little groundwater recharge exists with this management alternative. Figures 25 and 26 are contour plots of deviations from the maximum and minimum historical water table elevations respectively.

Alternative 13 - Long term recession curve

Under the existing water management procedures on the Snake River Fan the minimum water table level occurs just before the beginning of the irrigation season about May 1st. There are numerous shallow domestic wells in the area. Some concern exists that these wells will run dry if the water table is not recharged annually by the deep percolation of irrigation water. Without this recharge the water levels in the fan are expected to follow a depletion curve until a steady state is reached in which the inflow from the Snake River, Great Feeder and some valley subsurface flow equals the flow out of the area to the regional water table of the Snake Plain. To determine the depletion curve and the equilibrium water table of the Snake River Fan a simulation run was made in which after one season the only input to the area is seepage from the Snake River and the Great Feeder Canal.

The equilibrium water table is reached at a level averaging 5 feet below the historical minimum in the area around Rigby and 5 to 6 feet in the area north of the Great Feeder. In the vicinity of Ririe the minimum water table is 5 feet lower and where the local water table connects with the regional water table of the Snake Plain the levels are 6 to 7 feet lower. Since this boundary with the Snake Plain aquifer is influenced not only by the flow regime in the study area but also, in a lesser degree, by regional groundwater levels the minimum calculated water table may be conservative.

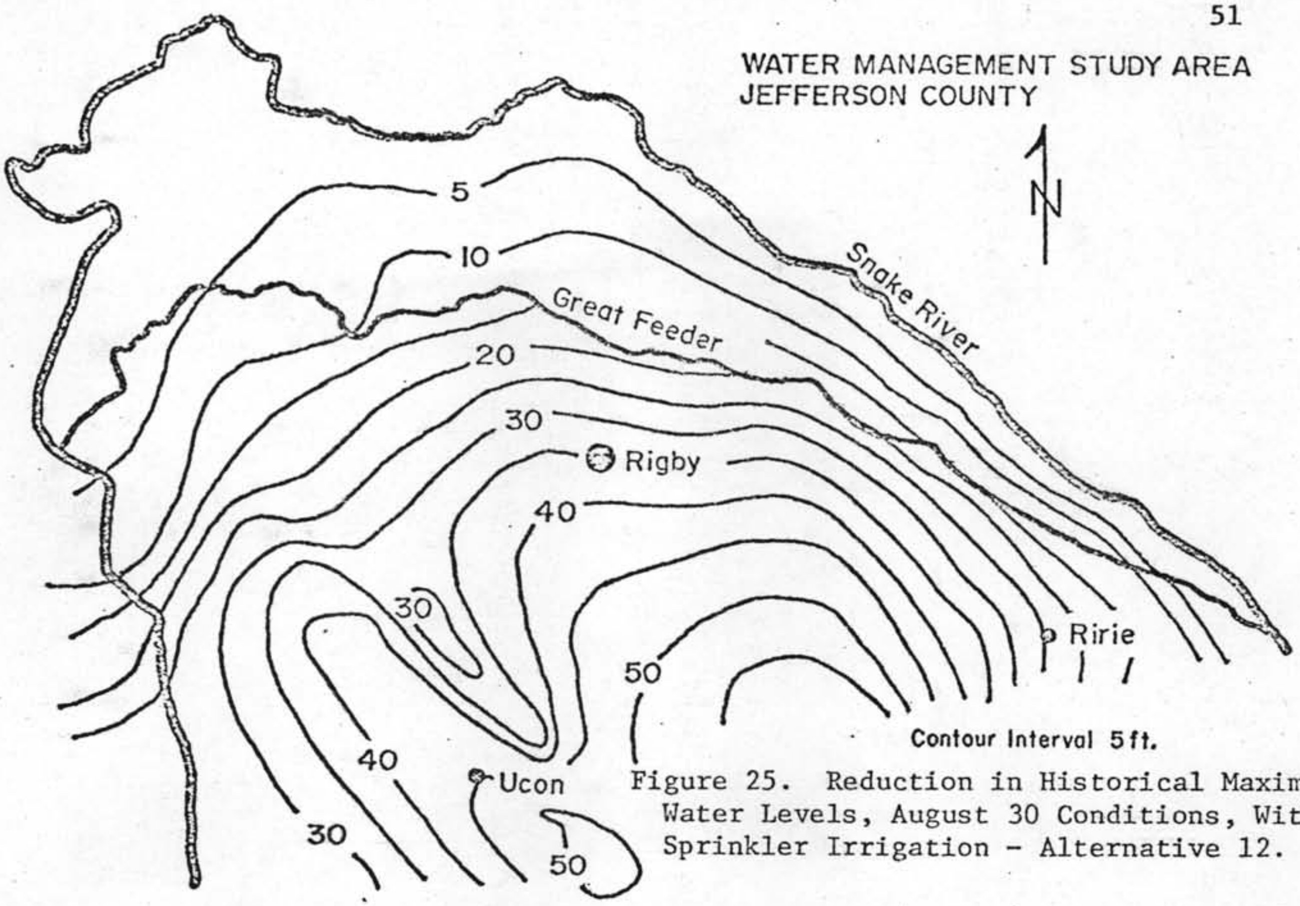

Figure 25. Reduction in Historical Maximum Water Levels, August 30 Conditions, With Sprinkler Irrigation - Alternative 12.

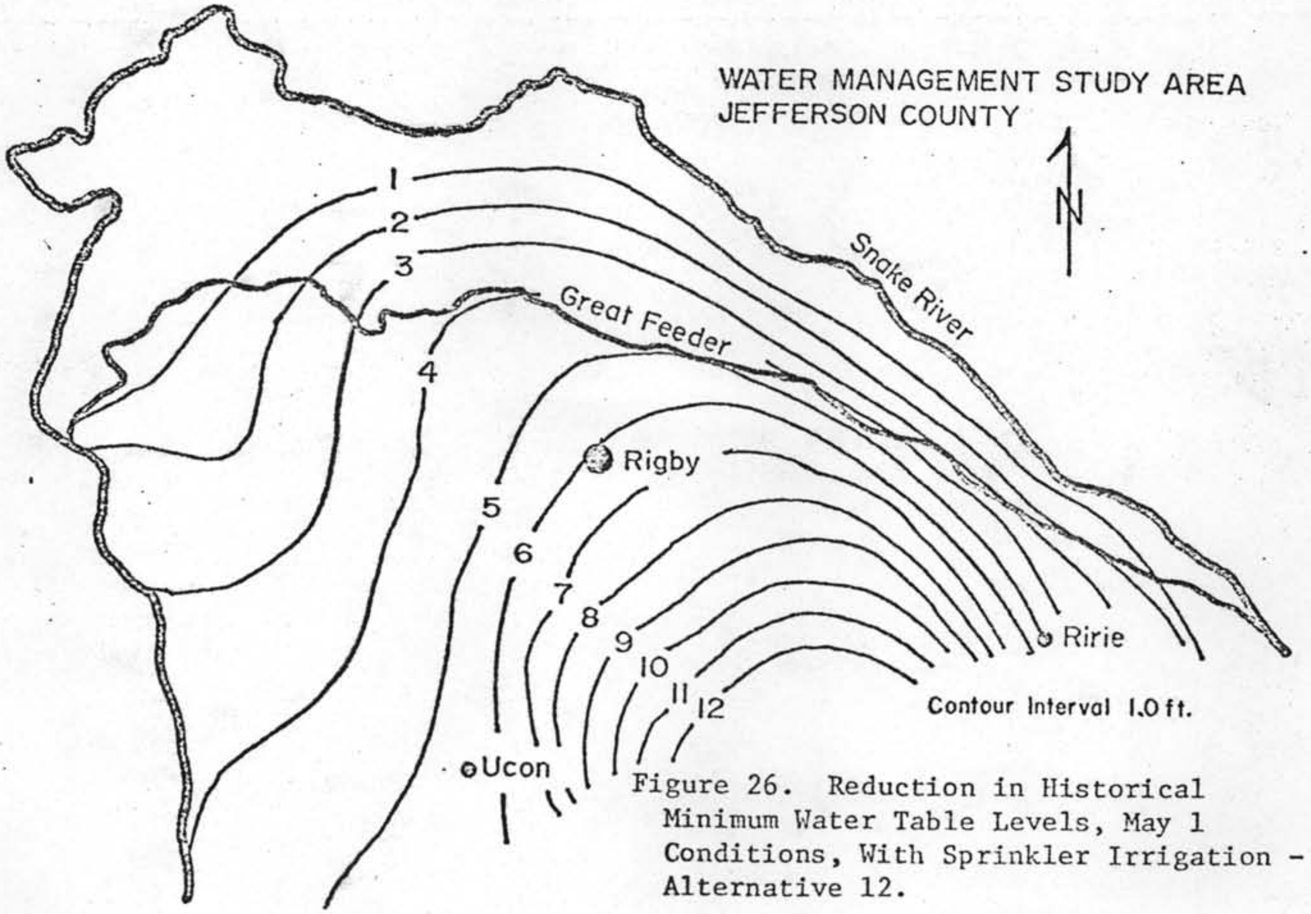

Figure 26. Reduction in Historical Minimum Water Table Levels, May 1 Conditions, With Sprinkler Irrigation - Alternative 12.

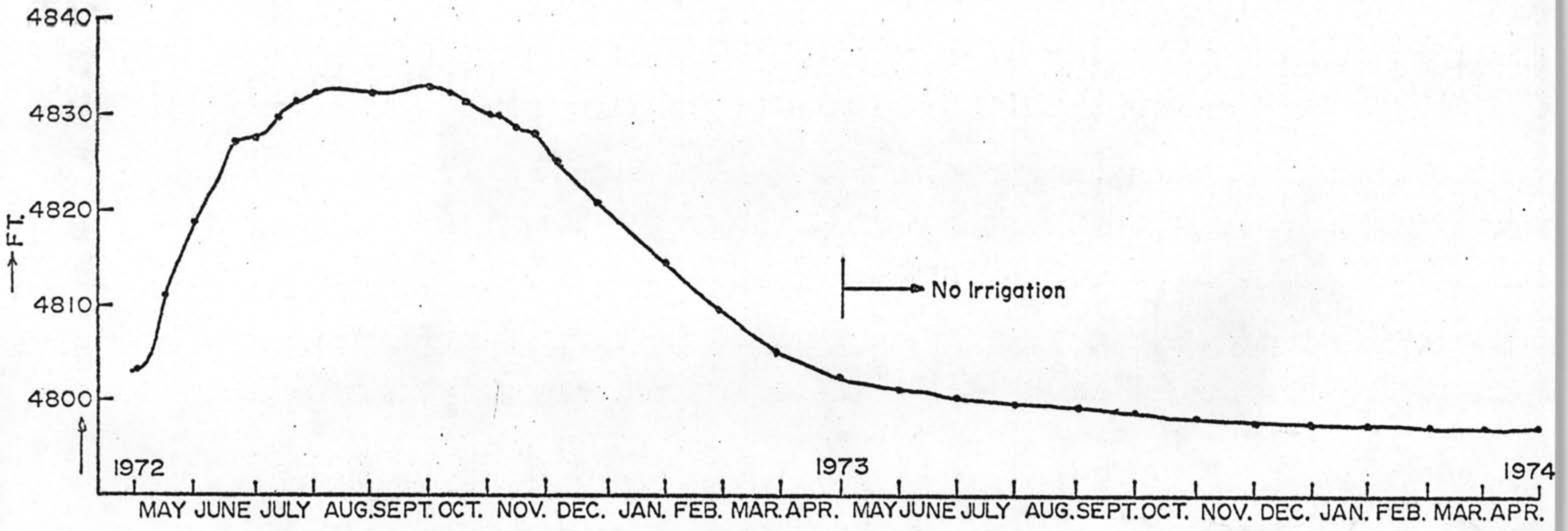

Figure 27. Simulated Seasonal Water Table and Recession Curve - Clark Well.

Figure 27 shows behavior of a representative well in the study area for the 1972 season and the computed recession curve and equilibrium water level resulting from a cessation of irrigation after one year.

Solutions to High Water Table Problems

Results of the management studies on the aquifer indicate that changes in the water surface elevation of the Snake River extended periods of time would not appreciable affect the aquifer and would not remedy the high water table problem in the Rigby area. Elimination of transmission losses by lining of all canals would reduce the maximum water rise 10 to 15 feet but may not be financially feasible. Selective lining of specific reaches of canals causes local reductions in the maximum water table rise. Local relief may also be achieved by well or well field operation; however, the quantities which must be pumped to achieve significant lowering of the water table are large and operation may not be economically feasible. Construction of an open drain near Rigby as has been proposed by local residents could lower the water table at Rigby by as much as 3 feet. Any drain constructed would necessarily be large because of the gravelly substrata and depth required to achieve significant lowering of the water table. It is estimated that to achieve the 3 foot reduction in water table elevation at Rigby a 100 cfs drain would be required, with an invert depth of 12 to 14 feet below ground surface.

The use of well and drains to remove water from the aquifer to alleviate the high water table problem are treatments of the symptoms and not the main causes of the problem. The most feasible solution is to reduce inputs to the aquifer, namely deep percolation from irrigation and canal losses.

A 20% reduction of the net irrigation diversion to the area would correct the high water table problem in both the Rigby and Ririe areas. Implementation of this reduction could be achieved either by system consolidation to reduce canal seepage, canal lining of specific reaches of canals, or decreasing of farm water use. Those parts of the area with shallow soils and high infiltration rates are most amenable to sprinkler irrigation. Conversion to closed system sprinkler irrigation for all or part of the area would solve the high water table problem. Minimum water table elevations under any alternative studied are not sufficiently decreased to jeopardize domestic well water supplies. The long term recession curve with zero input from irrigation indicates a lowering of the minimum water table of only 5 to 6 feet.

The effects of local changes of management in the Fan area on the regional water table of the Snake Plain aquifer were not the subject of this study.

Implementation of any alternative to alleviate the high water table problem in the study area will depend on the willingness of residents to cooperatively undertake a program and on the repayment capacity of the community to finance any venture.

CHAPTER V

UPDATING THE MODEL

Experience obtained from the application of the model to the Rigby area led to reconsideration of several program features.

Revision of the Flow Boundary

The alternating direction implicit method of solution for the finite differences equations that govern the groundwater flow requires two equations; the first equation implicit in the x-direction which is solved row by row, the second equation implicit in y-direction which is solved column by column. This leads to a system of tridiagonal equations in which the boundary equations have only two unknowns. The flow boundary as originally developed eliminated one unknown by assuming an approximative relationship between the average hydraulic gradients of consecutive half timesteps at the flow boundary nodes (Brockway, De Sonneville (1973). This method works satisfactorily if upper and lower limits for this function are introduced as well as limits for the head differences of succeeding node points close to the boundary.

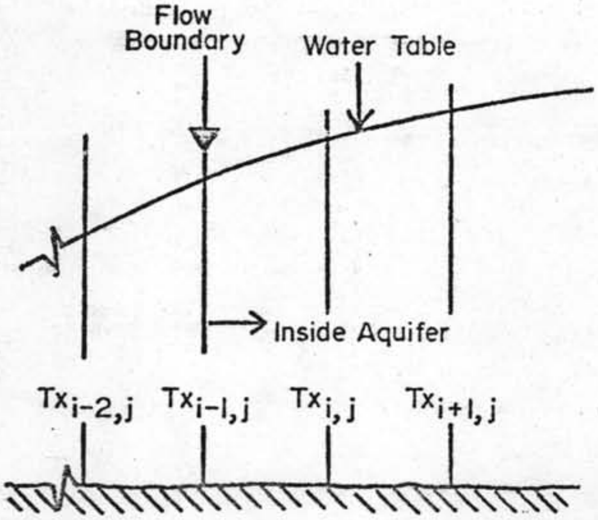

Figure 28. Representation of Flow Boundary.

Use of limiting factors reduces the variable gradient, variable head boundary to a boundary with a constant gradient in most unsteady flow conditions. Different treatments for the flow boundary were therefore developed that would eliminate the use of the imposed limits. Figure 28 is a schematic representation of the flow boundary, located at (i-1,j) with the interior nodes located at (i,j), (i+1,j)...... The head at (i-1,j) can be expressed in terms of the head values of the interior nodes by using forward differences techniques to approximate the differentials in the groundwater flow equation for that node. Solving implicitly for a row leads to

$$h_{i-1,j,k} = f \quad (h_{i,j,k}, h_{i+1,j,k}, h_{i-1,j+1,k-1}, h_{i-1,j,k-1}, h_{i-1,j-1,k-1}) \quad (24)$$

in which k is the present half timestep. For the interior node (i,j) the differential equation is solved using central differences leading to the generalized equation

$$A\, h_{i-1,j,k} + B\, h_{i,j,k} + C\, h_{i+1,j,k} = D \quad (25)$$

in which $D = g\ (h_{i,j+1,k-1}, h_{i,j,k-1}, h_{i,j-1,k-1})$ and

A, B and C are doefficients, substituted for all known terms. Substituting expression (24) into equation (25) leads to a boundary equation with two unknowns

$$B'\, h_{i,j,k} + C'\, h_{i+1,j,k} = D' \quad (26)$$

This procedure was tested for steady and unsteady state flow conditions. During the simulation the head values at the flow boundary diverged after a certain time period. Introduction of double precision for the variables increased the length of the stable period but head values still diverged. In other tests equation (24) was obtained by approximating the differentials of the flow equation with forward differences in which the truncation error is in the order of $0\{(\Delta x)^2\}$ instead of $0\{(\Delta x)\}$. This improved the calculation considerably but values still diverged if the simulation was run over an extended period of time.

In the next series of tests the head at the flow boundary was obtained in another way. Darcy's law and the continuity equation were applied to the flow boundary node (i-1,j) and the interior node (i,j), resulting in an equation replacing equation (24). This new expression substituted in equation (25) provided a boundary equation similar to equation (26), but with different coefficients B', C', and D'. This method proved to be stable. In tests involving a steady state condition the head at the flow boundary was stable. In tests involving an unsteady state condition in which nodes were either recharged or discharged the flow boundary would simulate the rise or decline of the water table very satisfactorily, however, in tests where the input was terminated after a specific period of time the head at the flow boundary node (i-1,j) would fail to return to its original starting value.

In the last series of tests a simplified expression was used, assuming a constant gradient at the flow boundary. Expression (24) then becomes;

$$h_{i-1,j,k} = h_{i,j,k} - (h_{i,j,1} - h_{i-1,j,1}) \tag{27}$$

in which $(h_{i,j,1} - h_{i-1,j,1})$ represents the gradient at the boundary which is obtained automatically from the initial water table configuration. Tests involving the constant gradient method resulted in a satisfactory steady and unsteady state simulation.

All tests were conducted in order to obtain a mathematically stable equation utilizing some varying gradient at the flow boundary to eliminate the six control variables necessary in the gradient-ratio method used in the model of the Rigby area. Several solutions were attempted. However, the only method resulting in a reasonable simulation without using control variables is the assumption of a constant (prescribed) gradient at the flow boundary.

The constant gradient flow boundary requires no input data to specify that gradient. The gradient for every node point along the flow boundary is obtained from the initial water table configuration. In this way it is possible for every boundary node to have a different constant gradient. The gradient may be changed by adjusting the initial hydraulic head values along the flow boundary for respective runs. This technique, which simplifies the input was tested successfully on the Snake River Fan aquifer and incorporated in the model.

Mass Balance Computation

The output of the model program consists basically of the numerical hydraulic head values for every half timestep on a grid point basis. To assist in adjustment of inputs and evaluation of the reasonableness of computed water table response a mass balance routine was incorporated in the model.

Inputs and outputs in a mass balance computation may be summed up as follows.

1. Input from irrigation diversion and irrigation canal seepage.
2. Input from precipitation.
3. Output from consumptive use.
4. Artificial recharge or withdrawal from well fields.
5. Reach gains or losses from streams and lakes perched above the aquifer (unsaturated flow).
6. Tributary valley underflow.
7. Flow across aquifer boundaries formed by streams and lakes hydraulically connected to the aquifer (saturated flow), $Q_h(i,j)$.
8. Flow across constant gradient boundary with variable head, $Q_h(i,j)$.
9. Leakage to or from an under or overlying water bearing formation, $Q_v(i,j)$.
10. Change in storage of water in the aquifer system (accretion term), $\Delta S(i,j)$.

The first six terms are generally known from field data or can be measured. They are calculated for every node point in a separate input program and stored under Q(i,j). Terms 7, 8, 9 and 10 are calculated implicitly in the model program.

Flow to Boundaries Formed by Hydraulically Connected Streams

Hydraulically connected lakes or streams provide one of the boundary conditions in the groundwater model known as the constant head boundary. If the hydraulic head along the stream or lake varies with time and is known from measurements, the 'constant' head along the stream or lake can be changed for every half timestep. Flow to or from this boundary varies as the head in the aquifer and/or the head in the lake or stream varies with time. A subroutine INOUT has been added to the model program which calculates the flow to or from every hydraulically connected stream node. Flow is calculated for every half timestep and the sum total for the simulation period is accumulated.

Figure 29 is a schematic representation of a constant head boundary. $H_c(i,j,k)$ is the head along the stream at half timestep k. $T_x(i,j)$ is the transmissibility of the aquifer beneath the stream. $h_{i+1,j,k}$ is the hydraulic head at node point (i+1,j) at half timestep k.

The specific flux across the boundary in x-direction from time k to k+1 for node point (i,j) is

$$q_x[\text{ft/day}] = - K_{i,j} \frac{(h_{i+1,j,k+1}+h_{i+1,j,k}) - [H_c(i,j,k+1)+H_c(i,j,k)]}{2 \Delta x} \quad (28)$$

The total volume of flow per node from time k to k+1 can be expressed by

$$Q_x = (\Delta t/2)\Delta y b_{i,j} q_x \ [\text{ft}^3]$$

where

$\Delta t/2$ = length of the half timestep [days].

Δy = width of the flow region in x-direction [ft].

$b_{i,j}$ = aquifer depth at the location of the hydraulic head [ft].

The total quantity of flow in y-direction can be expressed in the same manner.

$$Q_y = (\Delta t/2)\ \Delta x b_{i,j} q_y$$

where

Δx = width of the flow region in y-direction [ft]

Depending on the orientation of the coordinate axes relative to the

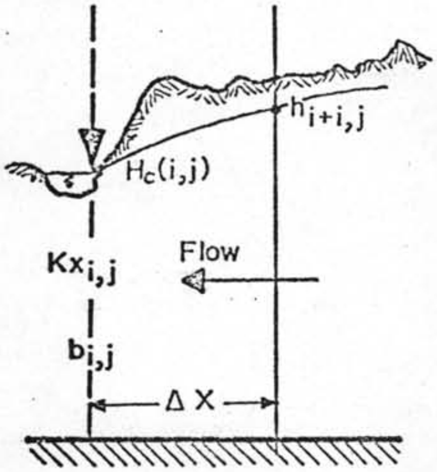

Figure 29. Schematic Representation of Constant Head Boundary.

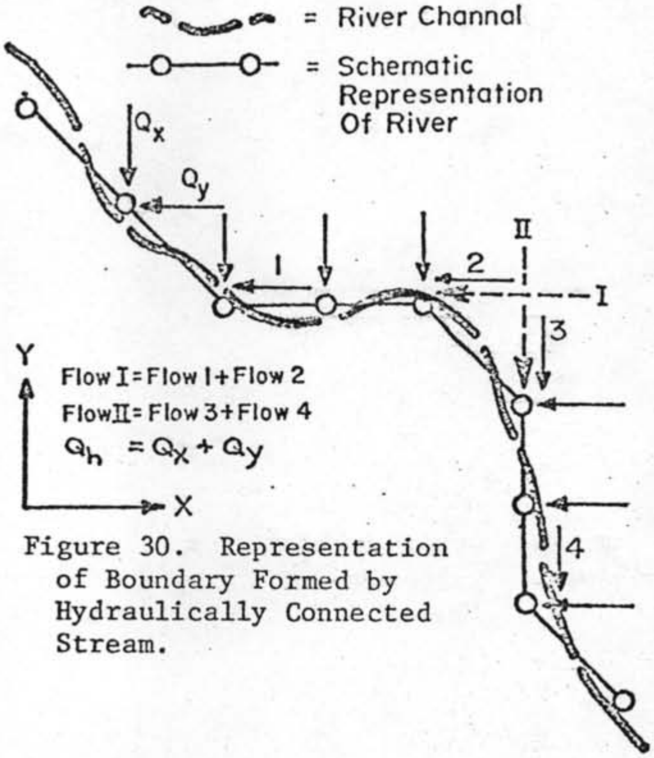

Figure 30. Representation of Boundary Formed by Hydraulically Connected Stream.

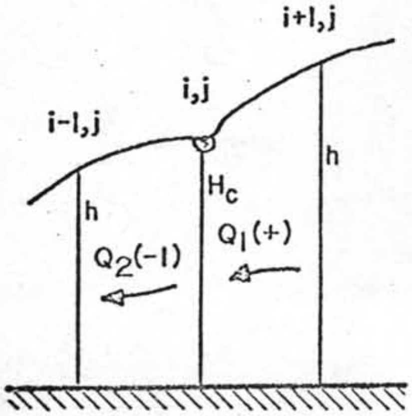

Figure 31. Representation of Stream Inside Boundaries, Gaining on One Side, Losing on the Other Side.

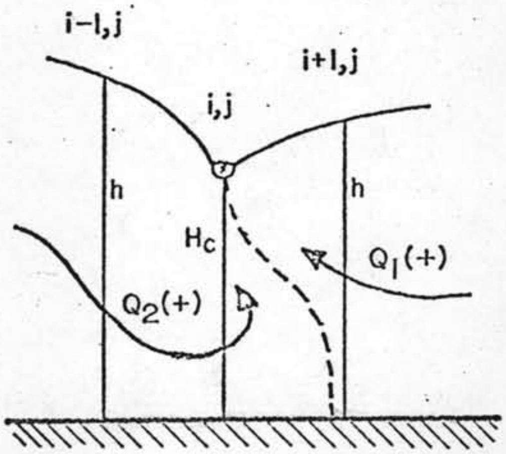

Figure 32. Representation of Stream Inside Boundaries, Gaining From Both Sides.

constant head boundary, the total flow Q_h across the boundary is composed of Q_x, Q_y or both, as illustrated in Figure 30. An irregular boundary like the one shown in Figure 30 has the following distribution of node point flows: Flow I leaves the system actually in two portions (1) and (2), each calculated with 1/2 Δy as the flow width; Flow II leaves the system via flow (3) and (4), each calculated with 1/2 Δx as the flow width.

If the lake or stream is not part of the boundary but located inside the aquifer system, the routine to calculate flow to or from a stream can be utilized to calculate reach gains or losses (i.e. the flow which is added to or is leaving the aquifer system).

In Figure 31 the total flow from node (i+1,j) to node (i,j) is $Q_1(+)$; the flow from node (i,j) to node (i-1,j) is $Q_2(-)$. The total flow which is added to the system or leaving the system is $(Q_1 + Q_2)$ which is the reach loss or gain respectively.

Figure 32 shows the case of a stream or drain where Q_1 as well as Q_2 is flowing to the stream. A line can be drawn from the bottom of the stream to the aquifer bottom across which no flow occurs. Subsequently the flow leaving the system is in this case $(Q_1 + Q_2)$.

The subroutine INOUT will calculate the flow across a constant head boundary if this boundary is located on the periphery of the aquifer. If the boundary is a lake or stream inside the aquifer the reach gains or losses of the streams will be calculated. In case a drain is operated in the aquifer the routine will calculate the drain flow as long as the flow is diverted to the drain. If the flow becomes negative, the drain will be deactivated. (See DRAIN subroutine, page 64).

Flow Across the Constant Gradient Boundary

In an aquifer employing a constant gradient boundary along part of the periphery, the INOUT subroutine can be utilized to calculate the flow across this boundary. With a constant gradient at the boundary the flow varies with the change of the hydraulic head at the boundary which results from the changing flow patterns in the aquifer. This flow is calculated in the same way as flow across a hydraulically connected stream boundary.

Calculation of the Leakage Term

A leaky aquifer system is simulated by using the leakage term in which the head in the underlying or overlying water bearing formation is assumed to be constant. The leakage increases or decreases with the variation of the head in the main aquifer, which is calculated for every half timestep.

The specific leakage flux for node point (i,j) from time k to k+1 is obtained from equation (29)

$$q_v[\text{ft/day}] = -\frac{K_v(i,j)}{2b_c(i,j)}\,[2h_c(i,j) - h_{i,j,k+1} - h_{i,j,k}]\ [\text{ft/day}] \qquad (29)$$

Where

$\frac{K_v(i,j)}{2b_c(i,j)}$ = leakance factor of the confining layer [1/day]

$h_c(i,j)$ = hydraulic head in the water bearing formation [ft]

$h_{i,j,k}$ = hydraulic head in the aquifer at t = k [ft]

The total volume of leakage from time k to k+1 for node point (i,j) can be expressed as

$$Q_v(i,j)\ [\text{ft}^3] = \Delta x\Delta y(\Delta t/2)\ q_v\ \ [\text{ft}^3]$$

in which

$\Delta x\Delta y$ = nodal area [ft^2]

$\Delta t/2$ = length of half timestep [days]

$Q_v(i,j)$ can also be written as follows (using equation 29)

$$Q_v(i,j) = (\Delta t/2)\ [\Delta x\Delta y\text{Fac}_{i,j}\ (h_{i,j,k+1}+h_{i,j,k}) - 2\Delta x\Delta y\ \text{Fac}_{i,j}h_c(i,j)]$$

Where

$\mathrm{Fac}_{i,j} = \dfrac{K_v(i,j)}{2b_c(i,j)}$, the leakance factor which is one of the input arrays of the model program. The quantity $[2\Delta x\Delta y\ \mathrm{Fac}(i,j)\ h_c(i,j)]$ is a constant for every simulation and al already computed quantity in the flow equation known as QC(i,j) as explained in De Sonneville (1972). Therefore

$$Q_v(i,j) = (\Delta t/2)\ [\Delta x\Delta y\ \mathrm{Fac}_{i,j}\ (h_{i,j,k+1}+h_{i,j,k}) - QC(i,j)]$$

This quantity is calculated for every node point for every half timestep and printed out as a lump sum over all the nodes in the aquifer. The sum total over the simulation period is also calculated and printed out.

Accretion Term

Water is stored or lost in the aquifer if the hydraulic head is increased or decreased respectively. This change in storage per nodal area from time k to k+1 can be expressed as

$$\Delta S(i,j) = -\ \Delta x\Delta y\ S_{i,j}\ (h_{i,j,k+1} - h_{i,j,k}) \tag{30}$$

Where

$\Delta x\Delta y$ = nodal area [ft^2]

$S_{i,j}$ = storage coefficient for node point (i,j) [dimensionless]

$(h_{i,j,k+1} - h_{i,j,k})$ = rise or decline of the hydraulic head from time k to k+1

This quantity is calculated for every node point for every half timestep and printed out as a lump sum over all nodes in the aquifer. The sum total over the simulation period is also calculated and printed out.

If the simulation period is chosen such that the hydraulic head values at the start and end of the simulation are the same, the accretion term is reduced to zero, as is the case in a steady state or equilibrium state simulation.

With the knowledge of these terms a mass balance computation can be undertaken to provide an additional procedure for checking the aquifer system.

For many aquifers data about the storage coefficient is scarce. If all other terms are known and the change in aquifer head is known from the model calculation, the mass balance computation may provide an additional way to obtain a general idea of the magnitude of the storage coefficients in the aquifer. In general the sum of all terms discussed above over all nodes in the aquifer system should balance according to the following equation:

$$\sum_{i,j} [Q(i,j) + Q_h(i,j) + Q_v(i,j) - \Delta S(i,j)] = 0 \tag{31}$$

Where

$Q(i,j)$ = external source term for node (i,j)

$Q_h(i,j)$ = flow across aquifer boundaries or reach gains or losses for node (i,j)

$Q_v(i,j)$ = leakage for node (i,j)

$\Delta S(i,j)$ = storage in the aquifer for node (i,j)

Subroutine DRAIN

A rising water table will normally intersect a surface drain at some point and then spread its influence along the drain. To simulate this behavior the DRAIN routine compares the calculated head values and stipulated drain elevations at every individual node point. Also the INOUT routine is used in the decision process whether to activate or deactivate the drain. The operation of the DRAIN subroutine can be divided in two parts dependent upon the state of activation the drain was in during the previous half timestep.

For example, assume a drain is located in an aquifer with the initial water table below the stipulated drain elevation. The water table rises as a result of a particular irrigation application in the area to a level above the drain elevation and then declines as a result of the termination of irrigation. In the initial stage of the simulation the calculated water levels at the drain location are below the drain elevation. For every half timestep the calculated head values at the drain node are compared with the drain elevation at that node. If at some half timestep the calculated head value is greater or equal to the drain elevation the DRAIN routine changes that node point to a constant head node. In the next half timestep the drain node functions as a constant head while the head values at nodes surrounding the drain are still rising. With the drain activated the INOUT subroutine calculates for every timestep the 'reach gain' or flow to the drain for every activated drain node. As long as the flow calculated is positive (flow towards drain) the drain remains activated for the next half timestep calculation. When the head values surrounding the drain are declining as a result of a termination of input, at some half timestep the calculated flow will become negative (drain is losing water). At that moment the DRAIN routine will eliminate the constant head boundary at that node and deactivate the drain for the calculation of the next half timestep for which again calculated head values and drain elevation are compared. All comparisons are made on a node point basis.

It must be kept in mind that this is a simplification of what happens in the field. It is quite possible that a drain is losing water supplied upstream. This is partly accounted for in the routine since the routine will not deactivate the drain until negative flows have been calculated. This flow serves as an input to the aquifer for that half timestep. However, water lost in that part of the drain which is above the calculated water table may be lost via unsaturated flow and cannot be adequately simulated in the model. This unsaturated flow is much smaller in magnitude than the flow occurring in a drain hydraulically connected with the aquifer. Because the constant head boundary is eliminated at places where the calculated water table is below the drain an error is introduced but the error introduced in not accounting for the unsaturated flow (seepage loss) is much smaller than the error, introduced in over accounting for this flow occurring in case the flow is calculated as suturated flow from a hydraulically connected stream. Despite the simplification the DRAIN subroutine worked very satisfactorily in tests with this procedure. The program prints out the location and the number of nodes which are acting as a drain for every half timestep. The subroutine is simple and requires as input only the location and stipulated elevation of the drain.

Variable Water Level Boundary

One of the boundary conditions included in the program is the constant head boundary. It is also possible to have a boundary along which the water level is changing according to the regulation of a reservoir, lake or stream. The assumption is that the water body has a fully saturated connection with the groundwater aquifer which must be verified in the field. Rivers with a hydraulic connection can be divided into separate reaches, each with its own stage.

Input for this feature is an array specifying the location of the reaches and the change in water level for every half timestep for every reach. In many cases the influence of a changing river level on groundwater levels is minimal so that this feature seems superfluous. In some cases however, with rapid changes in river stage or large changes in reservoir level, this influence cannot be neglected and should be incorporated in the modeling effort.

Revision of the Calibration Routine

In the study of the Snake River Fan four hydrogeological parameters were adjusted alternately; the hydraulic conductivity, storage coefficient, leakance factor and initial head differences between the local and regional groundwater table. Parameter changes were made based on the difference between calculated and historical water table values. The calculated head values were most sensitive to adjustments of the last three mentioned parameters. Calculated head values were least sensitive to adjustment of the hydraulic conductivity values according to this procedure.

Although the overall result showed a satisfactory simulation of the historical water table after final calibration, another way of adjusting parameters, resulting in a faster, more efficient calibration is described in the following section.

Adjustment of the Transmissibility Values

With any kind of calibration, uniqueness of the parameter solution must be secured. Flow equations are utilized to calculate the hydraulic head, given a set of geohydrological parameter values. The objective in solving the single parameter inverse problem is to determine a parameter function from a set of observed hydraulic heads. In the case of determining a transmissibility distribution the necessary condition for uniqueness is that, in addition to the hydraulic heads, the transmissibility, T, must be known along a surface crossed by all streamlines in the region (Nelson, 1962, 1968). The idealized case of one-dimensional steady state flow can be described by

$$\frac{\partial}{\partial x}\left[T_x\left(\frac{\partial h}{\partial x}\right)\right] = 0$$

Where

T_x = transmissibility in direction of flow

$\frac{\partial h}{\partial x}$ = hydraulic gradient in direction of flow

Implying the boundary condition $T_\tau = T_o$ yields (Figure 33)

$$T_o\left(\frac{\partial h}{\partial x}\right)_o = -\ q_o$$

or

$$T_{(\tau)} = \frac{-q_o}{\left(\frac{\partial h}{\partial x}\right)_o} \qquad (32)$$

q_o = discharge through line τ where T is known

$\left(\frac{\partial h}{\partial x}\right)_o$ = hydraulic gradient at line τ

As can be seen the transmissibility is a direct function of the hydraulic gradient. The case of one dimensional flow can be expanded easily to a two dimensional system in which (Figure 34)

$$T_{(\tau)} = T_o \qquad \text{or}$$

$$T_{(\tau)} = \frac{-q_o}{\left(\frac{\partial h}{\partial s}\right)_o}$$

$\frac{\partial h}{\partial s}$ = the gradient along the streamline and the transmissibility is a function of the hydraulic gradient of the streamline.

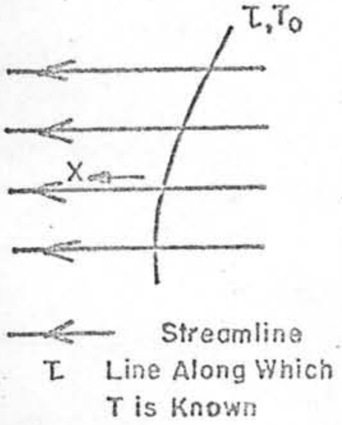

Figure 33. One Dimensional Flow. On Line τ, $T = T_o$

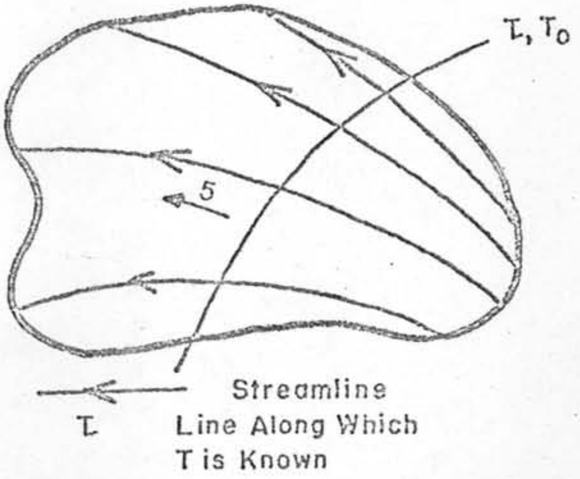

Figure 34. Two Dimensional Flow. On Line τ, $T = T_o$

Most times this boundary condition is from practical considerations difficult to satisfy. Since transmissibility and flux are related by

Darcy's law, this uniqueness criterion can be stated alternately in terms of flux, which crosses a line cutting all stream lines in the region. Such a line can be the circumference of a well; hence the well discharge provides a sufficient boundary condition for a unique solution. Using either boundary condition a set of observed hydraulic head values is required which will lead to an exact solution.

The procedure proposed here is an approximative method in which elements of the above procedures are included. Uniqueness of the transmissibility distribution is dependent on the boundary condition which is represented here by the presence in the modeled aquifer of withdrawal or recharge areas. The method is explained for an aquifer with an equilibrium state water table configuration.

The transmissibility calibration utilizes the calculated head values resulting from model application which uses as initial head values the historically observed equilibrium head values. Other inputs are an approximated transmissibility distribution and the external inputs to the aquifer which are calculated in the input program. The model is run until an equilibrium state is reached with the initial parameter values. Then, the transmissibility value of every node point is adjusted, based on the ratio of historical to calculated hydraulic gradient of the groundwater flow of that node point.

Since the recharge or discharge at the node points (inputs to the aquifer) form an integral part of the calculation of the equilibrium head values and hence influence the hydraulic gradient ratios of the individual points, these 'input' locations and amounts represent the controlling factor, or the boundary condition in the determination of a unique transmissibility value.

The method is an iterative method in which after every adjustment the deviations between calculated and historical head values are determined and the sum of squares of deviations is calculated. Successive adjustments of the transmissibility values are made until no decrease in sum of squares is observed. Since the sum of squares of deviations is the test criterion, the resulting transmissibility values will always be approximations because the sum of squares will never be exactly zero, although the values will be reasonable. The importance of the presence of recharge or discharge nodes in the aquifer will be illustrated in the following examples.

Consider an idealized case of a one-dimensional unconfined non-homogeneous, isotropic aquifer. Figure 35 shows an aquifer with a historic head distribution represented by the solid line and an initially uniform conductivity distribution.

The first run of the model computes steady state head values represented by the dashed line in Figure 35. Based on the ratios of historical and calculated hydraulic gradient the transmissibility (T) values will be adjusted until the dashed line matches the solid line.

In this example, because no nodes with recharge or discharge are present, no unique solution for the transmissibility distribution will be obtained since different initial T-values will result in a different final T-distribution.

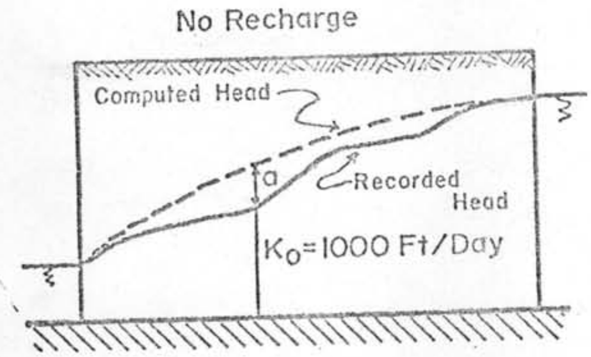

Figure 35. Flow Through Aquifer Without Recharge.

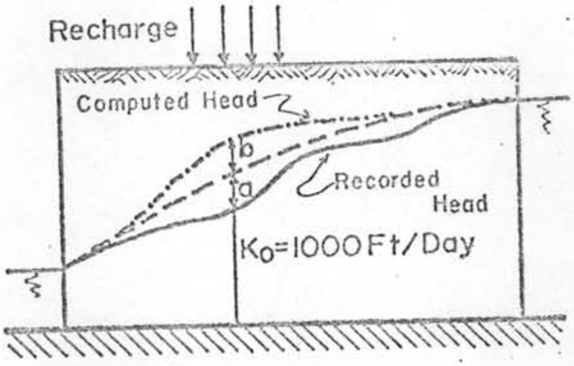

Figure 36. Flow Through Aquifer With Recharge.

However, the situation is entirely different in the presence of some nodes with input. Consider the same aquifer, same initial K-values, a similar water table gradient to be simulated but with nodes in the aquifer, Figure 36, which have a certain rate of artificial recharge. The first run of the model will produce a rise of the head values of which part 'a' is similar to that of the first example. Superimposed on this rise is an additional rise 'b' caused by the recharge. Rise 'a' and 'b' are interrelated because both are dependent on the magnitude of T. Since this is an equilibrium state situation, the total rise (a+b) is dependent on the absolute magnitude of the T-values at the node points, not the relative T-values. In this case only one transmissibility distribution will result in a match of calculated and historical water table, independent of the initial T-values. In this iterative technique convergence will be obtained faster if the initial estimates of the T-values are already in the neighborhood of the true T-values. It is possible that a match of the historical water table may not be obtained because the sum of squares of deviation may start to increase in other areas of the model resulting in an increase in the overall sum of squares. This can occur if initial estimates of K-values are grossly in error. It is therefore of importance that initial estimates of conductivity values are in the neighborhood of the exact values. The adjustment of the T-values is based on the single parameter inverse problem of an inhomogeneous aquifer with a given head distribution.

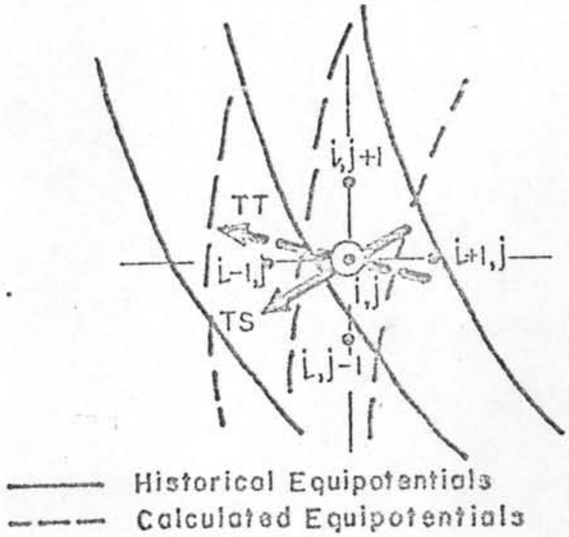

Figure 37. Representation of Two-Dimensional Flow. The T-Values are Adjusted Using the Ratio of the Gradients Along the Calculated and Historic Streamlines, $TT_{i,j}$ and $TS_{i,j}$ Respectively.

The energy dissipation method (Nelson, 1969) obtains the T-values by integrating along the streamline, the function of T and head distribution. The proposed method utilizes the relation between hydraulic gradient and transmissibility by adjusting the T-values on a grid point basis by multiplying the original T-value by the ratio of calculated and historical gradient along the flow line for that node point. See Figure 37.

$$T^{n+1}_{i,j} = T^{n}_{i,j} \frac{TT_{i,j}}{TS_{i,j}}$$

Where

n = number of iterations

$TT_{i,j} = \left(\frac{\partial h}{\partial s}\right)^{n}$ = gradient obtained from head values calculated with $T^{n}_{i,j}$ values.

$TS_{i,j} = \left(\frac{\partial h}{\partial s}\right)^{o}$ = historically observed gradient (constant

Let $R = \frac{TT_{i,j}}{TS_{i,j}}$ In the digital model the head values are discrete points and the hydraulic gradient of the streamline is approximated as follows:

$$TT_{i,j} = \frac{\partial h}{\partial s_{i,j}} = \sqrt{\frac{(h_{i+1,j}+h_{i-1,j})}{2}^{2} + \frac{(h_{i,j+1}+h_{i,j-1})}{2}^{2}} \tag{34}$$

In order to allow for a gradual adjustment of the transmissibility values the ratio is multiplied by a damping factor VV such that

$$T^{n+1}_{i,j} = [1 + VV(R-1)]\ T^{n}_{i,j} \tag{35}$$

VV is a factor between 0 and 1 and a function of the sum of squares of deviations (SS) of preceding runs. If for instance the decrease of SS is very slight compared to SS of the previous run, VV will approach 1 to allow for a maximum change of T. This method of adjusting T was tested in an example of one dimensional steady state flow as illustrated in Figure 38.

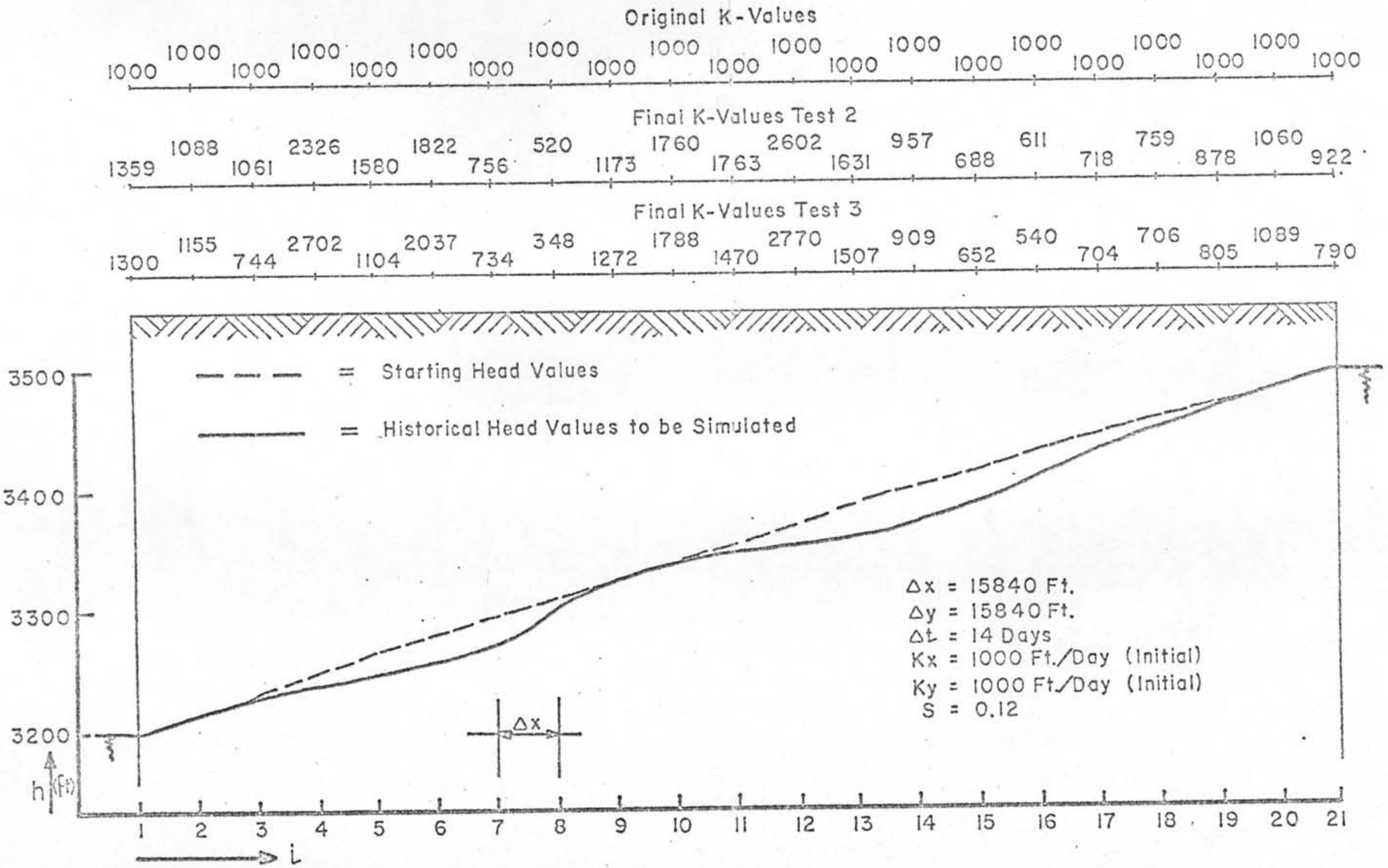

Figure 38. Schematic Representation of One Dimensional Steady State Flow. T-Values are Adjusted to Simulate Historical Response.

Initial hydraulic conductivity values were 1000 ft/day. Initial head values were determined analytically and are represented by the dashed line in Figure 38. The solid line represents the historical water table configuration to be duplicated by calibration of the transmissibility values.

In this stage of the calibration trials the damping factor VV was kept constant at 1.0. Figure 39 shows a graph of the decrease in standard deviation versus the number of iterations for Test 1.

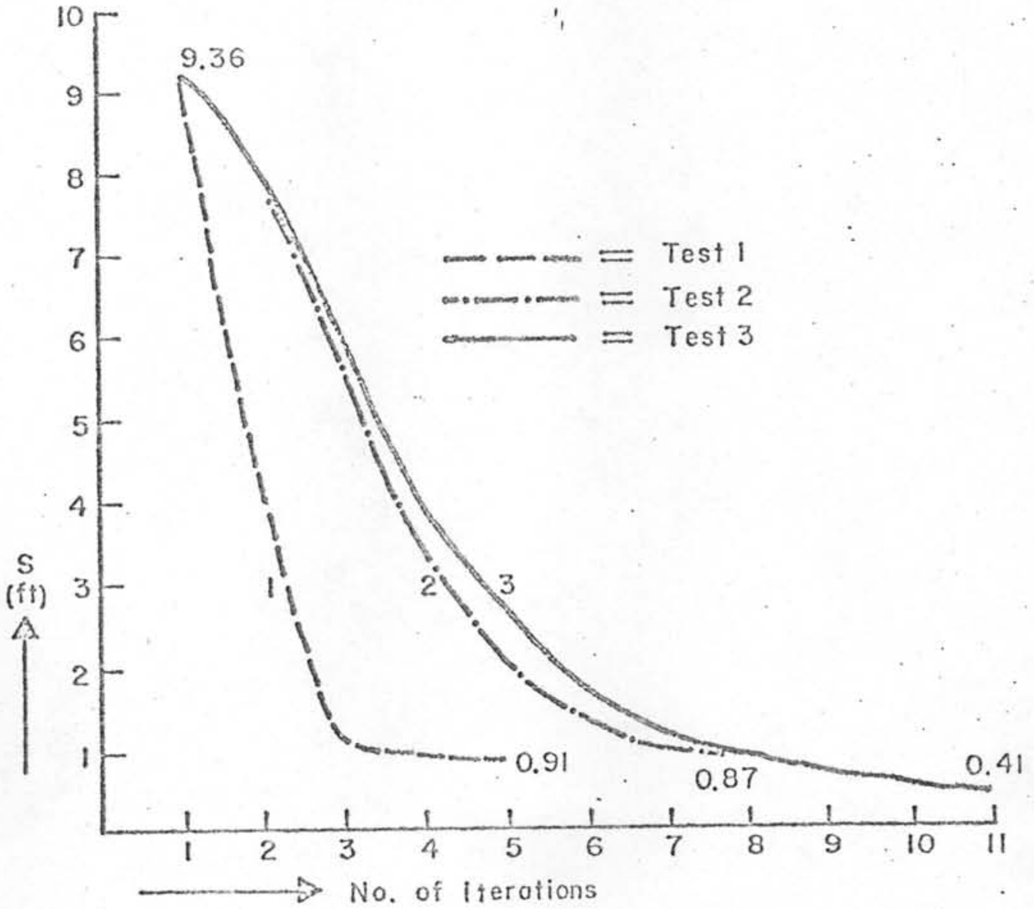

Figure 39. Decrease in Standard Deviation as a Result of the Adjustment of the T-Values for Three Tests.

The next test used the same data as Test 1 except for the damping factor which was set at 0.4 for all iterations. Figure 39 shows the result of Test 2 for which the standard deviation is smaller than Test 1 and shows a more gradual decrease in standard deviation. Although the standard deviation decreased considerably the result was not satisfactory. Adjustments of the transmissibility on a grid point basis did not sufficiently decrease S. The adjustment described above gives equal importance to the transmissibility of every node point in the calculation of the hydraulic head values. In the finite difference approximation of the flow equation however, the head value for every node point is calculated using a weighted average of the node point T-value and the surrounding nodes.

Assume the simplified case of two dimensional steady state flow in a nonhomogeneous unconfined isotropic aquifer. Then the flow equation can be written as

$$\frac{\partial T}{\partial x}\frac{\partial h}{\partial x} + T\frac{\partial^2 h}{\partial x^2} + \frac{\partial T}{\partial y}\frac{\partial h}{\partial y} + T\frac{\partial^2 h}{\partial y^2} = 0$$

Substituting finite differences yields

$$T_1\, h_{i-1,j} - (T_1+T_2)\, h_{i,j} + T_2\, h_{i+1,j} + T_3\, h_{i,j-1}$$

$$- (T_3+T_4)\, h_{i,j} + T_4\, h_{i,j+1} = 0 \qquad (33)$$

in which

$$T_1 = (T_{i-1,j} + T_{i,j})/2$$

$$T_2 = (T_{i+1,j} + T_{i,j})/2$$

$$T_3 = (T_{i,j-1} + T_{i,j})/2$$

$$T_4 = (T_{i,j+1} + T_{i,j})/2$$

Solving for head at node point (i,j) in equation (33) yields

$$h_{i,j} = \frac{(T_1 \, h_{i-1,j} + T_2 h_{i+1,j} + T_3 \, h_{i,j-1} + T_4 h_{i,j+1})}{(T_1 + T_2 + T_3 + T_4)} \qquad (34)$$

or, the hydraulic head at node (i,j) is the weighted sum of the surrounding node point head values divided by the weighted averages of the transmissibility values at mid point between the nodes. The denominator of expression (34) multiplied by 2, is called T^W

$$T^W = (T_{i-1,j} + T_{i+1,j} + T_{i,j-1} + T_{i,j+1} + 4 \, T_{i,j})$$

Thus, in the calculation of the head value of node point (i,j) $T^W_{i,j}$ is the representative transmissibility value and not $T_{i,j}$, indicating that the gradient ratio R should be used to adjust $T^W_{i,j}$ to arrive at a better simulation of the historic water table. A different calibration routine can therefore be used.

First, for iteration the weighted transmissibility values for every node point are calculated.

$$T^{W,n}_{i,j} = T^n_{i-1,j} + T^n_{i+1,j} + T^n_{i,j-1} + T^n_{i,j+1} + 4 \, T^n_{i,j} \qquad (35)$$

If $T^{W,n}$ did not result in a sum of squares of deviations that would represent a satisfactory simulation, for the next calibration run the weighted T-values ($T^{W,n+1}$) are adjusted using the ratio of calculated and historic gradients.

$$T^{W,n+1}_{i,j} = [1 + VV \, (R-1)] \, T^{W,n}_{i,j} \qquad (36)$$

Then, temporary individual T-values ($IT^{n+1}_{i,j}$) are calculated using the same gradient ratio:

$$IT^{n+1}_{i,j} = [1 + VV \, (R-1) \,] \; T^n_{i,j} \qquad (37)$$

Since the individual T-values are changed, a weighted average T-value calculated with these new individual T-values will be different from the one calculated in expression (36), which is the desired weighted average T^W.

In order to maintain the desired weighted T-values, individual nodal T-values are readjusted in order to compensate for the change in surrounding nodal T-values. In expression (35) $T^{W,n+1}_{i,j}$, obtained in equation (36), is substituted for $T^{W,n}_{i,j}$ and temporary individual T-values (IT) are substituted for the first four terms on the right hand side of equation (35). The equation is solved for $T^{n+1}_{i,j}$.

$$T^{n+1}_{i,j} = \left[T^{W,n+1}_{i,j} - (IT^{n+1}_{i-1,j} + IT^{n+1}_{i+1,j} + IT^{n+1}_{i,j-1} + IT^{n+1}_{i,j+1})\right] /4 \qquad (38)$$

Since for every node point (i,j) the T-values of the surrounding node points will also be adjusted, the final weighted average will still be somewhat different from the desired T^W.

This procedure was tested using the same data as used in Test 1 and Test 2. The damping factor VV was kept at 0.4, as in Test 2. The results of Test 3 are also shown in Figure 39 which indicates this method succeeded in reducing the final standard deviation by 50% to 0.46 foot which is considered satisfactory. Figure 38 shows the final K-values resulting from Test 2 and Test 3. The weighted average adjustment increased the differences between the nodal T-values but improved the overall match of the historical water table configuration.

Adjustment of the Leakance Factor

In a leaky aquifer system, two factors determine the amount of leakage; the leakance factor of the impeding layer and the hydraulic head difference that exists between the main aquifer and the underlying or overlying water bearing formation. These two parameters are seldom known on a grid point basis, therefore simulated head values in a leaky aquifer system most times differ from the historically measured head values.

An automated adjustment procedure based on deviations of both historical and calculated head values at a certain timestep from the head values at the start of the simulation is suggested as follows for the leakance factor.

Since the leakance factor is by definition a positive quantity, the direction of the leakage depends on the sign of the head difference, ΔH.

$$\Delta H = (h_{i,j} - h_{c\ i,j})$$

where

$h_{c\ i,j}$ = head in the underlying or overlying water bearing formation.

$h_{i,j}$ = head in the main aquifer to be modeled.

If ΔH is positive water leaks out of the main aquifer and tends to decrease the hydraulic head in the main aquifer. If ΔH is negative water leaks into the main aquifer and tends to increase the hydraulic head in the main aquifer. The sign of the head difference influences the direction of the change of the leakance factor.

Depending on the sign of the head difference and the relative magnitudes of the rise or decline of the historical and calculated heads from the start of the simulation,the adjustment can be separated in several cases as shown in Figures 40 and 41.

Figure 40 shows 6 combinations a,b,c,d,e, and f, which apply to a positive head difference (water leaking out of the aquifer). Figure 41 shows similar combinations applying to a negative head difference (water leaking into the aquifer). D_1 represents the difference between calculated head and initial head value, while D_2 represents the difference between historical head and initial head value. Utilizing the above leakance factor $Fac_{i,j}$ is adjusted as follows

$$Fac_{i,j}^{n+1} = [\ 1 + VV(R-1)\ ]\ \ Fac_{i,j}^{n} \text{ or } Fac_{k,j}^{n+1} = C_2\ Fac_{i,j}^{n} \tag{39}$$

where

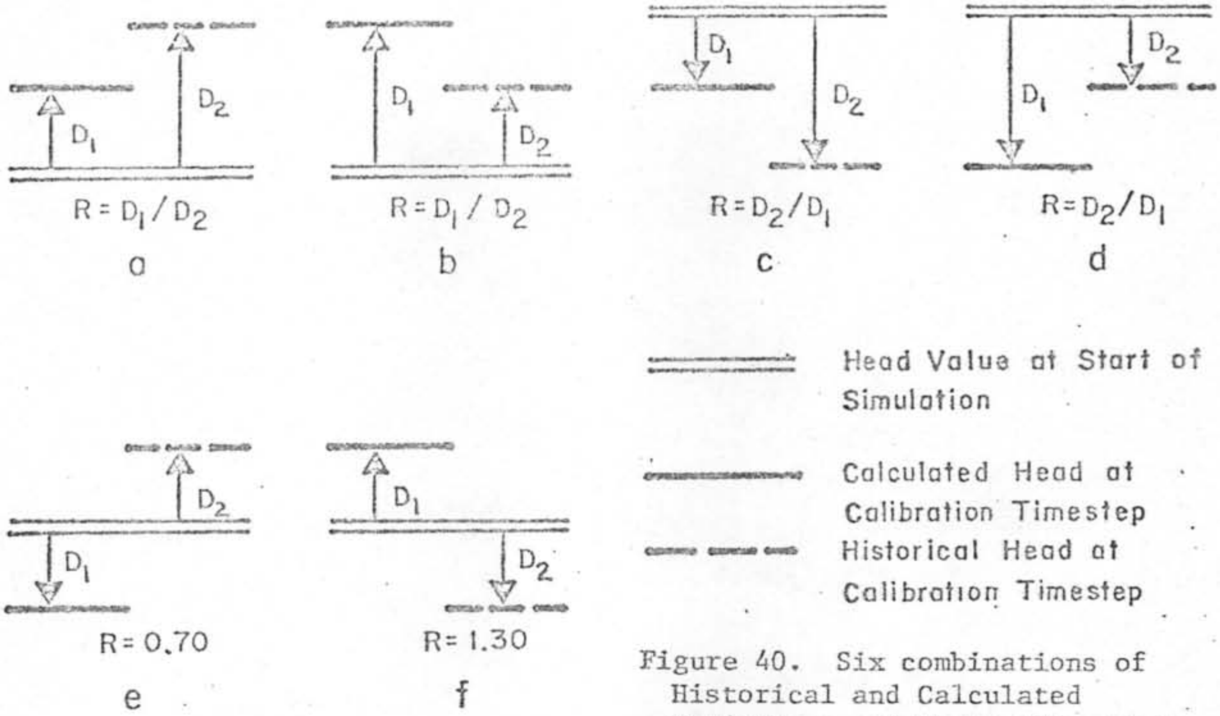

Figure 40. Six combinations of Historical and Calculated Deviations and Their Adjustment Ratio R (Head Difference is Positive).

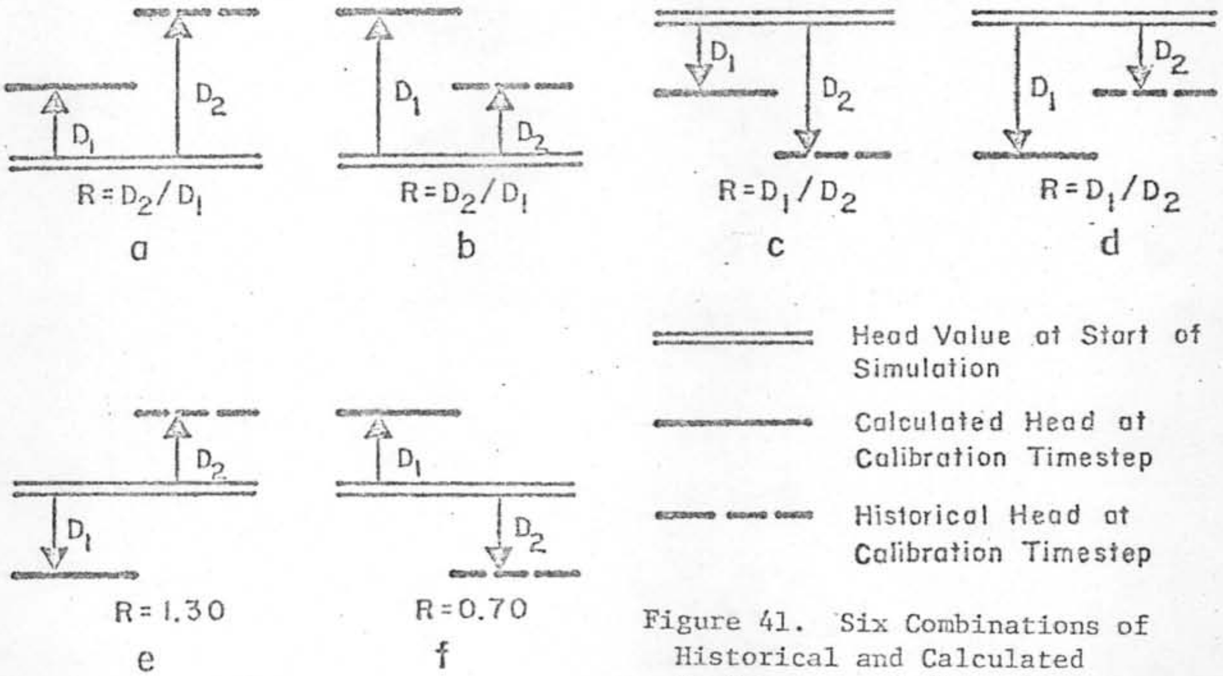

Figure 41. Six Combinations of Historical and Calculated Deviations and Their Adjustment Ratio R (Head Difference is Negative).

n is number of iterations

VV is a damping factor, identical to the one used in the adjustment of transmissibility values ($0 < VV < 1$)

R is defined as the ratio (D_1/D_2) or the ratio (D_2/D_1)

$$C_2 = 1 + VV\ (R-1);\ \text{if } R > 1 \text{ then } C_2 > 1;\ \text{if } R < 1 \text{ then } C_2 < 1$$

In the presence of a positive head difference the following adjustments are made:

For combinations 40a and 40b, $R = D_1/D_2$

In combination 40a, the calculated head is lower than the historical head; the leakance factor is decreased so that less water is taken out of the aquifer. For combination 40b the calculated head is higher than the historical head; the leakance factor is increased so that more water is taken out of the aquifer.

For combinations 40c and 40d, $R = D_2/D_1$

In combination 40c, the calculated head is higher than the historical head; the leakance factor is increased so that more water is taken out of the aquifer. For combination 40d, the calculated head is lower than the historical head; the leakance factor is decreased so that less water is taken out of the aquifer.

For combinations described above the deviations were both either negative or positive. In case the deviations have a different sign as is shown in combination 40 e and 40 f, the adjustment on the basis of the ratio is erroneous. For example if $D_1 = - D_2$, $R = - 1$ or the leakance factor becomes negative. Even with the absolute value of the ratio (then $R = 1$) the result will be a constant leakance factor.

Therefore if the deviations have an opposite sign the leakance factor is multiplied by a predetermined constant C_1 or C_3. For combination 40e the calculated head value is lower than the historical head value; the leakance factor is decreased so that less water is taken out of the aquifer.

$$Fac_{i,j}^{n+1} = C_1\ Fac_{i,j}^{n},\quad 0 < C_1 < 1$$

For combination 40f the calculated head value is higher than the historical head value; the leakance factor is increased so that more water is taken out of the aquifer.

$$\mathrm{Fac}_{i,j}^{n+1} = C_3\ \mathrm{Fac}_{i,j}^{n}\ ,\quad 1 < C_3 < 2$$

In order to have the same rate of change for all combinations the multiplication factor used in combinations 40a, b, c and d, that is $C_2 = [1 + VV(R-1)]$ has limits such that $C_1 < C_2 < C_3$

An additional problem occurs for combinations in which the leakance adjustment causes a decrease in leakage i.e., combinations 40a, 40d and 40e. For these three combinations the calculated head is lower than the historical head. The situation may occur in which the leakance factor in subsequent adjustments is decreased until the leakance factor is practically zero. If the calculated head is still substantially below the historical head several conclusions may be drawn.

1. Historic water table data are incorrect. In that case additional data collection or review is needed.
2. Calibration shows that there is no leakage present for this particular node. The discrepancy between historical and calculated head is caused by errors in other input data or parameters; additional data refinement is necessary.
3. The initial head difference, positive for combinations 40a, 40d and 40e, is incorrect and should be negative. A negative head difference causes water to leak into the aquifer. Instead of decreasing the leakance factor, in this case the factor is increased i.e., more water is added to the aquifer. This will result in an increase of calculated head, and ultimately to a simulation of the historical water table.

It is obvious that the initial choice - a positive or negative head difference - is important. An automatic sign change in head difference is not incorporated in the program since this conceals the possible causes of a discrepancy in calculated and historical head. Instead, an initial choice based on geological information is made for the head difference and the leakance factor is then calibrated. The results of the calibration show locations in the model where the available data may be incorrectly interpreted. Nodal points which show a combination of a zero final leakance factor and a substantial difference in calculated and historical head indicate where a re-examination of the input data is necessary. Changes can then be made based upon this re-evaluation.

The adjustment of the leakance factor in the presence of a negative head difference is similar to that for a positive head difference. For every combination the adjustment is exactly the reverse of the adjustment in the presence of a positive head difference as shown in 41a-f.

Adjustment of the Initial Head Difference

The other parameter that determines the amount of leakage is the head difference between the main aquifer and the underlying or overlying water bearing formation. An initial head difference is read in for every leakage node. The head in the under or overlying formation is kept constant while the leakage varies in time as the head in the main aquifer changes.

The adjustment of the head difference is done similarly to that of the leakance factor. An identical table can be made up with the same sets of combinations, the first set representing the adjustment with a positive head difference, the second with a negative head difference. (See Figures 40 and 41). For this combination the initial head difference is adjusted as follows

$$\Delta H^{n+1}_{i,j} = [1 + VV\ (R-1)]\ \ \Delta H^{n}_{i,j} \tag{40}$$

$$C_2 = 1 + VV\ (R-1)\quad ,\ R = D_1/D_2 \text{ or } D_2/D_1$$

Similarly the combinations of deviations resulting in a decrease of the leakage i.e., combinations 40a, d, e (with positive head difference), or 41b, 41d, 41f (with negative head difference) will require careful examination of the input data to decide on the sign of the head difference.

It is important to understand that in adjusting the leakance factor or the head difference any water table configuration at some timestep can be matched. The two parameters are adjusted and the sign of the initial head difference may be changed until enough water is taken out or added to the aquifer to match historical behavior. In other words water, or for that matter, lack of it is simply 'generated'. In order not to create a sourceof water that in reality does not exist, available data about the area have to be scrutizized, and the calculated magnitude of leakage judged on reasonableness, especially for aquifers which are in an equilibrium state (non-changing water table in time). In that case the leakage parameters are calibrated on the equilibrium water table values. Since the historical water table values at any other timestep are the same, the calculated water table at any timestep will always match the historical water table configuration. Therefore, only a one point check on the simulation is available in an equilibrium state calibration.

In case of an unsteady state aquifer condition the adequacy of the simulation over the entire period can be evaluated. The parameter values may be adjusted at the timestep for which the historical water table reaches a maximum. If by adjusting the parameters the calculated maximum approaches the historical maximum water table but at the same time the calculated minimum head values deviate increasingly from the historical minimum water table, the conclusion may be drawn that the adjusted parameter values are not reasonable.

Therefore, in the unsteady state calibration the sum of squares of deviations of both maximum and minimum water table are used as criterion to adjust the parameters. If the overall sum of squares continues to decrease (better fit overall over the total simulation period) additional parameter adjustments should be made.

Adjustment of the Storage Coefficient

For aquifers in a transient state, the decline or rise of the water table depends on the water withdrawn or added to the aquifer. The magnitude of that rise or decline is related to the value of the storage coefficient. A large storage coefficient tends to dampen the change in hydraulic head while a small storage coefficient amplifies the change in hydraulic head.

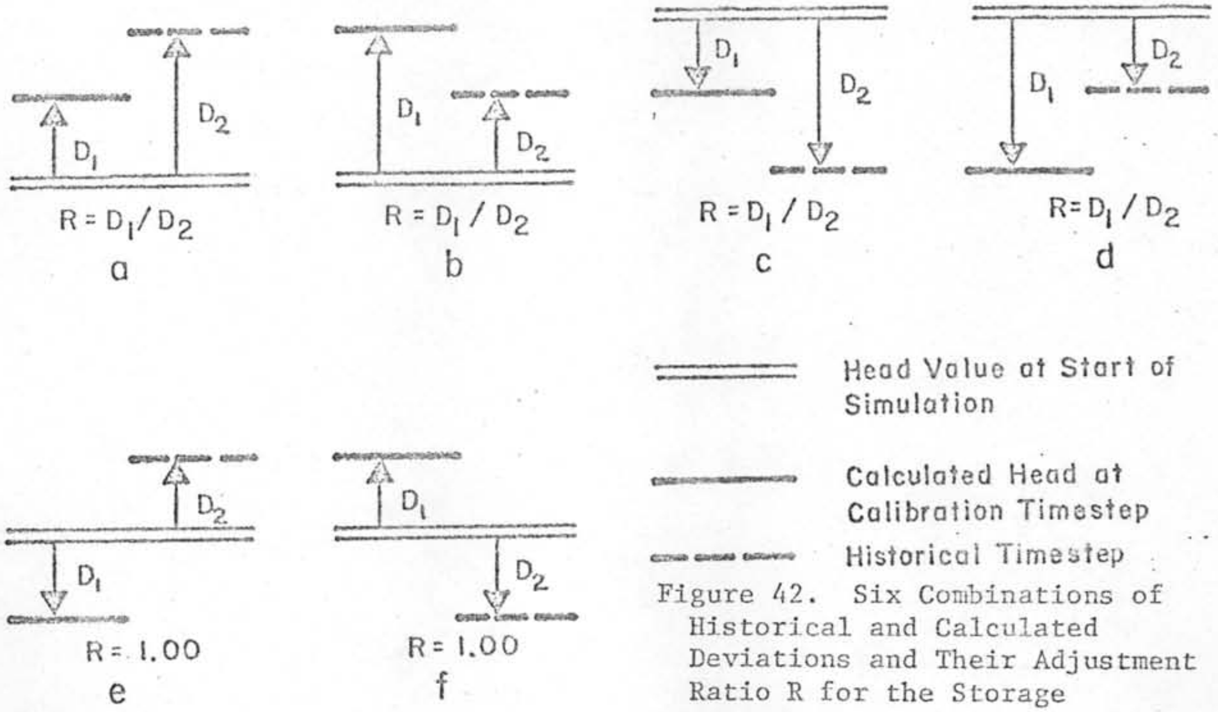

Figure 42. Six Combinations of Historical and Calculated Deviations and Their Adjustment Ratio R for the Storage Coefficient.

Figure 42 shows in six combinations of historical and calculated head deviations from the start of simulation to the calibration timestep. In the first four combinations the change of hydraulic head calculated in the model is in the same direction as the historical change and has the same sign as the input generated for this node point. For these combinations the storage coefficient is adjusted as follows

$$S_{i,j}^{n+1} = [1 + VV(R-1)]\ S_{i,j}^{n} \tag{41}$$

in which $R = (D_1/D_2)$

Generally for any node point the hydraulic head will rise if the resultant effective input to that node is positive. This effective input is a combination of the external input to the node point and a net groundwater flow to the node. The hydraulic head will decline if the resultant effective input is a withdrawal of water from the nodal area. Historic water table changes are naturally compatible with whatever the historic input was to that node. This means that if the historic change of hydraulic head is in the opposite direction of the calculated change of hydraulic head either the sign of the calculated input is not the same as the sign of the historical input or the historic head change is incorrectly interpreted from well records.

This situation is presented in combinations e and f, Figure 42. In this case it is unreasonable to change the storage coefficient of a node point without reevaluating the data available for this node point. Therefore, in the calibration routine for these combinations the storage coefficient is not changed but kept constant.

$$S^{n+1}_{i,j} = S^{n}_{i,j}$$

Again the calibration routine indicates the problem areas in the aquifer. Areas for which the storage coefficient does not change and the difference between historical and calculated hydraulic head does not decrease need to be reevaluated.

Two other cases are considered. Areas that completely lack well records are assumed to have no change in historical aquifer head. If available data indicate a net effective input to that area no logical way for determining a storage coefficient is present. In fact, any coefficient will suffice. This situation is represented in Figures 42b and 42d, with D_2 equal to zero.

The ratio (D_1/D_2) approaches infinity and suggests an unrealistic change. For these two combinations the storage coefficient is kept constant for lack of any better alternative. In areas with this situation occurring, the historical rise or decline may be set equal to the rise or decline calculated with the initial storage coefficient.

Resume of the Parameter Adjustment

The specific flow patterns in space and in time of an aquifer are the result of a combined value of two sets of aquifer parameters.

The first set, Set A, is related to the geohydrological properties of the aquifer and can be expressed in four parameters.

A
1. Transmissibility
2. Leakance factor of an impeding layer
3. Head difference between aquifers
4. Storage coefficient

The second set, Set B, is related to the water management of the area and the climatological conditions and can be expressed in the following way.

B
1. Inputs from irrigation diversion, pumping recharge, seepage.
2. Inputs from precipitation and evapotranspiration

Simulation of the historical behavior of an aquifer is dependent on the knowledge and accuracy of the aquifer data sets.

A. Hydrogeological parameters
B. External inputs to the aquifer
C. Historic water table data from wells

If data sets A and B are accurately known no calibration is necessary and simulation is readily achieved. If A or B are unknown calibration of the respective parameters is necessary and data set C is required. Historical water table data are a necessity for calibration. The choice of which data set to adjust depends upon the relative reliability of the data sets. If the geohydrological parameters are better known than the external inputs, the external inputs are adjusted until no further improvement is observed. If the external inputs are better known that the geohydrological parameters, the geohydrological parameters are adjusted.

Whichever data set is changed, caution must be exercised to stay within the reasonable range of parameter values. In the first data set uniqueness of the transmissibility values depends largely on the existence of external input/output in the aquifer; adjusting the leakance factor and head difference may lead to the generation of incorrect input, while the storage coefficient values depend greatly on the accuracy of historical water table data.

For most aquifers data set A, the geohydrological parameters, is least known. Most aquifers, especially the ones on which irrigation occurs, have data on irrigation diversion, pumping, crop distribution, temperature, precipitation etcetera to determine more or less accurately those external inputs to the aquifer. The adjustment of the geohydrological parameters is carried out in an automated way in a calibration routine. Depending on the complexity of the aquifer structure substantial differences in nodal parameter values may occur. In order to obtain parameter values for the node points which are of justifiable magnitude upper and lower limits are imposed upon the parameters, based upon the geological information of the aquifer. Calibrating the geohydrological parameters in this automated way has the additional result that it indicates where external inputs or historical water table data may be incorrect.

Adjustment of data set B, the external inputs, must be done very cautiously. It is unreasonable to double the input from irrigation for a particular node point just to match the historical head change, especially if specific data on irrigation diversion does not warrant this change. The nodal inputs are generated in a separate input program.

In case a reevaluation of the input is necessary, the components which make up the input term are investigated for their accuracy and the necessary changes made. With this new input a new calibration run is made. The fact that the input consists of several components makes it preferable not to adjust the lump sum of these components in an automated way. For instance, if the amount of precipitation is the weak component of the input an automatic adjustment of this one component may lead to a tripling of the precipitation in one node and drastic reduction of precipitation for a neighboring node in the same climatological area. A better match may be obtained but the precipitation distribution is highly unreasonable and not consistent with the climatic region. The same can be said for changes in irrigation diversions and evapotranspiration. An automatic change of these components would be based on individual head differences as is the case for the first data set, and not on climatological differences, better data on crop distributions, or improved irrigation diversion records. A logical way is to inspect the data of these components, suggest a general change in a component value for a certain area based on location characteristics. The input program is then operated again to provide a new lump sum for every node point and another calibration may be attempted. In this way a spatially erratic distribution of input components is avoided.

The automatic calibration routine adjusts four geohydrological parameters. Interaction between these parameters prevents a separate adjustment of these parameters unless a time sequence in the aquifer history can be chosen in which hydrogeological parameters can be separated.

For aquifers without a leaky aquifer interaction the only parameters considered for change are the transmissibility values and the storage coefficient. In those cases it is sometimes possible to select the time sequences in which the other parameter can be neglected. An aquifer may show a cyclic behavior of the water table caused by yearly irrigation practices. The amplitude of this cycle may be constant or nearly constant for every year. The average water table elevation for every node point over the yearly cycle combined with the yearly total input to every node creates a situation in which the aquifer is in an equilibrium state. In this equilibrium state the hydraulic head remains constant in time for every node and the term in the flow equation which contains the storage coefficient and the change of head in time is not applicable. In this case the only parameter to be considered for adjustment is the transmissibility. After the transmissibility values are calibrated a time sequence may be chosen in which the yearly rise of the water table due to water management is being simulated. This rise is dependent on the magnitude of the transmissibility values and the storage coefficients. In case the transmissibility values were adjusted in a simulation where the historical head values were in true equilibrium and the inputs were sufficiently known, the T-values would also represent the correct values in the unsteady state simulation. This allows for the adjustment of the storage coefficient as the only parameter. If the T calibration was a quasi-steady state condition both parameters have to be adjusted.

In case the aquifer has a leaky condition or in case no time sequences can be found that enable the separation of the geohydrological parameters, these parameters have to be adjusted more or less simultaneously. The extreme case is presented in the form of an unsteady state leaky aquifer for which all four parameters are to be changed. Then, a time period is chosen in which the historical water table behavior shows a definite minimum and maximum as shown in Figure 43.

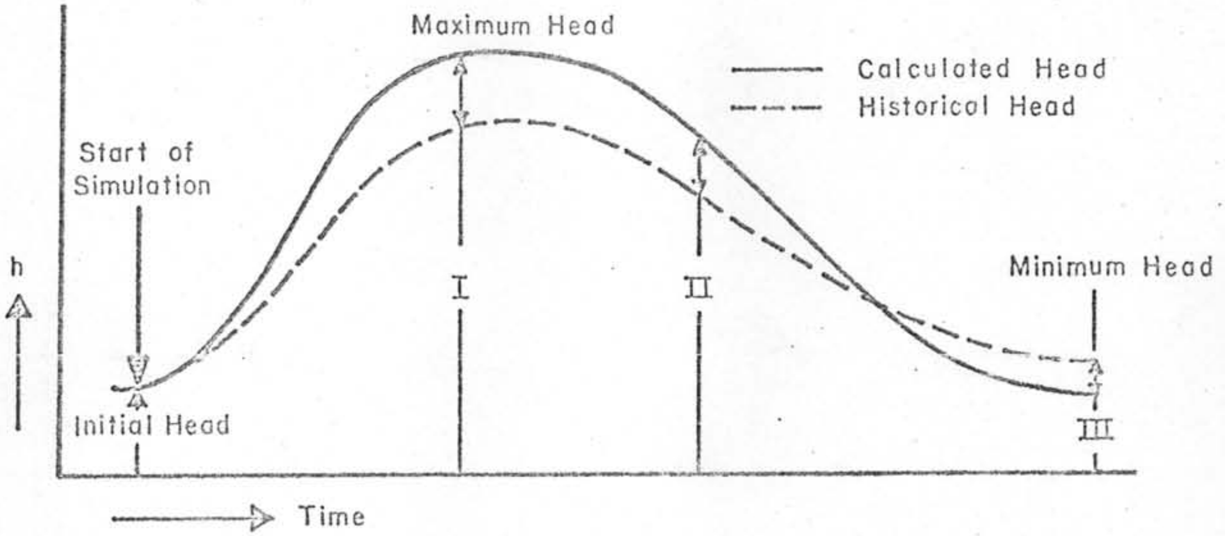

Figure 43. Schematic Representation of Simulated and Historical Water Table Behavior (Unsteady State Multi-Parameter Calibration).

Starting the simulation with the minimum head values and in case the best fit priority is given to the maximum water table the calibration changes the parameters to simulate the maximum water table in the aquifer occurring at timestep I, and the parameters are adjusted based on the deviations occurring at this timestep. For timesteps I, II, and III, the deviations of calculated from historical water table are calculated as well as the sum of squares of deviations, summed up over the three timesteps for the entire aquifer (overall SS). The first parameter is adjusted three times after which the parameter values which resulted in the minimum overall SS are stored. After that the parameter is changed back to its original value and the second parameter is adjusted, as well as the third and fourth. In a multi-parameter calibration such as this the number of successive adjustments of one parameter is kept small in order to equalize the influence of every parameter in the adjustment procedure. After all parameters are changed three times a simulation run is made with the new optimum values for every parameter. The resultant overall SS of this simulation is stored and a second calibration cycle is started in which all parameters are again changed. The resultant overall SS of the second set of optimum parameter values is compared with that of the previous cycle. As long as this sum of squares is decreasing successive calibration cycles are made. If the overall SS of the optimum set of parameter values starts to increase, the calibration is terminated and the set of optimum parameter values which resulted in the least overall SS is punched out on cards to be used as the final parameter values.

Depending on the kind of simulation (steady state or unsteady state simulation) and the kind of calibration (single-parameter or multi-parameter calibration) different procedures in calibration have to be followed. Since every procedure is carried out in an automated way the calibration routine, called 'COMPAR', requires control variables that govern the type of simulation and calibration.

These control variables are read in the main program and are the only necessary variables to provide a correct calibration.

A summary and explanation of the control variables and the input data of the model and the formats under which they are entered in the program are included in Appendix B.

CHAPTER VI

APPLICATION OF MODEL TO SNAKE PLAIN AQUIFER

Introduction

The eastern Snake River Plain in southern Idaho extends roughly 200 miles eastward and north eastward from Bliss to about Ashton, and is bounded on the northeast and south by mountain ranges and alluvium-filled intermontane valleys and on the west by an area of broad lava capped plateaus. The plain is underlain by a series of successive basalt flows that include interflow beds of pyroclastic and sedimentary materials. This series contains the Snake Plain aquifer which is one of the highest yielding aquifers in the United States.

The boundary of the aquifer represented by the dashed line in Figure 44, is drawn at the foot of the surrounding mountains and across the mouths of the tributary valleys and encompasses an area of 13,000 square miles. The main river is the Snake River which enters the area at Heise. A short distance further downstream it is joined by Henrys Fork, the major tributary, which drains the upper part of the Snake River plain. The Snake River flows near the southern boundary of the plain and enters a canyon at Milner Lake that reaches a depth of up to 400 feet. Smaller rivers enter the valley from the north and south. The aquifer lies mainly north of the Snake River from Bliss eastward to Blackfoot and covers about 9,000 square miles. North of Blackfoot the aquifer lies on either side of the river, including the Snake River Fan and part of the area above the Mud Lake region.

The groundwater flow is generally from east to west and southwest and follows the longitudinal axis of the plain. The groundwater body is augmented by snow melt in the northeastern part of the plain and seepage from rivers which are fed by snow melt in the surrounding mountains. Some rivers in the northern part of the aquifer lose all their water as they follow their course to the center of the plain. Subsurface flow from the tributary valleys add to this while a substantial recharge results from extensive irrigation diversions along the Snake River. The majority of the groundwater leaves the aquifer in two groups of springs.

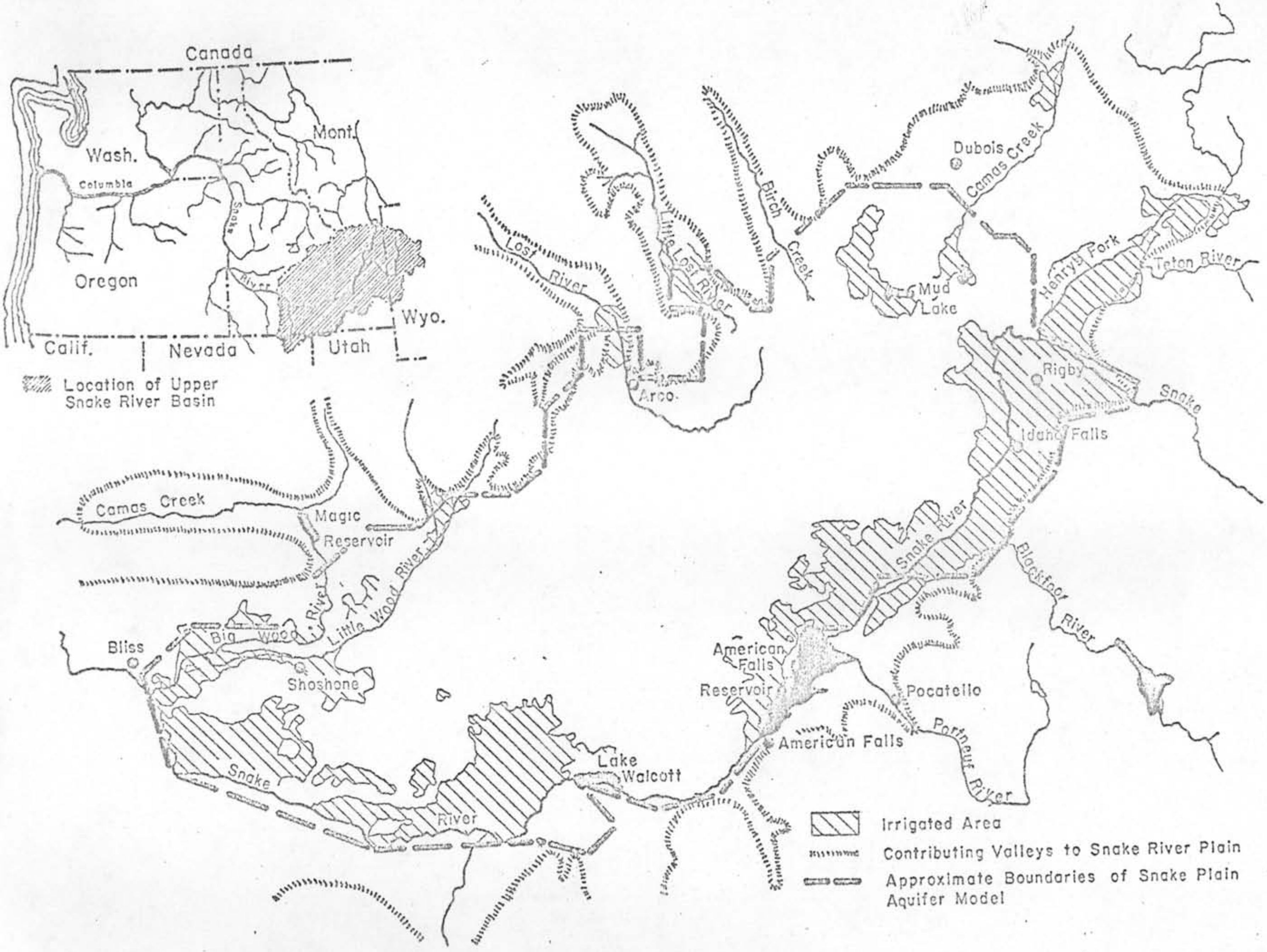

Figure 44. Snake Plain Aquifer and General Location Map.

The first area is the group extending from the mouth of the Blackfoot River to a short distance below American Falls. The second group, the Thousand Springs Area, extends from below Milner to King Hill, where the groundwater discharges into a canyon several hundred feet deep. Because of irrigation development the Snake River flows are controlled by regulatory and storage reservoirs. During years which have above normal runoff water passes downstream unused. Interest has developed in the possibility of artificially recharging the groundwater aquifer with river water during years which have above normal runoff.

Several reports have been written about the geology and water resources on the plain by Russell (1902), and about the geology and groundwater conditions by Stearns and others (1925). Reports researching the practicality of artificial recharge include the Special Report about the Snake Plain Recharge Project by the Bureau of Reclamation (1962), and progress reports on the application of an analog model to the Snake Plain aquifer including transient state investigations (Ribbens, Mantei and Phillips, 1969). Most information described above and hereafter were obtained from these reports. Although the general extend and properties of the aquifer are known, it is so large and thick that there are many areas for which data on the distribution of basalt flows and interbedded sedimentary deposits that control the movement of water are very scarce.

A recent progress report prepared by the United States Geological Survey in cooperation with the Idaho Department of Water Administration investigates two such areas; the Mud Lake Region and the general area between Aberdeen and Arco (Crosthwaite, 1969-1970). Mundorff and others (1964) evaluated the quantity of groundwater available for irrigation in the Snake River Basin and included a flow net analysis. Thomas (1969) evaluated the inflow to the Snake River between Milner and King Hill. The most recent resume on the possible future developments of the water resources on the Snake Plain is found in the preliminary report of the Interim State Water Plan for the State of Idaho (1972). The requirements of water for the Upper Snake River Basin for the year 2020 (this includes lands in tributary valleys adjacent to the regional aquifer, north and south of the Snake River) may be summed up as follows:

1. Municipal - Domestic Water

   Based on historical trends an increase from 27 billion to 79 billion gallons is expected, which is an increase of 160,000 ac ft, mostly from groundwater.

2. Industrial Water

   Based on employment records an increase from 20 billion to 70 billion gallons is expected, which is an increase of 132,000 ac ft, mostly from groundwater.

3. Trout Farms

   Since irrigation developed at the Snake Plain aquifer outflow from Thousand Springs has increased from 4,000 to over 6,500 cfs. At present 246,400 ac ft flow through commercial fish farms. Alternate uses of water in the Snake Plain aquifer will change the outflow of the springs which in turn will affect the fish farms.

4. Agriculture

   Presently, on the plain and on lands adjacent to the plain 2,327,000 acres of land are irrigated of which 638,000 acres are served by groundwater. Lands which are potentially irrigable include: 507,000 acres of Class 1; 1,723,000 acres of Class 2; and 1,393,000 acres of Class 3. Irrigated lands could increase from 2,327,000 to 5,950,000 acres. Areas which need supplemental water encompass 400,000 acres.

   If future planning would call for the development of the Class 1 lands, with an irrigation application of 5 ac ft/ac (gravity diversion) an additional 2,500,000 ac ft are needed.

5. Water Quality

   With industrial growth and increase of agricultural animals surface water will carry a higher load of waste. This either requires increased waste treatment or higher minimum flows. Recreation will increase considerably in the future. Both recreation and water quality enhancement will have great impact on the water resource.

Groundwater is a major resource of the plain. If the average value of the storage coefficient is 0.10 the aquifer contains about 520,000,000 ac ft of water per 1,000 feet of depth. Discharges from the aquifer to the Snake River at Thousand Springs average 4.7 to 4.9

million ac ft annually. This large body of groundwater could be used to fulfill future demands of water, since the surface water supply of the Snake River and tributaries will not be sufficient, especially in the dry years. The conjunctive use of the surface water supply and groundwater supply can be represented in the form of artificial recharge of the aquifer from surface water during wet years. During these years the groundwater body stores surplus water that may be used to supplement the surface water for irrigation demands in dry years and/or supply water to newly developed irrigated lands. Increasing need for water necessitates a higher efficiency of water use. Some of the questions related to management planning involve the way the flow of Thousand Springs, vital to Idaho's trout industry, will change when more water is pumped from the aquifer; what effect a major irrigation development will have on the water table elevations in other areas of the Snake River plain; in which way the groundwater levels will be affected by efficiency improvement of existing surface irrigation systems; and how different management plans would affect the flow to and from the Snake River. In order to find the answers to these and other questions the finite difference model, described previously is applied to the Snake Plain aquifer. The model must be able to simulate the historical behavior of the aquifer in order to predict with certain reliability the effects on the water table of different water management changes. The success of the simulation depends on the amount of data available, such as the input to the aquifer and the values of the hydrogeological parameters.

Calibration

If data about the geohydrological parameters are limited simulation of historical behavior cannot be achieved without a calibration of the parameters. Although some of the components that comprise the source term for the Snake Plain aquifer are not as well defined as other components, the input to the aquifer can be calculated fairly well from available data.

This is not the case for the geohydrological parameter values.

The Snake Plain aquifer contains a series of basaltic lava flows which include interflow beds composed of pyroclastic and sedimentary materials (Norvitch and others, 1969). Groundwater flow takes place in the interflow zones which are interconnected via vertical rock joints and fault zones. On a microscopic scale the groundwater flow system is anisotropic and non homogeneous.

On a macroscopic scale a model unit or nodal area for which a head value is to be calculated comprises several square miles and the effects of anisotropy tend to be minimized. On this scale the aquifer can be regarded as an isotropic aquifer with transmissibility values representing the average value for the nodal areas over the entire thickness of aquifer. Although the Snake Plain aquifer contains some areas with a perched water table interest is focused on the management of the regional water table. The aquifer has its base on the bedrock several thousands of feet below the land surface and is considered a non-leaky aquifer, leaving two parameters that describe the properties of the flow system; the transmissibility values and the storage coefficients. General information about these two parameters is available but the sheer size of the aquifer to be modeled leaves many areas for which only estimates exist so that calibration of these two parameters is necessary.

The calibration was done in two steps, representing two time sequences that were chosen to calibrate the parameters. The first time sequence represents an equilibrium water table condition. The term in the flow equation containing the storage coefficient becomes zero and the water table configuration is determined by the transmissibility values and the input to the aquifer. Historic well data indicate that during the period from 1963 to 1968 the average yearly water table values were nearly constant. The 1966 water year October 1965 - October 1966 was taken as a representative year. With yearly half timesteps and applying to every half timestep the same 1966 inputs a steady state (or equilibrium state) condition was created in which initial transmissibility values were adjusted to match the average 1966 water year water table elevations. Since the 1966 water table is not a true steady state this steady state calibration served to bring the T-values close to their true values.

After adjusting the transmissibility values the second step involves the adjustment of the storage coefficients and a readjustment of the transmissibility values for which a period was chosen that shows a varying water table configuration in time. Because the adjustment of the storage coefficient is based on the relative changes between calculated and historic head a better calibration and more reliable storage coefficient values are obtained if the adjustments are made in a time period for which the inputs are most accurately known and the historical head values show a maximum variation. The historical inputs from year to year are difficult to obtain and less reliable for early years of the simulation. The rise or decline of the average head values in the Snake Plain from year to year is in the order of 0.0 to 1.0 foot, which is very small. Therefore for the second calibration step a one year time period was chosen from April 1, 1966 to April 1, 1967, divided into two-week half timesteps. The model was calibrated to simulate the historical seasonal fluctuations of the water table as caused primarily by the irrigation in the area. Inputs were generated for every half timestep of the 1966-1967 period. During this time period, rise or decline in the water table ranges from 0 to 20 feet, a more pronounced change which will lead to a more sensitive adjustment of the storage coefficients.

System Simulation

Figure 45 shows a map of the study area on which a square grid is superimposed in which $\Delta x = \Delta y = 5{,}000$ meters (about three miles).

Boundaries of the Study Area

Figure 45 shows also the location and type of the boundaries in the study area. In the northern part and northeast the aquifer is bounded by the mountains and therefore delineated as an impermeable boundary. This impermeable boundary extends into the tributary valleys such as the Little Wood, Little Lost and Big Lost basins in order to provide an area to distribute the tributary valley underflow.

The impermeable boundary in the northeast terminates the aquifer

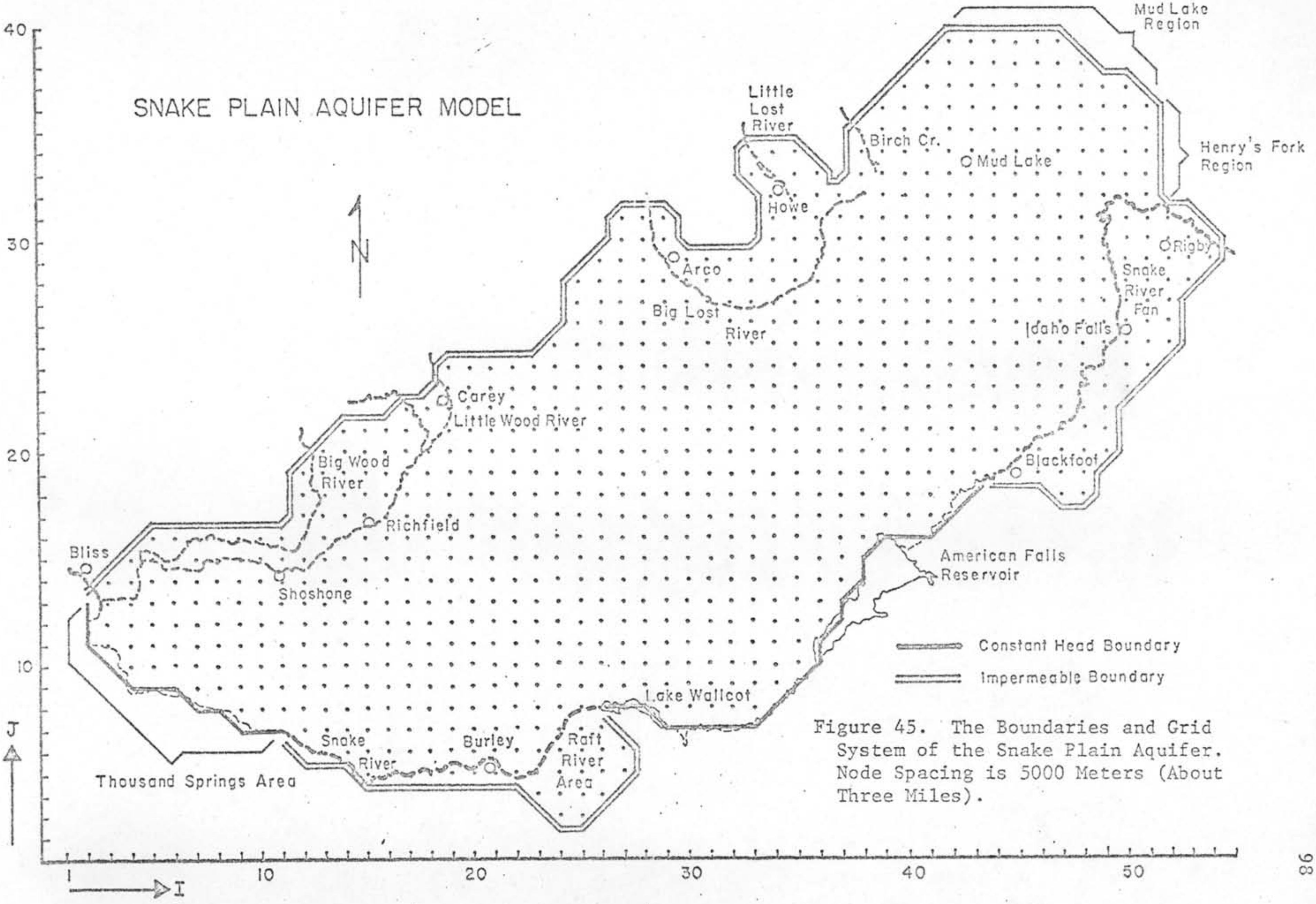

Figure 45. The Boundaries and Grid System of the Snake Plain Aquifer. Node Spacing is 5000 Meters (About Three Miles).

about 10 miles north and east of the Mud Lake region, going south to the confluence of the Henrys Fork with the Snake River. The groundwater inflow from the northeast (flowing to the Mud Lake region), and to the east (flow from the Henrys Fork region) is distributed over nodes along this boundary. The impermeable boundary borders the Snake River Fan area following the eastern foothills and extending for a little ways in the Blackfoot drainage area.

West of Blackfoot the Snake River is hydraulically connected and serves as a constant head boundary from Blackfoot to Minidoka Dam, west of Lake Walcott. The groundwater flow is to the river and American Falls Reservoir, where a significant amount of water returns to the river via a group of springs. Along Lake Walcott groundwater flow is to the aquifer. West of Lake Walcott the Raft River Basin is approximated by a rectangular shaped valley where the groundwater flow from the Raft River basin is distributed. The boundary between the Raft River basin and Kimberly is not clearly defined. In order to model this area an artificial no-flow (impermeable) boundary is drawn perpendicular to the equipotential lines.

Between Kimberly and Bliss the aquifer is terminated by the Snake River Canyon where the groundwater leaves the Snake Plain via a large group of springs. This boundary can be simulated in several ways. One way is to substitute an impermeable boundary along the canyon wall and situate artificial wells at the locations of the springs with the same pumping rate as the spring discharge. Some problems may be encountered since the spring discharge is a function of the water management in the Snake Plain and an adequate correlation between the springs and some water management parameter is not readily available.

Another way is to substitute an impermeable boundary and impose a leaky aquifer condition on the nodes along the boundary in which the head in the leaky aquifer represents the average level of the springs. By adjusting the leakance factor, the total leakage is made equal to the springs discharge. This solution also has a drawback. In case of a steady state calibration where the transmissibility values are adjusted in an automatic way, the leakance factors for this special case have to be adjusted manually to allow for the correct flows across the boundary.

Actually, the combination of impermeable boundary and a leaky aquifer condition at these nodes effectively induces a constant head boundary one grid mesh outside the boundary at the level of the springs. The leakage is then equivalent to the flow that would be calculated via hydraulic gradients and transmissibilities along this boundary.

It is therefore logical to represent this boundary by a constant head boundary at the average level of the water table along the canyon wall. The flow across this hydraulic boundary is matched with the spring flow by adjusting the transmissibility values at the constant head boundary nodes; because the transmissibility values are already adjusted in the calibration, the matching of the spring outflow is automatically incorporated.

Steady State Calibration

In the steady state single-parameter calibration initial transmissibility values were adjusted to match the average 1966 water table elevations as explained on page 67 . The values assigned to the control variables of the main program and calibration routine for this type of calibration are given in Table 2.

Table 2. Control Variables for the Steady State Calibration - Snake Plain Aquifer

Card 1	Card 2	Card 3	Card 4	Card 5	
MICX=0	NPX=0	NQ=4	NROW=55	NVA1=1	NV(3)=0
MICY=0	NPY=0	FLUX=0.0	NCOL=40	NVA2=1	N10=1
XA=13.70	NSUI=0		DELT=130.0	NVA3=1	JI=10
YA=10.65	NSU2=0		DELX=16404.2	NSTOP=1	JK=10
NIM=60	NSU3=0		DELY=16404.2	ITE=40	VP=0.40
NSER=0			LTS=131	NST=1	SREST=500.
			NTS=131	NV(1)=0	REST=900.
			NRIVER=0	NV(2)=0	RMI=150.

The first four cards contain the control variables of the main program while the control variables of the fifth card specify the type of calibration. A precise explanation of these control variables is given in Appendix B.

Data Collection for the Study Area

The input of the model consist of one dimensioned variable and 15 arrays in the following order: NREACH(I,J), DELH(K,I), DRAIN(I,J), Q(I,J), KX(I,J), KY(I,J), FAC(I,J), PPI(I,J), S(I,J), NCX(I,J), NCY(I,J), NN(I,J), SURF(I,J), Z(I,J), PHI(I,J,1), APHI(I,J,1). A precise explanation of the variables and the format under which they are entered in the program is included in Appendix B.

NREACH(I,J): Identifies a hydraulically connected stream node by reach number. Only read in if stream node has a changing water level, which is not the case in the steady state calibration.

DELH(K,I) : Represents the change in water level from t = I-1 to t = I for stream node within reach No. K, not applicable here.

DRAIN(I,J) : Denotes the drain level for node (I,J). No drains are specified in the Snake Plain aquifer, since the large grid spacing (three miles) makes that precise location impossible.

Q(I,J) : Represents the external source term which is generated in a separate input program discussed on page .

KX(I,J) : Represents the hydraulic conductivity in x-direction.

KY(I,J) : Represents the hydraulic conductivity in y-direction. As explained on page 9 , the aquifer is considered to be isotropic, unconfined and non-homogeneous, making KX(I,J) equal to KY(I,J). The initial transmissibility values for the calibration were obtained from the United States Geological Survey as determined from the flow net drawn by Mundorff and others (1964). Assuming an average aquifer thickness of 5,000 feet, the hydraulic conductivity values were obtained by dividing the transmissibility values by 5,000. The aquifer thickness is believed to range between 3,000 and 6,000 feet, but actual values are not essential for the calculation of the hydraulic head since transmissibility values are used.

FAC(I,J) : Represents the leakance factor in case of a leaky aquifer.

The Snake Plain aquifer is considered a non-leaky aquifer making FAC(I,J) a zero array.

PPI(I,J) : Represents the initial head difference between main aquifer and underlying or overlying water bearing formation , not applicable for the Snake Plain aquifer.

S(I,J) : Denotes the storage coefficient in the aquifer. In an equilibrium state calibration the values assigned to this array are not essential. A uniform storage coefficient of 0.15 is entered.

NCX(I,J) : Array denoting the boundary type for row wise calculation of the nodal head values. The values for this array are derived from the boundary conditions imposed on the aquifer. A detailed explanation is given in Appendix B. Figure 46 represents the NCX array of the Snake Plain aquifer.

NCY(I,J) : Array denoting the boundary type for the column wise calculation of the head values. Derived in a similar way as the NCX array. Figure 47 represents the NCY array of the Snake Plain aquifer.

NN(I,J) : Denoting the confined or unconfined character of the groundwater flow. The major part of the aquifer shows an unconfined flow condition. The inter flow between the basalt layers may show artesian conditions but looked upon in a macroscopic way over the entire depth of aquifer, the system is considered to be unconfined. Therefore NN(I,J) is a zero array.

SURF(I,J) : Represents the land surface elevation for each node point, which were interpolated from topographic maps.

Z(I,J) : Represents the aquifer bottom elevation necessary to calculate the aquifer thickness utilized in the calculation of the transmissibility values. The aquifer bottom elevations were obtained by subtracting 5,000 feet from the initial water table elevations.

PHI(I,J,1) : Represent the initial water table elevation for every node point. A computer contour plot program was adapted to generate alpha-numeric contour lines from a network of 205 wells, monitored by the United States Geological Survey. Figure 48 shows the location of the wells being used. Of the 205 wells 98 are current observation wells, measured monthly, bi-monthly, or every half year, 30 wells are discontinued observation wells and 77 wells are inventory wells most of which have only been measured once. The computer contour plot was drawn with 1966 water year average well levels. From this contour plot the nodal values were interpolated. Figure 49 shows a map with the 1966 average water table contours at 20 foot intervals.

APHI(I,J,1): Represents the historical water table values to be simu-

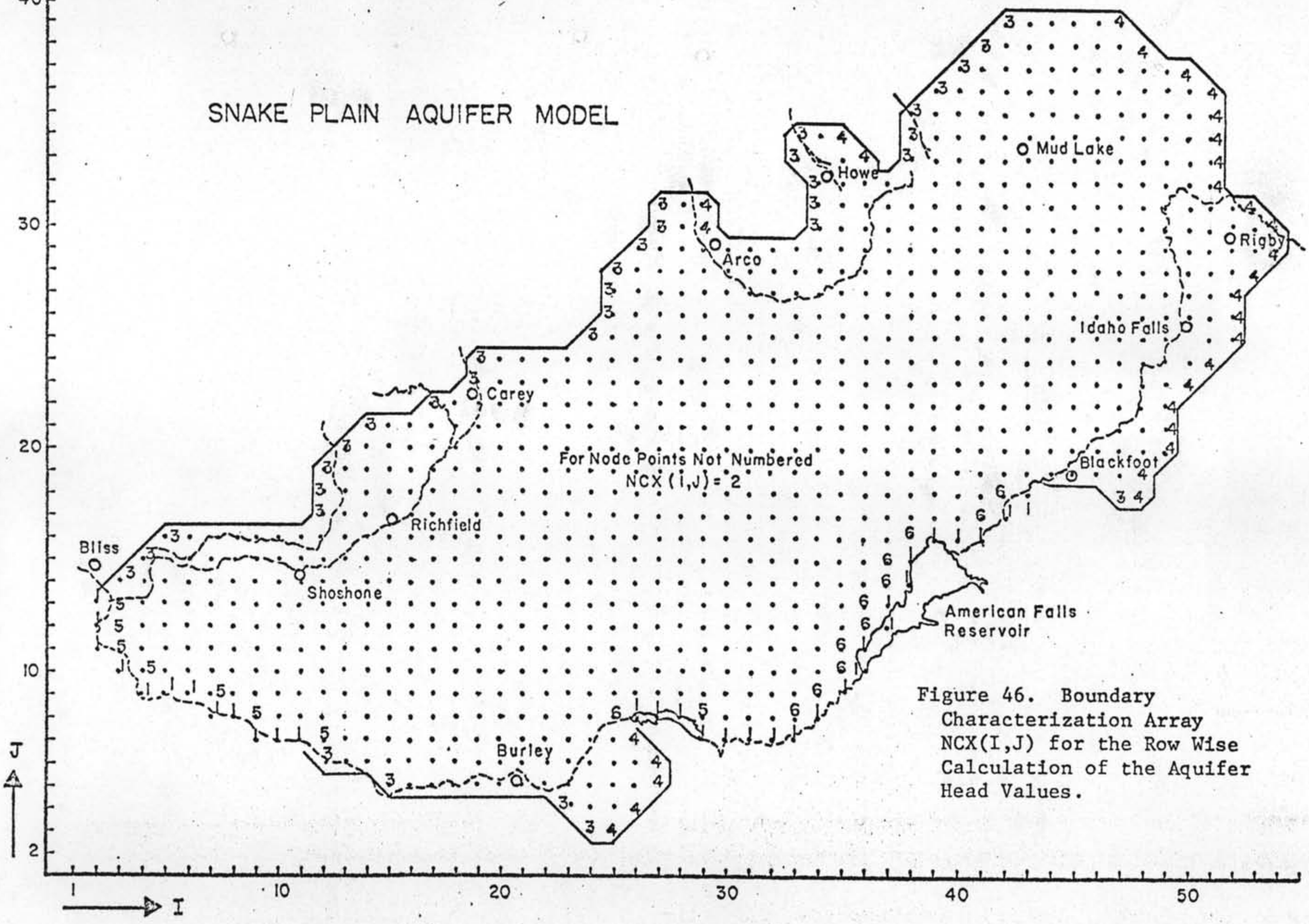

Figure 46. Boundary Characterization Array NCX(I,J) for the Row Wise Calculation of the Aquifer Head Values.

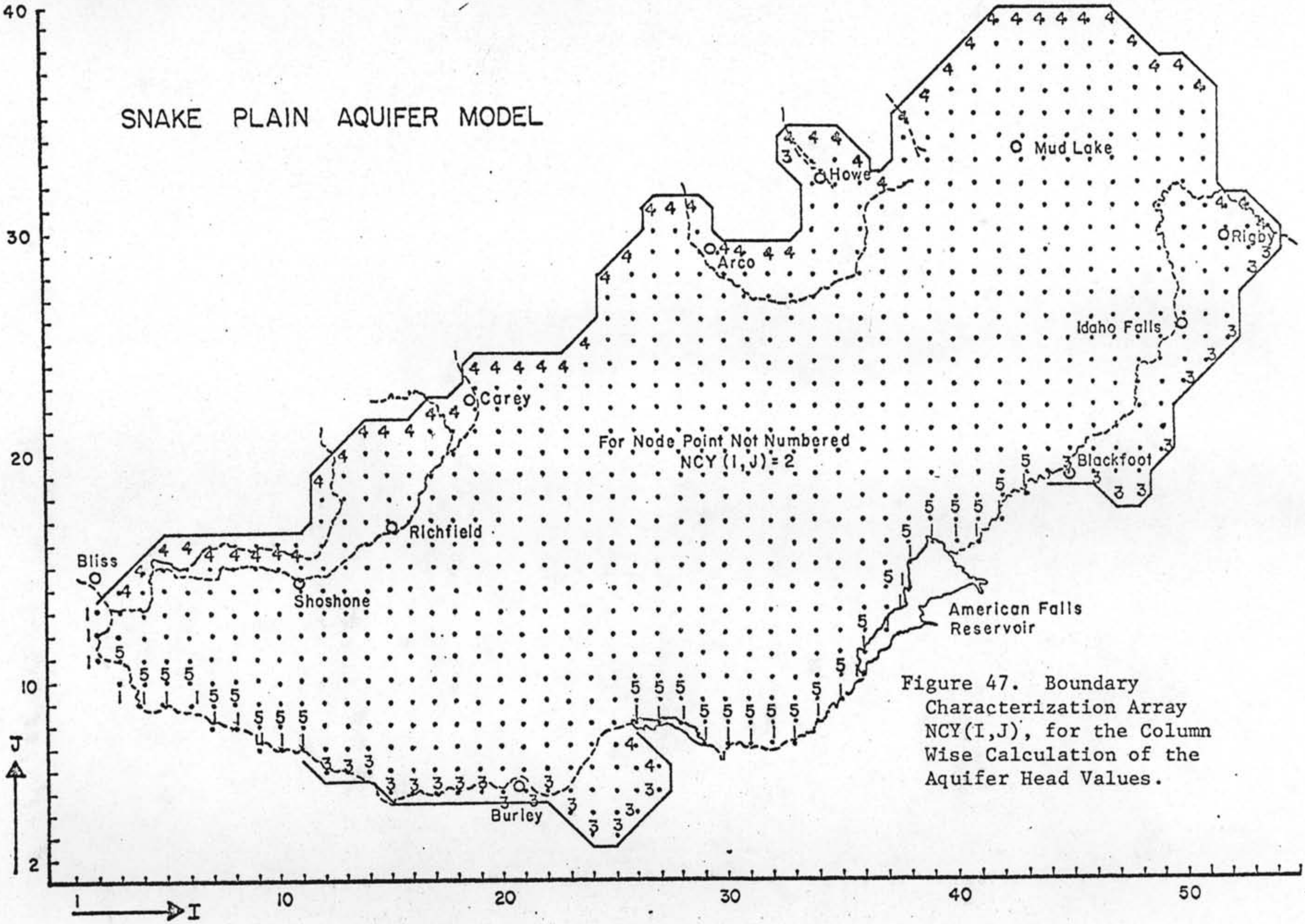

Figure 47. Boundary Characterization Array NCY(I,J), for the Column Wise Calculation of the Aquifer Head Values.

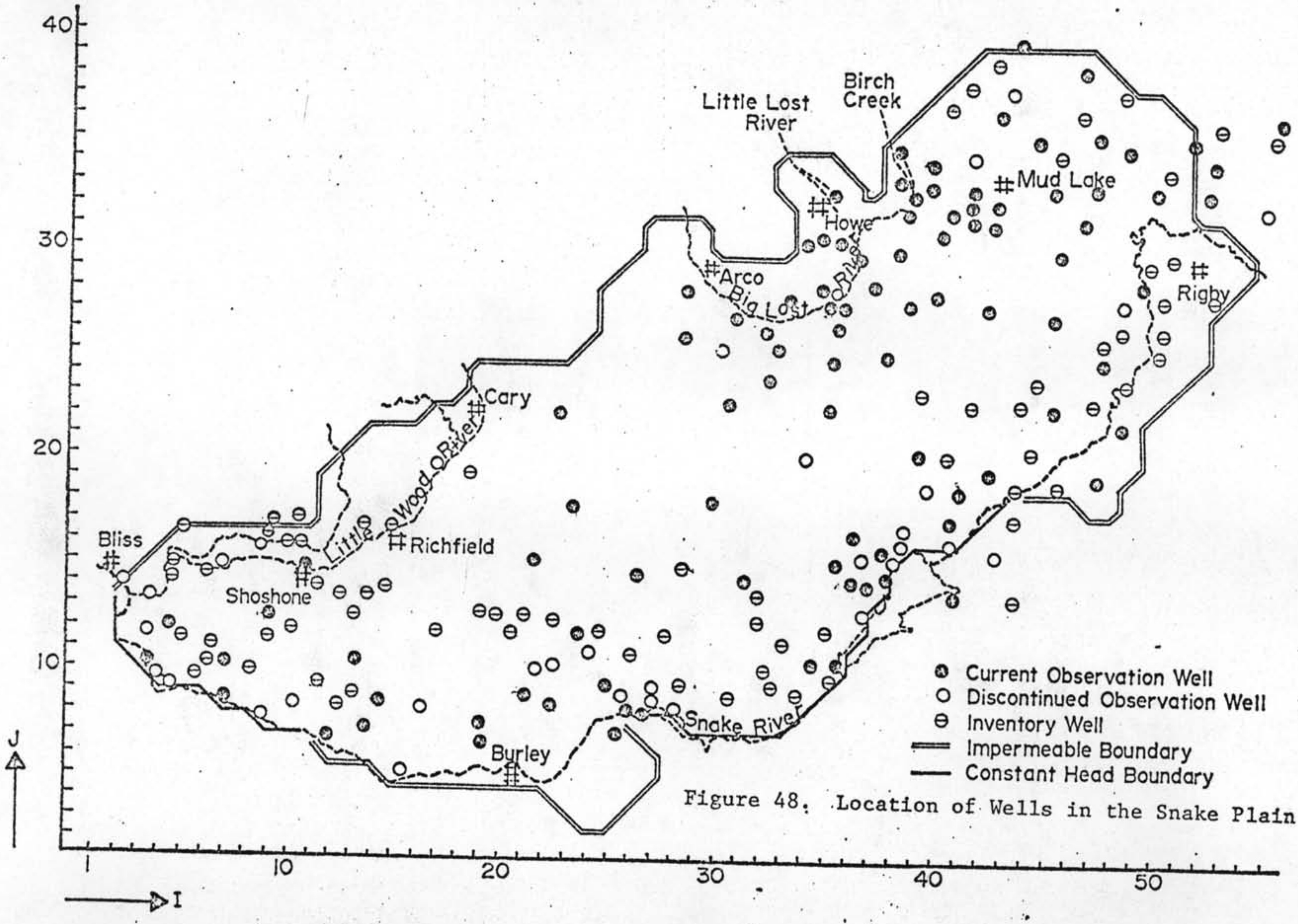

Figure 48. Location of Wells in the Snake Plain Aquifer.

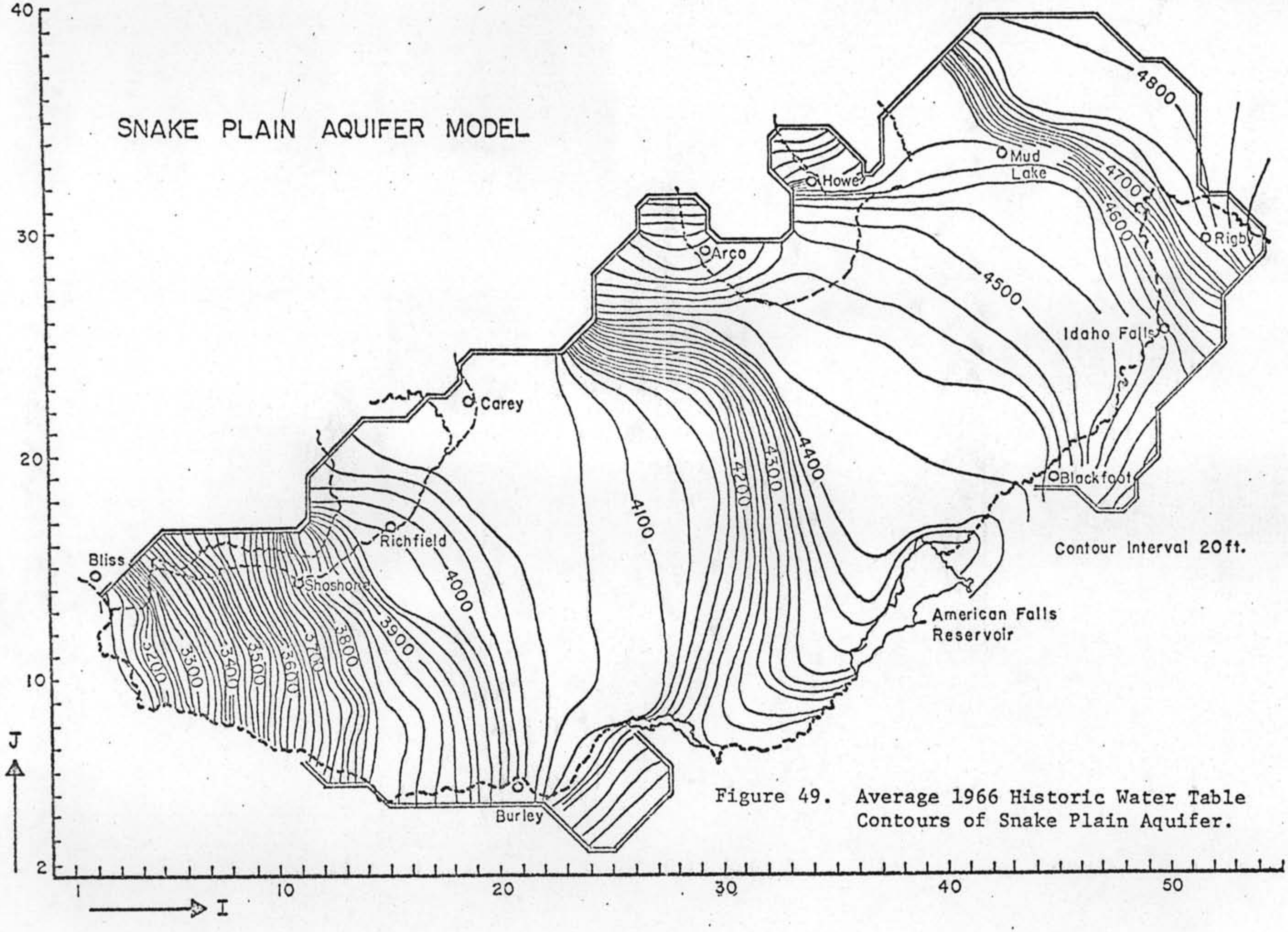

Figure 49. Average 1966 Historic Water Table Contours of Snake Plain Aquifer.

lated in the calibration (see Appendix B). Since this is an equilibrium state calibration the historical head values are the 1966 water year average water table elevations APHI(I,J,1) = PHI(I,J,1).

Inputs for 1966 Steady State Calibration

1966 data were used to generate the source term Q(I,J) for every node point. General information about the study area and the data necessary to calculate the inputs follow hereafter. A detailed explanation of the variables that serve as input to the input program and the format under which they are entered are included in Appendix A.

As described by Norvitch and others (1969) irrigation began on the plain in the latter half of the nineteenth century. The Carey Act of 1894 and the Federal Reclamation Act of 1902 provided the primary incentives and the means for a rapid growth of irrigation in the early 1900's.

Most of the easily accessible, arable land on the plain was developed by the mid-1920's. The first lands irrigated were those to which water could by conveyed by gravity flow in canals adjacent to the streams. Since the mid-1920's and continuing until about the last 1940's, a tapering off occurred in the growth of irrigated acreage. In the late 1940's, a resurgence in land development was brought about by use of groundwater for irrigation. By late 1965, an estimated 40% of all irrigated lands within the plain was either wholly or supplementally supplied by groundwater pumped from wells.

Irrigated Acreages

For the year 1966 the Soil Conservation Service, in its Type IV study, planimetered for every nodal area in the aquifer the surface water irrigated acres and groundwater irrigated acres from irrigation district maps. Total acreage was also delineated for each district. Groundwater irrigated acreages and surface water irrigated acreages not organized in districts were recorded under SUREST(I,J) and GRREST(I,J) (see Appendix A). Table 3 shows a list of the irrigation districts on the Snake Plain

Table 3. Irrigation Districts in the Snake Plain Aquifer with Return Flow Percentages, Calculated and Recorded Acreages and 1966 Total Diversions.

Dist. No.	Name	Return Flow (%)	Recorded Acreage	Calculated Acreage	1966 Div. (ac ft)
1	Texas Feeder	71	10,000	10,570	78,300
2	Butte and Market Lake Canal	10	20,000	31,720	92,400
3	Liberty Park Irr. Co.		-	-	
4	Reid	71	5,500	2,560	54,700
5	Lenroot Canal Co.	71	3,100	3,170	33,900
6	Sunnydell Irr. District	71	3,780	4,620	44,800
7	Island Irrigation	31	5,500	3,230	52,400
8	Parks & Lewisville	31	7,000	7,980	104,700
9	North Rigby Irr. & Canal	31	1,400	1,370	17,700
10	Rigby Canal & Irr.	31	4,000	3,310	58,500
11	Dilts	31	580	560	7,900
12	East Labelle Irr. Co.	31	3,000	2,270	38,800
13	Clark and Edwards	31	1,800	2,010	23,500
14	Lowder Slough Canal Co.	31	1,000	1,110	13,600
15	Burgess Canal Irr. Co.	12	22,000	25,450	291,600
16	Harrison Canal Irr. Co.	10	13,000	10,550	169,600
17	Rudy Irr. Canal Co.	10	5,000	5,270	72,400
18	Farmers Friend Irr. Co.	10	10,000	11,170	119,800
19	Enterprise Canal	10	5,200	5,220	48,500
20	Butler	31	1,110	1,000	12,600
21	Progressive Irr. Dist.	10	33,000	35,350	284,600
22	Poplar Irr. District		-	-	
23	Osgood (Utah Idaho Sugar)	0	6,200	6,290	16,000
24	Owners Mutual (Kennedy)		-	-	
25	West Side Mutual		-	-	
26	Shattock Irr. Co. (Kennedy)	23	2,700	3,750	10,600
27	Idaho Irr. District	33	35,860	37,330	325,100
28	Martin Canal Co.		-	-	
29	Great Western Porter (New Sweden)	23	30,220	31,630	238,000
30	Woodville Canal Co.	23	2,350	2,330	26,900

Table 3.-Continued

Dist. No.	Name	Return Flow (%)	Recorded Acreage	Calculated Acreage	1966 Div. (ac ft)
31	Snake River Valley Irr. District	33	21,520	23,950	209,600
32	The New Lava Side Ditch	10	6,000	6,080	44,200
33	Corbett Slough Ditch Co.	33	6,000	5,250	61,100
34	Blackfoot Irr. Co.	33	15,000	15,790	97,800
35	Peoples Canal & Irr. Co.	32	20,000	21,690	139,600
36	Aberdeen Springfield Co.	32	63,000	73,960	388,500
37	Riverside Ditch Co.	10	5,000	3,100	39,100
38	Danskin Ditch Co.	10	6,000	6,000	58,300
39	Wearyrick Ditch Co.	10	1,600	1,550	16,300
40	Trego Ditch Co.	10	1,620	1,670	21,200
41	Watson Slough Ditch & Irr.	10	3,000	3,150	35,900
42	Fort Hall Indian Res. New Land (Fort Hall Michaud)	15	8,693	11,490	30,200
43	Michaud Flats Project	15	6,720	11,780	26,300
44	Minidoka Irr. District	21	72,000	80,130	499,700
45	Burley Irr. District	21	48,000	54,170	383,100
46	Minidoka No. Side Pumping	0	14,520	19,270	57,000
47	Milner Low Lift	0	13,470	13,470	70,100
48	North Side Canal Co.		160,000	182,580	1,317,900
49	Riley	23	900	900	6,190
50	North Gooding	0	38,316	38,316	151,800
51	South Gooding	20	23,620	23,620	125,300
52	Dietrich Tract	0	13,950	13,950	74,400
53	Richfield Tract	0	24,100	24,100	119,300
54	Shoshone Tract	0	10,240	10,240	54,600
55	Shoshone to South Gooding	0	3,640	3,640	9,500
56	Little Wood	0	11,330	11,330	112,200
57	Milner Gooding	0	21,900	21,900	133,490
58	West Labelle Long Island	31	10,500	16,030	153,200
59	Little Lost	0	12,160	12,160	52,900
			862,819	947,766	

↳ indicates that irrigation districts have combined diversions.

aquifer, with the total acreage per district as recorded in the reports of the water master for District 1 of the State of Idaho, and the calculated acreage obtained by the Soil Conservation Service. Estimated return flow percentages for each district are also included in Table 3. The calculated acreage is an average of 9.8% higher than the recorded acreage. The calculated total acres per district are used in the input program calculations. Irrigation districts 1 through 49 are denoted by the same serial number as they are reported in the District 1 records. Irrigation districts 50 through 59 are arbitrary numbers assigned to districts for this study. Districts 50 through 57 are irrigation districts in the Wood River Basin.

Surface Water Diversions

Irrigation diversions for the first 49 districts are recorded in the reports of Water District 1. These measurements, performed by District 1 and U. S. Geological Survey personnel, are obtained primarily by periodic current metering of the canals at selected rating sections and reporting of daily staff gage readings by water masters. Table 3 shows the total diversion per district for the 1966 water year. The table also shows the annual percentage of return flow obtained from the river operations model of the Idaho Water Resources Board.

Table 4. Irrigation District Denoted by Model Number and Reach Combination.

Model No.	Name	Reach Combination	I.W.R.B. No.
50	North Gooding	North Gooding (Hist.Div.N.G.)	110
		Thorn Creek to Gooding	120
		Div. #100 Use	100
51	South Gooding	South Gooding	251
		Div. South Gooding to Gooding	252
52	Dietrich Tract	Dietrich Div. - F waste	220
		Div. below Dietrich canal	221
53	Richfield Tract	Richfield use	210

Table 4 - Continued

54	Shoshone Tract	Hist. Div. North Shoshone	81
55	Little Wood	Div. Little Wood near Carey	17
56	Shoshone to South Gooding	Historic Diversion Shoshone to above South Gooding	250
57	Milner Gooding	Milner Gooding Canal Use between Milner and Gooding	

The irrigation diversions for the districts of the Wood River Basin are composed of several river reach diversion records as compiled by the Idaho Water Resource Board. Table 4 shows the respective irrigation districts and the combination of reach diversions for every district. The total surface water irrigated acreage adjacent to the Little Lost River is lumped together as Irrigation District 59. The irrigation diversions for this district are represented by the total diversion from the Little Lost river near Howe.

Irrigation districts 16, 18, 21, 27, 31, 33 and 34 are located in the eastern part of the Snake Plain where the hydrology is fairly complex. In order to distribute the correct amount of water in the area shown in Figure 50, data from the District 1 reports and total water use figures as calculated by the Idaho Water Resources Board are combined. For the area inside the solid line, Area I, monthly total water use figures are calculated by the Idaho Water Resource Board from inflow and outflow records and denoted by Use 1. This area includes the Idaho Irrigation District, the Snake River Valley district and parts of the Farmers Friend, Harrison and Progressive Irrigation Districts. The total irrigation diversions of Harrison, Farmers Friend, and Progressive Irrigation Districts are known from the District 1 reports and denoted by the numbers 16, 18 and 21.

The amounts of water flowing into the solid line area, denoted by 16',18' and 21' are obtained from the study on the Rigby Fan area (Brockway and De Sonneville, 1973). For 1966, Use 2, Use 3 and Use 4 are calculated by subtracting 16' from 16, 18' from 18, and 21' from 21. Combining above information a total water use figure (Use ^{o}) can

Use 1= Total Water Use in Area I
Use 2= 16-16'
Use 3= 18-18'
Use 4= 20-20'

Use^{0} = Use1 + Use2 + Use3 + Use4

Irr. No.	Districts in Area I	Percentage of Use^{0}
16	Harrison	= 15.5%
18	Farmers Friend	= 10.8%
21	Anderson and Eagle Rock	= 25.5%
27	Idaho Irr. District	= 28.6%
31	Snake River Valley	= 19.6%

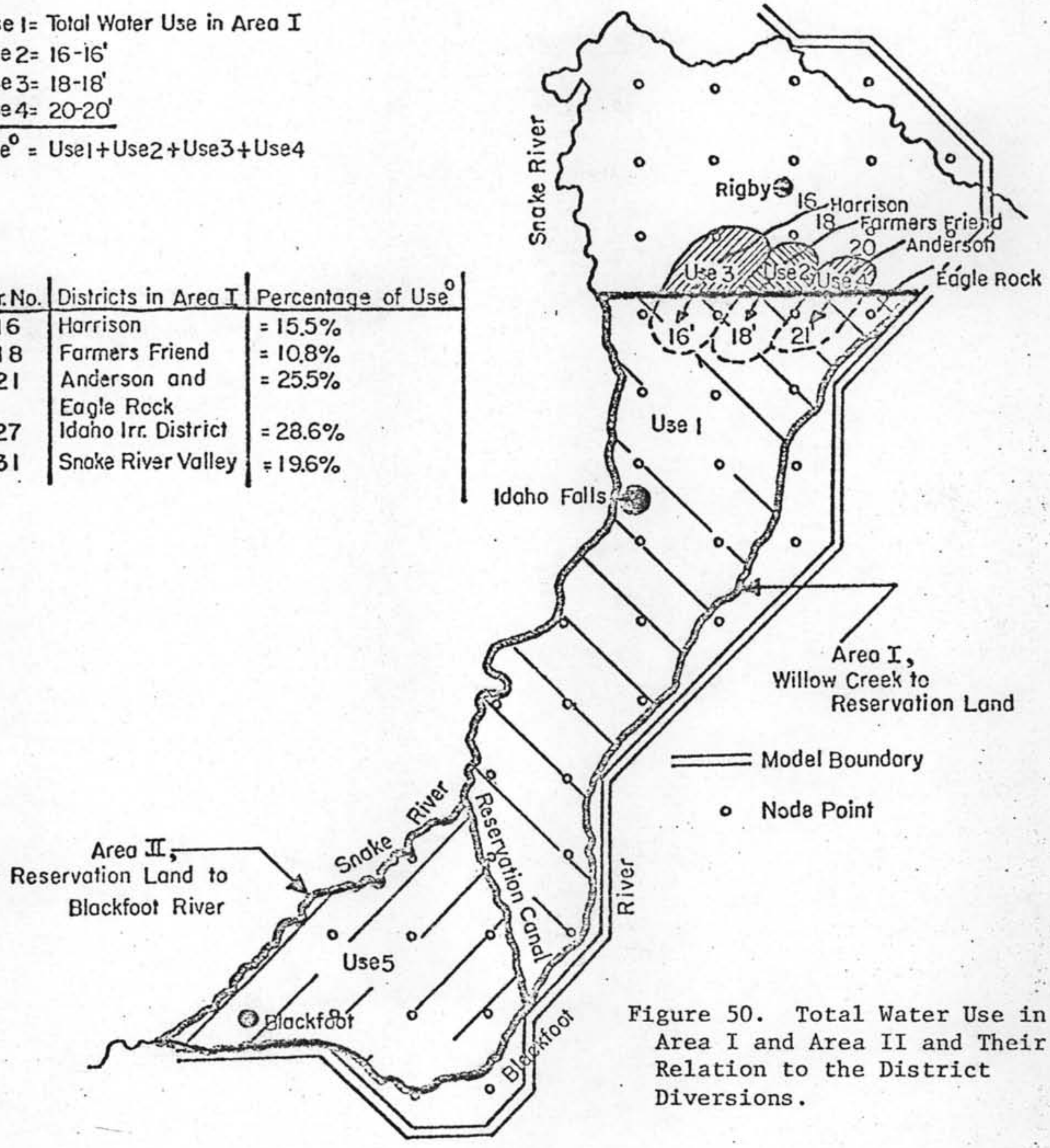

Figure 50. Total Water Use in Area I and Area II and Their Relation to the District Diversions.

Irr. No.	Districts in Area II	Percentage of Use 5
33	Corbett	= 30%
34	Blackfoot	= 40%
		24% is irrigated by non-organized lands

accurately be calculated for the area composed of Area I and the shaded areas according to equation

$$\text{Use}^{0} = \text{Use } 1 + \text{Use } 2 + \text{Use } 3 + \text{Use } 4$$

The irrigation diversions of the districts in this area, as recorded in the District 1 reports are replaced by their respective percentages of Use^{o}. These percentages are based on the relative magnitude of the historic diversions of the districts.

The percentage figures shown in Figure 50 are assumed to be constant and independent of the magnitude of Use^{o} figure and are used in the steady and unsteady state calibration.

The same procedure was used for the second area bounded by the Reservation Canal and the Blackfoot River, Area II; total water use calculated for this area is called Use 5. For 1966, the historic diversions for the Corbett and Blackfoot irrigation districts amounted to 30% and 46% of Use 5. The balance, 24%, is the total use on non-organized irrigated lands in the area. Again the irrigation diversions for the Corbett and Blackfoot Irrigation districts are replaced by percentages of Use 5, as shown in Figure 50.

Ground Water

The groundwater pumped for irrigation usually exceeds the amount of water consumed by the crops. It is assumed that the volume difference between gross pumpage and consumptively used water in the groundwater irrigated areas returns as recharge to the aquifer at the same time the pumping occurs. This means that the groundwater irrigated areas can be treated just by calculating the consumptive use from crop distribution and climatological data.

Consumptive Use

The Snake Plain aquifer is divided into climatic regions as delineated in University of Idaho Agricultural Experiment Station

Bulletin 516 (Sutter Corey, 1970). The Idaho Water Resources Board operates a computer program that for every climatic region calculates the consumptive use from climatological data and crop distribution records for the region. The assumption was made that for every nodal area in the climatic region the crop distribution is the same. The 1966 consumptive use for the climatic regions in the Snake Plain aquifer area is given in Table 5. The values in Table 5 represent the evapotranspiration occurring in the irrigation season. Because of a lack of data these values do not take into account the off season evaporation and consumptive use.

Table 5. Consumptive Use, and Precipitation for the Climatic Regions of the Snake Plain Aquifer (University of Idaho Agricultural Experiment Station Bulletin 516)

Climatic Region No:	1966 CU (inches)	1966 Precipitation (inches)
1	19.92	9.78
10	19.08	11.12
11	18.72	15.82
15	18.72	15.10
17	20.16	8.87
18	19.08	9.32
22	19.08	9.18
29	22.56	11.14
32	22.32	8.21
38	21.84	10.22
41	21.36	9.58

Precipitation

Precipitation records are available for each of the climatic regions from University of Idaho Agricultural Experiment Station Bulletin 516 (Sutter,Corey 1970). The 1966 precipitation which is assumed to be uniformly distributed over the climatic region is listed in Table 5. Mundorff and others (1964) show that more than half the Snake Plain receives less than 10 inches of precipitation annually. Marginal parts of the plain, notably the Craters of the Moon area and the extreme northeast end of the plain, receive as much as 20 inches annually. Throughout much of the plain the soil absorbs all the precipitation during the growing season and most of the precipitation during the non growing season. In 10 to 20 percent of the plain, the soil cover is thin or absent where a larger part of the precipitation may reach the water table.

Many estimates have been made of the quantity of precipitation that becomes groundwater recharge on the Snake River plain. Mundorff and others (1964) divided the Snake Plain into four areas and determined the percentage of effective precipitation, given in Table 6.

Table 6. Percentage of Precipitation That is Effective on Snake Plain

		Percentage
Area 1	Central part of Snake Plain	3
Area 2	Craters of the Moon	26
Area 3	Big Bend Ridge*	33
Area 4	South Side of Snake Plain (Lake Walcott - American Falls area and northeast along southeast side of Snake River)	16

*The Big Bend Ridge area is not located within the boundaries of the study area.

The study area was divided into these areas of equal percentage of effective precipitation. In the nongrowing season the precipitation is multiplied by the percentage figures of Table 6. For agricultural lands the precipitation was considered to be 100% effective during the growing season. Combined with the consumptive use the resulting output equals the crop irrigation requirements. Figure 51 denotes the location of the areas with different percentages of effective precipitation. The number corresponds with the area numbers in Table 6.

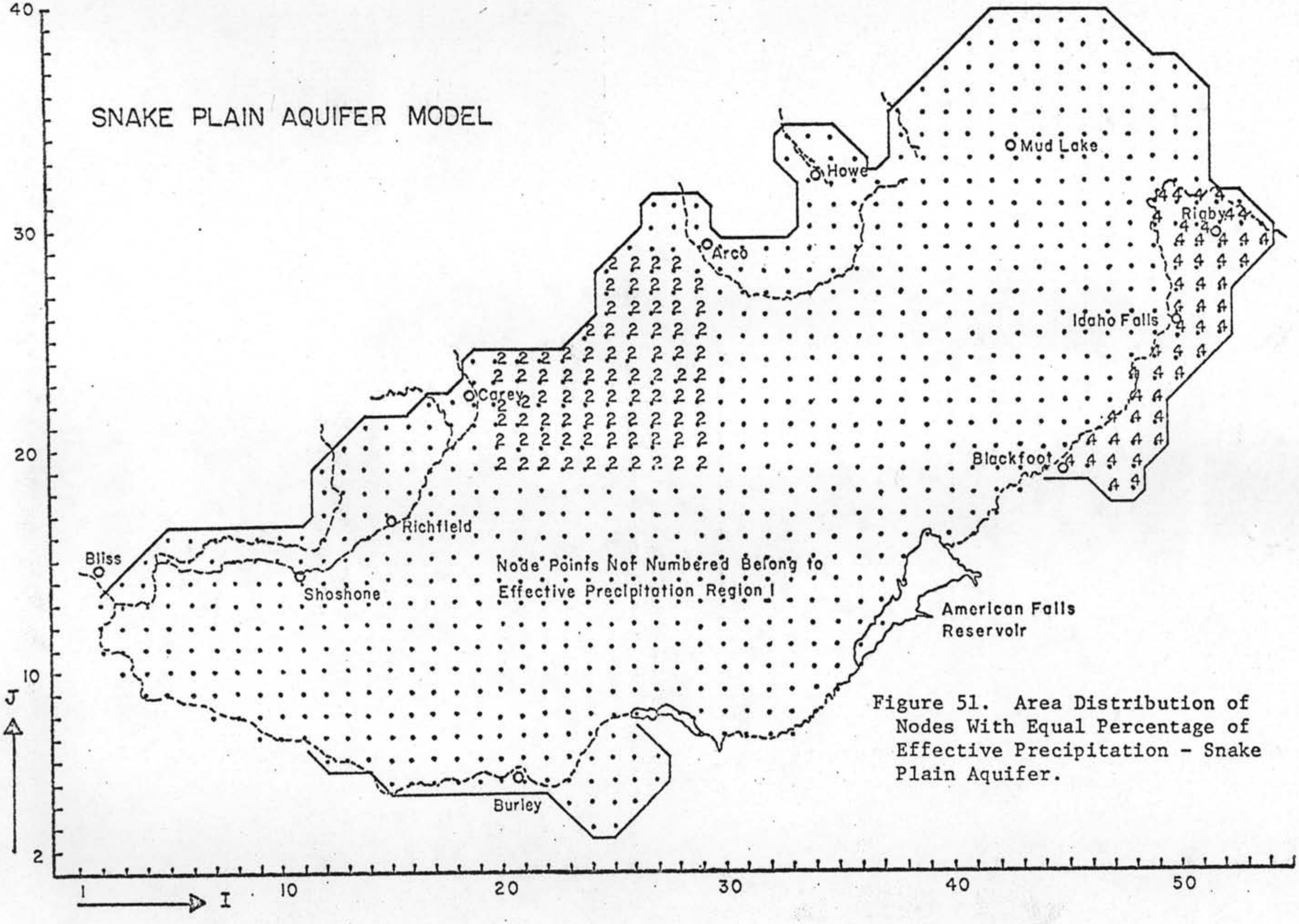

Figure 51. Area Distribution of Nodes With Equal Percentage of Effective Precipitation - Snake Plain Aquifer.

Tributary Valley Underflow

Figure 52 shows the study area with the aquifer boundaries and the tributary basins denoted by Roman numerals. For each of the valleys the nodes over which the valley underflow is distributed are denoted by corresponding Arabic numerals. Table 7 shows the tributary valley and the groundwater underflow estimates for 1966, and the number of nodes over which the flow is distributed. For lack of more accurate data these total flows were assumed to be the same for every year.

Table 7. 1966 Tributary Valley Underflow for Snake Plain Aquifer Used in the Steady State Calibration

Tributary Valley Number	Name	Underflow	No. of Nodes Over Which Flow Is Distributed
I	Big Wood	0	1
II	Silver Creek	38,000	2
III	Little Wood	53,000	2
IV	Big Lost	91,000	4
V	Little Lost	100,000	4
VI	Birch Creek	70,000	2
VII	Mud Lake*	420,000	13
VIII	Henrys Fork*	725,000	10
IX	[Rigby Area]	1,203,300	23
X	Blackfoot	25,000	2
XI	[Raft River]	130,000	7

The area denoted by [–] is not really a tributary valley. It can be considered as an area for which the lump sum of upgradient irrigation diversions, precipitation, and groundwater flow is denoted as groundwater underflow, this to confine the study area to the model boundaries.

The underflows for tributary valleys I, II, and III were taken from Mundorff and Crosthwaite (1964), and Castelin and Chapman (1972). For the Big Lost Basin the total net water yield from above Mackay Reservoir

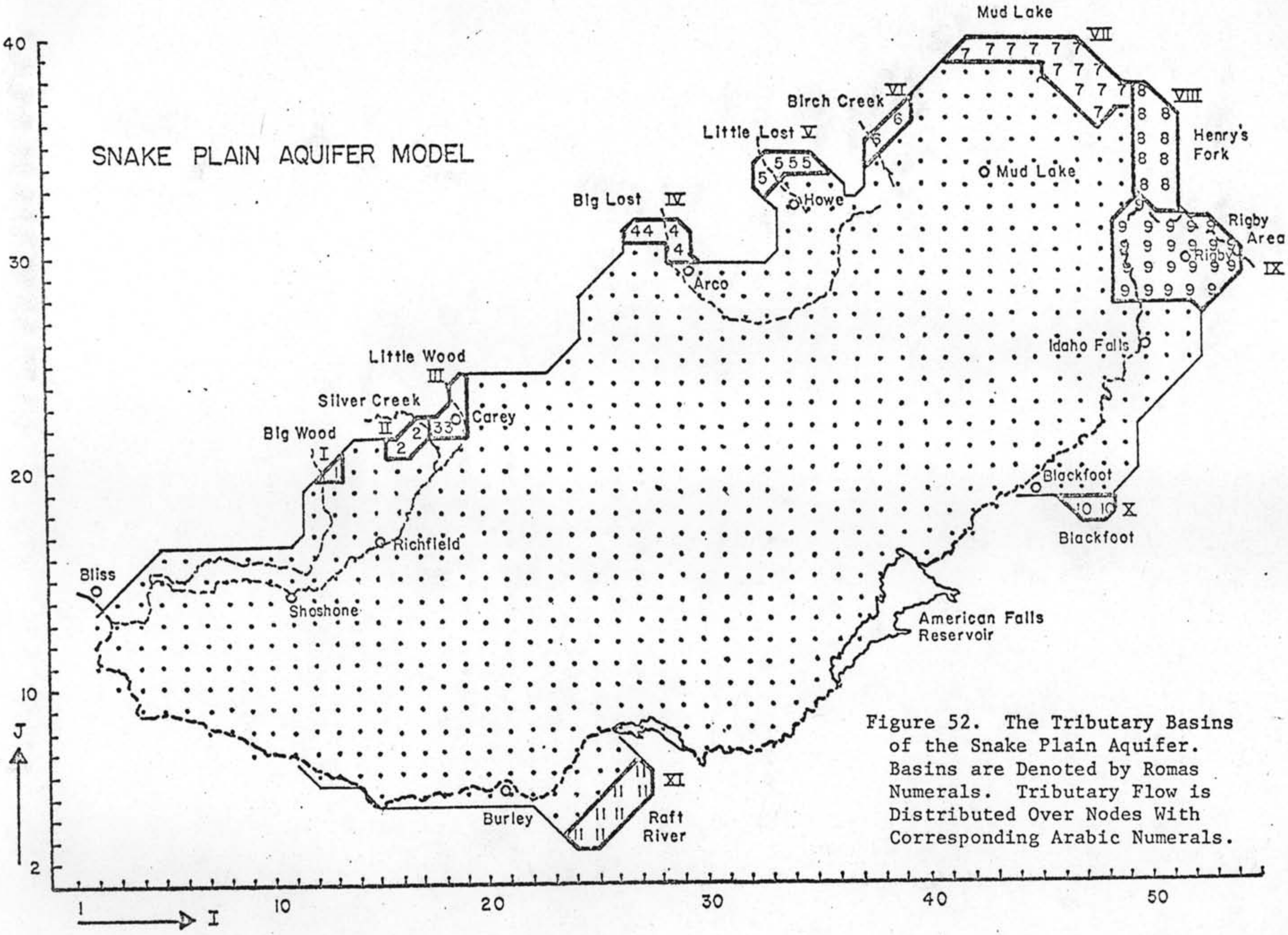

Figure 52. The Tributary Basins of the Snake Plain Aquifer. Basins are Denoted by Romas Numerals. Tributary Flow is Distributed Over Nodes With Corresponding Arabic Numerals.

is 280,000 ac ft. Figure 53 is a schematic representation of Mackay Reservoir located outside the model boundaries.

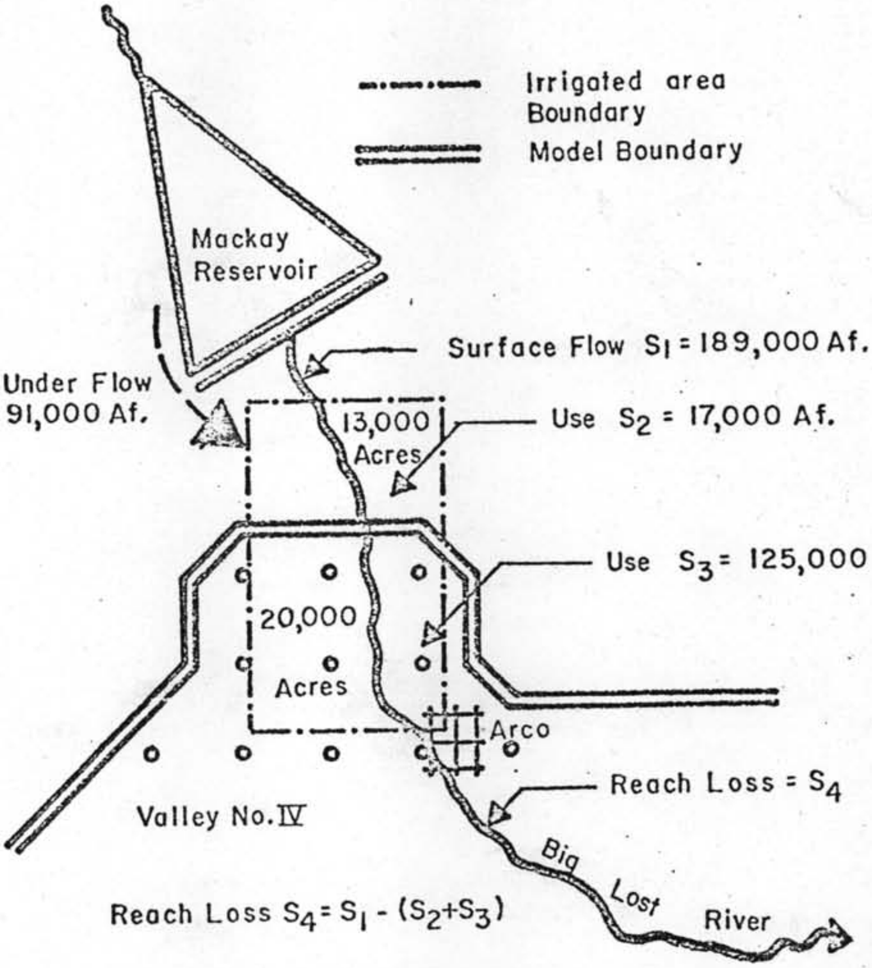

Figure 53. Schematic Representation of Mackay Reservoir, Located OUtside the Model Boundaries.

At Mackay Reservoir the net water yield composed of groundwater underflow of 91,000 ac ft and a surface water flow of 189,000 ac ft (S1).

Below Mackay Reservoir irrigation is practiced on approximately 33,000 acres of which 20,000 acres are located within the model boundaries. If the irrigation requirement can be estimated as 1.3 ac ft/ac the total water consumed from surface flow S1 is 17,000 ac ft (S2). For the 20,000 acres inside the model boundaries an average irrigation application of 6.25 ac ft/ac was assumed which results in a total input of 125,000 ac ft (S3). The total flow from the Big Lost Basin is :

Underflow : 91,000 ac ft
Irrigation diversion : 125,000 ac ft
Surface water flow = S1-(S2+S3) : 47,000 ac ft

The surface water flow in the Big Lost River is lost via seepage to the Snake Plain aquifer.

The underflow for the Little Lost basin was estimated at 100,000 ac ft and for the Birch Creek basin at 70,000 ac ft. Basins VII and VIII denoted by an asterisk in Table 7 are not really tributary valleys. They are denoted as such in order to distribute the groundwater flow from the northeast part of the aquifer towards the Mud Lake sub-area and the total contribution from the Henrys Fork and Egin Bench areas.

The total groundwater flow to the Mud Lake sub-area was estimated at 500,000 ac ft. The evaporation loss from lake surfaces was estimated at 80,000 acre feet (Stearns, Bryan, and Crandall, 1939) which is subtracted from this amount, leaving 420,000 ac ft as groundwater underflow. The total underflow from the Henrys Fork and Egin Bench areas was obtained from Crosthwaite and others (1970) and comprises 725,000 ac ft. The underflow for the Blackfoot River basin was estimated at 25,000 ac ft.

Tributary valleys IX and XI represent a special case. Valley IX, the Snake River Fan was the subject of a comprehensive study, described in Chapter IV. From this study a fairly accurate figure for total groundwater recharge was obtained. Rather than calculating the input again from the different components the total input of 1,113,000 ac ft was distributed equally over all nodes in the Rigby area. For these nodes the value of INPUT(I,J) is 1 as explained on page 20 of this report. The same can be said for the Raft River basin. The hydrology and irrigation management of this area is very complex and it would be difficult to calculate the inputs from groundwater underflow and irrigation diversions separately. Nace and others (1961) estimated the total flow from this area, reaching the Snake Plain aquifer at 130,000 ac ft. This flow is distributed over the nodes in the Raft River basin. For these nodes INPUT (i,j) = 1.

River Reach Gains and Losses

For the purpose of this study only that part of the river reach gain or loss attributable to the ground water system is defined as reach gain or loss. Figure 54 shows the location of 17 separate reaches. Reaches 1 - 6 are located along the Snake River; reaches 7 - 14 are located in the general area of the Wood River.

The gains and losses were calculated from the historic diversion records in the River Operation Study for Idaho (Unpublished data, Idaho Water Resource Board) for reaches 1 - 6. The reach gains or losses for reaches 10 - 14 were obtained from unpublished study conducted by the Idaho Water Resource Board. Table 8 shows a list of the separate reaches, the reach name, the gain or loss, and the number of nodes over which the gains and losses are distributed.

For the reaches in the Wood River Basin the table shows also the relation between the number given to the reaches in the aquifer model and the reach numbers used in the Wood River basin study. All reaches except for reach 3 and 4 (denoted by an asterisk) have an unsaturated connection with the aquifer. The reach gains and losses for reach 3 and 4 are calculated in the main program via the INOUT subroutine.

The reach gain for reach 4, from Blackfoot to Neeley, represents the gain from the north side of this reach. The model boundary is drawn along this reach and for mass balance computations in the aquifer only groundwater flow from the north is of interest.

Reach 15 represents the Milner Gooding Canal from its diversion point at the Snake River to a gaging station above Little Wood River. Along this canal lands are irrigated from diversions out of the canal. These lands are lumped together as the Milner Gooding Irrigation District (District 57). In 1966 the total flow at Little Wood gaging station was 190,700 ac ft less the total flow at the diversion point at the Snake River, of which 30% is believed to be lost as canal seepage denoted as reach loss for reach 15. The rest of the loss 133,500 ac ft is denoted as the total irrigation diversion of the Milner Gooding Irrigation District.

The reach loss of the Big Lost River is 47,000 ac ft (see page 120). The reach loss of the Little Lost River is totally appropriated in the form of irrigation diversion for the Little Lost Irrigation District,

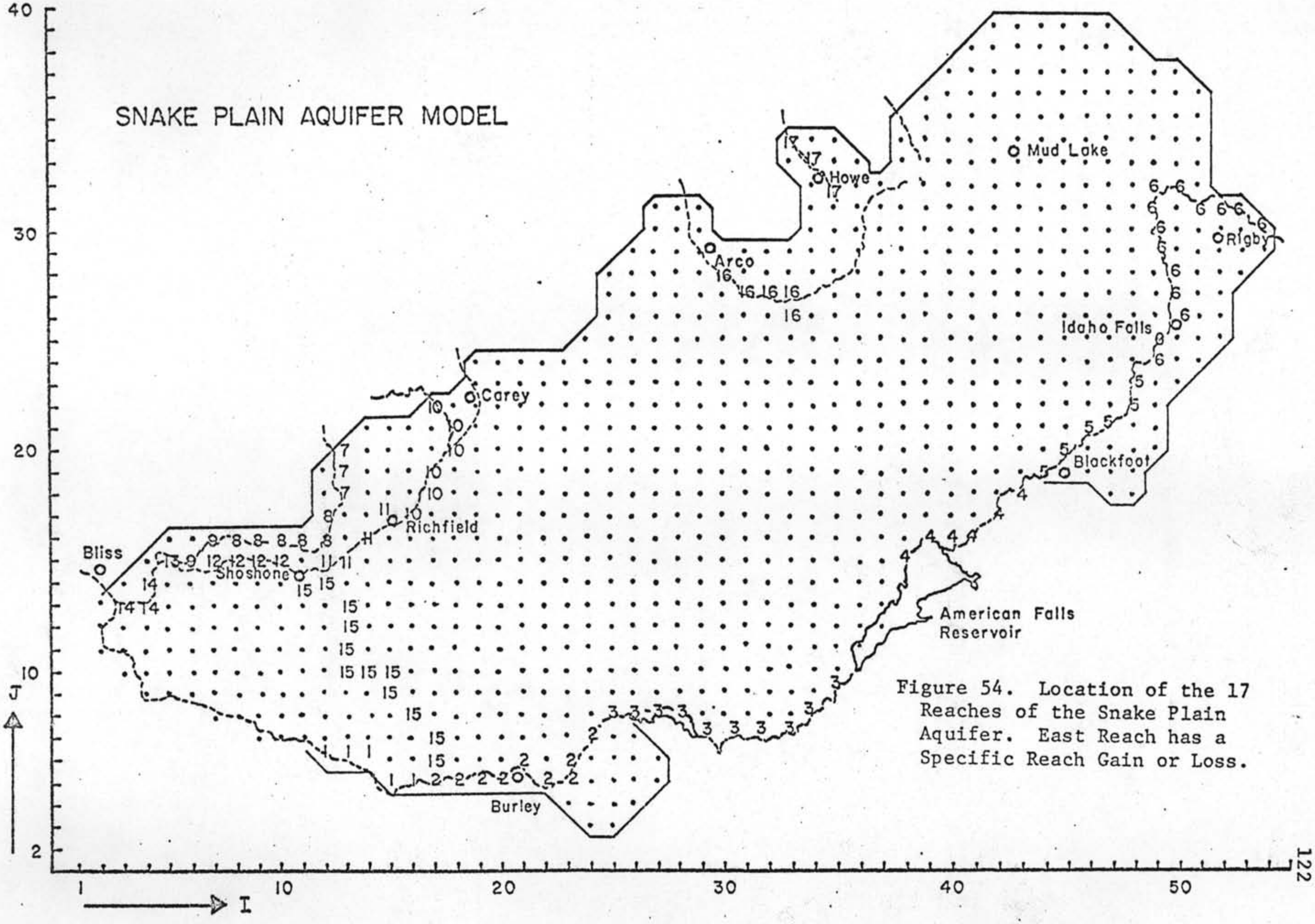

Figure 54. Location of the 17 Reaches of the Snake Plain Aquifer. East Reach has a Specific Reach Gain or Loss.

Table 8. 1966 Reach Gains and Losses of the Snake Plain Aquifer Used in the Steady State Calibration.

Reach No.	Reach Name	Gain/loss af	No. of nodes over which gains and losses are distributed
1	Milner-Kimberly	270,500	5
2	Minidoka - Milner	0	10
3	Neeley - Minidoka*	72,200	10
4	Blackfoot - Neeley*	924,800	6
5	Shelley - Blackfoot	-274,500	6
6	Heise - Shelley	-278,000	15
7	Magic Res. - Shoshone Canal	-30,200	4
8	Above Thorn Cr. - Below Shosh.C.	0	6
9	Big Wood above Thorn Cr. - Gooding	4,900	2
10	Little Wood above Picabo - Richfield	-11,000	6
11	Little Wood nr.Richfield - above M.G. Canal	-14,100	4
12	Little Wood nr. Gooding - L.W. above 1. Milner C.	-13,600	4
13	Reach 14	-30,000	1
14	Little Wood Gooding - Big Wood West of Gooding	0	3
15	Milner Gooding at Snake - above L.W.	-57,210	12
16	Big Lost River	-47,000	5
17	Little Lost River	0	3

*Reaches denoted by an asteric are hydraulically connected streams. The gains of these reaches are calculated in the main program.

Gains to river are positive, losses are negative.

Comparison of model reach numbers and IWRB reach numbers for the Big Wood Basin.

Model		IWRB	Model		IWRB
7	-	28	11	-	22
8	-	8	12	-	24 & 25
9	-	12	13	-	14
10	-	19	14	-	26 & 27

denoted as District 59.

Summary of Steady State Source Term Q(I,J)

The data described above was utilized in the separate input program to calculate the total net input for every node point within the aquifer boundaries of the 1966 water year. The calculated sum totals over all the nodes within the model boundaries are as follows.

For 1966 a total of 1,508,800 acres was irrigated of which 944,100 acres are irrigated by surface water. 93% or 874,700 acres are lands organized in districts, 7% or 69,400 acres are non-organized lands. A total of 564,700 acres are irrigated from groundwater. The total net diversion (gross diversion-return flow) for all surface water irrigated lands amounts to 5,452,000 ac ft which result in an average net irrigation application rate of 5.8 ac ft/ac. The total consumptive use over the surface water and groundwater irrigated lands was calculated as 2,607,100 ac ft which amounts to an average consumptive use of 1.72 ac ft/ac or 20.7 in/ac. A total of 644,400 ac ft of precipitation was calculated as effective recharge to the aquifer of which 404,300 ac ft fell during the growing season on the cropped lands. The remaining 240,100 ac ft fell as effective precipitation on the rest of the plain.

The sum total of all inputs, including the tributary groundwater underflow and the input from reach gains and losses, excluding the contribution from the hydraulically connected reaches is 5,831,000 ac ft. In the steady state calibration every year has the same input. The head values with the aquifer in an equilibrium state do not change in time, indicating no change of storage in the aquifer. This means that the total input of 5.83 million ac ft must also leave the area. Groundwater flow leaves the area in two places, the Thousand Springs area (King Hill to Kimberly) and the Minidoka to Blackfoot reach. If the historical gain for the Minidoka - Blackfoot reach is correct (924,800 + 72,200 = 997,000 ac ft) the total flow leaving the springs in the Thousand Springs area may be calculated from the input program as

Flow Thousand Springs = 5,831,500 - 997,000 = 4,834,500 ac ft

or an equivalent discharge rate of 6,622 cfs

Thomas (1969) calculated the average flow from the springs area as 4,700,000 ac ft. Both figures are subject to error in measurement but satisfactorily agree. The total input calculated may then be regarded as a realistic figure with which the calibration of the transmissibility values can be undertaken.

Adjustment of the Transmissibility Values

The model was run using the initial T-values and the 1966 yearly input for every half timestep until an equilibrium condition was obtained. Then, T-values were adjusted based on the hydraulic gradients of the historic and calculated equilibrium water tables. The new T-values were used in the next run to obtain a new equilibrium water table which matches the historic water table more closely. Again T-values were adjusted and convergence to the historical water table configuration was tested on the sum of squares of deviations, SS, between calculated and historical head values over all nodes in the aquifer. The calibration was terminated after the SS starts to increase.

Until now, the new routine to adjust the T-values was tested on a hypothetical aquifer with simple boundaries and one dimensional flow. The Snake Plain aquifer is considered a two-dimensional flow system with complex boundaries. Several tests were necessary to determine the most efficient way to calibrate the model on a complex system and to finalize the T-adjustment routine.

Figure 55 is a schematic representation of the aquifer with its boundaries. Most of the boundaries, as denoted by the double line are impermeable boundaries where the hydraulic head is not prescribed. Boundaries which in a physical sense constitute a constant head are the Thousand Spring area, Boundary 1, and the Snake River between Minidoka Dam and Blackfoot, Boundary 2. These are the only places where the aquifer is defined with hydraulic head values. In the first calibration trials it was feared that these two places did not sufficiently define the aquifer. Therefore a third constant head boundary was created, Boundary 3, in the northeast part of the aquifer. It was argued that, since this is an equilibrium state calibration in which the hydraulic

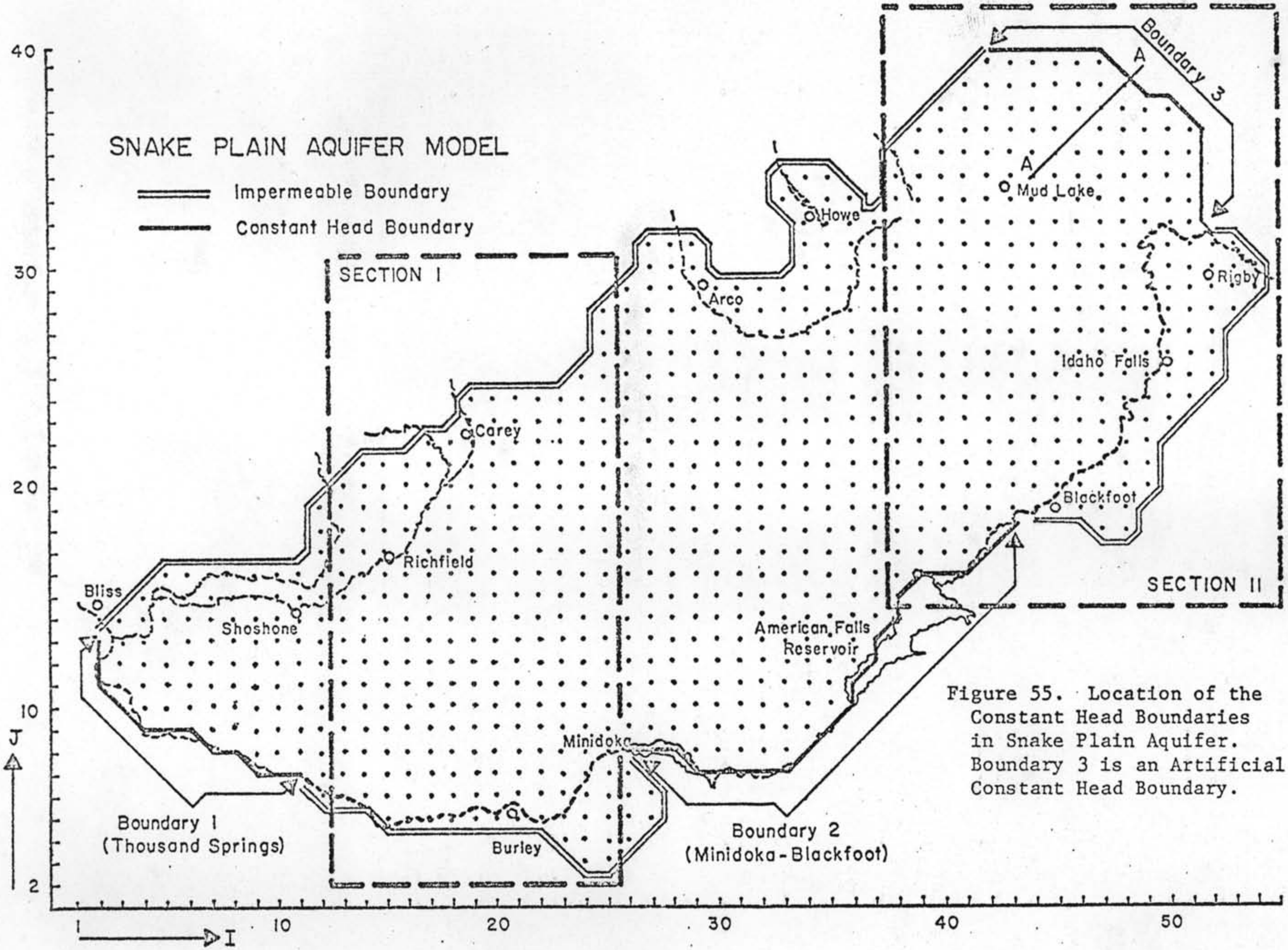

Figure 55. Location of the Constant Head Boundaries in Snake Plain Aquifer. Boundary 3 is an Artificial Constant Head Boundary.

head values do not change in time, the impermeable boundary may be replaced by an artificial constant head boundary.

In test 1, instead of T-values the hydraulic conductivity (K) values were adjusted based on the gradient ratios of the individual node points, and readjusted in a second step to keep the calculated optimum weighted average K-values the same. The adjustment of K is based on the gradient ratio as is the case when T is adjusted. However,if for two node points with a different aquifer depth the K-values are adjusted by the same percentage, the resultant T-values will have a disproportional change. Since the T-values are used in the calculation of the head values rather than the K-values, the adjustment of K-values may lead to a slower convergence. As is shown in the initial testing (see page 77) the second step in the adjustment, using weighted average K-values, tends to increase the difference in K-values between nodes. Test 1 showed that the application of the second adjustment step right from the start of the calibration increased the sum of squares of deviations in an early stage of the calibration. Therefore the calibration routine was divided into two parts. In the first part only the nodal K-values are adjusted with gradient ratios for every iteration. This is done until the sum of squares of deviations starts to increase. In the second part of the calibration nodal K-values are adjusted with the gradient ratio and readjusted to keep the calculated optimum weighted average K-values the same. This readjustment takes place in a stage when the new K-values are close to the optimum values (no decrease in sum of squares). After the sum of squares increases for the second time the calibration is terminated. Figure 56 represents the results of three tests. Test 2 shows the decrease in standard deviation (S) over the aquifer against the number of iterations by changing the K-values. The arrow denoted by W indicates the beginning of the second part of the calibration in which the weighted average K-values are utilized. For Test 2 the second step did not result in a decrease of S. The simulation with the original K-values, determined by a node point interpolation of an existing transmissibility contour map (Mundorff and others, 1964) resulted in a standard deviation of 53.7 feet. Final K-values resulted in a minimum standard deviation of 4.4 feet.

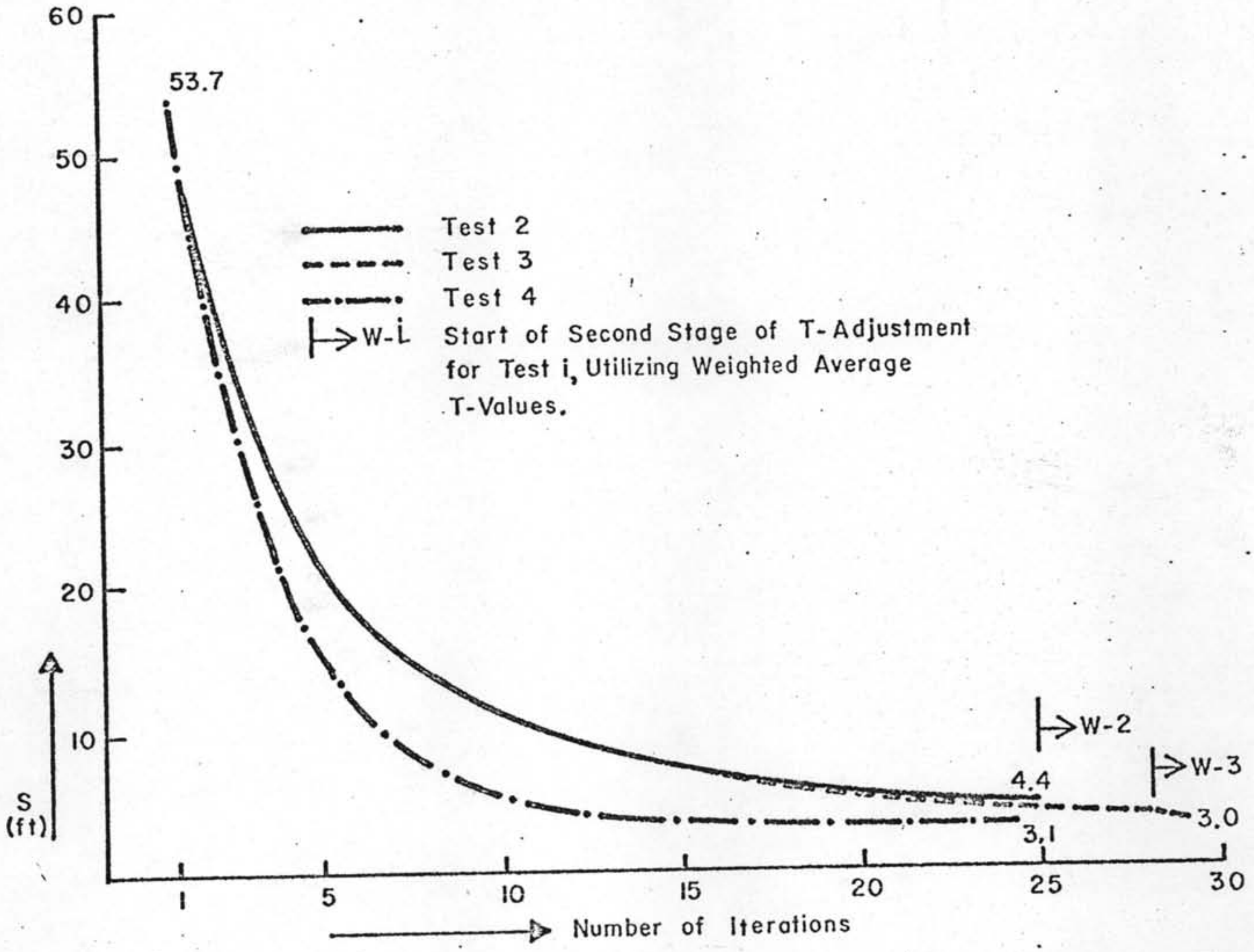

Figure 56. Decrease in Standard Deviation as a Result of the Adjustment of the T-Values in the Steady State Calibration for Test 2, 3 and 4.

In Test 3, instead of changing the K-values the T-values were adjusted. In this way the change in T-values automatically incorporates the change in aquifer depth that occurs as the calculated head approaches the historical head. In Test 3 the standard deviation decreased to a minimum of 3.17 feet at iteration 28. The weighted average adjustment decreased it further to 3.00 feet.

Although the resultant S was sufficiently small the final T-array resulting from Test 3 showed that for some nodes the resulting T-values were unreasonably low as indicated by the circled numbers in Table 9. Table 9 represents the T-values resulting from Test 3, Section I, as indicated in Figure 55. For the second and third test the magnitude of change of the K-values and T-values was not confined within prescribed limits. For node points in the aquifer which show the greatest deviations from historical water table values the change for every iteration is large. It is possible that for these node points the T-values are over-adjusted. This over-adjustment leads to very low or very high T-values in the early stage of the calibration.

In order to let the adjustments occur with equal emphasis so that all node points obtain their optimum T-values at the same stage of the calibration an upper and lower limit is imposed on the new nodal values in three ways. The first limit can be described by

$$1 - A < \frac{T^{n}_{i,j}}{T^{n-1}_{i,j}} < 1 + A$$

Where

T^{n} = the new transmissibility value of node point (i,j)

$T^{n-1}_{i,j}$ = the transmissibility value of the previous iteration

A = maximum allowable change as a decimal fraction

The second limit imposed upon the new T-value originates from the consideration that the T-values of surrounding nodes should be in the same order of magnitude as the nodal value. For a node surrounded by four nodes the average weighted transmissibility is

$$\overline{T}_{i,j} = (4T_{i,j} + T_{i-1,j} + T_{i+1,j}\ T_{i,j-1} + T_{i,j+1})/8$$

$$0.5 < \frac{T_{i,j}}{\overline{T}_{i,j}} < 2.0$$

Table 9. Final Transmissibility Values in ft.^{2}/day for Section I, Resulting from Test 3, with No Limits Imposed

30	0	0	0	0	0	0	0	0	0	0	0	0	0
29	0	0	0	0	0	0	0	0	0	0	0	0	0
28	0	0	0	0	0	0	0	0	0	0	0	0	16
27	0	0	0	0	0	0	0	0	0	0	0	0	53
26	0	0	0	0	0	0	0	0	0	0	0	0	36
25	0	0	0	0	0	0	0	0	0	0	0	2	17
24	0	0	0	0	0	0	213	206	770	155	37	13	52
23	0	0	0	0	0	0	740	1619	④	646	14	52	67
22	0	0	0	0	15433	1777	7359	318	6265	2124	356	206	300
21	0	3679	38810	④	15393	16668	4296	9804	4415	9281	11008	1069	1550
20	4338	2281	①	5379	1995	2077	④	6176	5273	(73)	12734	5793	5732
19	186	193	32	374	466	1119	8983	2066	16742	11921	8532	4381	2936
18	762	652	263	103	625	461	1436	12495	4004	10039	9314	4466	3547
17	325	520	487	1522	1034	934	628	4086	12296	4795	5422	1676	1407
16	165	491	2546	1601	1923	2591	777	1736	4318	④	691	3048	552
15	975	4220	1347	5507	2406	2865	1013	1828	343	4675	47693	④	1747
14	1546	1172	4617	㉓	3291	1013	2507	1512	4577	2163	1663	187	1089
13	974	④	445	2310	1229	1416	862	1667	2195	2694	3270	1880	1500
12	610	783	352	991	615	878	688	710	1002	1108	1781	1214	1983
11	551	1549	910	1472	894	702	500	249	675	2032	294	3502	3975
10	528	1053	1038	1516	620	492	333	290	514	1401	442	2189	3462
9	777	101	901	1136	527	215	375	83	290	499	170	1488	2830
8	976	369	2050	1306	368	322	206	134	222	321	143	433	74
7	575	714	1574	362	563	291	311	211	199	225	84	168	190
J ↑ 6	5	329	1258	613	881	308	358	221	196	187	148	66	49
5	0	0	192	413	820	370	243	192	44	88	71	38	78
4	0	0	0	0	0	0	0	0	0	0	74	154	66
3	0	0	0	0	0	0	0	0	0	0	0	90	18
2	0	0	0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0	0	0	0
	13	14	15	16	17	18	19	20	21	22	23	24	25
	→ I												

Table 10. Final Transmissibility Values in ft^2/day for Section I, Resulting from Test 4, with Limits Imposed

J ↑	13	14	15	16	17	18	19	20	21	22	23	24	25
30	0	0	0	0	0	0	0	0	0	0	0	0	0
29	0	0	0	0	0	0	0	0	0	0	0	0	0
28	0	0	0	0	0	0	0	0	0	0	0	0	21
27	0	0	0	0	0	0	0	0	0	0	0	0	50
26	0	0	0	0	0	0	0	0	0	0	0	0	39
25	0	0	0	0	0	0	0	0	0	0	0	1	12
24	0	0	0	0	0	0	179	206	751	162	38	10	59
23	0	0	0	0	0	0	655	1350	92	419	11	56	64
22	0	0	0	0	4993	2929	5777	640	4071	1783	184	190	349
21	0	6549	23392	1230	20308	6806	3187	5150	4217	5981	8581	1016	1306
20	2471	1612	330	339	1395	1563	790	5227	4662	1229	10816	4664	4852
19	316	165	67	225	359	756	7270	2298	14905	10683	7627	3805	2539
18	541	566	179	121	634	353	1396	10927	3933	8534	8806	3868	3107
17	246	442	432	1254	916	843	547	3726	11087	4841	4932	1648	1518
16	118	434	2136	1443	1825	2251	763	1414	3695	1795	4352	2302	1459
15	855	3821	1300	5031	2280	2537	1134	1349	2505	4751	2517	3308	2576
14	1388	985	3966	366	2872	909	2081	1479	2945	2372	2332	1838	3053
13	814	57	383	2059	1090	1255	805	1516	2180	2377	3199	2137	1382
12	634	654	319	869	559	830	674	714	1050	1144	1922	1348	1784
11	457	1329	812	1362	875	706	522	282	759	2324	441	4131	4528
10	490	1020	982	1502	687	536	393	365	637	1886	582	3089	4473
9	748	139	850	1238	683	257	499	130	391	698	263	1895	5729
8	918	292	2021	1705	410	402	249	143	251	321	128	384	54
7	705	1078	2160	438	654	344	325	235	214	254	84	156	163
6	40	353	1538	702	1079	365	415	259	225	216	188	66	47
5	0	0	155	408	839	365	248	197	48	103	78	46	67
4	0	0	0	0	0	0	0	0	0	0	89	168	64
3	0	0	0	0	0	0	0	0	0	0	0	101	14
2	0	0	0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0	0	0	0

→ I

The third limit only involves the second stage of the calibration where the individual T-value is readjusted to maintain the optimum weighted average transmissibility (T^W, see page 77) as determined with the gradient ratio. The readjusted T-value should not deviate more than 20% from the T-value obtained after the first two limits have been imposed. In all tests the second and third limits were kept constant, while the value A of the first limit, denoting the maximum allowable change per interation was investigated to determine the optimum value.

For Test 4, A was set to 0.3. Despite the limits the calibration resulted in a faster conversion in the initial stage of the calibration as shown in Figure 56. The imposed limits smoothed the adjustment of the T-values. The final standard deviation was 3.1 feet, slightly larger than that of Test 3, but was obtained with a more reasonable T-distribution as is presented in Table 10. Comparing the circled values of Table 9, Test 3, with Table 10, Test 4, shows clearly that the imposed limits reduced the over-adjustment of the T-values to a minimum.

Test 5 and Test 6 were run with an equal to 0.40 and 0.45 respectively. Table 11 gives the values for the constant A and the final standard deviation S.

Table 11. Maximum Allowable Adjustment A Versus Standard Deviations for Four Tests.

	A	S (feet)
Test 4	0.30	3.10
Test 5	0.35	3.00
Test 6	0.40	3.25
Test 7	0.35	3.40

Table 11 shows that Test 5 with A = 0.35 resulted in the minimum standard deviation of 3.0 feet. In Tests 4, 5 and 6 the T-values of the constant head nodes were adjusted in the same way as the interior node points. Since at the constant head boundary the simulated as well as the historical water tables are fixed at the same level the difference in gradient is greater for nodes closer to the boundary. This is indicated by the angle between the historical and calculated gradient represented by α, β and γ, Figure 57, for nodes (i,j), (i+1,j) and (i+2,j) respectively.

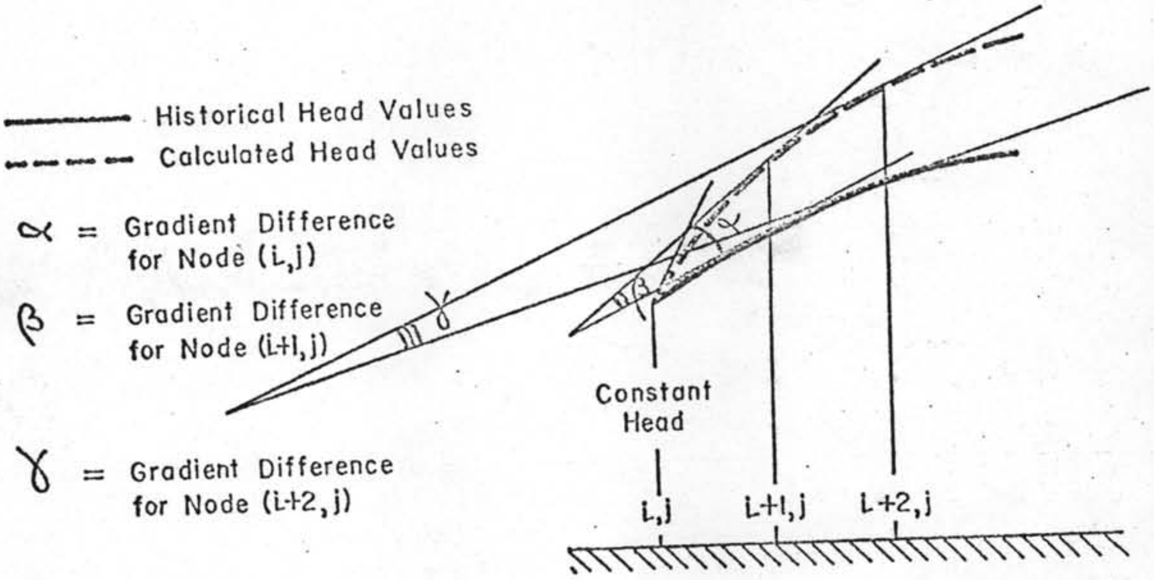

Figure 57. Constant Head Boundary and the Gradient Difference Between Historical and Calculated Water Table for Nodes Close to the Boundary.

Over-adjustment of the T-values is most likely to occur there so that the described limits especially apply to the constant head boundary nodes. To investigate whether this over-adjustment can be minimized another way, in Test 7 with A = 0.35, the constant head T-values were not adjusted with the gradient ratio but were calculated as an average of the interior nodal values close to the constant head boundary nodes. The result was similar to Tests 4, 5 and 6 but the final standard deviation was higher than these tests, 3.40 feet,as shown in Table 11.

In comparing the final transmissibility values resulting from Test 7 with the initial values it appeared that especially in the northeastern part of the aquifer the adjusted T-values were much higher. The higher T-values were apparently caused by the introduction of the artificial constant head along the northeastern boundary of the aquifer. In order to define the influence of this artificial constant head on the

T-values, in the next test, Test 8, Boundary 3 in Figure 55 was replaced by the geohydrologically correct impermeable boundary. A calibration run was made, using as starting T-values the final T-values from Test 7. For Test 8 the constant head T-values were also adjusted by calculating the average of the surrounding interior nodal values. Since the final T-values from Test 7 were used as initial values minor adjustments of the T-values were expected. However, the initial run showed large differences between calculated and historical head values as indicated by the initial standard deviation of 65.0 feet, shown in Figure 58. The adjustment of the T-values in this calibration resulted in a final standard deviation of 3.8 feet. Most of the change in T-values took place in the northeastern part of the aquifer.

The differences may be explained in the following way: In the northeastern part of the aquifer relatively little water recharges the groundwater body. In imposing a constant head, the water table values initially calculated in Test 7 were lower than the historical water table because of the relative small recharge. This caused the calculated gradient to be greater than the historical gradient as shown in Figure 59, which represents a northeast-southwest transect over the boundary (line A-A in Figure 55). The calibration routine in this case increased the T-values to decrease the calculated gradient. The calculated gradient approached the historical gradient but the final T-values were high. The high T-values of Test 7 induced a large groundwater flow into this area, which is historically incorrect because little water recharges the area. The constant head boundary is a hydraulic connection and theoretically supplies all the water needed to sustain the flow. The final transmissibility values resulting from Test 7 were also 10-20% higher in the western part of the aquifer than the final transmissibility values resulting from Test 8 because in Test 7 a larger amount of water was flowing through the system. The introduction of the constant head was an erroneous decision. The conclusion may be drawn that the correct hydrogeologic boundaries should always be imposed on the study area.

In Test 8 using the high final T-values from Test 7 and an impermeable boundary in the northeast, the small amount of water that recharges the northeastern part of the aquifer is transported out of the area very

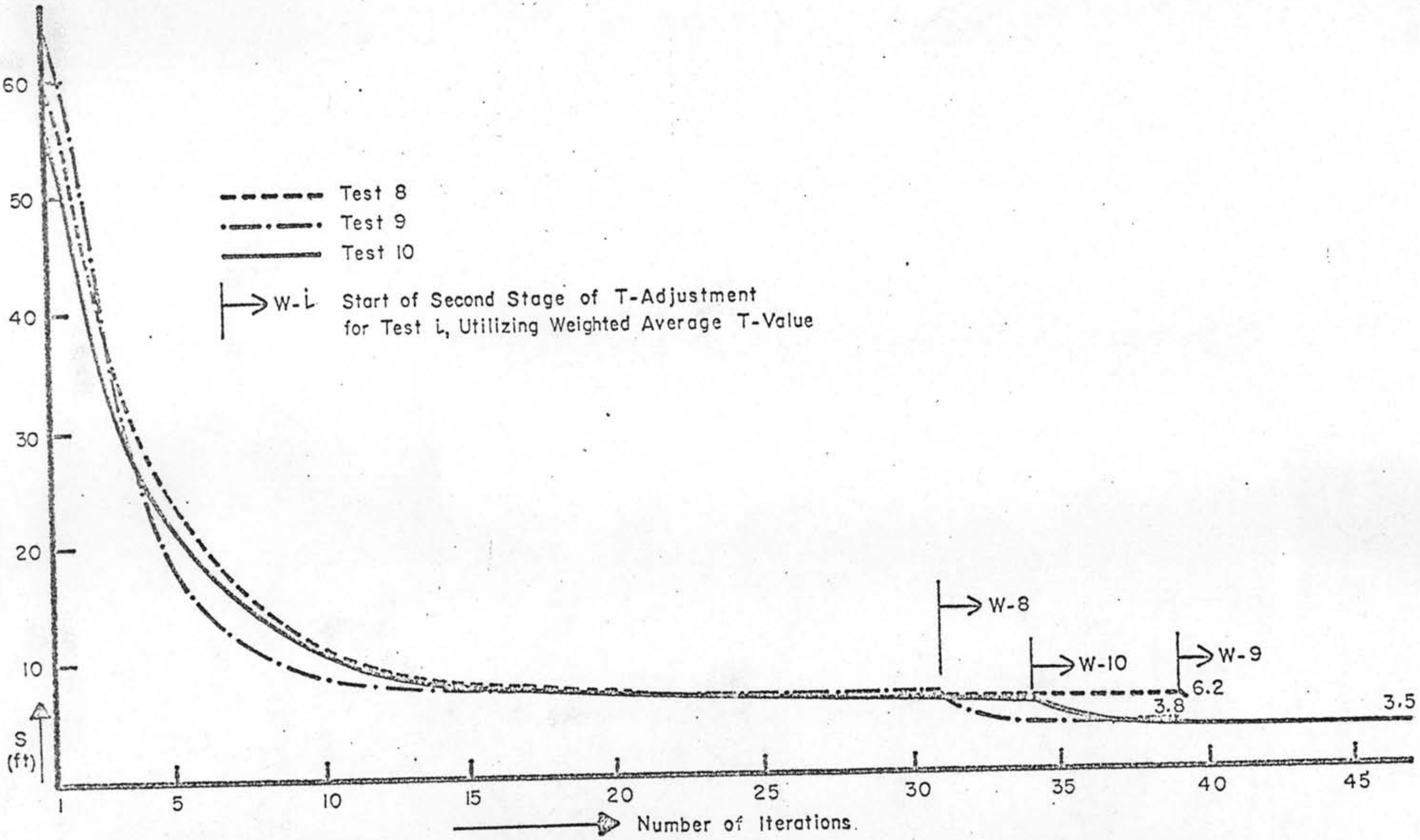

Figure 58. Decrease in Standard Deviation as a Result of the Adjustment of the T-Values in Steady State Calibration for Test 8, 9 and 10.

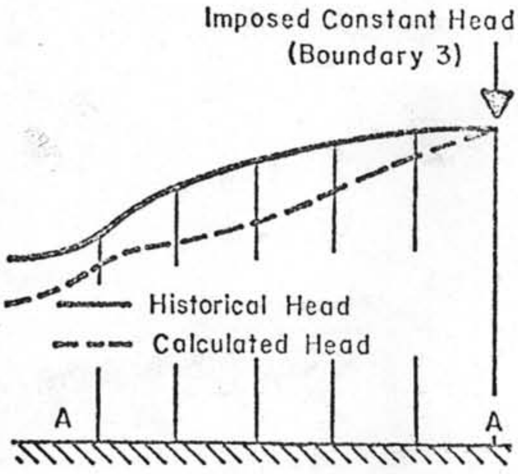

Figure 59. Representation of Historic and Calculated Head Values for Transect A-A Using Artificial Constant Head Boundary (Test 7).

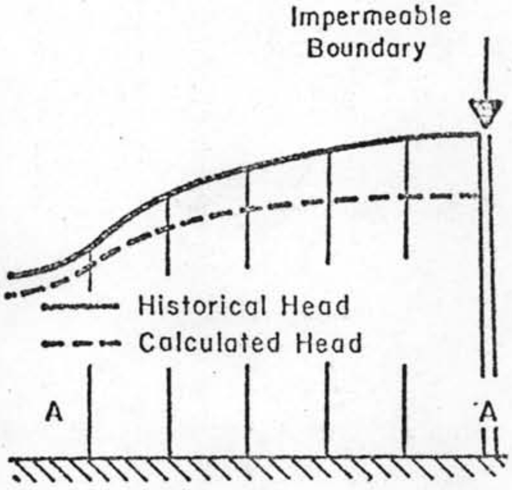

Figure 60. Representation of Historical and Calculated Head Values for Transect A-A Using Impermeable Boundary (Test 8).

rapidly.

This causes the water table values to drop dramatically as shown in Figure 60 and is indicated also by the 65.0 ft. initial standard deviation shown in Figure 58. In this calibration the routine decreased the T-values, retaining the water longer in the area, and subsequently increasing the water table elevations. The final transmissibility values of Test 8 are considered to be correct.

To investigate the uniqueness of the solution using different initial T-values the calibration was repeated in Test 9. Test 9 used the original starting T-values as compared to Test 8 in which high initial T-values were used. Figure 58 shows the decline in standard deviation for Test 8 and 9. The calibration for Test 9 was terminated at the start of the second stage of the calibration (iteration No. 40). The full calibration was not completed since the maximum number of iterations was reached. Final T-values were compared of Test 8 and 9 at the end of the first stage of the calibration before the start of the weighted average adjustment routine. For Test 8 this is iteration 31 and for Test 9 iteration 39 as indicated in Figure 58.

The standard deviation of iteration 31, Test 8, differs only slightly from iteration 39, Test 9. This indicates that water table

elevations for

corresponding node points in the two tests are sufficiently close so that a comparison of either T-values or K-values is immaterial. Because only K-values are printed out for every iteration these values are compared. Table 12 shows the initial K-values for representative nodes in Section II of the aquifer for Test 8.

Table 13 shows the initial K-values for Section II of Test 9. Table 14 shows the final K-values resulting from the first stage of the calibration for Section II of Test 8, while Table 15 shows the final T-values for Section II of Test 9.

The results show that despite highly different initial K-values, the final K-values for both tests are in very good agreement. For the majority of the node points the difference of the K-values is within 5%. For some node points the differences are 12%. The iterative technique developed here does not provide a mathematically unique solution but results in K-values that are generally repeatable within 5% limits. Test 8 shows also that with the introduction of the impermeable boundary in the northeastern part of the aquifer, the second stage of the calibration, has a more pronounced effect on decreasing the standard deviation as indicated in Figure 58.

For Test 8 and 9 the program adjusted the T-values at the constant head boundary as an average of the interior neighboring nodes. Test 5 showed that adjusting the constant head boundary T-values the same way as the interior nodes results in a smaller standard deviation. A final test, Test 10, was run utilizing this feature and the result is shown in Figure 58. The final standard deviation of 3.5 feet is 0.3 foot smaller than Test 8. The transmissibility distribution resulting from Test 10 is therefore accepted as the final result of the steady state calibration of the Snake Plain aquifer. Figure 61 is the computer drawn contour map of the equilibrium head values calculated in the model with the original T-distribution. Figure 62 is the historic water table contour map of the Snake Plain aquifer drawn from the available well data and shows also a computer drawn contour map of the calculated equilibrium head values resulting from the final T-distribution of Test 10. Agreement between the historical water table contours and calculated water table contours is very good. For the major part of the aquifer the differences between

Table 12. Initial Conductivity Values in ft/day for Section II, Test 8 - Steady State Calibration

40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
39	0	0	0	0	2468	1734	2154	2596	1638	740	0	0	0	0	0	0	0	0
38	0	0	0	818	1120	281	2599	4881	1331	166	75	0	0	0	0	0	0	0
37	0	0	1770	234	172	648	8333	2236	268	781	79	222	66	0	0	0	0	0
36	0	4657	8797	503	66	192	534	621	314	1785	251	355	324	165	0	0	0	0
35	1822	4907	1323	154	26	63	44	63	74	146	52	360	433	78	0	0	0	0
34	4082	758	1961	941	88	100	113	52	67	156	117	180	415	88	0	0	0	0
33	584	585	745	685	415	356	137	79	72	57	174	227	310	34	0	0	0	0
32	79	484	658	254	373	319	673	586	197	30	120	71	149	48	0	0	0	0
31	149	188	668	1063	1320	1905	2086	4423	1363	107	100	157	38	107	93	60	0	0
30	365	812	1450	2280	404	1384	3269	3744	1677	274	140	171	23	99	82	185	146	0
29	863	2000	1457	1725	1784	3956	3462	1966	1383	720	85	238	54	43	80	31	55	0
28	1818	925	1078	1577	2794	2165	1620	1345	336	334	380	73	138	24	121	30	0	0
27	816	726	2622	2804	1945	2915	2129	711	1175	228	216	130	133	76	73	0	0	0
26	1674	1718	884	1611	2652	1376	1080	1368	695	319	288	221	85	53	46	0	0	0
25	1438	1602	3673	2341	1286	1086	877	696	1867	1364	729	526	67	63	23	0	0	0
24	2386	2597	2176	2403	1205	865	795	1312	597	776	264	77	50	63	0	0	0	0
23	2029	4494	3499	1052	1238	941	331	289	335	207	29	8	7	0	0	0	0	0
22	2635	3741	2148	8716	7592	2734	286	55	123	173	22	14	0	0	0	0	0	0
21	2331	3715	4285	6562	1683	2170	218	39	71	23	11	11	0	0	0	0	0	0
20	3311	10201	3933	5838	5438	1362	224	48	16	10	1	3	0	0	0	0	0	0
J 19	19616	1767	755	2223	644	692	115	52	30	24	1	0	0	0	0	0	0	0
↑ 18	7886	1135	4413	3755	2456	1418	0	0	0	104	8	0	0	0	0	0	0	0
17	1760	390	609	574	1248	0	0	0	0	0	0	0	0	0	0	0	0	0
16	461	556	550	556	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	459	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
14	279	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55

→ I

Table 13. Initial Conductivity Values in ft/day for Section II, Test 9 - Steady State Calibration

J	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55
40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
39	0	0	0	0	53	187	481	401	428	428	0	0	0	0	0	0	0	0
38	0	0	0	53	53	428	962	775	535	535	401	0	0	0	0	0	0	0
37	0	0	53	53	267	1069	1069	1069	1069	1069	508	321	214	0	0	0	0	0
36	0	241	321	107	134	508	1069	1069	1069	1069	695	401	267	267	0	0	0	0
35	187	535	535	134	134	134	134	160	294	294	294	535	374	401	0	0	0	0
34	294	508	481	267	134	134	134	134	134	134	134	535	454	214	0	0	0	0
33	267	294	267	187	134	134	134	134	134	134	134	401	428	134	0	0	0	0
32	187	187	134	134	214	241	214	401	134	134	134	134	214	107	80	80	0	0
31	187	160	134	267	374	695	615	348	374	134	107	80	80	53	27	27	27	0
30	241	267	267	294	401	722	856	802	401	294	107	53	27	27	13	13	13	0
29	348	321	374	428	588	722	749	749	561	267	134	27	13	13	7	7	0	0
28	454	374	454	561	642	882	829	722	561	294	187	27	27	7	7	0	0	0
27	508	401	481	561	508	668	936	749	508	294	214	53	27	13	7	0	0	0
26	401	401	401	461	535	508	615	829	481	267	267	134	27	13	7	0	0	0
25	428	401	454	535	535	454	401	802	561	267	214	187	27	13	0	0	0	0
24	1069	695	508	401	401	401	374	535	668	267	187	80	13	13	0	0	0	0
23	1604	1577	1016	802	508	401	374	401	535	267	134	53	27	0	0	0	0	0
22	1203	1738	1604	1283	1016	829	454	321	214	107	53	27	0	0	0	0	0	0
21	1310	1497	1871	1631	1577	1470	535	160	107	53	27	27	0	0	0	0	0	0
20	2085	2139	2460	2406	2246	1791	428	107	53	27	27	27	0	0	0	0	0	0
19	2673	2673	2673	1845	1470	882	160	53	53	27	27	0	0	0	0	0	0	0
18	2673	2353	1069	909	642	267	0	0	0	53	27	0	0	0	0	0	0	0
17	1898	882	481	535	267	0	0	0	0	0	0	0	0	0	0	0	0	0
16	802	454	401	401	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	508	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
14	401	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

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Table 14. Final Conductivity Values in ft/day for Section II, Test 8 - Steady State Calibration

40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
39	0	0	0	0	109	27	38	41	41	27	0	0	0	0	0	0	0	0
38	0	0	0	175	179	90	283	267	303	74	58	0	0	0	0	0	0	0
37	0	0	328	43	22	89	969	929	155	596	76	275	118	0	0	0	0	0
36	0	714	1184	91	12	30	83	81	55	463	69	265	153	462	0	0	0	0
35	405	682	254	29	4	5	6	7	10	45	29	210	270	66	0	0	0	0
34	563	159	411	83	9	9	9	4	23	49	47	142	287	124	0	0	0	0
33	74	103	115	80	22	23	10	12	18	25	105	194	155	52	0	0	0	0
32	14	83	187	36	27	16	37	124	92	25	83	64	103	197	0	0	0	0
31	15	27	100	157	85	111	246	1476	969	105	63	140	30	90	52	68	0	0
30	44	139	388	287	70	256	1370	2413	1620	226	132	173	34	115	93	331	254	0
29	134	546	274	200	261	1638	2076	1453	1406	801	124	258	64	35	75	46	48	0
28	521	139	137	348	1226	1100	1071	794	308	296	356	68	118	30	125	31	0	0
27	150	152	698	943	959	1465	1327	569	1234	208	215	119	140	84	66	0	0	0
26	483	513	280	688	1325	776	649	1092	581	258	273	225	88	45	55	0	0	0
25	450	387	1238	988	586	593	766	816	2107	1395	812	535	64	75	21	0	0	0
24	558	1046	870	1075	668	667	974	1393	573	766	248	70	56	56	0	0	0	0
23	277	1372	1089	557	665	1022	338	233	264	208	38	11	11	0	0	0	0	0
22	570	1530	816	3932	5608	2866	285	56	134	169	19	18	0	0	0	0	0	0
21	412	1001	3180	4288	1781	1710	202	43	69	21	12	5	0	0	0	0	0	0
J 20	1293	6912	1631	2745	3786	1493	229	52	18	12	3	6	0	0	0	0	0	0
19	7828	1243	464	1357	647	775	116	52	29	34	2	0	0	0	0	0	0	0
18	2928	488	1494	1569	2002	1378	0	0	0	101	11	0	0	0	0	0	0	0
17	965	148	187	253	1116	0	0	0	0	0	0	0	0	0	0	0	0	0
16	207	174	195	221	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	148	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
14	95	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55

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Table 15. Final Conductivity Value in ft/day for Section II, Test 9 - Steady State Calibration

J	38	39	40	41	42	43	44	45	46	47	48	49	50	51	52	53	54	55
40	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
39	0	0	0	0	180	25	35	37	41	34	0	0	0	0	0	0	0	0
38	0	0	0	151	158	60	263	279	301	62	69	0	0	0	0	0	0	0
37	0	0	272	36	18	74	820	691	116	428	59	220	112	0	0	0	0	0
36	0	726	984	76	10	27	70	63	52	487	67	243	176	433	0	0	0	0
35	468	642	181	22	3	5	5	6	12	49	38	311	329	56	0	0	0	0
34	608	141	297	68	7	9	7	4	23	58	58	168	262	95	0	0	0	0
33	85	105	98	63	17	20	9	13	19	33	117	216	137	48	0	0	0	0
32	15	83	125	27	24	14	31	139	105	28	89	78	72	204	0	0	0	0
31	21	34	94	121	68	91	262	1658	1106	117	78	125	36	101	48	72	0	0
30	54	145	269	208	66	261	1441	2706	1578	227	116	172	33	107	85	361	221	0
29	161	421	192	193	299	1655	2197	1373	1367	760	120	254	61	38	80	48	49	0
28	414	116	129	355	1225	1153	1094	895	312	307	364	70	124	30	123	28	0	0
27	154	125	604	941	979	1515	1355	535	1214	210	209	123	135	81	70	0	0	0
26	368	423	334	731	1343	834	760	1162	636	275	285	222	87	52	50	0	0	0
25	395	411	1247	969	628	686	795	712	2079	1370	786	538	67	69	21	0	0	0
24	540	970	900	1135	760	682	897	1430	555	762	254	71	53	58	0	0	0	0
23	285	1477	1171	569	716	1014	342	251	278	212	39	11	10	0	0	0	0	0
22	632	1433	932	3883	6099	2699	272	57	126	168	21	16	0	0	0	0	0	0
21	318	1113	3361	4794	1691	1783	197	46	66	22	10	8	0	0	0	0	0	0
20	1374	7426	1801	3057	3389	1346	228	50	17	12	2	5	0	0	0	0	0	0
19	8337	1330	473	1267	601	815	120	53	29	34	2	0	0	0	0	0	0	0
18	4296	578	1427	1612	1879	1337	0	0	0	102	11	0	0	0	0	0	0	0
17	1124	166	172	251	1054	0	0	0	0	0	0	0	0	0	0	0	0	0
16	247	192	196	222	0	0	0	0	0	0	0	0	0	0	0	0	0	0
15	177	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
14	112	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
13	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0

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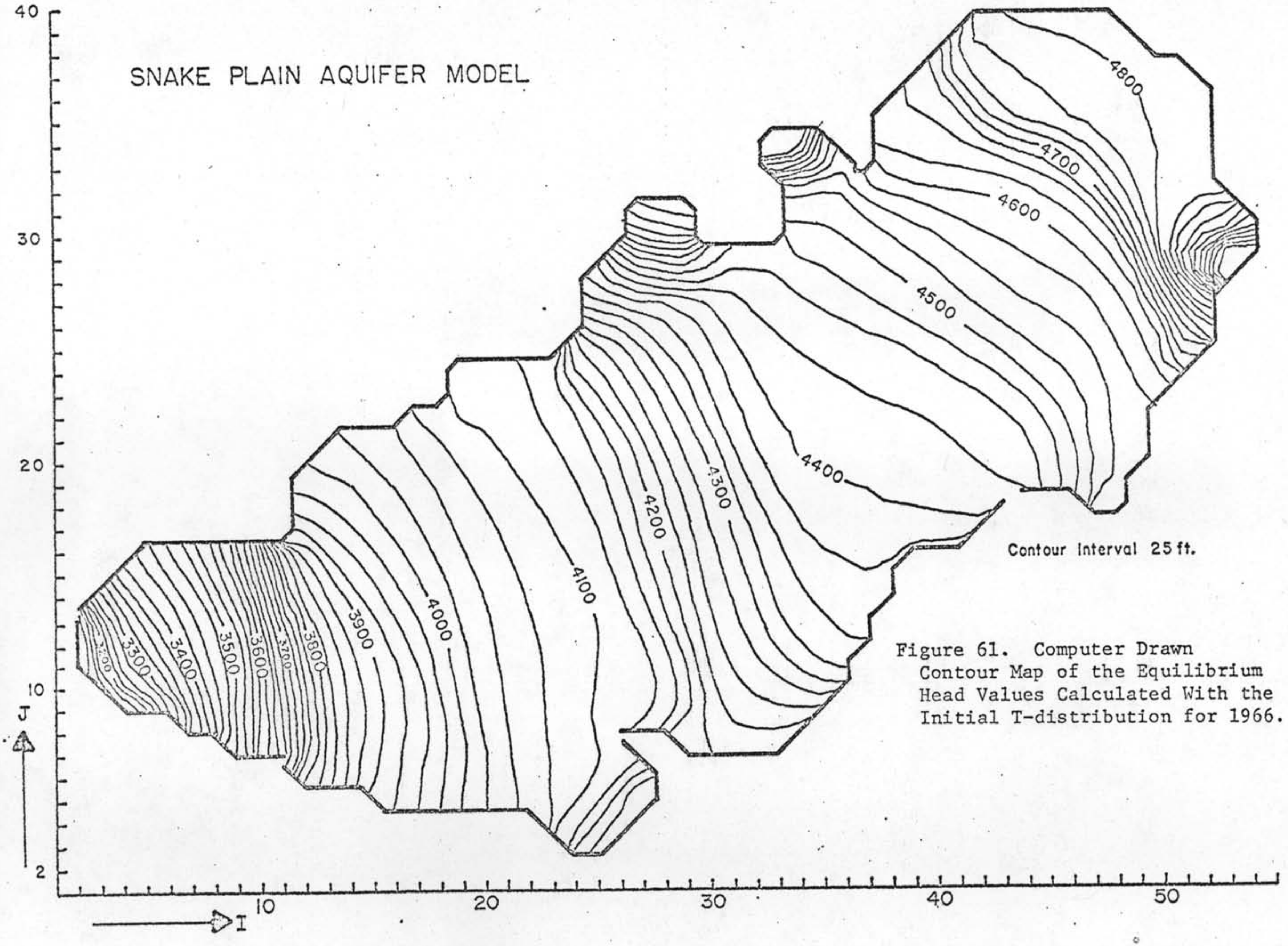

Figure 61. Computer Drawn Contour Map of the Equilibrium Head Values Calculated With the Initial T-distribution for 1966.

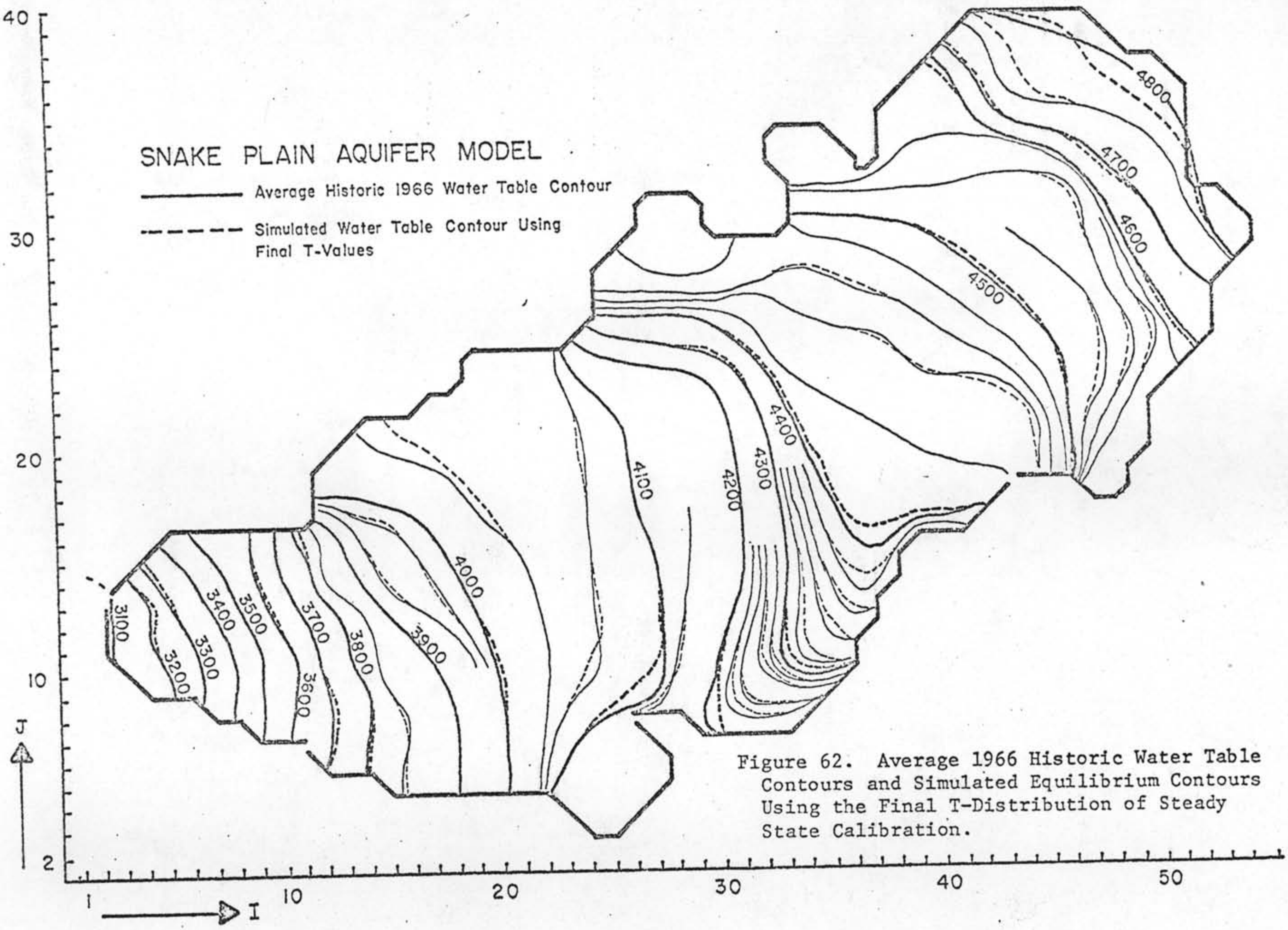

Figure 62. Average 1966 Historic Water Table Contours and Simulated Equilibrium Contours Using the Final T-Distribution of Steady State Calibration.

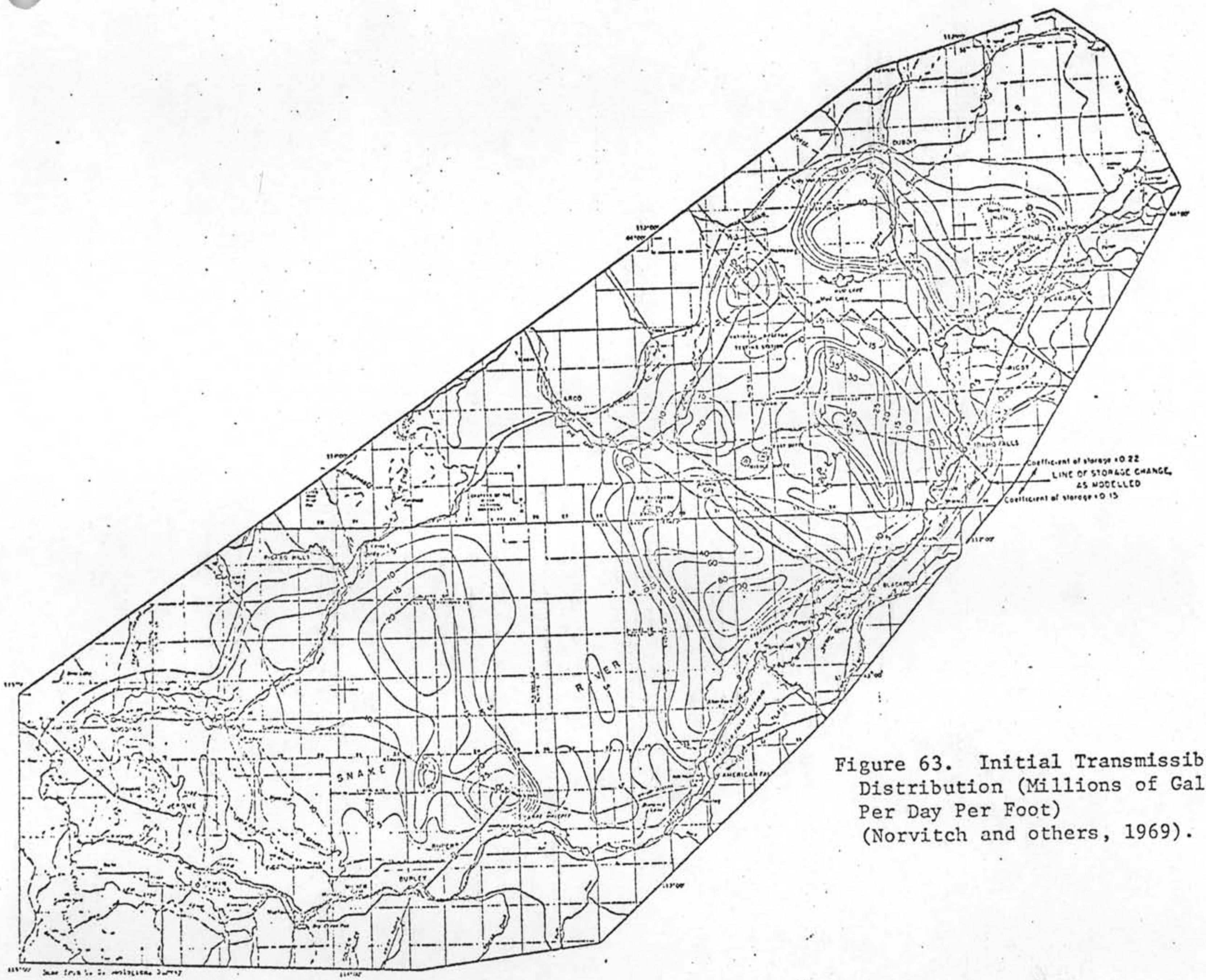

Figure 63. Initial Transmissibility Distribution (Millions of Gallons Per Day Per Foot) (Norvitch and others, 1969).

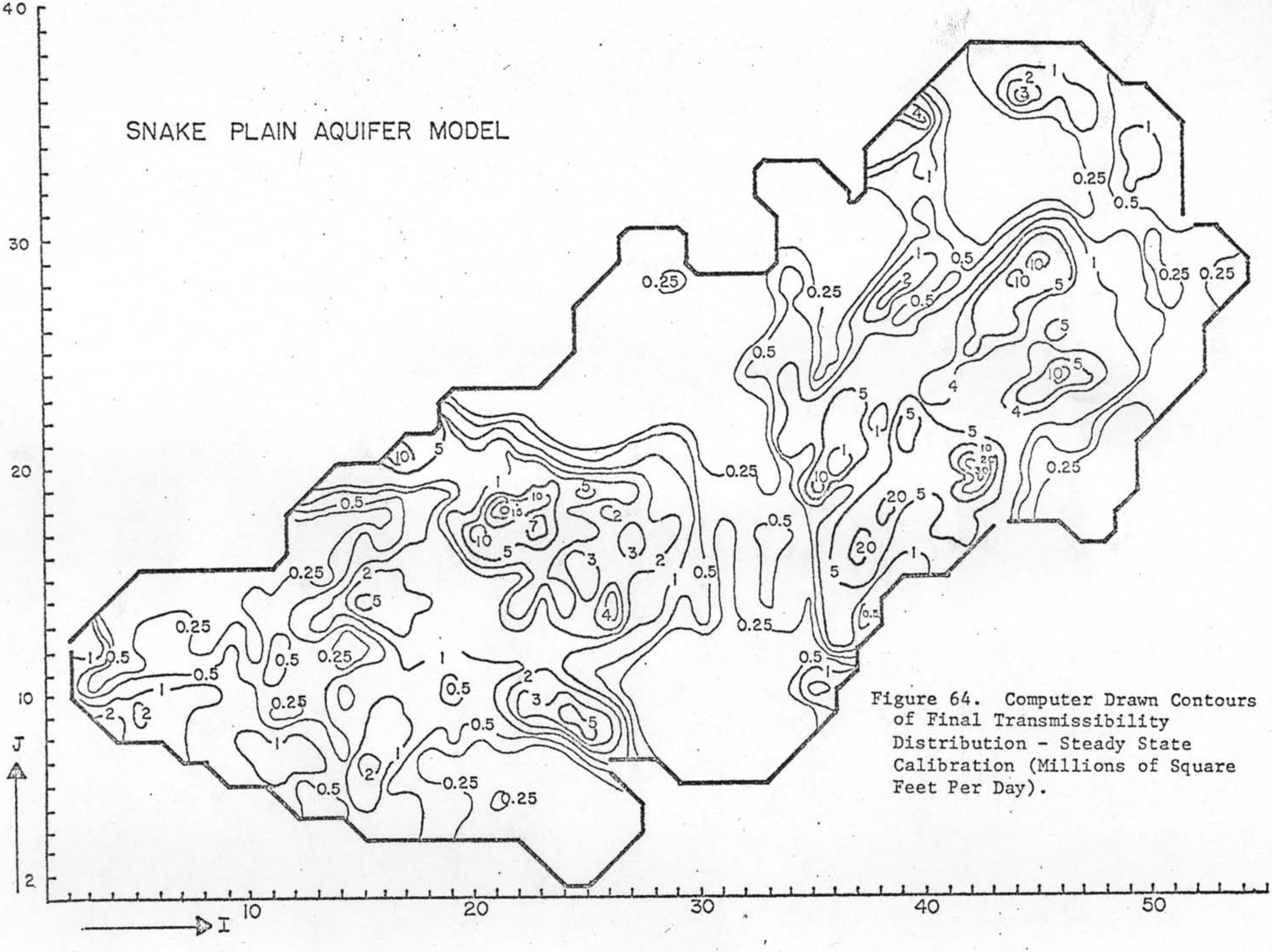

Figure 64. Computer Drawn Contours of Final Transmissibility Distribution - Steady State Calibration (Millions of Square Feet Per Day).

the historical and simulated contour lines are not discernable. Figure 63 is the transmissibility contour map in millions of gallons per day per foot, from which the original T-values were interpreted (Norvitch and others 1969), while Figure 64 is the calculated final T-distribution in millions of square feet per day (1 ft^2/day = 6.7 g/day ft). The high and low T-values are generally concentrated in the same regions but the magnitude of the T-values differs substantially.

Mass Balance-Steady State Calibration

The final steady state calibration, resulting in a standard deviation of 3.5 feet produced a T-distribution to be used as initial T-values in the unsteady state calibration. Before engaging in the unsteady state calibration a closer look was given to the mass balance calculated for the steady state calibration. For the equilibrium state simulation the storage accretion tends to zero. The Snake Plain aquifer is considered a non-leaky aquifer making the total leakage to or from the aquifer zero. This reduces the mass balance equation as described on page 64 to

$$\sum_{i,j} [Q(i,j) + Q_h (i,j)] = 0 \tag{42}$$

Where

$Q(i,j)$ = external source term for node (i,j)

$Q_h(i,j)$ = flow across boundary formed by hydraulically connected stream for node (i,j).

Equation 42 implies that the total input to the aquifer calculated in the input program $Q(i,j)$ must equal the total flow across the hydraulically connected stream nodes, $Q_h(i,j)$, as calculated in the INOUT subroutine. The left part of Table 16 shows the calculated steady state flows resulting from Test 10. The total net flow from all stream nodes amounts to 7.47 million ac ft, while the total input to the aquifer amounts to 5.83 million ac ft, a difference of 27%. The flow leaving the aquifer can be divided into three specific areas as denoted in the table. The subtotal of the Thousand Springs area amounts to 4.46 million ac ft while the input program (see page 124) indicates an amount of 4.83 million ac ft leaving the aquifer at Thousand Springs, a difference of only 8%.

The Minidoka-Neeley reach includes Lake Walcott. The lake loses water to the aquifer calculated at 64,000 ac ft, while the flow to the Snake River from the area directly east of Lake Walcott to Neeley is calculated at 63,000 ac ft, making the net total 1,300 ac ft. This is the total flow crossing the boundary in the Minidoka-Neeley reach from the north side. The historical measured reach gain for the Minidoka-Neeley reach is obtained from unpublished data by the Idaho Water Resource Board and amounts to 72,200 ac ft which includes the inflow from both sides of the river. No records are available that measure the total reach gain of the area directly east of Lake Walcott to Neeley so that the calculated figures are difficult to check against the historical records. From water table maps (Mundorff and others, 1964) it is shown that the gradient of the water table south of the Snake River is toward the river. Assuming that the gain from the south equals the gain from the north the calculated total reach gains would amount to -64,900 + 2 x 63,600 = 62,500 ac ft. This is a highly speculative number but it is in the range of the historically measured 72,200 ac ft. Since the magnitude of the river-aquifer exchange in this reach is very small compared to the total mass balance the effect of errors in this area is estimated as minimal.

The third reach is the Neeley-Blackfoot reach, including American Falls reservoir. The calculated flow from the north side of the Snake River to this reach is 3.0 million ac ft while the historically calculated flow (unpublished data - Idaho Water Resource Board) is 0.924 million ac ft, a difference of more than 300%. Figure 65 represents a map of the Minidoka-Neeley and Neeley-Blackfoot reach and the steady state water table contours. The numbers represent the calculated steady state flow to the respective node points in ac ft. For the Neeley-Blackfoot reach most node points show flows of which the relative magnitude is as hydrologically would be expected. The flows near the southwestern part of the reservoir are small, increasing in the northeast

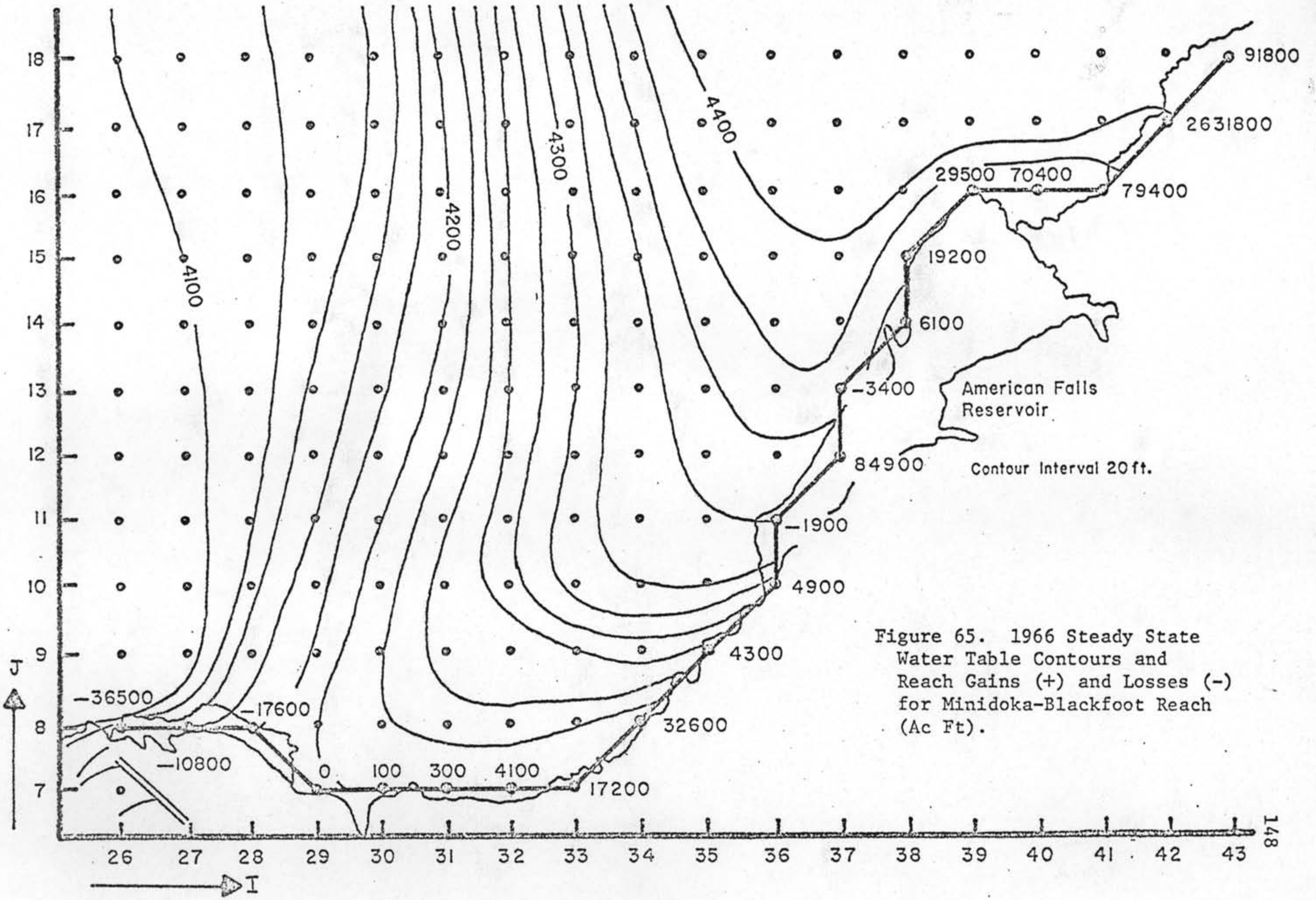

Figure 65. 1966 Steady State Water Table Contours and Reach Gains (+) and Losses (-) for Minidoka-Blackfoot Reach (Ac Ft).

Table 16. 1966 Steady State Flows Across Hydraulically Connected Boundaries.

Flows with Final T-values of Test 10.			Flows with adjusted T-values	
	Node	Flow(ac ft)	Flow (ac ft)	
Thousand	(2,11) =	13259	16040*	
	(2,12) =	28321	31302*	
	(2,13) =	314472	365186*	
Springs	(3,10) =	1268975	1291044*	
	(4, 9) =	654240	722845*	
	(5, 9) =	980713	1046399*	
Area	(6, 9) =	586216	667521*	
	(7, 8) =	108201	117088*	
	(8, 8) =	229266	273779*	
	(9, 7) =	137220	154276*	
	(10, 7) =	20290	28647*	
	(11, 7) =	121650	146602*	
Subtotal		4462800	4860700	4815000 2/
Minidoka	(26, 8) =	-36530	-29204	
	(27, 8) =	-10773	-11287	
	(28, 8) =	-17584	-17645	
to	(29, 7) =	5	4	
	(30, 7) =	130	124	
	(31, 7) =	324	319	
Neeley	(32, 7) =	4121	4092	
	(33, 7) =	17203	17137	
	(34, 8) =	32598	32481	
	(35, 9) =	4300	4286	
	(36,10) =	4947	4932	
Subtotal		-1300	5200	
Neeley	(36,11) =	-1851	-1938	
	(37,12) =	84886	84178	
	(37,13) =	-3437	-3696	
to	(38,14) =	6052	6017	
	(38,15) =	19189	19032	
	(39,16) =	29469	29581	
Blackfoot	(40,16) =	70441	72748	
	(41,16) =	79448	82361	
	(42,17) =	2631753	536068*	
	(43,18) =	91824	102775*	
Subtotal		3006700	927100	924800 1/
Total		746[illegible]	5793100	5831500 2/

*Stream node for which the transmissibility is manually adjusted.
1/Total for that reach as estimated by Idaho Water Resource Board.
2/Total flow as calculated in mass balance of Input Program.

direction, being maximal east and north of the reservoir. This seems correct since the major portion of the reach gain originates from springs northeast of the reservoir and those discharging into the reservoir. This is confirmed also by the general shape of the groundwater contour lines.

The flow of one node point, node point (42,17) seems to be out of proportion to the other nodal flows and amounts to 2.63 million ac ft. The groundwater flow to or from these hydraulically connected nodes is a function of the hydraulic gradient and the transmissibility of the constant head node. As discussed on page 75 the hydraulic head in the aquifer is calculated using weighted average T-values instead of individual T-values. The first head value calculated is the head one node spacing inside the constant head or stream boundary. The flow to the stream node may be readjusted by changing the T-value of the constant head node. However, this will affect the weighted average T-value of the interior node, causing a change in the hydraulic gradient that will tend to offset the T-adjustment. In order to maintain the original flow pattern when the T-value of the constant head node is adjusted,the interior node of this constant head node must also be adjusted. The necessity for this procedure is effectively demonstrated for node point (42,17).

Table 17 represents the T-values in thousands of square feet per day for the Neeley-Blackfoot reach. The calculated flow to this node point is 2.63 million ac ft. To simulate the historical reach gain for this reach the flow has to be reduced from 3.01 million ac ft to 0.92 million ac ft. Only the flow to node point (42.17) is out of proportion and the total necessary change may be obtained by changing the T for this node point, an extreme adjustment, reducing the nodal flow from 2.63 million ac ft to 0.543 million ac ft. Therefore the T-value was decreased accordingly from 31.6 million ft^2/day to 6.5 million ft^2/day. A test simulation was run with the new T without adjusting the interior nodal T-value. The resulting head values calculated for the interior nodes close to this node point were 6 to 8 feet higher. Although the flow calculated for node point (42,17) was decreased significantly, the increased gradient partly offset the reduction. The increased gradient was a result of a reduction of the weighted average T-values of node

Table 17. Final Steady State T-Values for Neeley-Blackfoot Reach in 1 000 ft^2/day.

J				
23	1718	2255	5294	1789
22	16945	27328	16823	1298
21	10304	12809	13337	905
20	6123	42868	5992	1088
19	9847	9691	3510 (3847)	567
18	5059	21548 (34656)	5985 (4519)	0
↑ 17	2450	31617 (4770)	0	0
16	311	0	0	0
15	0	0	0	0
→ I	41	42	43	44

(——) = T-values within parentheses are manually adjusted

point (42,18). In order to maintain the same gradient the weighted average T for node point (42,18) should be maintained at the same value. The weighted average T-value for node point (42,18) with the original T-values is (see Table 17).

$$1000\ [4(21{,}548) + (31{,}617) + (9{,}691) + (5{,}985) + (5{,}059)] =$$

$$138.5 \text{ million } ft^2/day$$

Substituting 6.5 million ft^2/day for node point (42,17) and maintaining the same weighted average transmissibility for (42,18) the new T-value for node point (42,18) was calculated as follows

$$T\ (42,18) = 1000\ [(138{,}544) - (6{,}513) - (9{,}691) - (5{,}985) -$$

$$(5{,}059)]\ /4 = 27.8 \text{ million } ft^2/day$$ as compared to the original value of 21.5 million ft^2/day.

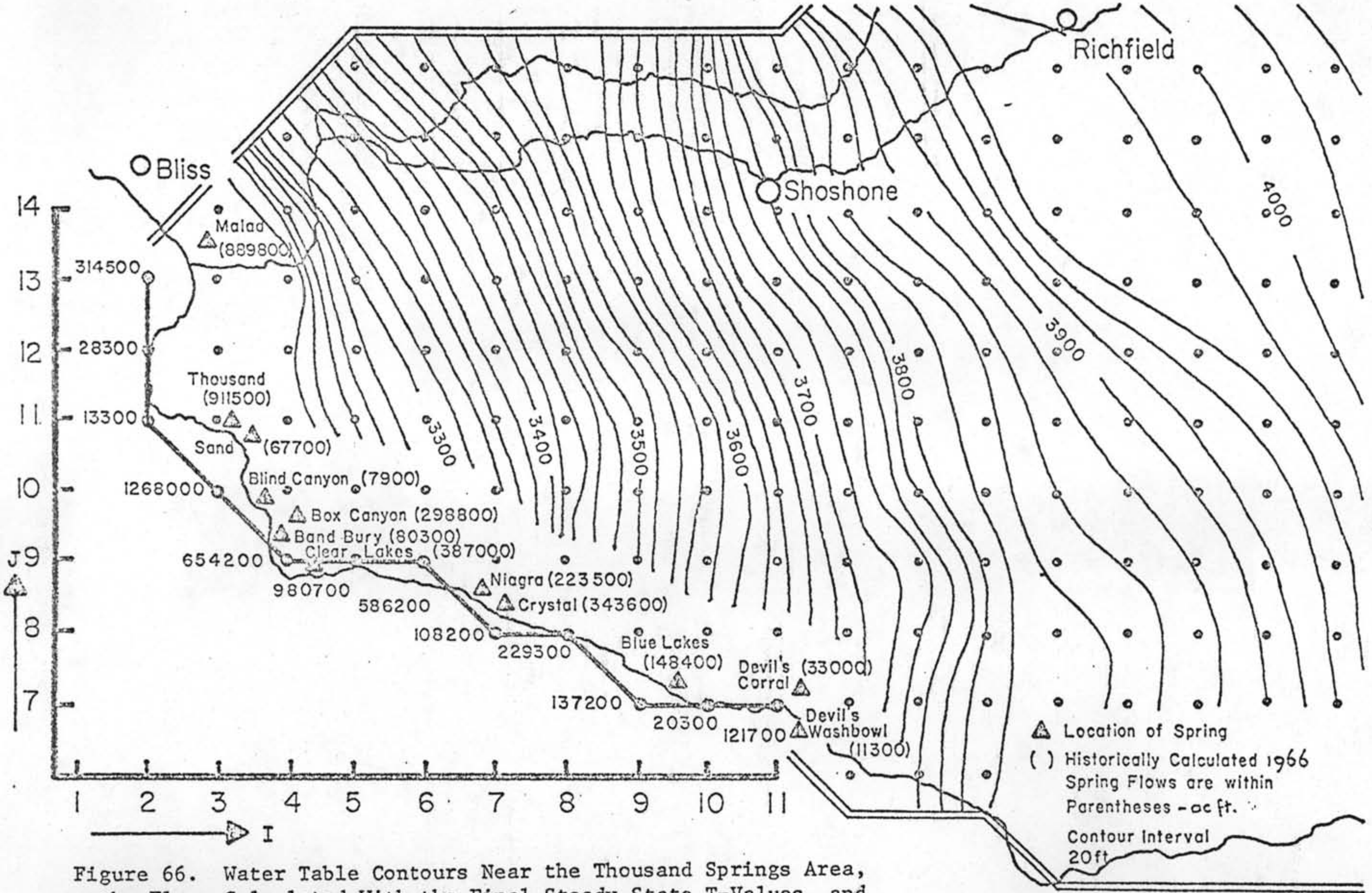

Figure 66. Water Table Contours Near the Thousand Springs Area, the Flows Calculated With the Final Steady State T-Values, and the Historically Calculated Spring Flows (Ac Ft).

With the new T-value a new steady state equilibrium was obtained in which the change of head values close to this node were less than a foot, a satisfactory result. The flow for node point (42,17) was reduced from 2,631,753 ac ft to 686,084 ac ft. The total flow for the Neeley to Blackfoot reach was still 15% more than the historically calculated total of 924,800 ac ft. A final adjustment was made for the transmissibility values of nodes (42,17), (42,18), (43,18), and (43,19). The results are shown in Table 17. The figures within parentheses represent the readjusted T-values. Table 16 represents the original and adjusted calculated flows for the reach node points and the historical total. The adjusted and historical total flow are within 0.2%, a satisfactory result.

No adjustments were made for the Minidoka to Neeley reach. The total flow of the last reach denoted as the Thousand Springs area was 8% lower than that indicated by the mass balance in the input program. Figure 66 represents this area and shows flows calculated with the final T-values of Test 10 and the location and magnitude of historically measured flows of the actual springs in parentheses as reported in Idaho Department of Water Administration, Water Bulletin No. 9 (1969). The designated flows do not represent the total flow since only the flows are calculated which were in an accessible location. In comparing the concentration of the historic flows and the relative magnitude of the calculated flows it is apparent that the relative distribution of the calculated flows follows the historical spring flows quite accurately. Therefore it was decided that the 8% increase necessary to simulate the historical total was to be obtained by increasing the T-values of all stream nodes in this reach by the same percentage and to compensate the interior nodal T-values accordingly. In this simulation after adjustment to boundary T-values the resulting flow calculated in the INOUT subroutine amounted to 4.86 million ac ft, 0.5% off the historical flow. Table 18 represents a table of the final T-values of Test 10 while the readjusted T-values in parentheses for the Thousand Springs reach and Table 16 represents the original and adjusted calculated flows for the three reaches. The new value calculated for the net outflow, Q_h, amounts to 5.79 million ac ft. The total input to the aquifer was calculated at 5.83 million ac ft, a difference of 0.3%.

Table 18. Final Steady State T-Values for the Thousand Springs Area in 1000 ft^2/day.

17	0	0	0	0	0	0	0	0	0	0
16	0	0	0	9	14	68	209	73	78	103
15	0	0	13	59	36	160	330	82	503	244
14	0	184	41	292	387	352	63	283	227	397
13	995 (1145)	1535 (1432)	145	287	362	245	80	601	112	711
12	86 (97)	121	81	102	313	237	425	595	273	809
11	39 (50)	276 (180)	968	1032	1526	1096	1588	832	564	400
10	0	3614 (4279)	965 (760)	686 (573)	1912 (1814)	1064	421	686	446	89
9	0	0	1407 (1677)	4667 (5220)	1607 (1892)	700 (596)	828 (748)	803	1687	530
8	0	0	0	0	0	284 (340)	491 (589)	622 (523)	628 (527)	1801 (1698)
7	0	0	0	0	0	0	0	320 (381)	244 (298)	472 (564)
6	0	0	0	0	0	0	0	0	0	0
J ↑ 5	0	0	0	0	0	0	0	0	0	0
→ I	2	3	4	5	6	7	8	9	10	11

(———) T-values within parentheses are manually adjusted

Readjustment of the T-values along the stream boundary did have some influence on the standard deviation over the whole aquifer but was kept at a minimum with the compensation of the interior nodal T-values. The standard deviation with the original T-values was 3.50 feet while the standard deviation calculated with the adjusted T-values amounted to 3.55 feet, an increase of 1.4%. The refinement of the constant head transmissibility values balanced the aquifer system without changing the flow pattern,expressed in the water table contour lines and the readjusted T-distribution was used in the unsteady state calibration.

Unsteady State Calibration

The steady state calibration provided an approximate transmissibility distribution that enabled a satisfactory simulation of the average water table conditions in the Snake Plain aquifer. Very little is known of the storage coefficients (Thomas, 1969), so that an area distribution similar to that of the T-values is not available. For this reason, a uniform storage coefficient was used for the entire area of the Snake Plain. To calibrate the storage coefficient most effectively, a time period was chosen in which the magnitude of the changes in water table elevations was adequate to allow a calibration of the storage coefficients.

In the unsteady state calibration the rise and decline of the water table in a 12 month period,caused primarily by recharge from irrigation was simulated. April 1, 1966 was chosen for the initial water table values, representing the time that the majority of the node points are at the starting point of the normally cyclic water table variations. The simulation was run for one year with two week half timesteps during the period that recharge of the aquifer is greatest (the irrigation season). During the non-irrigation season monthly half timesteps are utilized, the last half timestep ending at March 31, 1967. Table 19 is a time table of the simulation. K represents the half timestep number.

The time period between August 16 and August 31, half timestep 11, is the period in which the majority of the node points obtain their maximum or minimum water table elevations. For this simulation two timesteps are chosen for which deviations between historic and calculated water table values are computed. These are half timestep 11, the maximum,and half timestep 21, the end of the cycle, making NV(1) = 11 and NV (2) = 21. For these two half timesteps historic water table values for every node point have to be tabulated in arrays APHI(i,j,1) and APHI(i,j,2). The objective is to simulate as well as possible the maximum rise or decline of the aquifer occurring at timestep 11. For that reason the deviations at half timestep 11 are used to adjust the parameter values. The total number of half timesteps is 21, while the number of half timesteps with the original timestep length of 30.41 days is 17.

Table 19. Time Table for Unsteady State Calibration

Initial Starting Condition: April 1 Water Table (K=1)

	day	K (half timestep number)	
April 1		1	
April	1-15	2	Δt_1 = 30.41 days
	16-30	3	↓
May	1-15	4	
	16-31	5	
June	1-15	6	
	16-30	7	
July	1-15	8	
	16-31	9	
August	1-15	10	
	16-31	11	———— APHI(1), NV(1) = 11
September	1-15	12	
	16-30	13	
October	1-15	14	
	16-31	15	
November	1-15	16	
	16-30	17	(LTS)
December	1-31	18	Δt_2 = 60.83 days
January	1-31	19	↓
February	1-28	20	
March	1-31	21	(NTS) ———— APHI(2), NV(2) = 21
NTS = 21	20 half timesteps		
LTS = 17			

Data Collection for the Study Area

All the input arrays for the main program have the same values assigned as in the steady state calibration except for the following;

NREACH(I,J): For the unsteady state calibration therewere three stream nodes that show a time-varying water table, each with a different variation. These are nodes (36,10), (36,11) and (37,12) located along the southeast side of American Falls Reservoir.

DELH(K,I) : Represents the change in water level from $\tau = I-1$ to $\tau = I$ for reach K. Values wereread in the three stream nodes for every half timestep.

Q(I,J) : The source term for every node point wasread from tape for every half timestep and generated in the input program.

KX(I,J)
KY(I,J) : For these arrays the final hydraulic conductivity values resulting from the steady state calibration and the boundary adjustment are substituted.

S(I,J) : In the equilibrium state calibration the values assigned to the storage coefficient, S(I,J), are not essential and were assigned the value 0.15. In the unsteady state it was decided that a storage coefficient of 0.10 as initial estimate more closely represented the storage coefficient of the aquifer.

PHI(I,J,1) : The initial historic water table elevation for every node point, in this case the April 1, 1966 water table elevations, were obtained by nodal interpolation from a computer generated water table contour map.

APHI(I,J,1): The historic August 31, 1966 water table elevations were used for comparison and adjustment of parameters in the calibration. The water table elevations were obtained in the same way as the April 1, 1966 water table elevations.

APHI(I,J,2): The March 31, 1967 historic water table elevations represent the end of the simulation cycle.

The historic seasonal fluctuations of the water table caused by natural recharge and irrigation water application have a cyclic character, the magnitude of which is nearly constant each year. Well data indicate that the difference in water levels between the April 1, 1966 water table and the April 1, 1967 water table is less than one foot and in the order of magnitude of the interpolation error occurring in the determination of the nodal values from a water table contour map. For this reason the hydraulic head values for March 31, 1967 were set equal to the initial hydraulic head values, APHI(I,J,2) = PHI(I,J,1).

Input 1966-1967 Unsteady State Calibration

The input program calculates for every node point for every half timestep the total source term Q(I,J). Two data sets were required for the input program (see page Appendix A). Data Set 1 is identical to that of the steady state calibration. Data Set 2 is related to the water distribution per half timestep. The arrays which make up Data Set 2 are identical to Data Set 2 of the steady state calibration, but have to be prepared for every half timestep. The following data was required for every half timestep.

1 - Total diversion per district.

2 - Return flow percentage. For every half timestep the same return flow percentage was used, equal to that of the yearly average. If more specific information is available, the return flow percentage can be changed for every half timestep.

3 - Consumptive use for each climatic region.

4 - Precipitation for each climatic region.

5 - Artificial recharge or discharge for each node. This allows for simulation of recharge projects. For this study this term was zero for every half timestep.

6 - Change in irrigated acreage per node. No changes occurred in 1966-1967; therefore the changes were zero for every half time step.

7 - Tributary valley groundwater underflow.

The data for the underflows of the respective valleys is only available as average yearly estimates from various reports. For the unsteady state calibration it was necessary to divide the underflow in two weekly and monthly totals. The yearly total might be distributed as the runoff hydrograph but this would be speculative since little information is available about lag times between runoff and the groundwater underflows. In these trials, for lack of better data, the base flow was distributed equally over all half timesteps. Two areas deserve special attention. Both are considered artificial valleys and the total of base flow and irrigation diversion was distributed over specific nodes. The first valley is the Henrys Fork, designated as Valley No. 8 (see Figure 52, page 118). The average 1966 underflow was calculated at 725,000 ac ft of which 225,000 ac ft is denoted as base flow distributed equally over all half timesteps. The remaining 500,000 ac ft originates from irrigation diversions in the Henrys Fork and Egin Bench area and was distributed according to the diversions of the St. Anthony Canal, one of the major irrigation canals in that area. The second valley is the Rigby area, Valley No. 9. From the Rigby area study (Brockway and De Sonneville, 1973) a total of 1,203,000 ac ft was determined as recharge of which 90,000 ac ft is considered to be base flow equally distributed over all half timesteps. The remaining 1,113,300 ac ft originates from irrigation and is distributed according to the mass flow hydrograph of four major canals in the area; the Farmers Friend, Anderson, Eagle Rock and Harrison Canals.

8 - Reach gains and losses which were determined for every half timestep.

The program generates a tape that serves as input to the model program. It also provides a printout that lists for every half timestep diversions per district, the water use per climatic region, underflows, reach gains or losses and a table of terms that comprise the net input per node contributed by each component. It also provides a summary of half timestep totals over all nodes in the aquifer, as well as the totals over the entire simulation period.

Summary of Unsteady State Source Term Q(i,j)

For the April 1, 1966 to April 1, 1967 year the following total inputs were calculated for the Snake Plain aquifer model area.

The total net diversion for all surface water irrigated lands amounts to 5,305,555 ac ft resulting in an average net application rate of 5.53 ac ft/ac. The total consumptive use over the surface water and groundwater irrigated lands was calculated as 2,579,000 ac ft. A total of 742,641 ac ft of precipitation was calculated as effective recharge to the aquifer of which 541,147 ac ft fell during the growing season on the cropped lands. The sum total of all these inputs, including the tributary groundwater underflow and the input from reach gains and losses, excluding the contribution from the hydraulically connected reaches was

5,667,920 ac ft.

Single Parameter Calibration

In the first three tests of the unsteady state calibration the rise and decline of the water table in a 12 month period, caused primarily by irrigation practices, was simulated adjusting the storage coefficient as only parameter. The values assigned to the control variables of the main program (card 1-4) and calibration routine (card 5) for the unsteady state single-parameter calibration is given in Table 20. A precise explanation of the control variables is given in Appendices A and B.

Table 20. Control Variables for the Unsteady State Single-Parameter Calibration - Snake Plain Aquifer

Card 1	Card 2	Card 3	Card 4	Card 5	
MICX = 0	NPX = 0	NQ = 5	NROW = 55	NVAI = 4	NV(3) = 0
MICY = 0	NPY = 0	FLUX = 0.0	NCOL = 40	NVA2 = 4	N10 = 1
XA = 13.7	NSUI = 0	MU = 1.00	DELT = 30.41	NVA3 = 1	JI = 10
YA = 10.65	NSU3 = 0		DELX = 16404.2	NSTOP = 1	JK = 10
NIM = 60	NSU3 = 0		DELY = 16404.2	ITE = 40	VP = 0.40
NSER = 0			LTS = 17	NST = 2	SREST = 500
			NTS = 21	NV(1) = 11	REST = 0.0
			NRIVER = 3	NV(2) = 21	RMI = 0.0

The storage coefficients wereadjusted based upon the relative difference between the August 30 and April 1 head values. For the first test the head values were obtained from generated contour maps with 20 foot contour intervals. The differences in head values for most node points in the aquifer is less than a foot or 1/20 contour interval. Some surface water irrigated areas show a rise of 8-9 feet. However, most of the larger differences occur in areas where the groundwater gradient is steeper so that the shift in contour lines between the April and August water tables is small. Errors equal in magnitude to the actual water table rise may result either from subjectiveness in drawing contour lines or in interpolation of nodal values from contour lines. In areas with small deviations the interpolation might be ambiguous and the total error involved may result in a decline of the water table while the true historic deviation should have been a rise of the water table. In the Rigby-Ririe area (all surface water irrigation) the local groundwater table showed large rises (6 times the contour interval) for most node points in the aquifer and the calculated head difference was in the same direction as the recorded differences which allowed for a proper adjustment of the S-values.

In the Snake Plain aquifer some areas show declining water tables caused by pumped irrigation. Other areas show rising water tables, in the order of 4-25 feet, caused by surface water irrigation. A simulation was run with the initial uniform storage coefficients to obtain an array of calculated rises and declines of water table values. Even if the storage coefficients are incorrect the array shows the general trend of the water table. Problem areas were indicated where the calculated head differences were in the opposite direction from the interpolated head differences. For those node points no rational storage coefficient adjustment can be proposed (as explained on page 85) and the S-values were kept at the initial value. For this aquifer the inputs per node point were believed to be more accurate than any other parameter, suggesting that the historic water table values were incorrectly interpolated. If the historic rises and declines were to be represented as deviations the standard deviation calculated from the August and April water table maps would be 4.99 feet, which is relatively small. This presents another problem in that this average variation is only 1.5 feet larger than the standard deviation between calculated and historic equilibrium values in the 1966 steady state calibration (3.55 feet).

This standard deviation of 3.55 feet represents the average discrepancy between the calculated and the historic average 1966 water table. The April 1, 1966 water table shows for the majority of the node points minimal differences with the average 1966 water table. In an unsteady state simulation with as starting head values the April 1, 1966 head values,the T-values obtained from the steady state calibration may introduce variations in head values that could change the sign of the calculated differences between the August and April dates. The magnitudes of these variations are such that they could turn a calculated rise into a decline, while the historic tabulated difference may be obscured by errors, as explained above, the combination of which may result in an ambiguous comparison between historic and calculated head values.

It could be argued that the deviations resulting from the steady state calibration do not influence the unsteady state simulation because the April 1, 1966 water table is a water table at a different date. In case of the Snake Plain aquifer the difference between the 1966 average water table and the April 1, 1966 is minimal for the majority of the node points, indicating that the deviations resulting from the optimum T-values in the steady state simulation would also appear if the T-values were to be used in the unsteady state calibration. In order to eliminate the error introduced by the T-values the deviations resulting from the steady state calibration were added both to the April 1, and August 30 water tables.

Above discussion leads to the same conclusions reached in the Resume of the Calibration (page 86); the calibration routine will adjust efficiently the initial parameter values and obtain a good match of the historic water table only if accurate water table records are available. The calibration routine represents a considerable improvement; it eliminates to a great extent the need for accurate initial geohydrological parameter values, necessary for models that do not incorporate such a routine, but water table records are needed and should be more accurate in areas where the variations in head values, seasonal or yearly, are small. The Snake Plain aquifer is an excellent example of this data dilemma; it is a large aquifer with high T-values which reduce the response of the water table to water input, making a calibration more difficult.

The first test utilized the interpolated April 1, and August 30 water table maps which were adjusted by adding to both dates the deviations resulting from the optimum steady state T-values. For every iteration the sum of squares of deviations was calculated for the August 30, 1966 and March 31, 1967 head values and added together. The calibration was initially continued till the total sum of squares, a measure of the fit over the total simulation period, started to increase. August 30 = half timestep 11, for first iteration resulting in SS(11,1); March 31 = half timestep 21, for first iteration resulting in SS(21,1). The total sum of squares of deviations over the simulation period for simulation 1 is:

$$SSS(1) = SS(11,1) + SS(21,1)$$

The storage coefficients were adjusted based on the deviations of the 11th half timestep because best fit priority was given to this half timestep. Decrease in SS(11,2) might result in an increase of SS(21,2) such that SSS(2) > SSS(1). In general, if SSS(k) > SSS(k-1) the calibration is terminated if in addition SS(11,k) < SS(21,k). However, if the sum of squares of the priority timestep is larger than the other half timestep sum of squares [SSS(k) > SSS(k-1) but SS(11,k) > SS(21,k)], the calibration is continued until the priority timestep sum of squares is less than or equal to sum of squares of the other timestep. In this way the discrepancy between historical and calculated head values is equally divided over the two timesteps of the simulation period.

The second test used the same rationale in the calibration but a different method was used in obtaining the August 30 water table values. In order to eliminate the error introduced when corresponding node points on the two water table maps are not interpolated in a consistent way the computer contour plot program was run not for the August 30 well elevations but for the water level differences occurring between August 30 and April 1. This provides a contour plot of differences which more accurately describes the absolute differences between the two water tables because it eliminates inconsistent interpolation for the same node at two different dates. For Test 2, the initial April 1, 1966 water table values and March 31, 1967 water table values were identical to the values of Test 1. The August 30 water table values were obtained by adding the interpolated (August - April) differences to the April 1, 1966 water table values. Also for Test 2, to both water tables the deviations

Table 21. Unsteady State Calibration of Storage Coefficient, Tests 1 and 2 (Steady State Deviations Added to Water Table)

Test 1		Test 2
Using August and April 1 Water Table Contours		Using April 1 Water Table and Contour Plot of Differences Between August and April
SS(11,1) = 0.2560 05		SS(11,1) = 0.1761 05
SS(21,1) = 0.1198 05	S_c = 5.36	SS(21,1) = 0.1198 05
SSS(1) = 0.3758 05		SSS(1) = 0.2959 05
SS(11,2) = 0.2217 05		SS(11,2) = 0.1525 05
SS(21,2) = 0.1198 05	S_c = 5.14	SS(21,2) = 0.1205 05
SSS(2) = 0.3415 05		SSS(2) = 0.2730 05
SS(11,3) = 0.1896 05		SS(11,3) = 0.1311 05
SS(21,3) = 0.1283 05	S_c = 5.05	SS(21,3) = 0.1283 05
SSS(3) = 0.3179 05		SSS(3) = 0.2594 05
SS(11,4) = 0.1678 05		SS(11,4) = 0.1185 05
SS(21,4) = 0.1404 05	S_c = 5.0	SS(21,4) = 0.1371 05
SSS(4) = 0.3082 05		SSS(4) = 0.2556 05
SS(11,5) = 0.1532 05		SS(11,5) = 0.1114 05
SS(21,5) = 0.1497 05		SS(21,5) = 0.1451 05
SSS(5) = 0.3029 05*		SSS(5) = 0.2564 05
SS(11,6) = 0.1475 05		
SS(21.6) = 0.1565 05		
SSS(6) = 0.3040 05		
S_{11} = 3.91 ft.		S_{11} = 3.44 ft. S_h = 4.99 ft.
S_{21} = 3.86 ft.		S_{21} = 3.70 ft. S_c = 5.0 ft.

S_{11} = standard deviation between calculated August 30, 1966 head values and historic August 30 head values.

S_{21} = standard deviation between calculated March 31, 1967 head values and historic March 31 head values.

S_c = average calculated rise and decline between April 1, 1966 and August 30, 1966, expressed as a standard deviation.

S_h = average historic rise and decline, expressed as a standard deviation.

*simulation resulting in minimum total sum of squares.

resulting from the steady state calibrationwere added. Table 21 shows the results of Test 1 and Test 2, in which only the storage coefficient is adjusted.

A comparison is made of the sum of squares of deviations SS, of the two half timesteps and the total sum of squares, SSS, over the simulation period. The sum of squares of the simulation using the initial storage coefficient values are SS(11,1), SS(21,1) and SSS(1) respectively. For both tests the same March 31, 1967 water table values were used as well as the same initial storage coefficients so that SS(21,1) of Test 1 equals SS(21,1) of Test 2. It is clearly shown that some of the interpolation error was eliminated by using the contour plot of differences to obtain the August 30 water table values since SS(11,1) of Test 2 less than SS(11,1) of Test 1. For both tests the initial priority timestep SS was larger than the other timestep SS. For Test 1 the total sum of squares decreased until the 6th iteration. SSS(6) > SSS(5) and the calibration was terminated because for iteration 6, SS(11,6) < SS(21,6). Test 2 shows similar results but reaches the optimum storage coefficient values one iteration earlier as indicated by asterisk. Test 2 using the contour plot of differences shows better results. S_{11} represents the standard deviation between the calculated and historic August 30, 1966 head values while S_{21} represents the standard deviation for March 31, 1967. S_c represents the average calculated rise and decline between the starting head values, April 1, 1966 and the head values at August 30, 1966, expressed as a standard deviation while S_h represents the average historic rise and decline of the aquifer expressed as a standard deviation.

The optimum calibration for Test 2 resulted from the storage coefficient values of the 4th simulation. S_{11} = 3.44 feet, S_{21} = 3.70 feet while S_c decreased from 5.36 feet to 5.0 feet (S_h = 4.99 feet). The results show that the average simulated rise or decline in the aquifer approaches the average historic rise and decline but that there are deviations in fit at individual node points. The scatter around the August 30 historical water table amounts to a standard deviation of 3.44 feet, around the March 31 water table the scatter amounts to 3.70 feet standard deviation.

Table 22. Calibration of Storage Coefficient, Test 3 (Steady State Deviations Not Added)

SS(11,1) = 0.2090 05
SS(21,1) = $\underline{0.1539}$ 05 S_c = 5.78
SSS(1) = 0.3629 05

SS(11,2) = 0.1808 05
SS(21,2) = $\underline{0.1532}$ 05 S_c = 5.51
SSS(2) = 0.3340 05

SS(11,3) = 0.1567 05
SS(21,3) = $\underline{0.1587}$ 05 S_c = 5.32
SSS(3) = 0.3154 05

SS(11,4) = 0.1425 05
SS(21,4) = $\underline{0.1648}$ 05 S_c = 5.19
SSS(4) = 0.3073 05

SS(11,5) = 0.1351 05
SS(21,5) = $\underline{0.1706}$ 05* S_c = 5.10
SSS(5) = 0.3057 05

SS(11,6) = 0.1334 05
SS(21,6) = $\underline{0.1765}$ 05 S_c = 5.12
SSS(6) = 0.3100 05

S_{11} = 3.67 ft. S_h = 4.99 ft.

S_{21} = 4.13 ft. S_c = 5.10 ft.

*simulation resulting in minimum total sum of squares.

For the remaining tests the contour plot of differences between the August and April water tables was used. To check whether the addition to the water tables of the deviations resulting from the steady state T-values indeed decreased the sum of squares of deviations, a calibration was performed without this addition (Test 3, Table 22). In comparing Test 3 with Test 2 it is observed that the initial SS is greater for Test 3 while S_c is also larger. The final optimum storage coefficient resulted in a higher standard deviation indicated by S_{11} = 3.67 feet and S_{21} = 4.13 feet.

Multi-Parameter Calibration

In the first three tests, only the storage coefficient S was adjusted. The sum of squares of deviations continued to decline for the priority timestep, August 30, but increased for March 31, eventually increasing the total sum of squares. In the steady state calibration, it was possible to separate the parameters T and S because the terms involving S in the differential equation, governing equilibrium state flow, vanish.

For the unsteady state simulation, the rise and decline of the water table are dependent on the diffusivity of the aquifer which is a function of T and S and it would be possible to separate the parameters if the T-values were adjusted in a true steady state condition, i.e., the 1966 average historic water table used in the steady state calibration was truly in equilibrium state with the 1966 historic input. Well records show that the average 1966 water levels were lower than the 1965 average water levels, indicating a slightly transient situation, and warranting a further adjustment in the T-values resulting from the steady state calibration. In the steady state calibration in which the total 1966 input was applied for every half timestep, an equilibrium state was created and the T-values were adjusted by comparing the calculated equilibrium gradients with the historic quasi-equilibrium gradients. The reason for adjusting T in a steady state calibration was to improve the T-distribution from estimates to a distribution more closely resembling the true T-values. In a multi-parameter unsteady state calibration in which both T and S are simultaneously adjusted it would be possible to use the originally estimated T distribution, as far as the adjustment of T is concerned. T-values are adjusted based on the ratio of historic and calculated gradients and a 'unique' or repeatable final T-distribution is independent on the type of calibration. However, using the initial starting T-values would unfavorably influence the adjustment of S in this calibration.

Use of original T estimates would cause large errors in initially calculated head value and would result in a 50-foot standard deviation (as shown in the steady state calibration). This would obscure completely the average water table variation of 5 feet occurring between April 1, 1966 and August 30, 1966. The resulting direction of the calculated head differences between the two dates would be erroneous. Because the adjustment of S is based on the ratio of calculated and historic head differences and is only adjusted if the two have the same sign the adjustment of S would be invalid.

The values assigned to the control variables of the main program for the unsteady state multi-parameter calibration where T and S are changed are the same as those of the single-parameter calibration, given in Table 20. The values assigned to the control variables of the calibration routine are also the same as shown in Table 20, except for the variables NVA1(=1), NVA3(=3), NSTOP(=6) and ITE(=3).

Table 23 shows the results of Test 4, multi-parameter calibration. A multi-parameter calibration consists of a number of calibration cycles. In one calibration cycle three simulations (iterations) per parameter are run, the first simulation using the optimum parameter values of the previous calibration cycle. For the first calibration cycle these are the initial parameter values. For Test 4 the T-values were adjusted first. After two adjustments (three simulations) the T-values were changed back to the original T-values and the S-values were adjusted. The optimum values for T and S of the first calibration cycle serve as starting parameter values of the second calibration cycle. SUNST(i) represents the total sum of squares using the optimum parameter values of the (i-1)th calibration cycle. The number of calibration cycles is continued until SUNST(i) increases. For the 4th calibration cycle the SSS(1) of the T-values and SSS(1) of the S-values were the smallest of the three,indicating no improvement in the 4th cycle. This is indicated in SUNST(5), representing the total sum of squares resulting from the optimum T and S values of the 4th cycle. SUNST(4) represents the minimum sum of squares, or, the optimum values of the 3rd cycle, denoted by the asterisks, resulted in the least sum of squares.

S_{11} = 2.46 feet compared to 3.44 feet of Test 2 (changing only S)
S_{21} = 3.53 feet compared to 3.70 feet of Test 2 (changing only S)
S_c = 4.83 feet and S_h = 4.99 feet

Table 23. Unsteady State Calibration of T and S, Test 4
(Steady State Deviations Added)
Storage coefficient Limits (0.01 ≤ S ≤ 0.300)

Changing T		Changing S	
SUNST(1) = 0.2959 05			
Cycle I_A		Cycle I_B	
	SS(11,1)=0.1761 05		SS(11,1)=0.1761 05
SSS(1)=.2959 05	SS(21,1)=0.1198 05	SSS(1)=.2959 05	SS(21.1)=0.1198 05
	SS(11,2)=0.1580 05		SS(11,2)=0.1525 05
SSS(2)=.2958 05	SS(21,2)=0.1377 05	SSS(2)=.2730 05	SS(21,2)=0.1205 05
	SS(11,3)=0.1267 05		SS(11,3)=0.1311 05
SSS(3)=.2877 05	SS(21,3)=0.1610 05	SSS(3)=.2594 05	SS(21,3)=0.1283 05
SUNST(2) = 0.2421 05			
Cycle II_A		Cycle II_B	
	SS(11,1)=0.9755 04		SS(11,1)=0.9755 04
SSS(1)=.2421 05	SS(21,1)=0.1456 05	SSS(1)=.2421 05	SS(21,1)=0.1456 05
	SS(11,2)=0.9699 04		SS(11,2)=0.8916 04
SSS(2)=.2537 05	SS(21,2)=0.1567 05	SSS(2)=.2363 05	SS(21,2)=0.1471 05
	SS(11,3)=0.7421 04		SS(11,3)=0.8197 04
SSS(3)=.1977 05	SS(21,3)=0.1237 05	SSS(3)=.2363 05	SS(21,3)=0.1543 05
SUNST(3) = 0.1915 05			
Cycle III_A		Cycle III_B	
*	SS(11,1)=0.6702 04		SS(11,1)=0.6702 04
SSS(1)=.1915 05	SS(21,1)=0.1245 05	SSS(1)=.1915 05	SS(21,1)=0.1245 05
	SS(11,2)=0.7066 04	*	SS(11,2)=0.6081 04
SSS(2)=.2182 05	SS(21,2)=0.1476 05	SSS(2)=.1858 05	SS(21,2)=0.1250 05
	SS(11,3)=0.6689 04		SS(11,3)=0.5762 04
SSS(3)=.2233 05	SS(21,3)=0.1564 05	SSS(3)=.1892 05	SS(21,3)=0.1381 05
SUNST (4) = 0.1858 05**			
SUNST (5) = 0.1960 05			

S_{11} = 2.46 ft. S_{21} = 3.53 ft. S_c = 4.83 ft. S_h = 4.99 ft.

SUNST(i) - total sum of squares using the optimum parameter values of the (i-1)th calibration.

* simulation resulting in minimum total sum of squares for a calibration cycle.

**minimum total sum of squares of this unsteady state calibration.

Table 24. Unsteady State Calibration of T and S., Test 5
(Steady State Deviations Not Added)
Storage Coefficient Limits (0.001 ≤S≤ 0.200)

Changing T	Changing S
SUNST(1) = 0.3629 05	
Cycle I_A	Cycle I_B
SS(11,1)=0.2090 05 SSS(1)=.3629 05 SS(21,1)=0.1539 05	SS(11,1)=0.2090 05 SSS(1)=.3629 05 SS(21,1)=0.1539 05
SS(11,2)=0.1976 05 SSS(2)=.3795 05 SS(21,2)=0.1818 05	SS(11,2)=0.1808 05 SSS(2)=.3340 05 SS(21,2)=0.1532 05
SS(11,3)=0.1655 05 SSS(3)=.3754 05 SS(21,3)=0.2099 05	SS(11,3)=0.1567 05 SSS(3)=.3154 05 SS(21,3)=0.1587 05
SUNST(2) = 0.3214 05	
Cycle II_A	Cycle II_B
SS(11,1)=0.1279 05 SSS(1)=.3214 05 SS(21,1)=0.1936 05	SS(11,1)=0.1279 05 SSS(1)=.3214 05 SS(21,1)=0.1936 05
SS(11,2)=0.1289 05 SSS(2)=.3413 05 SS(21,2)=0.2115 05	SS(11,2)=0.1197 05 SSS(2)*=.3158 05 SS(21,2)=0.1961 05
SS(11,3)=0.9790 04 SSS(3)*=.2640 05 SS(21,3)=0.1661 05	SS(11,3)=0.1165 05 SSS(3)=.3261 05 SS(21,3)=0.2097 05
SUNST(3) = 0.2587 05**	
Cycle III_A	Cycle III_B
SS(11,1)=0.9064 04 SSS(1)=.2587 05 SS(21,1)=0.1680 05	SS(11,1)=0.9064 04 SSS(1)=.2587 05 SS(21,1)=0.1680 05
SS(11,2)=0.1047 05 SSS(2)=.3203 05 SS(21,2)=0.2156 05	SS(11,2)=0.8852 04 SSS(2)=.2614 05 SS(21,2)=0.1729 05
SS(11,3)=0.8900 04 SSS(3)=.2866 05 SS(21,3)=0.1976 05	SS(11,3)=0.8615 04 SSS(3)=.2676 05 SS(21,3)=0.1814 05
SUNST(4) = 0.2587 05	

S_{11} = 3.01 ft. S_{21} = 4.1 ft. S_c = 5.1 ft. S_h = 4.99 ft.

SUNST(i) - total sum of squares using the optimum parameter values of the (i-1)th calibration.

*simulation resulting in minimum total sum of squares for a calibration cycle.

**minimum total sum of squares of this unsteady state calibration.

The table shows that the individual sum of squares of the priority timestep (August 30) always decreases in adjusting the storage coefficients. The improvement in standard deviation of the multi-parameter calibration, Test 4, compared to Test 2, S-calibration only, is 28% for August 30 and 4.5% for the March 31, 1967 date.

For Test 4 the S-value adjustments were limited to a maximum value of $S_{max} = 0.30$. A number of nodal points had final S-values ranging from 0.24 to 0.29. Well pump tests show S ranging between 0.04 and 0.22 over the Snake Plain aquifer (Norvitch, 1969). The maximum S-value that would be a realistic figure was set to 0.20 for all additional tests. In Tests 5 and 6, an attempt was made to more accurately define the effect of the addition to the water tables of the deviations resulting from the optimum steady state T-values. For both tests $0.001 \leq S \leq 0.200$. Test 5 utilized the unadjusted head values while Test 6 utilized the adjusted head values for the August and March water tables. The results of Tests 5 and 6 are shown in Table 24 and Table 25 respectively.

For Test 5 as well as Test 6, SUNST(3) was the minimum total sum of squares, indicating that the optimum values of the 2nd calibration cycle resulted in the minimum sum of squares. Using the unadjusted head values in Test 5 resulted in a higher standard deviation for August and April. The difference between the results of Test 4 (Table 23) and Test 6, (Table 25) both using the adjusted head values is minimal. This indicates that the lower maximum limits imposed on S in Test 6 has only minor influence on the calibration. In Table 26 the initial and final standard deviation for the August and March dates for Tests 5 and 6 are compared. For Test 5 the decrease in standard deviation for August 31 is 1.55 feet, for Test 6 1.57 feet, while for both tests the increase in standard deviation for March 31, 1967 is small. Essentially the comparisin shows that the absolute decrease in standard deviation was not influenced by the omission or addition of the steady state deviations.

A closer examination of Tables 24 and 25 reveals that most of the decrease in total standard deviation was caused by the final adjustment of the T-values. Results of Tests 5 and 6 show that use of April 1 and August 30 water tables without adjustment for steady state deviations resultedin a higher final standard deviation. Differences between the

Table 25. Unsteady State Calibration of T and S, Test 6
(Steady State Deviations Added)
Storage Coefficient Limits (0.001≤ S ≤0.200)

Changing T	Changing S
SUNST(1) = 0.2959 05	
Cycle I_A	Cycle I_B
SS(11,1)=0.1761 05 SSS(1)=.2959 05 SS(21,1)=0.1198 05	SS(11,1)=0.1761 05 SSS(1)=.2959 05 SS(21,1)=0.1198 05
SS(11,2)=0.1580 05 SSS(2)=.2958 05 SS(21,2)=0.1377 05	SS(11,2)=0.1525 05 SSS(2)=.2730 05 SS(21,2)=0.1205 05
SS(11,3)=0.1267 05 SSS(3)=.2877 05 SS(21,3)=0.1610 05	SS(11,3)=0.1311 05 SSS(3)=.2594 05 SS(21,3)=0.1283 05
SUNST(2) = 0.2431 05	
Cycle II_A	Cycle II_B
SS(11,2)=0.9755 04 SSS(1)=.2431 05 SS(21,2)=0.1456 05	SS(11,1)=0.9755 04 SSS(1)=.2431 05 SS(21,1)=0.1456 05
SS(11,2)=0.9699 04 SSS(2)=.2537 05 SS(21,2)=0.1567 05	SS(11,2)=0.9089 04 SSS(2)*=.2397 05 SS(21,2)=0.1488 05
SS(11,3)=0.7421 04 SSS(3)*=.1979 05 SS(21,3)=0.1237 05	SS(11,3)=0.8731 04 SSS(3)=.2477 05 SS(21,3)=0.1604 05
SUNST(3) = 0.1950 05**	
Cycle III_A	Cycle III_B
SS(11,1)=0.6865 04 SSS(1)=.1950 05 SS(21,1)=0.1263 05	SS(11,1)=0.6865 04 SSS(1)=.1950 05 SS(21,1)=0.1263 05
SS(11,2)=0.7214 04 SSS(2)=.2224 05 SS(21,2)=0.1502 05	SS(11,2)=0.6610 04 SSS(2)=.1966 05 SS(21,2)=0.1305 05
SS(11,3)=0.6839 04 SSS(3)=.2276 05 SS(21,3)=0.1592 05	SS(11,3)=0.6307 04 SSS(3)=.2014 05 SS(21,3)=0.1383 05
SUNST(4) = 0.1950 05	

S_{11} = 2.62 ft. S_{21} = 3.55 ft. S_c = 4.89 ft. S_h = 4.99 ft.

SUNST(i) - total sum of squares using the optimum parameter values of the (i-1)th calibration cycle.

*simulation resulting in minimum total sum of squares for a calibration cycle.

**minimum total sum of squares of this unsteady state calibration.

Table 26. Comparison of Unsteady State Calibrations, Test 5 and Test 6

Test 5 Calibration without steady state deviations added	Test 6 Calibration with the steady state deviations added
August 31 SS(11,1) S_{11} Initial 0.2090 05 4.57 ft. Final 0.9046 05 3.01 ft. Decrease = + 1.56 ft.	August 31 SS(11,1) S_{11} Initial 0.1761 05 4.19 ft. Final 0.6865 05 2.62 ft. Decrease = +1.57 ft.
March 31 SS(21,1) S_{21} Initial 0.1539 05 3.92 ft. Final 0.1680 05 4.09 ft. Increase = − 0.18 ft.	March 31 SS(21,1) S_{21} Initial 0.1198 05 3.46 ft. Final 0.1263 05 3.55 ft. Increase = -0.09 ft.
Total percentage of node points not adjusted because of sign difference in calculated and historic deviation Initial = 47% Final = 38% Decrease = 9%	Total percentage of node points not adjusted because of sign difference in calculated and historic deviation Initial = 50% Final = 36% Decrease = 14%

April 1, the August 30 and the average 1966 water table are very minimal for a large part of the aquifer so that very little improvement can be achieved in T-values that were optimal for the steady state calibration. This means that part of the 3.55 foot standard deviation resulting from the steady state calibration cannot be removed in the unsteady state calibration and will be reflected in the standard deviation determined using the time varying response of the water table. Test 5 shows this clearly since even with the greater deviations the T-values could not be improved to the extent that they eliminate the deviations from the steady state calibration. Since the same decrease in SS was achieved for both tests the question arises as to which optimum T and S values should be used.

For those node points that have opposite signs for the calculated and historical head differences occurring between August and April the S-value will be kept constant at the initial value. In the first iteration for Test 5, the initially unadjusted S-values cover 47% of the aquifer (Table 26). This decreased to 38% in the final iteration. For Test 6 the decrease was from 50% to 36%. The result for Test 6 is slightly better than Test 5. A closer look at the S-distribution reveals that the locations of the unadjusted S-values in Test 6 differ from those of Test 5. This is caused by the superposition of the steady state deviations upon the calculated deviations resulting from input. The result is that the direction of the deviation is not a function of the input. The locations of areas where differences in the sign of the deviations occur are arbitrary and not dependent on the unsteady state behavior of the aquifer. Therefore the optimum T and S values of Test 6 were considered as the final values. In this test, the steady state deviations were eliminated by adding them to the historic water table values and the storage coefficients are adjusted based upon calculated differences, which are a function of the magnitudes of the inputs per node and not obscured by the deviations remaining from the steady state calibration. For Test 6 the final total number of nodes with unadjusted S-values amounted to 36% which is a considerable part of the aquifer. For that part of the aquifer the historic water table values could not be defined close enough to permit a correct calibration, mostly occurring in that part of the aquifer where the well level differences between the two dates are small and the number of wells are scarce.

Mass Balance Unsteady State Calibration

In the unsteady state calibration the T-values and S-values were adjusted. In comparing the final transmissibility maps from the steady and unsteady state simulation the differences are slight for the majority of the aquifer. The largest changes occurred in irrigated areas where

T-values were refined to more closely simulate the unsteady state behavior of the aquifer.

Calculated flow from the Thousand Springs area for the unsteady state calibration was 4.94 million ac ft where the same flow for steady state calibration was 4.87 million ac ft. The Minidoka-Neeley reach shows a calculated inflow to the Snake River of 1800 ac ft. The historic reach gain from the north in the Neeley-Blackfoot reach for the April 1, 1966 - March 31, 1967 year is 1,085,000 ac ft while the calculated flow is 1,257,514 ac ft, a difference of 14%. The terms that make up the mass balance equation are:

Q_h = flow cross hydraulic boundaries = 6,194,400 ac ft
ΔS = storage depletion of the aquifer = 544,800 ac ft
Q = total external input to the aquifer = - 5,603,100 ac ft

The mass balance equation is

$$Q + Q_h - \Delta S = - 5{,}603{,}100 + 6{,}194{,}400 - 544{,}900 = 46{,}400 \text{ ac ft}$$

which means that the total outflow from the aquifer exceeded the total input by 46,400 cu ft or 0.8%.

Lack of data and incorrectly estimated historic water table rises close to American Falls reservoir may have led to an over adjustment of some T-values as is indicated by the 14% difference between calculated and historical flow for that reach. For two stream nodes along the Neeley to Blackfoot reach, (42,17) and (43,18), and the two inside nodes (42,18) and (43,19), the T-values were adjusted to maintain the same average weighted T-values and to similate the historic flow more precisely without changing the mass balance or the general flow pattern in the aquifer. The T-values for these four nodes were adjusted an average of 16%. Table 27 gives the results of this adjustment. Since the T-values were manually adjusted for only two stream nodes most of the nodal flows remained the same. For the Neeley-Blackfoot reach the adjusted total flow is 1,090,000 ac ft which is within 0.5% of the historically calculated flow. The mass balance equation with the new figures is

Table 27. Unsteady State Flows Across Hydraulically Connected Boundaries

Flow with T-values Resulting from Unsteady State Calibration			Flow with readjusted T-values	
	Node	Flow (ac ft)	Flow (ac ft)	
Thousand	(2,11) =	31049	31046	
	(2,12) =	44710	44669	
	(2,13) =	494543	494605	
	(3,10) =	1107755	1107725	
Springs	(4, 9) =	763845	763793	
	(5, 9) =	1168004	1168115	
	(6, 9) =	601918	601894	
	(7, 8) =	135170	135090	
Area	(8, 8) =	267789	267758	
	(9, 7) =	149406	149403	
	(10, 7) =	16665	16666	
	(11, 7) =	157776	157786	
Subtotal		4938600	4938600	
Minidoka	(26, 8) =	-43138	-43165	
	(27, 8) =	-15665	-15678	
	(28, 8) =	-20170	-20162	
	(29, 7) =	-8	-7	
to	(30, 7) =	285	285	
	(31, 7) =	574	569	
	(32, 7) =	4743	4691	
	(33, 7) =	15357	15397	
Neeley	(34, 8) =	36092	36022	
	(35, 9) =	8441	8630	
	(36,10) =	11689	11669	
Subtotal		-1800	-1750	
Neeley	(36,11) =	-8279	-8278	
	(37,12) =	35769	35769	
	(37,13) =	4637	4664	
	(38,14) =	7030	7034	
to	(38,15) =	31962	31983	
	(39,16) =	51109	51216	
	(40,16) =	56546	56712	
	(41,16) =	110830	111369*	
Blackfoot	(42,17) =	657333	569617*	
	(43,18) =	310587	229915	1085000[1]
Subtotal		1257514	1090000	
Total Outflow (Q_h)		6194400	6026800	
Total Inflow (Q)		-5603100	-5603100	
ΔS		-544900	-531000	
Error in Balance		46400 (0.7%)	-107300 (1.7%)	

*Stream nodes for which the transmissibility is manually adjusted.

1/Total for that reach as estimated by the Idaho Water Resource Board.

$$Q + Q_h - \Delta S = - 5{,}603{,}100 + 6{,}026{,}800 - 531{,}000 = - 107{,}300 \text{ ac ft}$$

indicating that the total outflow from the aquifer was 107,300 or 1.9% less than the total input + the storage depletion of the aquifer which is a satisfactory result. Even though the manual change of T-values on these specific nodes increased the mass balance error, it was justified because of the closer simulation of the historical reach gains. The change in T-values did not change the total sum of squares as calculated in the unsteady state calibration. The term S amounted to 531,000 ac ft, indicating a storage loss of water in the aquifer during the 1966 year. For the unsteady state simulation the historical March 31, 1967 head values were assumed to be equal to the April 1, 1966 water table, however the calculated head values for March 31, 1967 indicated an average decline of the water table of 0.88 feet since April 1, 1966. With the loss of 531,000 ac ft and an average decline of 0.88 feet over the 5,714,312 acres of the modeled area, an average storage coefficient of $S = 0.106$ can be calculated.

The standard deviation from historical head values for August 30 is 2.63 feet while the standard deviation for March 31 is 3.55 feet. For the historical March 31, 1967 head values the April 1, 1966 head values were used. For all the 93 observation wells in the modeled area the measured March 31, 1967 levels were compared with the April 1, 1966 levels. The average historical decline of the water table over the 93 wells amounted to 0.65 feet indicating that the aquifer indeed lost water during this period. The observation wells are not spread evenly over the aquifer so that the 0.65 foot decline cannot be regarded as the average historic decline in the entire aquifer. The results confirm qualitatively the results found in the calibration. If the actual March 31, 1967 water table had been used as reference level to calculate the deviations from historical head values the standard deviation for March 31 would be less than 3.55 feet. This means that the simulation was actually closer than indicated by the standard deviation.

A revision of the historic March 31, 1967 water table would not have any influence on the actual calibration of the parameters since they were adjusted based on the deviations occurring at August 30 half time-step. Table 28 shows the final transmissibility values resulting from the unsteady state multi-parameter calibration while Table 29 shows the final storage coefficients resulting from this calibration.

Table 28. Final Transmissibility Values
Unsteady State Multi-Parameter Calibration. 1000 ft^2/day.

40	0	0	0	0	0	0	0	0	0	0	0	0	0
39	0	0	0	0	0	0	0	0	0	0	0	0	0
38	0	0	0	0	0	0	0	0	0	0	0	0	0
37	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0
35	0	0	0	0	0	0	0	0	0	0	0	0	0
34	0	0	0	0	0	0	0	0	0	0	0	0	0
33	0	0	0	0	0	0	0	0	0	0	0	0	0
32	0	0	0	0	0	0	0	0	0	0	0	0	0
31	0	0	0	0	0	0	0	0	0	0	0	0	0
30	0	0	0	0	0	0	0	0	0	0	0	0	0
29	0	0	0	0	0	0	0	0	0	0	0	0	0
28	0	0	0	0	0	0	0	0	0	0	0	0	0
27	0	0	0	0	0	0	0	0	0	0	0	0	0
26	0	0	0	0	0	0	0	0	0	0	0	0	0
25	0	0	0	0	0	0	0	0	0	0	0	0	0
24	0	0	0	0	0	0	0	0	0	0	0	0	0
23	0	0	0	0	0	0	0	0	0	0	0	0	0
22	0	0	0	0	0	0	0	0	0	0	0	0	0
21	0	0	0	0	0	0	0	0	0	0	0	0	1672
20	0	0	0	0	0	0	0	0	0	0	0	908	232
19	0	0	0	0	0	0	0	0	0	0	212	208	58
18	0	0	0	0	0	0	0	0	0	0	35	172	101
17	0	0	0	0	0	0	0	0	0	0	38	159	278
16	0	0	0	9	15	57	243	61	73	169	248	83	368
15	0	0	22	62	44	156	318	120	423	254	525	566	1103
14	0	308	62	345	339	376	72	274	267	375	761	1544	272
13	1493	1231	154	292	334	292	137	450	147	617	475	585	77
12	133	177	120	134	318	267	460	639	284	820	202	615	277
11	89	377	885	1164	1476	1201	1593	732	630	361	320	451	1261
10	0	2925	661	740	2028	1045	378	727	513	118	210	480	941
9	0	0	1606	5018	1619	488	883	939	1713	631	1572	687	222
J 8	0	0	0	0	0	347	532	606	539	1392	905	1085	477
↑ 7	0	0	0	0	0	0	0	370	311	616	406	317	746
6	0	0	0	0	0	0	0	0	0	0	150	138	445
5	0	0	0	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0	0	0	0
	2	3	4	5	6	7	8	9	10	11	12	13	14

→ I

Table 28.-Continued

40	0	0	0	0	0	0	0	0	0	0	0	0	0
39	0	0	0	0	0	0	0	0	0	0	0	0	0
38	0	0	0	0	0	0	0	0	0	0	0	0	0
37	0	0	0	0	0	0	0	0	0	0	0	0	0
36	0	0	0	0	0	0	0	0	0	0	0	0	0
35	0	0	0	0	0	0	0	0	0	0	0	0	0
34	0	0	0	0	0	0	0	0	0	0	0	0	0
33	0	0	0	0	0	0	0	0	0	0	0	0	0
32	0	0	0	0	0	0	0	0	0	0	0	0	0
31	0	0	0	0	0	0	0	0	0	0	0	0	61
30	0	0	0	0	0	0	0	0	0	0	0	0	42
29	0	0	0	0	0	0	0	0	0	0	0	149	145
28	0	0	0	0	0	0	0	0	0	0	9	44	67
27	0	0	0	0	0	0	0	0	0	0	21	66	36
26	0	0	0	0	0	0	0	0	0	0	18	50	75
25	0	0	0	0	0	0	0	0	0	2	5	22	33
24	0	0	0	0	845	252	224	79	16	7	29	35	31
23	0	0	0	0	3184	636	618	702	72	36	26	29	34
22	0	0	2035	6664	3442	666	3046	1215	314	74	141	204	377
21	13790	6009	9175	1449	2286	1784	2832	2032	4170	486	448	719	1034
20	880	412	940	272	943	2401	4293	3712	10931	3383	5271	3780	4371
19	113	93	568	977	10373	3241	14475	9757	9284	2461	2383	1558	2821
18	637	325	467	1117	2610	12141	3862	7210	7574	3121	3883	2547	5096
17	620	482	2239	940	1268	6758	9397	2400	4244	1884	1915	2312	2503
16	2632	2913	1257	1808	1032	1023	1338	1337	2086	2288	1177	2831	1493
15	13712	4731	1809	1863	1868	1374	1935	4867	1521	4911	1526	5177	1057
14	1120	4491	1664	1116	1418	1317	1560	2608	1753	2635	3377	3648	1569
13	162	459	2344	1820	822	1375	1309	1332	1835	1921	2289	2048	1636
12	724	533	913	546	654	351	722	713	1407	504	1521	525	1117
11	771	1333	937	343	496	186	900	1765	1867	947	390	292	90
10	776	1390	897	825	316	515	669	2798	2359	5805	1813	1985	130
9	1209	1514	605	210	474	96	219	379	491	913	5831	492	52
J 8	2464	833	193	224	112	53	80	121	75	93	375	99	25
7	2122	290	414	206	366	242	193	197	147	20	50	332	0
6	1864	567	1066	359	547	220	290	142	164	30	43	133	122
5	273	267	709	364	254	168	55	47	83	39	55	55	27
4	0	0	0	0	0	0	0	0	46	193	56	41	0
3	0	0	0	0	0	0	0	0	0	100	21	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0	0	0	0
	15	16	17	18	19	20	21	22	23	24	25	26	27

I

Table 28.- Continued

J ↑

40	0	0	0	0	0	0	0	0	0	0	0	0	0
39	0	0	0	0	0	0	0	0	0	0	0	0	0
38	0	0	0	0	0	0	0	0	0	0	0	0	0
37	0	0	0	0	0	0	0	0	0	0	0	0	465
36	0	0	0	0	0	0	0	0	0	0	0	1383	3953
35	0	0	0	0	0	0	0	0	0	0	2457	2963	1128
34	0	0	0	0	0	30	20	29	0	0	3065	663	1123
33	0	0	0	0	0	222	126	82	18	0	283	346	433
32	0	0	0	0	0	0	42	18	5	7	54	362	857
31	27	31	0	0	0	0	174	101	27	9	45	133	243
30	186	168	0	0	0	0	514	300	70	21	186	638	1557
29	277	414	144	76	19	98	1123	202	28	50	567	2511	881
28	36	51	131	41	10	51	448	472	61	248	1831	449	348
27	71	81	74	23	34	260	2230	304	110	1106	444	440	2120
26	66	84	133	204	205	595	953	365	114.	550	1856	1645	1680
25	70	80	63	113	544	403	1222	179	400	2233	1446	1210	4946
24	59	80	87	58	71	253	1463	439	1538	7332	1607	4506	4430
23	70	168	61	112	62	149	1015	2174	3542	3982	1204	4721	4870
22	768	985	180	237	88	243	2233	746	2980	5177	2121	6760	3136
21	2555	831	205	81	305	79	333	1165	5323	3051	1189	4917	10385
20	1621	1390	575	236	400	121	523	6472	1268	3110	11915	18354	4751
19	1369	1897	399	239	277	275	265	1052	914	7593	20429	5254	2239
18	2703	2005	487	135	342	154	243	423	3902	25736	8768	1125	5465
17	2684	1355	559	121	381	220	329	512	8861	32379	8180	502	493
16	2079	645	743	135	436	218	507	305	3387	2320	577	116	175
15	1542	338	698	171	345	217	467	228	2248	756	98	0	0
14	963	196	666	227	328	301	465	280	1017	215	33	0	0
13	177	158	182	133	74	195	67	201	1054	186	0	0	0
12	125	92	114	148	131	138	245	1359	298	1330	0	0	0
11	19	15	38	59	151	105	886	677	102	0	0	0	0
10	14	39	14	68	28	99	128	46	26	0	0	0	0
9	11	6	12	48	90	22	16	11	0	0	0	0	0
8	76	9	36	7	17	88	65	0	0	0	0	0	0
7	0	1	6	3	18	77	0	0	0	0	0	0	0
6	0	0	0	0	0	0	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0	0	0	0
	28	29	30	31	32	33	34	35	36	37	38	39	40

→ I

Table 28.-Continued

J	41	42	43	44	45	46	47	48	49	50	51	52	53	54
40	0	0	0	0	0	0	0	0	0	0	0	0	0	0
39	0	100	55	342	170	177	87	0	0	0	0	0	0	0
38	51	342	333	1391	1520	1194	459	147	0	0	0	0	0	0
37	69	129	430	3196	1756	257	1499	329	550	409	0	0	0	0
36	246	33	47	239	265	399	1613	688	1430	1189	214	0	0	0
35	87	11	18	21	20	61	383	338	2612	1938	210	0	0	0
34	118	13	27	25	23	134	487	365	1131	1612	167	0	0	0
33	152	14	42	20	76	149	186	662	1778	636	260	0	0	0
32	100	19	25	136	796	516	130	563	576	476	323	0	0	0
31	281	91	159	1376	14067	7029	543	342	583	144	623	211	272	0
30	610	340	1289	4120	19608	12137	945	550	1072	144	612	427	258	114
29	233	1091	6218	16086	11143	6489	2470	364	1255	372	113	302	94	270
28	1098	5736	9838	6707	4594	1948	2298	691	225	725	134	666	225	0
27	4201	4595	6437	9005	2893	4542	1851	1039	344	772	475	322	0	0
26	4298	5675	3962	3995	3764	1838	1835	974	1078	563	155	186	0	0
25	4038	2455	2941	4315	4968	8314	6956	3012	2250	451	294	72	0	0
24	4848	3345	3198	6086	7649	2528	3580	778	509	250	235	0	0	0
23	3179	3366	5298	2209	1462	766	760	264	41	35	0	0	0	0
22	15667	26956	8808	1193	363	999	939	93	39	0	0	0	0	0
21	9999	29529	24157	1591	195	452	88	15	5	0	0	0	0	0
20	5395	58714	7062	2667	222	68	35	4	6	0	0	0	0	0
19	14890	5909	4993	559	279	123	28	11	0	0	0	0	0	0
18	2910	20045	4609	0	0	0	175	57	0	0	0	0	0	0
17	1783	4245	0	0	0	0	0	0	0	0	0	0	0	0
16	414	0	0	0	0	0	0	0	0	0	0	0	0	0
15	0	0	0	0	0	0	0	0	0	0	0	0	0	0
14	0	0	0	0	0	0	0	0	0	0	0	0	0	0
13	0	0	0	0	0	0	0	0	0	0	0	0	0	0
12	0	0	0	0	0	0	0	0	0	0	0	0	0	0
11	0	0	0	0	0	0	0	0	0	0	0	0	0	0
10	0	0	0	0	0	0	0	0	0	0	0	0	0	0
9	0	0	0	0	0	0	0	0	0	0	0	0	0	0
8	0	0	0	0	0	0	0	0	0	0	0	0	0	0
7	0	0	0	0	0	0	0	0	0	0	0	0	0	0
6	0	0	0	0	0	0	0	0	0	0	0	0	0	0
5	0	0	0	0	0	0	0	0	0	0	0	0	0	0
4	0	0	0	0	0	0	0	0	0	0	0	0	0	0
3	0	0	0	0	0	0	0	0	0	0	0	0	0	0
2	0	0	0	0	0	0	0	0	0	0	0	0	0	0
1	0	0	0	0	0	0	0	0	0	0	0	0	0	0

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Table 29. Final Storage Coefficients Unsteady State Multi-Parameter Calibration - Dimensionless

J

40	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
38	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
37	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
36	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
35	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
34	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
33	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
32	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
31	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
30	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
29	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
28	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
27	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
26	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
25	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
24	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
23	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
22	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
21	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.10
20	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.05	0.03
19	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.05	0.06	0.03
18	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.04	0.05	0.04
17	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.06	0.10	0.05
16	0.0	0.0	0.0	0.0	0.13	0.16	0.11	0.05	0.20	0.10	0.19	0.20	0.09	0.05
15	0.0	0.0	0.0	0.08	0.20	0.13	0.16	0.05	0.20	0.05	0.07	0.05	0.05	0.07
14	0.0	0.0	0.10	0.06	0.16	0.20	0.10	0.20	0.10	0.19	0.07	0.07	0.05	0.10
13	0.0	0.10	0.19	0.05	0.20	0.09	0.10	0.20	0.20	0.10	0.05	0.10	0.20	0.13
12	0.0	0.10	0.04	0.06	0.04	0.05	0.04	0.15	0.13	0.11	0.20	0.07	0.16	0.08
11	0.0	0.10	0.06	0.07	0.10	0.11	0.03	0.20	0.05	0.07	0.10	0.10	0.11	0.08
10	0.0	0.0	0.10	0.10	0.07	0.08	0.10	0.07	0.05	0.11	0.10	0.04	0.20	0.16
9	0.0	0.0	0.0	0.10	0.10	0.10	0.04	0.03	0.17	0.10	0.19	0.20	0.07	0.05
8	0.0	0.0	0.0	0.0	0.0	0.0	0.10	0.10	0.13	0.20	0.14	0.06	0.09	0.04
7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.10	0.10	0.10	0.15	0.18	0.05
6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.20	0.20	0.20
5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	1	2	3	4	5	6	7	8	9	10	11	12	13	14

I

Table 29.–Continued

40	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
38	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
37	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
36	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
35	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
34	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
33	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
32	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.05	0.12
31	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.04	0.17
30	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.05	0.04	0.04
29	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.04	0.07	0.07	0.04
28	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.05	0.07	0.10	0.07
27	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.10	0.10	0.10	0.10
26	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.10	0.10	0.10	0.10	0.10
25	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.10	0.10	0.10	0.10	0.10
24	0.0	0.0	0.0	0.0	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10
23	0.0	0.0	0.0	0.0	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.09	0.10
22	0.0	0.0	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.20
21	0.10	0.05	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.07	0.10	0.10	0.10	0.20
20	0.07	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.07	0.08	0.04	0.04	0.13	0.20
19	0.04	0.03	0.10	0.10	0.20	0.20	0.20	0.20	0.20	0.20	0.10	0.08	0.20	0.20
18	0.04	0.04	0.10	0.10	0.20	0.20	0.15	0.07	0.04	0.14	0.11	0.10	0.09	0.20
17	0.09	0.05	0.10	0.10	0.06	0.17	0.10	0.05	0.05	0.15	0.05	0.05	0.10	0.10
16	0.15	0.17	0.11	0.10	0.12	0.07	0.07	0.07	0.04	0.16	0.06	0.09	0.14	0.10
15	0.06	0.17	0.17	0.10	0.11	0.09	0.07	0.07	0.08	0.12	0.09	0.07	0.06	0.10
14	0.05	0.17	0.17	0.13	0.05	0.07	0.04	0.05	0.04	0.06	0.04	0.03	0.05	0.10
13	0.11	0.10	0.10	0.10	0.07	0.07	0.07	0.05	0.06	0.09	0.04	0.10	0.10	0.10
12	0.07	0.10	0.10	0.20	0.08	0.05	0.10	0.06	0.11	0.17	0.08	0.04	0.05	0.10
11	0.11	0.10	0.10	0.10	0.10	0.07	0.04	0.09	0.08	0.10	0.12	0.09	0.05	0.10
10	0.20	0.10	0.10	0.07	0.10	0.09	0:07	0.09	0.09	0.07	0.09	0.05	0.16	0.10
9	0.14	0.20	0.10	0.12	0.04	0.07	0.05	0.15	0.08	0.05	0.06	0.10	0.17	0.20
J 8	0.20	0.15	0.10	0.16	0.06	0.07	0.10	0.14	0.10	0.10	0.13	0.10	0.10	0.10
7	0.10	0.07	0.10	0.10	0.10	0.10	0.10	0.10	0.20	0.20	0.05	0.05	0.0	0.0
6	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.10	0.20	0.13	0.03	0.05	0.05	0.0
5	0.07	0.07	0.10	0.10	0.10	0.10	0.10	0.10	0.20	0.05	0.04	0.10	0.10	0.0
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.06	0.04	0.07	0.10	0.0	0.0
3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.07	0.10	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	15	16	17	18	19	20	21	22	23	24	25	26	27	28

→ I

Table 29.– Continued

J ↑

40	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.20
38	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.20	0.11
37	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.17	0.20	0.20
36	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.10	0.06	0.14	0.14
35	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.10	0.10	0.14	0.12	0.13
34	0.0	0.0	0.0	0.0	0.09	0.11	0.12	0.0	0.0	0.10	0.07	0.08	0.20	0.20
33	0.0	0.0	0.0	0.0	0.04	0.14	0.20	0.13	0.0	0.07	0.10	0.10	0.10	0.10
32	0.0	0.0	0.0	0.0	0.0	0.05	0.06	0.04	0.03	0.05	0.10	0.08	0.20	0.05
31	0.11	0.0	0.0	0.0	0.0	0.03	0.07	0.03	0.03	0.05	0.10	0.10	0.03	0.10
30	0.16	0.0	0.0	0.0	0.0	0.10	0.16	0.10	0.05	0.10	0.10	0.10	0.10	0.10
29	0.11	0.15	0.04	0.10	0.10	0.05	0.05	0.05	0.10	0.16	0.05	0.10	0.10	0.10
28	0.04	0.05	0.05	0.10	0.04	0.04	0.03	0.10	0.10	0.14	0.05	0.10	0.10	0.05
27	0.08	0.08	0.18	0.11	0.10	0.06	0.04	0.10	0.10	0.06	0.14	0.05	0.05	0.05
26	0.10	0.10	0.04	0.10	0.07	0.13	0.06	0.10	0.10	0.12	0.15	0.11	0.10	0.05
25	0.10	0.10	0.10	0.04	0.04	0.05	0.05	0.10	0.09	0.09	0.10	0.06	0.15	0.09
24	0.10	0.05	0.05	0.05	0.03	0.05	0.10	0.10	0.04	0.10	0.14	0.05	0.10	0.06
23	0.15	0.04	0.04	0.05	0.05	0.10	0.07	0.04	0.06	0.07	0.07	0.10	0.10	0.20
22	0.08	0.05	0.05	0.04	0.07	0.10	0.07	0.04	0.07	0.10	0.10	0.10	0.10	0.16
21	0.09	0.08	0.13	0.07	0.05	0.10	0.03	0.10	0.13	0.20	0.20	0.20	0.07	0.07
20	0.10	0.07	0.10	0.20	0.17	0.11	0.10	0.20	0.10	0.19	0.20	0.16	0.10	0.07
19	0.10	0.17	0.10	0.10	0.10	0.20	0.03	0.11	0.10	0.13	0.20	0.08	0.10	0.07
18	0.10	0.20	0.10	0.10	0.08	0.10	0.10	0.09	0.10	0.20	0.05	0.10	0.10	0.07
17	0.10	0.20	0.18	0.10	0.20	0.10	0.10	0.08	0.10	0.05	0.04	0.08	0.05	0.10
16	0.10	0.20	0.09	0.10	0.20	0.10	0.10	0.10	0.07	0.03	0.10	0.10	0.10	0.0
15	0.10	0.11	0.05	0.10	0.20	0.10	0.10	0.10	0.06	0.10	0.0	0.0	0.0	0.0
14	0.10	0.07	0.03	0.03	0.17	0.08	0.20	0.06	0.03	0.10	0.0	0.0	0.0	0.0
13	0.10	0.10	0.07	0.07	0.10	0.05	0.20	0.20	0.10	0.0	0.0	0.0	0.0	0.0
12	0.10	0.10	0.10	0.05	0.12	0.10	0.04	0.10	0.10	0.0	0.0	0.0	0.0	0.0
11	0.07	0.10	0.10	0.10	0.07	0.10	0.13	0.03	0.0	0.0	0.0	0.0	0.0	0.0
10	0.12	0.12	0.10	0.07	0.07	0.05	0.10	0.03	0.0	0.0	0.0	0.0	0.0	0.0
9	0.03	0.13	0.10	0.07	0.07	0.05	0.10	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	0.10	0.10	0.06	0.20	0.20	0.10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	0.10	0.10	0.10	0.10	0.10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	29	30	31	32	33	34	35	36	37	38	39	40	41	42

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Table 29.—Continued

40	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
39	0.20	0.09	0.08	0.07	0.10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
38	0.09	0.10	0.10	0.10	0.10	0.10	0.0	0.0	0.0	0.0	0.0	0.0	0.0
37	0.10	0.10	0.10	0.10	0.20	0.10	0.10	0.20	0.0	0.0	0.0	0.0	0.0
36	0.06	0.10	0.07	0.10	0.10	0.10	0.10	0.20	0.20	0.0	0.0	0.0	0.0
35	0.07	0.07	0.20	0.10	0.13	0.10	0.10	0.20	0.20	0.0	0.0	0.0	0.0
34	0.10	0.09	0.15	0.20	0.10	0.10	0.10	0.10	0.20	0.0	0.0	0.0	0.0
33	0.07	0.20	0.20	0.08	0.06	0.10	0.10	0.10	0.20	0.0	0.0	0.0	0.0
32	0.04	0.10	0.07	0.04	0.20	0.10	0.10	0.20	0.14	0.0	0.0	0.0	0.0
31	0.10	0.07	0.10	0.10	0.10	0.10	0.20	0.05	0.14	0.14	0.09	0.0	0.0
30	0.10	0.10	0.10	0.10	0.10	0.10	0.07	0.08	0.13	0.13	0.10	0.06	0.0
29	0.10	0.10	0.10	0.10	0.20	0.06	0.05	0.07	0.15	0.11	0.09	0.09	0.0
28	0.10	0.10	0.10	0.10	0.20	0.20	0.08	0.06	0.10	0.09	0.07	0.0	0.0
27	0.07	0.07	0.03	0.04	0.04	0.20	0.05	0.05	0.08	0.06	0.0	0.0	0.0
26	0.05	0.10	0.10	0.10	0.15	0.19	0.04	0.07	0.08	0.04	0.0	0.0	0.0
25	0.11	0.11	0.07	0.13	0.09	0.09	0.04	0.06	0.07	0.04	0.0	0.0	0.0
24	0.14	0.20	0.10	0.09	0.10	0.07	0.05	0.08	0.06	0.0	0.0	0.0	0.0
23	0.20	0.10	0.06	0.11	0.05	0.07	0.11	0.04	0.0	0.0	0.0	0.0	0.0
22	0.10	0.10	0.10	0.14	0.09	0.10	0.05	0.0	0.0	0.0	0.0	0.0	0.0
21	0.10	0.10	0.12	0.18	0.19	0.10	0.05	0.0	0.0	0.0	0.0	0.0	0.0
20	0.07	0.06	0.20	0.17	0.16	0.06	0.04	0.0	0.0	0.0	0.0	0.0	0.0
19	0.06	0.20	0.18	0.20	0.19	0.20	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	0.10	0.0	0.0	0.0	0.10	0.20	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
J 8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
	43	44	45	46	47	48	49	50	51	52	53	54	55

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CHAPTER VII

SUMMARY AND CONCLUSIONS

Investigations of water resources systems involving groundwater simulation are generally data scarce projects. Insufficient geohydrologic data severely inhibits simulation of historical water table behavior.

In this study, a basic finite difference groundwater model developed in 1972 in a M.S. thesis by the author was expanded to include a parameter calibration routine. This routine automatically adjusts geohydrological parameters using historic aquifer response. The model and calibration routine as initially developed were applied to the Rigby-Ririe area which has a history of high water table or sub-water conditions caused by excessive input to the gravel aquifer. The calibration routine was effective in systematically adjusting geohydrologic parameters to fit historic water table responses of this area. Simulation of maximum water table rises of 35 to 50 feet for 1972 were achieved with a standard deviation of 1.25 feet between the calculated and historic water table elevations. A series of management studies were run on the aquifer and showed the model to be flexible in evaluating aquifer responses.

The results and experience obtained from the application to the Rigby area were used to improve the basic model and the calibration routine to more efficiently adjust the geohydrological parameters. Transmissibility values are adjusted based on the ratio of historic and calculated hydraulic gradients, an approximate method in which the uniqueness of the resulting parameter distribution is dependent on the presence in the modeled aquifer of withdrawal or recharge areas. The remaining geohydrological parameters, leakance factor, initial head difference between main and secondary aquifer, and storage coefficient, are adjusted based on the ratio of historic and calculated rise or decline in head values since the start of simulation. The basic model was expanded to include procedures to simulate open drains in the aquifer and the flow boundary was simplified to a constant gradient boundary with variable head. The model also calculates all the terms, summed over all node points, that make up the mass balance for the aquifer. This provides an additional check of the aquifer system important in evaluating the significance of the results and historic data.

The calibration routine is very efficient in adjusting the transmissibility values in the aquifer as shown in several tests. Using a single control card, one parameter or any combination of parameters in any sequence, for a steady or unsteady state simulation may be adjusted.

The model was applied to the Snake Plain aquifer, a large regional water table aquifer in southern Idaho covering about 9,000 square miles. Using the 1966 input data and the 1966 average water table elevation, a steady state calibration was run in which the transmissibility values were adjusted. The initial standard deviation resulting from the initial T-estimates was 58 feet which was reduced to 3.55 during the adjustment. The final T-distributions resulting from different estimates of initial T-values were repeatable to within 5% accuracy. The results of the calibration trials showed that it is necessary for the model to accurately simulate the properties of the flow system of the aquifer boundaries. For instance, selection of a constant head boundary to represent a fixed head impermeable boundary will impede the correct operation of the adjustment routine. Since the 1966 average water table elevations represent an arbitrary equilibrium condition, the T-distribution resulting from the steady state calibration cannot be used as the final distribution.

In the unsteady state calibration utilizing the seasonal fluctuations of the water table caused by irrigation application, the T-values and S-values were simultaneously adjusted. The calibration program again proved to be effective in reducing the standard deviations. It appeared that the calibration of T was more effective in reducing the sum of squares of deviations than the calibration of S. The reason for this lies in the method of adjusting T. T is adjusted for all nodes in the aquifer using the hydraulic gradients. The storage coefficient at each node is adjusted only if the historic and calculated differences in head values since the start of the simulation are of same sign. Because lack of accurate water table data this was the case for only 64% of the node points. For the other 36%, the calculated head differences were not consistent with the historic head differences and did not contribute to a decrease in standard deviation. Since the storage coefficients are adjusted in this particular manner a steady state calibration to adjust the T-values should proceed an unsteady state calibration. Use of the steady state adjusted T-distribution increases the occurrence of compatible historic and calculated head differences so that the initial adjustment of S in the unsteady state calibration is more efficient.

The unsteady state calibration resulted in a reduction of the standard deviation at the maximum rise (August 31, 1966) from 4.19 feet to 2.62 feet, (37%). The standard deviation for the timestep at the end of the simulation period, March 31, 1967, remained constant. The final T-values and S-values resulted in an average calculated August 31 water table 0.3 foot higher than the historic levels, while the calculated March 31, 1967 levels were 0.88 foot lower than the April 1, 1966 head levels. Historic well records show an average decrease of 0.65 foot between April 1, 1966 and March 31, 1967.

The groundwater flows from the Snake Plain aquifer mainly via two groups of springs, the first group located northeast of American Falls Reservoir, the second group known as the Thousand Springs area, located at the southwestern boundary of the aquifer. Both boundaries were simulated as constant head boundaries. Transmissibility values of these boundaries needed only minor adjustments to simulate historic spring flows.

The calibration routine allows the use of the model on aquifers where data on geohydrological parameters is limited. However it shows the need for accurate historic water level records as necessary data for calibration.

External inputs to nodes in the aquifer are not adjusted in the calibration procedure. Usually, the input consists of a number of terms that make the adjustment of a lump sum of inputs unreasonable.

The results of the calibration procedure are also very effective in indicating those parts of the aquifer where more water table data are needed or where inputs must be revised. The strength of the model and the calibration method developed lies in the fact that it is not limited to use on largely hypothetical aquifers in which the fluctuation in response to a pumped well or to a flood wave are used as observations to determine aquifer parameters. The calibration routine is an iterative technique that has passed the stage of laboratory tests with experimental data. Its validity is proven by the effective application to two aquifers in Idaho.

Meeting the first three objectives culminated in the development of a simulation tool consisting of an input program, a parameter identification routine and a groundwater model that effectively simulates historic behavior of aquifers. A major change in the modeling technique and calibration procedure probably will not be needed until this is necessitated by the experiences obtained from applying the model to other aquifers.

The final objective, to develop an operational model of the Snake Plain aquifer, give insight into the problems to be encountered in modeling and most recommendations are therefore related to possible ways to improve the parameter identification on the Snake Plain.

As noted earlier, availability of accurate water table data is most pertinent to the proper adjustment of parameter values. In the Snake Plain a fairly large number of wells are present, but the majority of them are situated in a relatively narrow band along the Snake River where irrigation is practiced. The remainder of the aquifer, especially the area along the northern boundaries, includes a very limited number of wells making it difficult to sufficiently define the water table. Although this area is mostly not suitable for irrigation, it may have importance as an area for artificial recharge since this part of the plain is not adjacent to hydraulic boundaries. It would be beneficial to have additional wells in this area.

In the steady state T-calibration of the Snake Plain aquifer, the external inputs to the system served as a boundary condition to insure a unique T-distribution. The input is composed of several terms, some of which are based on crude estimates for lack of better data. Because the magnitude of the inputs plays an important role in the calibration, updating of inputs is necessary, especially for the tributary valley underflow, precipitation on non-cropped areas of the plain and the off season evapotranspiration or evaporation occurring in the plain.

The groundwater flow from the tributary basins is presently known on an annual basis. The fluctuations in flow from year to year as a result of a heavy or light snowpack is not known. The tributary underflow calculated from the yearly estimates accounts for 1,242,000 ac ft or 20% of the total input. Fluctuations in underflow may amount to hundreds of thousands of acre feet. In the unsteady state calibration, a one-year period was simulated in which the underflow was equally distributed over all timesteps. In order to more accurately simulate the seasonal response of the aquifer, the time distribution of the underflows needs to be known. Some techniques need to be investigated to arrive at more accurate yearly estimates and time distribution of the flows during the season.

There are some areas in the Snake Plain for which the total groundwater base flow and irrigation application is presented as a lump sum, distributed over a number of node points. These are the Raft River Basin, the Rigby-Ririe area, and the Henrys Fork region. The model boundary in the northeast part of the aquifer is located just west of the Henrys Fork. In order to account for the irrigation application east of this boundary, the total of base flow and irrigation diversions was moved inside the model boundaries and distributed over a number of nodes. Along the boundary the historic rise of the head values during the irrigation season ranges from 3 to 5 feet, however the calculated rise was much higher since the total input was distributed over few nodes, resulting in high storage coefficient values necessary to simulate the historic rise. High storage coefficients indicate areas in the model where the input is not distributed correctly. There are several areas on the Snake Plain with this problem, and in order to refine the calibration results, attempts should be undertaken to determine correct methods to distribute these flows.

Presently an estimated 650,000 ac ft of precipitation recharges the aquifer, of which 240,000 ac ft falls on non-cropped lands covering 75% of the modeled area. For the non-cropped areas, the precipitation is assumed to be 3% effective, a highly speculative figure that certainly would not be uniform over such a vast area. Some techniques need to be investigated to more accurately determine the percentage of effective precipitation in the respective regions of the plain.

A severe lack of data exists concerning the off season evapotranspiration for large areas of the Snake Plain. For this study the amounts were assumed to be negligible but there is some indication that this assumption is not correct and studies should be initiated to provide insight into the significance of this input.

In the calibration of the Snake Plain aquifer the net withdrawal from groundwater irrigated areas was assumed to be equal to the crop irrigation requirement of the growing season assuming that water pumped in excess of this amount would return to the aquifer in the same place and time. However, this is not true for areas that are recently developed or areas that have deep soil cover. The amount of water withdrawn is a deciding factor in the identification of aquifer parameters and some different approach to account for the groundwater withdrawal should be taken.

The calibration can be improved by further refinement of existing data. In the calibration, the March 31, 1967 hydraulic head elevations were assumed to be equal to the April 1, 1966 head elevations. Well records indicate a decrease in water table values so that a new March 31, 1967 water table map should be prepared.

If during the continuation of the research better data become available that indicate major changes in the input to the aquifer, the total calibration procedure should be repeated to allow for a balanced adjustment of the parameters over all nodes in the aquifer. The steady state calibration should be undertaken, after which the multi-parameter unsteady state calibration should be completed.

The steady state calibration was performed with the 1966 inputs repeated for every half timestep. Better results may be produced if the average inputs of the 1966-1973 period are used. This average input should define more accurately the present condition of the Snake Plain aquifer system from which reference head values may be obtained.

The hydraulic relationship between the aquifer and rivers flowing through the aquifer changes from node to node. The Snake River is hydraulically connected in certain reaches, and there the exchange of water between the aquifer and the river is calculated adequately. In other reaches, the contribution to the aquifer from the river takes place via unsaturated flow. The contribution from these reaches as well as the flows of the hydraulically connected reaches are calculated in the river operation model applied to the Snake River by the Idaho Water Resources Board. The flows calculated in the groundwater model for the hydraulically connected reaches as they vary with the management of the plain can be used to firm up those determined in the river model. A more accurate evaluation of the reach gains of the unsaturated flow reaches that serve as external input to the aquifer model is therefore possible.

Both the groundwater and the river operation models use mutual data and are dependent on generated results. It indicates that linking of the aquifer model and the river model to provide a complete planning tool for basin studies would be beneficial.

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APPENDICES

APPENDIX A

CONTROL VARIABLES AND INPUT DATA FOR THE INPUT PROGRAM

Control Variables of the Input Program

The control variables of the input program are related to the grid system, size, duration of simulation and the general character of the aquifer to be modeled. They are read in on one card:

NROW = number of node points along a row

NCOL = number of node points along a column

DELT = duration of the initial full time step (days)

DELX = length of mesh in the x-direction (along the row) [ft.]

DELY = length of mesh in the y-direction (along the column) [ft.]

LTS = serial number of the last half time step for which the input is calculated using the initial time step value. After half time step no. LTS the time step is doubled. Used in the unsteady simulation.

NTS = serial number of the last half time step of the simulation for which the input is calculated. In case of a steady state aquifer simulation where the input for each node does not vary in time only one half time step is calculated, since the input for every half time step is the same. In that case LTS = NTS = 2. (to be consistent with the model program) In case of an unsteady state aquifer simulation the input per node is time dependent. The simulation period is divided into half time steps, each for which the input Q (I,J) is calculated. (K = 2,3, ..., NTS) (NTS-1) half time steps are calculated. In that case NTS > 2 and LTS > 2. If for the unsteady state every half time step has the same length LTS = NTS.

NIRRI = total number of organized irrigation districts in the aquifer.

NRE = total number of stream reaches in the aquifer for which reach gains and losses are recorded.

NVAL = total number of tributary valleys (including the artificial valleys) entering the aquifer.

NCLI = total number of climatic regions in the aquifer.

NR = total number of regions in the aquifer with a different percentage of effective precipitation.

Inputs to the Input Program

The input data consisting of arrays and dimensioned variables can be separated into two data sets. The first data set is read in only once, regardless of the number of half time steps for which input Q (I,J) is calculated. The second data set is directly related to the type of simulation. These data involve the amount of the irrigation diversions, return flows, crop use, precipitation, artificial recharges, valley underflows, reach gains and losses and changes in irrigated acreage from which the input for one half time step is calculated. In case of steady state simulation only one half time step is calculated,therefore the second data set is read in only once. In case of unsteady state simulation the second data set is read in successively as many times as the number of half time steps in the simulation (NTS-1).

Input data of Data Set 1

The variables of Data Set 1 are entered in the following order:

NCX(I,J) = array specifying by numbers, for boundary nodes the type of boundary and for nodes not at the boundary the location (inside or outside the aquifer boundaries). Mainly used in the groundwater main program to properly calculate the hydraulic head values. Used in the input program in relation to the calculation of the sum total of the net input of nodes inside the aquifer boundaries.

NODAL(I,J) = array specifying the climatic region for node point (I,J). Every climatic region has a number. For every node point in one region, NODAL (I,J) is equal to the region number.

NEFF(I,J) = array specifying by a number the node points which have equal effective precipitation. The aquifer is divided into areas each with a specific percentage of effective precipitation. For every node point in one such area NEFF(I,J) is equal to the area number.

RECH(K) = variable specifying the percentage of precipitation that is effective for recharge for area no. K. For node points inside area no. K, NEFF(I,J) = K.

NIR(I,J,N) = irrigation district identification for the Nth district in node (I,J). There is a maximum of four different districts per node. $N^{max} = 4$. Each district within the model boundaries is uniquely defined by its identification number.

ARWET(I,J) = total wetted area of canals in node point (I,J)[ac]

SURFAR(I,J,N) = surface water irrigated area of the Nth irrigation district in node (I,J)[ac]

GRWAR(I,J,N) = ground water irrigated area of the Nth irrigation district in node (I,J)[ac]

SUREST(I,J) = lump sum of surface water irrigated area of non-organized lands for node point (I,J)[ac]

GRREST(I,J) = lump sum of ground water irrigated area of non-organized lands for node point (I,J)[ac]

FACTOR(I,J) = average seepage rate for all irrigation canals in node point (I,J) [ft/day]

For every node point one card is read in with the coordinates of the node point (I,J) and the values of the last seven arrays described above, as long as at least one of the nodal values of the seven arrays is non zero. The arrays NIR(I,J,N), SURFAR(I,J,N), GRWAR(I,J,N), SUREST(I,J) and GRREST(I,J) are related as follows: Node point (I,J) may have a maximum of four organized districts, irrigation district No. K, L, M and N. The data for node point (I,J)are entered in four groups on one card, as shown in Table 30 by Group I, II, III and IV respectively.

Table 30. Irrigation Information for Node Point (I,J)

I	II	III	IV
NIR(I,J,1) = K	SURFAR(I,J,1) = A	GRWAR(I,J,1) = E	SUREST(I,J) = X
NIR(I,J,2) = L	SURFAR(I,J,2) = B	GRWAR(I,J,2) = F	GRREST(I,J) = Y
NIR(I,J,3) = M	SURFAR(I,J,3) = C	GRWAR(I,J,3) = G	
NIR(I,J,4) = N	SURFAR(I,J,4) = D	GRWAR(I,J,4) = H	

'A' represents the total surface water irrigated acreage for irrigation district K. If irrigation district No. K has ground water irrigated lands within the district boundaries, E represents the number of ground water irrigated acres for district K. If no ground water irrigated lands are located inside the boundaries of district K, E = 0. As long as either

A is non zero or E is non zero K is a non zero number, representing the district number.

If there is only one irrigation district in node point (I,J) the irrigation district number is entered under NIR(I,J,1), the surface irrigated area entered under SURFAR(I,J,1) and the ground water irrigated area entered under GRWAR(I,J,1). In that case L=M=N=B=C=D=0. In presence of two irrigation districts the numbers are entered under NIR(I,J,1) and NIR(I,J2) etc. The last values entered on the card are the surface water and ground water irrigated acres of the non organized lands. If none exist, X and Y are 0. There may be a situation in which no organized districts are present for node (I,J), but only non organized lands. In that case all values are zero except X and Y.

INPUT(I,J) = array specifying the amount of data available for node (I,J). If data is adequate INPUT(I,J) = 0 and the input term is calculated as follows

$$Q(I,J) = -WATER-SEEPAG + OUT - RAIN + PUMP - FLOWIN + SINKIN$$

If the data are inadequate to calculate all terms in above equation INPUT(I,J) = 1, and the input is calculated from an estimated sum-total, distributed over a number of nodes, similar to the calculation of the tributary valley underflow. The areas for which INPUT (I,J) = 1 must be identified by a 'tributary' valley number, NFLOW(I,J), (see calculation of the Input Program, page 20).

NFLOW(I,J) = array specifying by identification number the node belonging to a specific tributary valley (I,J). Each valley is given a unique number. For all nodes over which the underflow of valley No. L is distributed, NFLOW(I,J) = L. For nodes not in tributary valley NFLOW(I,J) = 0.

NREACH(I,J) = array specifying by identification number the node belonging to a specific reach. Each reach is given a unique number. For all nodes over which the gains/losses of reach No. M is distributed NREACH(I,J) = M. For nodes not on reach NREACH(I,J) = 0.

NUM(L) = variable specifying the number of nodes over which the ground water underflow of valley L is distributed.

NO(M) = variable specifying the number of nodes over which the gains or losses of reach M is distributed.

TOTAC(K) = total recorded surface water irrigated acreage for district K [ac]. The *recorded* acreage is used to calculate the irrigation rate for the district. In the input program, in scanning through the complete grid system, the total surface water irrigated acreage for the districts is calculated by adding up all the respective planimetered areas per node point, and stored under the variable CALCAC(K). Both variables are printed out. If case irrigation district K is located in its entirety within the grid system TOTAC(K) should equal CALCAC(K). If TOTAC(K) does not equal CALCAC(K) the recorded acreage TOTAC(K) for district No. K should be set equal to CALCAC(K). This is to obtain the correct irrigation rate (the irrigation inputs are calculated by applying the irrigation rate to the *planimetered* areas per node point)

If the irrigation district is partly located inside the grid system, CALCAC(K) represents only part of the district acreage and has no significance. The area represented by TOTAC(K) is then used to calculate the correct irrigation rates.

CH(K) = decimal fraction by which the net diversion of irrigation district K is multiplied. In case the historical diversions are to be used, CH(K) = 1.00 CH(K) is used in management trials to determine the effect of a reduced net diversion on the water table.

Input Data of Data Set 2

The variables of Data Set 2 are entered in the following order:

TOTDIV(K) = total surface water diversion for one half time step for irrigation district No. K. [ac ft]

PERC(K) = percentage of the total surface water diversion which is return flow for one half time step for irrigation district K. [%]

USE(L) = crop consumptive use for one half time step for climatic region No. L [inches]

PRECIP(L) = total precipitation for one half time step for climatic region No. L [inches]

SUMPRE(L) = total precipitation falling in the irrigation season for climatic region L. [inches] Only read in when half time step duration is one year.

In the course of the simulation of a historic time span some irrigation districts may have an increase in irrigated acreage; the non-organized irrigated lands may increase, or an increase may occur in the ground water irrigated acreage. To accommodate the respective increases, for every half time step these changes are read in. In the calculation routine these changes are added to the original variable values . The variables related to these changes are the following six variables:

IRN(I,J) = represents the identification number of the irrigation district for which the acreage changes during the half time step.

NREG(I,J) = climatic region identification number of the node for which a change takes place.

DNAMED(I,J) = change in surface water irrigated acreage from previous to present half time step for irrigation district 'IRN(I,J)' [ac]

DEREST(IJ) = change in surface water acreage from previous to present half time step for non-organized lands.[ac] Only one district per node per half timestep can be changed.

DELGR(I,J) = change in ground water irrigated acreage from previous to present half time step for organized and non-organized lands.[ac]

DARW(I,J) = change in wetted irrigation canal area from previous to present half time step for node point (I,J)[ac]

The remaining terms of Data Set 2 include

PU(I,J) = total recharge or withdrawal for one half time step for node (I,J). [ac ft]

UNDFLO(M) = total ground water underflow in one half time step for valley M. [ac ft]

REACH(N) = total reach gain or loss in one half time step for reach N. [ac ft]

Entering of the Data for the Input Program

The control variables are entered via one card under the format as shown in Table 31. Data Set 1 is entered under the formats as shown in Table 32, while Data Set 2 is entered under the formats as shown in Table 33.

Table 31. Format Specifications for the Control Variables - Input Program

Cards Required	Format	Variables	Remarks
1	(2I5,F5.2, 2F10.2,7I5)	NROW,NCOL,DELT DELX,DELY,LTS, NTS,NIRRI,NRE, NVAL,NCLI,NR	

Table 32. Format Specifications for Variables of Data Set 1 - Input Program

Cards Required	Format	Variables	Remarks
NCOL*(NROW/40)1/	(40I2)	(NCX(I,J), I = 1, NROW)	J = NCOL, 1 Start with top row,(max. J) Enter data from left to right (increasing I)
NCOL*(NROW/40)1/	(40I2)	(NODEL(I,J), I = 1, NROW)	J = NCOL, 1 Start with top row, (max. J) Enter data from left to right (increasing I)
NCOL*(NROW/80)1/	(80I1)	(NEF(I,J), I = 1, NROW)	J = NCOL, 1 Start with top row, (max. J) Enter data from left to right (increasing I)
(NR/9)1/	(9F5.2)	RECH(I)	I = 1, NR
XI	(I2,I3,4I2, 3X,F4.1, 10I5,F3.1, 5X,I2)	I,J, (NIR(I,J,K), K = 1,4), ARWET(I,J), (SURFAR(I,J,K), K = 1,4), (GRWAR(I,J,K), K = 1,4), SUREST(I,J) GRREST(I,J), FACTOR(I,J) LAST	XI = total number of nodes for which at least one of the variables is non zero. Enter one card per node. LAST = 0, except for on last card; then LAST = 99. If no nodes are present with some form of irrigation, enter dummy card with I = 1, J = 1, LAST = 99.
NCOL*(NROW/80)1/	(80I1)	(INPUT(I,J), I = 1, NROW)	J = NCOL, 1 Start with top row (max. J) Enter data from left to right (increasing I)
NCOL*(NROW/40)1/	(40I2)	(NFLOW(I,J), I = 1, NROW)	J = NCOL, 1 Start with top row (max. J) Enter data from left to right (increasing I)
NCOL*(NROW/40)1/	(40I2)	(NREACH(I,J), I = 1, NROW)	J = NCOL, 1 Start with top row (max. J) Enter data from left to right (increasing I)
(NVAL/26)1/	(26I3)	NUM(K)	K = 1, NVAL
(NRE/26)1/	(26I3)	NO(K)	K = 1, NRE
(NIRRI/11)1/	(11F7.0)	TOTAC(K)	K = 1, NIRRI Enter data without dec. point.
(NIRRI/26)1/	(26F3.2)	CH(K)	K = 1, NIRRI Enter data without dec. point.

1/ round off the ratio to upper whole number

Table 33. Format Specifications for Variables of Data Set 2 - Input Program[2]

Cards Required	Format	Variables	Remarks
(NIRRI/11)[1]	(11 F7.0)	TOTDIV(K)	K = 1, NIRRI Enter data without the decimal point.
(NIRRI/26)[1]	(26 F3.1)	PERC(K)	K = 1, NIRRI Enter data without the decimal point.
(NCLI/20)[1]	(20 F4.2)	USE(K)	K = 1, NCLI Enter data without the decimal point.
(NCLI/16)[1]	(16 F5.2)	PRECIP(K)	K = 1, NCLI Enter data without the decimal point.
(NCLI/16)[1]	(16 F5.2)	SUMPRE(K)	K = 1, NCLI Enter data without the decimal point. Enter only when DELT = 730.00
XP		I,J, PU(I,J), LAST	XP = total number of nodes with an input pumped wells. Enter one card per node. LAST = 0 except for last card; then LAST = 99 If no pumped wells are present enter dummy card with I = 1, J = 1, PU(I,J) = 0,LAST = 99.
XC	(I2, I3, 2 I5, 3 I5, F5.1, I2)	I, J, IRN(I,J),NREG(I,J), DNAMED(I,J), DEREST(I,J), DELGR(I,JO, DARW(I,J), LAST	XC is total number of nodes for which a change takes place. Enter one card per node. LAST is zero except on last card; then LAST = 99. If no nodes are present with a change enter a dummy card with I = 1, J = 1, LAST = 99.
(NVAL/11)[1]	(11 F7.0)	UNDFLO(K)	K = 1, NVAL Enter data without decimal point.

[1] round off the ratio to upper whole number.
[2] Enter Data Set 2 (NTS-1) times.

APPENDIX B

CONTROL VARIABLES AND DATA FOR THE GROUNDWATER MODEL

A summary and explanation of the control variables and input data in the sequence they are entered in the program follow. Formats and rules for entering data are presented later in Tables 5 and 6.

Control Variables of Main Program

The control variables of the main program are arranged on four cards.

1. Variables related to Microfilm Contour Plot.

If program is run on a computer with a microfilm printer available a subroutine 'CONPLT' is added to the program that will generate a contour map of the calculated head values at specified half time steps.

MICX = variable related to the half time step number. If MICX = 1, a contour plot will be generated for the even numbered half time steps (head values are calculated row wise; x - direction). If MICX = 0, no contour plot will be generated.

MICY = variable for which similar rules apply for the odd numbered half time steps (head values are calculated column wise, y - direction).

XA = variable specifying size of contour plot along X-axis. XA = 13.70 (Figure of 13.7 is specified for NRTS computer center, Idaho Falls).

YA = variable specifying size of contour plot along Y-axis.
YA = (NCOL/NROW)*(XA+1.0)

NIM = variable representing the desired number of contour intervals.

NSER = variable used in connection with MICY.
If MICY = 1 and NSER = 1, contour maps will be generated for half time steps 3, 5, 7, 9, . . . etc.
If MICY = 1 and NSER = 2, contour maps will be generated for half time steps 3, 7, 11, 15 . . . etc.

2. Variables related to Print Out.

NPX = variable related to half time step number.
If NPX = 1, calculated head values will be printed out as an array for every even numbered half time step (heads calculated row wise; x - direction).
If NPX = 0 the head values are not printed.

NPY = variable for which similar rules apply but for the odd numbered half time steps (head calculated column wise; y - direction)

NSU1 = variable denoting the number of the first half time step for which the depth of water is printed out as an array. (half time step 'NSU1')
If NSU1 = 0 no depth to water array is printed out.

Three odd numbered time steps may be specified at which the depth to the calculated water table from land surface is printed out as an array.

NSU2 = variable denoting the number of the second half time step for which the depth to water is printed out as an array.

NSU3 = variable for which similar rules are valid.

3. Variables related to the source term Q(I,J)

NQ = dummy variable specifying the way Q(I,J) is read in. The source term Q(I,J) represents the external input to the aquifer. Dependent on the value of dummy variable NQ, the source term is read in for different simulations in a different way.

Steady State Simulation.

A. NQ = 1: For specific node points the source term is read from cards (one for each node point) [ft^3/day], constant for each half time step.

B. NQ = 2: For all node points the source term is set equal to a common value 'FLUX' [ft^3/day], constant for all half time steps.

C. NQ = 3: For all node points the source term Q(I,J) is zero.

D. NQ = 4: The source term is calculated in a separate input program and written out on magnetic tape. This tape serves as input to the model program. The source term may be different for each node but constant for all half time steps [ft^3/day]

Unsteady State Simulation

A. NQ = 5: The source term is calculated in a separate input program and written out on magnetic tape. This tape serves as input to the model program. The source term may be different for each node and different for each half time step [ft^3/day]

FLUX = variable, the value of which represents the total input for one node per half time step in case NQ = 2 [ft^3/day]

MU = is decimal fraction multiplier for the source term Q(i,j) If MU = 1.00, simulation is run with historic source term. MU may be used in management trials to determine the effect of a changed input on the water table.

4. Variables related to gridsize, time step, duration of simulation and presence of time varying hydraulically connected streams.

NROW = number of nodes along row (X - direction)

NCOL = number of nodes along column (Y - direction)

DELT = length of the initial time step [days]

DELX = length of mesh in X - direction [ft]

DELY = length of mesh in Y-direction [ft]

LTS = serial number of the last half time step for which head values are calculated with the initial time step length. After this half time step, the time step length is doubled (may be used in unsteady state simulation)

NTS = serial number of the last half time step of the total period of simulation. The initial head values are read in under PHI(I,J,1); the first half time step calculated is PHI(I,J,2), while the head values at the end of the first full time step are denoted by PHI(I,J,3). LTS as well as NTS are chosen such that they represent the end of a full time step so that they are always odd numbers. The total simulation is divided into half time steps (k = 2, 3 . . ., NTS), and (NTS-1) half time steps head values are calculated. For steady state simulation LTS = NTS.
For unsteady state simulation LTS $\leq$ NTS.

NRIVER = number of hydraulically connected streams each with a specific time dependent water level. One hydraulically connected stream may represent a number of node points with the same time dependent water level or represent just one node.
NRIVER = 0 if there are no streams present with a time-dependent hydraulic head.

Control Variables-Calibration Routine COMPAR

There are 16 control variables that are related to the type of calibration (single- or multi-parameter calibration), type of simulation (steady or unsteady state simulation), the number of calibration cycles, parameter adjustments and other variables, necessary for a correct calibration procedure. They are NVA1, NVA2, NVA3, NSTOP, ITE, NST, NV(1), NV(2), NV(3), N10, JI, JK, VP, SREST, REST, RMI and are explained as follows.

The calibration procedure is composed of the single-parameter calibration and the multi-parameter calibration. Each calibration is subdivided in a steady state type simulation and an unsteady state type simulation.

There may be four geohydrological parameters subject to calibration, dependent on the values that the dummy variable NVAR will assume in the calibration procedure. If NVAR = 1, the transmissibility values (TX and TY) are changed. If NVAR = 2, the leakance factor (FAC) is changed. If NVAR = 3, the head difference between the main aquifer and the underlying or overlying water bearing formation (PPI) is changed. If NVAR = 4, the storage coefficient (S) is changed.

In order to allow adjustment of any selection of parameters the value of NVAR is determined in a DO-LOOP by choosing appropriate values for NVA1, NVA2 and NVA 3 as follows:

DO m NVAR = NVA1, NVA2, NVA3.

This means that variable NVAR = NVA1, (NVA1 + NVA3) or (NVA1 + 2NVA3)··· until NVAR = NVA2 respectively. It follows then that if NVA1 = NVA2 a single-parameter calibration is conducted. It follows then that if NVA1 < NVA2 a multi-parameter calibration is conducted.

Single-Parameter Calibration

In a single-parameter calibration NVA1 = NVA2 = i and NVA3 = 1; i = 1, 2, 3 or 4 dependent on the variable to be changed. Since only one parameter is adjusted only one calibration cycle is conducted. NSTOP represents the number of calibration cycles. In this case NSTOP = 1.

In the calibration cycle a maximum of ITE simulations (or iterations) are run (either steady or unsteady state simulations); this means that the parameter being calibrated is changed a maximum of (ITE-1) times. ITE is set to 40, a number dependent only on the number in the dimension statement.

For every simulation the sum of squares of deviations of calculated head values and historic head values is determined. The sum of squares of deviations for simulation n is SSS(n). The calibration is terminated if

1. The sum of squares for iteration n is less than a specified rest term, SREST. SSS(n) < SREST
2. The sum squares is decreasing; SSS(n) < SSS(n-1) but the maximum specified number of iterations reached; n = ITE
3. The sum of squares is increasing; SSS(n) > SSS(n-1).

For either case the parameter values that resulted in the least sum of squares are punched out on cards for future use.

The parameter adjustments are based on the ratio of calculated and historic hydraulic gradient (for T), or is based on historical and calculated deviations from starting head values (FAC, PPI, S) in which a damping factor is used to allow for a more gradual change of the parameter. The initial value of the damping factor is represented by VP where 0.0 < VP < 1.00. After the first simulation the damping factor is changed automatically in the program dependent on the ratio of successive sums of squares of deviations. If the decrease of sum of squares is very slight, the damping factor will approach unity to allow for as great as possible change in the parameter value.

Multi-Parameter Calibration

In a multi-parameter calibration NVA1 < NVA2 so that more than one parameter is adjusted, dependent on the values for NVA1, NVA2, and NVA3. For a multi-parameter calibration a total of NSTOP calibration cycles are made, each cycle consisting of ITE simulations (iterations) per parameter. NSTOP = 6 which is an arbitrary number. For most aquifers calibration will be completed within 6 cycles.

In every calibration cycle, every parameter selected via the values for NVA1, NVA2 and NVA3, is subjected to ITE simulations (steady state or unsteady state) in which every parameter is changed (ITE-1) times. In a multi-parameter calibration ITE is kept small; this in order to equalize the influence of every parameter in the decrease of sum of squares of deviations. (For the Snake Plain Aquifer in the unsteady state multi-parameter calibration ITE was set equal to 3).

Starting with the first parameter (NVAR = NVA1) ITE simulations are run for which the sum of squares of deviations (SSS) is calculated. The maximum number of simulations is run even if the sum of squares starts to increase. After the ITE simulations are run, the parameter values which resulted in the least sum of squares are stored and the parameter values are changed back to their initial values. Then the next parameter, parameter (NVA1 + NVA3), is changed (ITE-1) times and again optimum values for this parameter are stored and the parameter values are set equal to their initial values. This process is repeated for the last parameter to be changed (parameter NVA2). After the last parameter is changed the first calibration cycle is completed. The next calibration cycle has as starting values the optimum parameter values resulting from each of the ITE simulations of the previous calibration, and the sum of squares of deviations using these optimum values is determined under SUNST(m).

SUNST(m) = the total sum of squares using the optimum parameter values of the (m-1)th calibration cycle. The optimum values of the (m-1)th calibration cycle are the initial parameter values of the m^{th} calibration cycle and may be defined accordingly.

The whole process of adjusting the respective parameters for this calibration run is repeated. The sequence of calibration cycles is terminated if

1. The sum of squares for calibration cycle m is less than a specified rest term SREST. SUNST(m) < SREST
2. The sum of squares is decreasing; SUNST(m) < SUNST(m-1) but the maximum number of calibration cycles is reached: m = NSTOP.
3. The sum of squares is increasing; SUNST(m) > SUNST(m-1)

For either case the set of parameter values that resulted in the least sum of squares (SUNST) are punched out on cards for future use. For the multi-parameter calibration the same initial damping factor is used for every parameter; 0.0 < VP < 1.00.

It was pointed out that both the single-parameter calibration and the multi-parameter calibration are subdivided in a steady and unsteady state type simulation. The control variables that prescribe a single- or multi-parameter calibration are independent of this sub-division. The control variables that prescribe a steady state or unsteady state simulation are in turn independent of the calibration variables and are explained below. Figure 67 is a schematic representation of a steady state simulation for either type of calibration while Figure 68 is a schematic representation of an unsteady state simulation for either type of calibration.

For both simulations the historic head values for node point (i,j) are represented by APHI(i,j,k) and read in for every node point for specific time steps dependent on the simulation. The calculated head values are represented by PHI(i,j,k). The initial head values are read in under PHI(i,j,1).

Steady State Simulation in Relation to the Parameter Calibration

In case of steady state simulation (or equilibrium state simulation) parameters are changed based on the ratio of calculated and historic equilibrium hydraulic gradients (for T), or based on the ratio of differences between the calculated equilibrium head values and historical equilibrium head values (for FAC, PPI, S) at a certain time step. The historical equilibrium head values are constant for every time step, however, the calculated head values may show a response as shown by the dashed line in Figure 67. The model is run starting with the historical equilibrium head values, PHI(i,j,1), and the equilibrium inputs for every node point (i,j). If the geohydrological parameter distributions used for this simulation are incorrect the head values calculated in this simulation will show an unsteady state response (dashed line, Figure 67, drawn quite arbitrarily). Since the input is the same for every time step the system will eventually reach an equilibrium state. The system is said to have reached its equilibrium state if the absolute sum of head changes between two successive half time steps is less than a certain rest term DELTA, or:

$$\Sigma\, |\mathrm{PHI}(i,j,k) - \mathrm{PHI}(i,j,k-1)| < \mathrm{DELTA}.$$

For the first simulation DELTA is set equal to a relatively big number (REST) in order to keep the number of half time steps, necessary to reach an equilibrium state condition, as defined above, within a reasonable limit. DELTA then decreases to a minimum number (RMI) as the sum of squares of deviations at the calibration time step decreases.

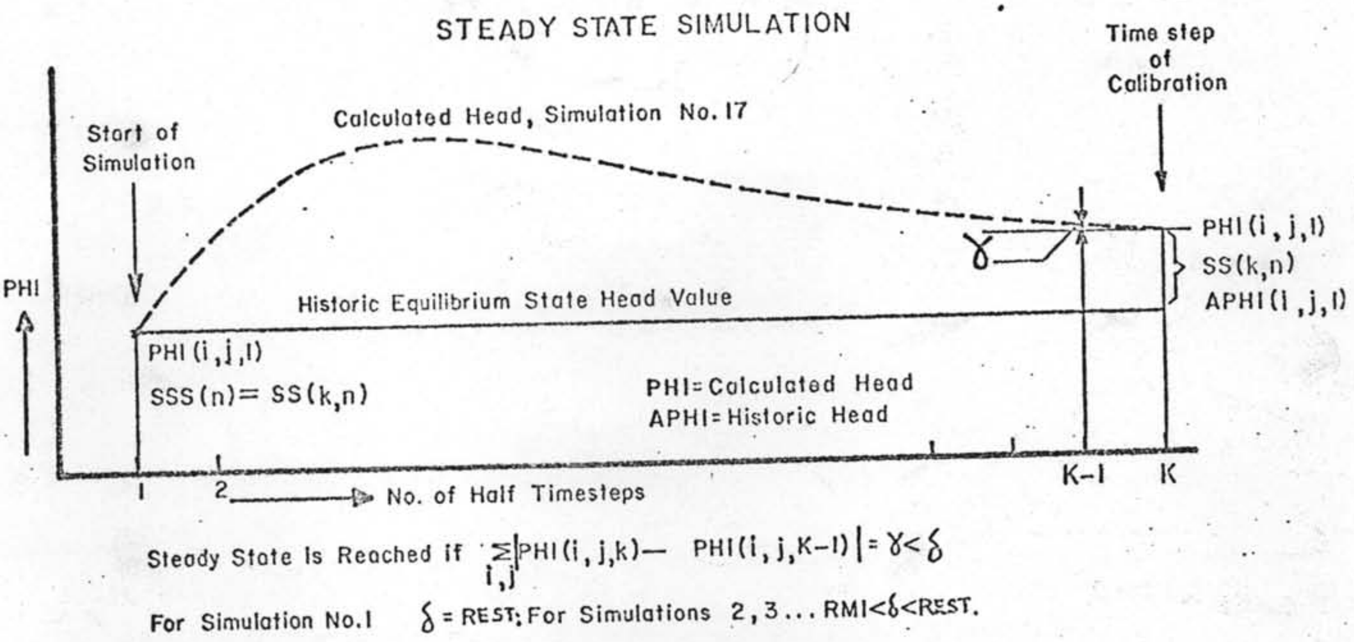


Figure 67. Calibration of Parameters in Steady State Simulation.

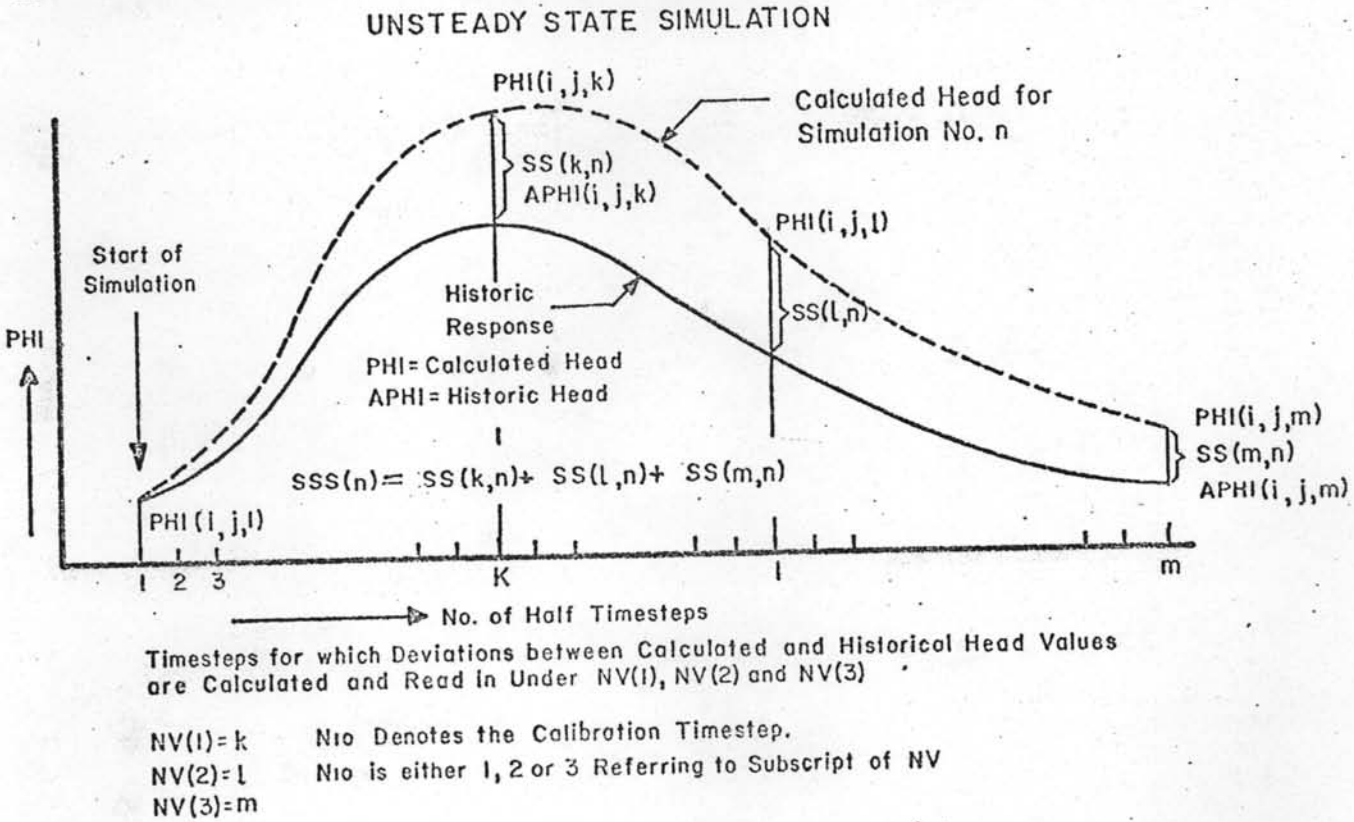


Figure 68. Calibration of Parameters in Unsteady State Simulation.

REST and RMI are read in.

$$RMI \leq DELTA \leq REST$$

In case of steady state simulation there is only one half time step at which the sum of squares of deviations is calculated, denoted by NST(=1). This is the half time step at which the equilibrium state is reached and also the half time step used for parameter adjustment. Let this be half time step No. k; then the total sum of squares of deviations for the n^{th} simulation used as a measure of fit is

$$SSS(n) = SS(k,n)$$

where

$$SS(k,n) = \sum_{i,j} [PHI(i,j,k) - APHI(i,j,k)]^2$$

k = half time step number at which the equilibrium state is reached.

n = n^{th} simulation (iteration) or $(n-1)^{th}$ parameter adjustment.

N10 is a variable denoting the half time step which is used to adjust the parameters. In the steady state calibration, for only one half time step, deviations between historical calculated head values are stored making N10 = 1. The remaining control variables, NV(1), NV(2) and NV(3) have meaning only in the unsteady state simulation. For this steady state simulation

NV(1) = 0

NV(2) = 0

NV(3) = 0

Unsteady State Simulation in Relation to the Parameter Calibration

In case of unsteady state simulation, the historic head values may show a response as schematically represented in Figure 68. The n^{th} simulation (iteration) is run starting with the initial historic head values PHI(i,j,1) and the calculated reponse may be presented as shown by the dashed line, Figure 68. This response results from the specific set of parameter values and unsteady state external inputs. To obtain a sum of squares of deviations which is more representative, more an indication of fit over the total period of the simulation, the sum of squares is composed of the individual sum of squares at specific half time steps of the period of simulation (See Figure 68). Dependent on the availability of historical data and the effort involved, two or three half time steps are chosen to be included in the overall sum of squares. In case of two half time steps, NST = 2 and the half time steps will generally represent the historic maximum and minimum head values in the aquifer. In case three half time steps are used, NST = 3 and the half time steps represent the half time steps denoting the historic maximum, and intermediate value and the historic minimum head values in the aquifer. The first half time step is half time step k; the second half time step is half time step ℓ; the third half time step is half time step m. The sequence numbers k and ℓ and m read in under the variables NV(1) and NV(2) and NV(3) respectively.

The overall sum of squares of deviations for the n^{th} simulation, used as a measure of fit is

$$SSS(n) = SS(k,n) + SS(\ell,n) + SS(m,n) \quad [ft^2]$$

where

$n = n^{th}$ simulation or (n-1)th parameter adjustment

k,ℓ,m = time step numbers for which deviations between historic and calculated head values are stored.

N10 is a control variable denoting which half time step of the three to use for adjustment of the parameters. Generally the half time step for which the differences between calculated and historical head values are used in the parameter adjustment will show the best agreement between historical and calculated head values. Dependent on considerations, different for every aquifer, this best fit may be most desirable for the first time step (maximum) then N10 = 1; the second time step (intermediate), then N10 = 2; or the third time step (minimum), then N10 = 3.

The rest terms REST and RMI are not used in the unsteady state calibration and therefore zero in this case.

Coupled to the calibration program is a microfilm plot-routine that, if available, will plot the deviations of calculated head value from historic head values at the designated half time steps against the number of simulations. Not every node is plotted since this would lead to an over abundance of output. Therefore two control variables are read in: JI and JK, which cause every JIth node in the row and every JKth node in the column of the grid system to be plotted as long as these points are located inside the model boundaries. A simple calculation may be carried out to determine the number of points resulting from this procedure. The total should not exceed 30 since this is the maximum allowed in the dimension statement related to this plot routine. The description of the control variables of the calibration routine COMPAR can be summed up in Table 34.

Table 34. Control Variables of Calibration Routine COMPAR

SINGLE-PARAMETER CALIBRATION	MULTI-PARAMETER CALIBRATION
NVA1 = NVA 2 = i , $1 \le i \le 4$	NVA1 < NVA2
NVA3 = 1	$1 \le$ NVA3 ≤ 3
NSTOP = 1	NSTOP = 6
ITE = 40	ITE = 3

For Either Calibration

VP = f $0.0 < f < 1.00$

SREST = limit for Total Sum of Squares [ft^2]

In Case of

Steady State Simulation	Unsteady State Simulation
NST = 1	NST = 3 or 2
NV(1) = 0	NV(1) = k
NV(2) = 0	NV(2) = 1
NV(3) = 0	NV(3) = m or o
N10 = 1	$1 \le$ N10 ≤ 3 or ($1 \le$ N10 ≤ 2)
REST = Upper Limit Head [ft]Change[ft]	REST = 0.0
RMI = Lower Limit Head Change [ft]	RMI = 0.0

For Either Calibration

JI = JI^{th} node in row to be plotted

JK = JK^{th} node in column to be plotted

Input Variables of the Groundwater Model

Input variables which are read in consist of one dimensioned variable and 15 arrays in the following order:

NREACH(I ,J) = variable denoting hydraulically connected stream node by reach number. Hydraulically connected streams which are denoted as boundaries are divided in reaches each reach is given a number. If NREACH(I,J) = 1, node point (I,J) is located in a reach with a constant hydraulic head. If the reach has a time-dependent hydraulic head, the reach identification number is greater than one. For instance, for node points located in reach 3, NREACH(I,J) = 3 and all nodes in Reach 3 have the same time-dependent changes in hydraulic head. A reach may consist of one or more node points.

The array NREACH(I,J), is initialized to 'one' at the beginning of the program, therefore NREACH(I,J) is read in only if some Reach (I,J) has a changing hydraulic head.

DELH(K,L) = variable representing the change in water level from half time step (L-1) to L for Reach K [ft]. It read in only if reaches with time-dependent head values are present. There are NRIVER Reaches with a changing hydraulic head. The first Reach is always Reach 2; (K = 2). The last Reach NRIVER + 1 (K = NRIVER + 1).

DRAIN(I,J) = variable denoting the drain elevation for that node point [ft], if a drain is present. If no drain is present DRAIN (I,J) = 0

Q(I,J) = the source term or external input to the aquifer and is either read in from cards (one card per node), set equal to a constant value for each node, or read in from a magnetic tape, dependent on the value of NQ (discussed in the control variables section, page 209). In either case Q(I,J) is calculated per half time step for every node in ft^3 and entered in the program as an average value applied during that half time step in $[ft^3/day]$.

KX(I,J) = array denoting the hydraulic conductivity for every node point in X - direction [ft/day].

KY(I,J) = array denoting the hydraulic conductivity for every node point in Y - direction [ft/day].

FAC(I,J) = array denoting the leakance factor of the impeding layer for every node point [1/day]. If no leaky aquifer condition is present FAC(I,J) = 0.0.

PPI(I,J) = array denoting the head difference between main aquifer and over or underlying water bearing formation at the start of the simulation [ft]. PPI(I,J) > 0. if head in main aquifer is greater than head in water bearing formation. If no leaky aquifer situation is present, PPI(I,J) = 0.

S(I,J) = array denoting the storage coefficient for every node point [dimensionless].

Boundary Characterization Arrays (NCX(I,J) and NCY(I,J)

In the alternating direction implicit method the nodal head values are calculated for every half time step, alternately row wise and column wise. In the row wise calculation the nodal head values are calculated along the row from boundary to boundary. Dependent on the type of boundary different flow equation coefficients are calculated. A similar procedure is followed for the column wise calculation of the nodal head values. In order to allow for complex boundary configurations two arrays are read in, one for the row wise calculation, the other for the column wise calculation of the head values. These arrays characterize the type of boundary and the general location of the node points by assigning numbers to each node.

NCX(I,J) = array denoting boundary type for the calculation of the nodal head values along a row (X - direction). For node point (I,J) if:

NCX = 0: Node point is located outside model boundaries.

NCX = 1: Node point is a hydraulically connected stream node (lake, river), representing a constant head boundary.

NCX = 2: Node point is located inside the model boundaries.

NCX = 3: Effective impermeable boundary is located one half grid spacing to the left of this node.

NCX = 4: Effective impermeable boundary is located one half grid spacing to the right of this node.

NCX = 5: Node point located one grid spacing to the left of this node is a hydraulically connected stream node (lake, river elevation) NCX(I-1,J) = 1.

NCX = 6: Node point to the right of this node is a hydraulically connected stream node (lake, river elevation) NCX(I+1,J) = 1.

NCX = 7: Node points one grid spacing to the left and one grid spacing to the right of this node are hydraulically connected stream nodes (lake, river elevation). NCX(I-1,J) = 1 and NCX(I+1,J) = 1.

NCX = 8: Node point is a constant gradient boundary node at the start of a row (left to right).

NCX = 9: Node point is a constant gradient boundary node at the end of a row (left to right).

NCX =10: If successive node points along the row are constant gradient boundary nodes (constant slope boundary parallel to row) all these nodes are characterized by NCX = 10.

The respective boundary conditions for the row wise calculation are illustrated in figures 69-72. Figure 69 shows the rivers on start and end of row. Figure 70 represents a combination of NCX = 5, NCX = 6 and NCX = 7. Figure 71 represents a row with an impermeable boundary on the start and beginning of a row.

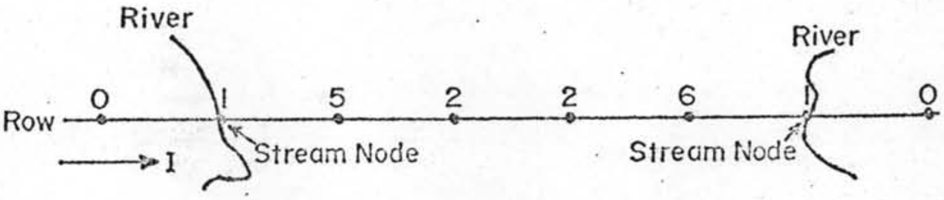


Figure 69. Rivers at Beginning and End of Row.

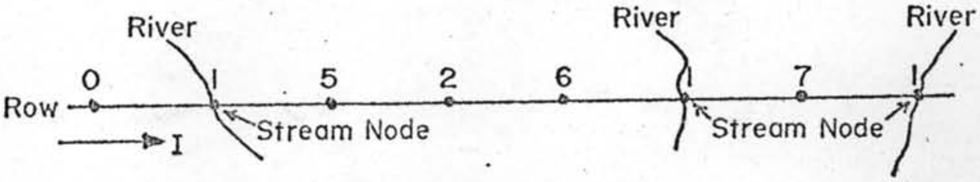


Figure 70. Three Rivers Crossing Row.

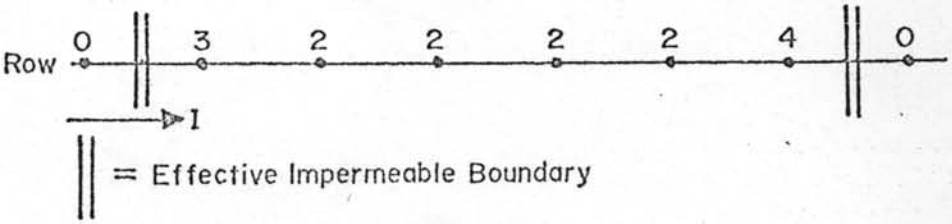
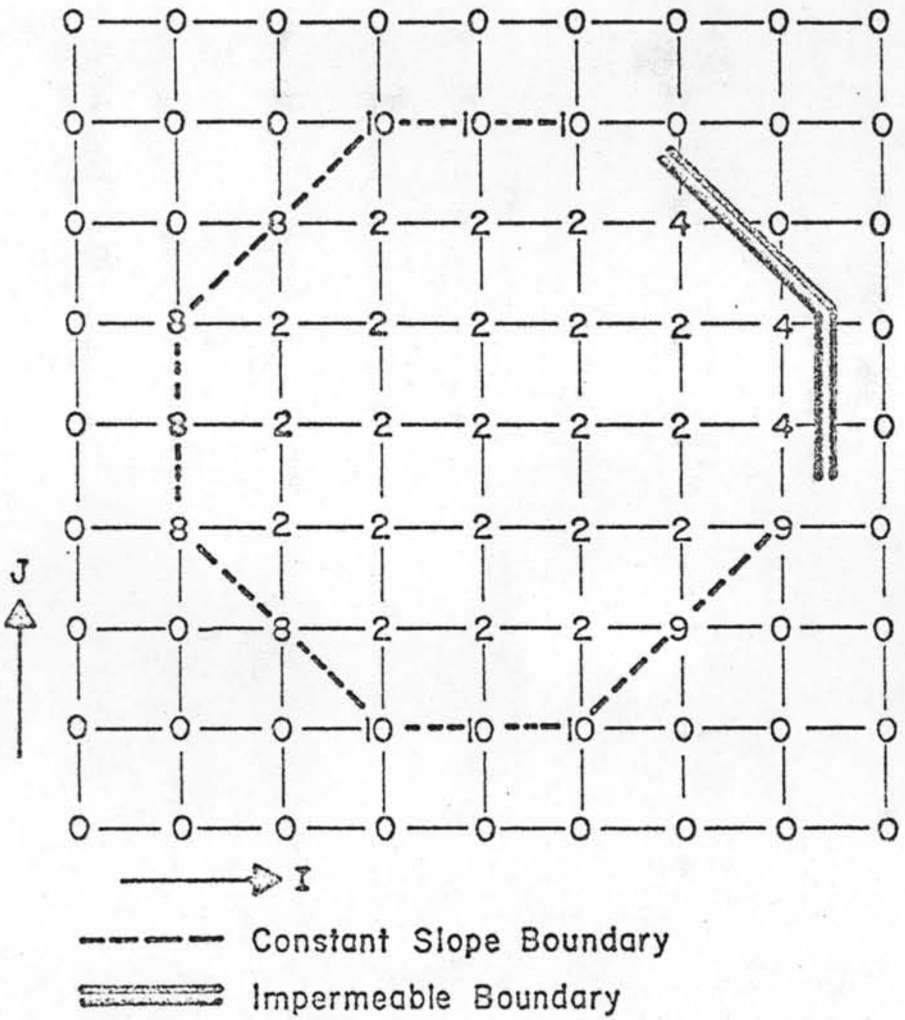
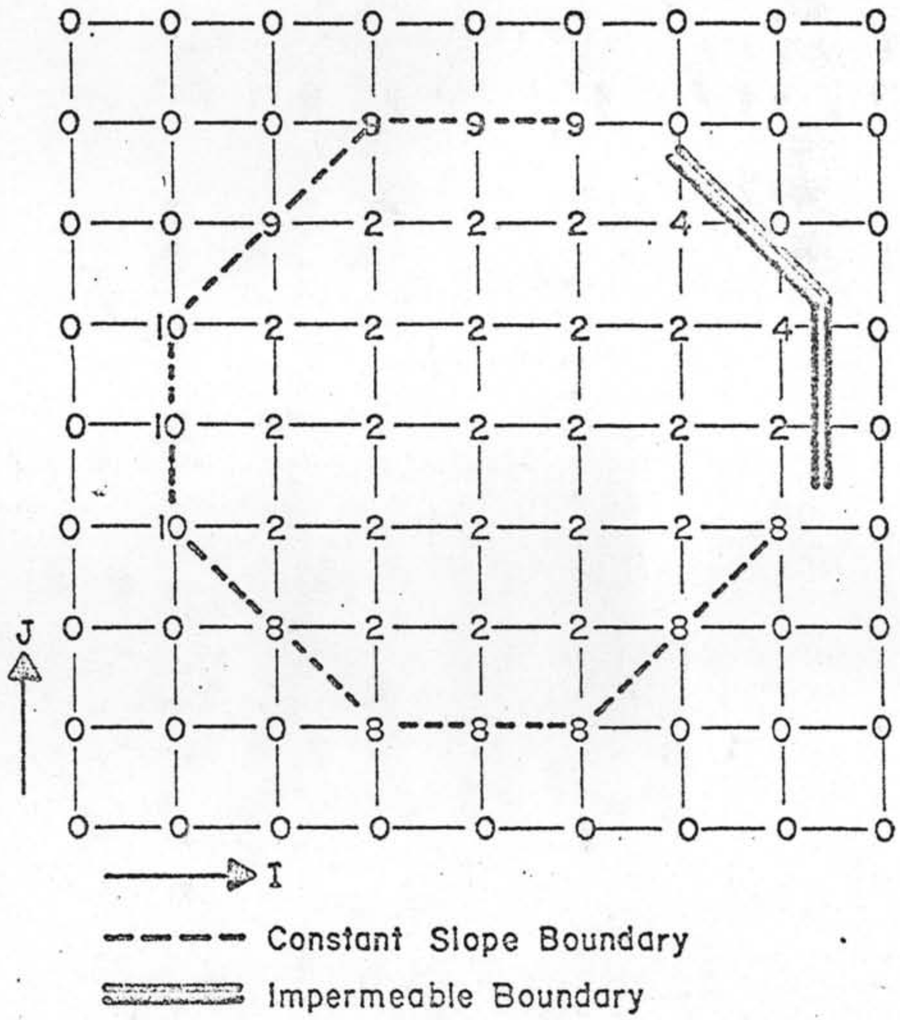


Figure 71. Three Impermeable Boundaries.



0	0	0	0	0	0	0	0	0
0	0	0	10	10	10	0	0	0
0	0	8	2	2	2	4	0	0
0	8	2	2	2	2	2	4	0
0	8	2	2	2	2	2	4	0
0	8	2	2	2	2	2	9	0
0	0	8	2	2	2	9	0	0
0	0	0	10	10	10	0	0	0
0	0	0	0	0	0	0	0	0

Figure 72. NCX - Array for Aquifer With Constant Slope (Gradient) Boundaries (Head Values Are Calculated Row Wise).



0	0	0	0	0	0	0	0	0
0	0	0	9	9	9	0	0	0
0	0	9	2	2	2	4	0	0
0	10	2	2	2	2	2	4	0
0	10	2	2	2	2	2	2	0
0	10	2	2	2	2	2	8	0
0	0	8	2	2	2	8	0	0
0	0	0	8	8	8	0	0	0
0	0	0	0	0	0	0	0	0

Figure 73. NCY - Array for Aquifer With Constant Slope (Gradient) Boundaries (Head Values Are Calculated Column Wise).

Figure 72 illustrates the numbering system for the constant gradient boundary for the row wise calculation and represents an aquifer octagonal in shape. A constant gradient boundary is located on all sides except for the northeast corner which represents an impermeable boundary. To determine NCX for node point (I,J) every row can be considered separate from the other rows. NCX = 10 for a constant gradient boundary parallel to the row.

NCY(I,J) = array denoting the boundary type for the calculation of the nodal head values along the column (Y - direction. (For node point (I,J), if:

NCY = 0: Node point is outside model boundaries. If NCX(I,J) = 0, NCY(I,J) = 0

NCY = 1: Node point is a hydraulically connected stream node (lake, or river), representing constant head boundary. If NCX(I,J) = 1, NCY(I,J) = 1.

NCY = 2: Node point is inside the model boundaries.

NCY = 3: Effective impermeable boundary is located one half grid spacing below this node.

NCY = 4: Effective impermeable boundary is located one half grid spacing above this node.

NCY = 5: Node point one grid spacing below this node is a hydraulically connected stream node (lake, river elevation) NCY(I,J-1) = 1

NCY = 6: Node point one grid spacing above this node is a hydraulically connected stream node (lake, river elevation) NCY(I,J+1) = 1.

NCY = 7: Node point one grid spacing below and one grid spacing above this node is a hydraulically connected stream node (lake, river elevation) NCY(I,J-1) = 1 and NCY(I,J+1) = 1.

NCY = 8: Node point is a constant gradient boundary node at the start of a column (bottom to top).

NCY = 9: Node point is a constant gradient boundary at the end of a column (bottom to top).

NCY =10: If successive node points along the column are constant gradient boundaries nodes (constant gradient boundary is parallel to column) all these nodes are characterized by NCY = 10.

The respective boundary conditions for the column wise calculations are illustrated in Figures 74, 75 and 73. To determine NCY for node (I,J) every column can be considered separate from the other columns.

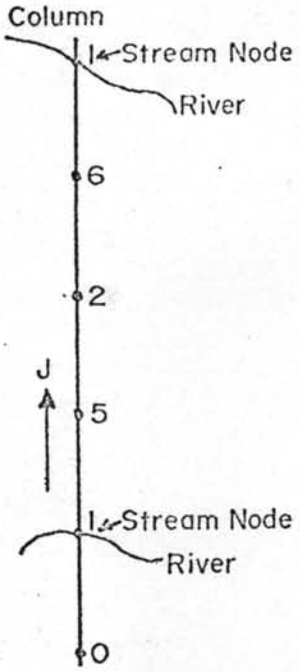


Figure 74. Constant Heat at Start and End of Column.

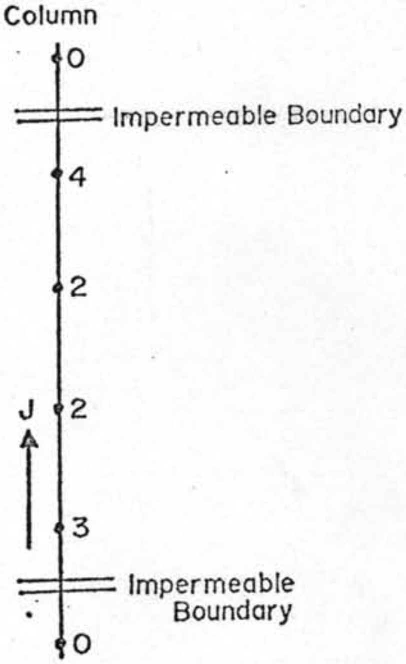


Figure 75. Impermeable Boundary at Start and End of Column.

Figure 74 represents a column with a constant head at the start and the end of the column. Figure 75 represents a column with an impermeable boundary at the beginning and at the end of the column. Figure 73 represents an octagonal aquifer with a constant slope boundary on all sides except for the northeast corner which presents an impermeable boundary. The figure is similar to Figure 72 but illustrates the number system for the column wise calculation of the head values in the aquifer. The remaining input variables to the main program include

NN(I,J) = array denoting the confinedness of the aquifer. If NN(I,J) = 0 the node is located in an unconfined aquifer. If NN(I,J) = 1, the node is located in a confined aquifer.

SURF(I,J) = array denoting the land surface elevation with as reference mean sea level [ft]. Used only to calculate the depth of water if desired.

Z(I,J) = array denoting the aquifer bottom elevation with as reference mean sea level [ft]. Used to calculate the depth of aquifer for each node point, necessary to calculate the transmissibility values from the conductivity values. For node point in confined aquifer Z(I,J) represents the depth of the main aquifer.

PHI(I,J,1) = array denoting the initial hydraulic head values in the aquifer at the start of the simulation. Reference is mean sea level [ft].

APHI(I,J.,NST) = array used for the calibration. It denotes the historically measured head values at specific half time steps of the simulation. For steady state simulation one value is read in for each node; NST = 1 and APHI(I,J,1) represents the historic equilibrium head values. For unsteady state simulation historic head values at two or three half time steps are read in dependent on the value of NST (discussed in the control variables section). The historic head values are used in combination with the calculated head values to adjust the geohydrological parameters.

Entering of the Data

The control variables of the main program (first four cards) and the calibration routine (card 5) are entered as shown in Table 35.

The values for the input variables are entered as shown in Table 36.

Table 35. Format Specifications for the Control Variables of the Groundwater Model.

Cards Required	Format	Variables	Remarks
1	(2I5,2F5.2,2I5)	MICX,MICY,XA,YA, NIM,NSER	If micro film printer is not available enter blank card
1	(5I5)	NPX,NPY,NSU1, NSU2,NSU3	
1	(I5,F10.0,F5.2)	NQ,FLUX,MU	
1	(2I5,F5.2,2F10.2, 3I5)	NROW,NCOL,DELT, DELX,DELY,LTS, NTS,NRIVER	
1	(12I5,F5.2, 3F5.0)	NVA1,NVA2,NVA3, NSTOP,ITE,NST, NV(1),NV(2),NV(3), N10,JI,JK,VP, SREST,REST,RMI	

Table 36. Format Specifications for Input Variables of the Groundwater Model.

Cards Required			
XR	(I2,3I3)	I,J, NREACH(I,J), LAST	XR = total number of nodes which have time dependent hydraulic heads. Enter one card per node. LAST = 0 except on last card to be entered: then LAST = 99 Omit all these cards if NRIVER = 0.
NRIVER*[(NTS-1)/20]1/	(20 F4.2)	(DELH(K,I), I = 2,NTS)	K = 2,NRIVER+1; I = 2,NTS Omit all these cards if NRIVER = 0
XD	(I2,I3,I5,I3)	I,J, DRAIN(I,J, LAST	XD = total number of nodes with a prescribed drain level. Enter one card per node. LAST = 0 except on last card to be entered: then LAST = 99 If no drain nodes are present: Enter one card with: I = 1, J = 1, DRAIN(I,J) = 0, LAST = 99.
XQ	(I2,I3,F10.2, I2)	I,J,Q(I,J), LAST	XQ = total number of nodes for which an input is read in. Enter only if NQ = 1 (one card per node) LAST = 0 except on last card; then last = 99 Omit all cards if NQ = 1
NCOL*(NROW/13)1/	(13F6.1)	(KX(I,J),I=1,NROW)	J = NCOL, 1.Start with top row (maximum J). Enter data from left to right (increasing I). Enter data without decimal point.
NCOL*(NROW/13)1/	(13F6.1)	(KY(I,J),I=1,NROW)	J = NCOL, 1.Start with top row in array (maximum J) Enter data without decimal point.
NCOL*(NROW/16)1/	(16F5.5)	(FAC(I,J),I=1,NROW)	J = NCOL, 1.Start with top row in array (maximum J) Enter data without decimal point.

1/round off the ratio to upper whole number.

Table 36.—continued

Cards Required	Format	Variables	Remarks
NCOL*(NROW/26)[1/]	(26F3.1)	(PPI(I,J),I = 1,NROW)	J = NCOL, 1 . Start with top row in array. (maximum J) . Enter data without decimal point
NCOL*(NROW/26)[1/]	(26F3.3)	(S(I,J), I = 1,NROW)	J = NCOL, 1 . Start with top row in array (maximum J) Enter data without decimal point
XK	(I2,I3,F5.0,I3)	I,J, KX(I,J), LAST	Used for hydraulically connected stream nodes Enter one card per node point. Values of KX(I,J) entered here will preempt the value for KX(I,J) and KY(I,J) read in under the array form above. This allows to make trial adjustments of the K-values at the hydraulically connected boundary nodes for flow simulation, without changing the original K-values. If minor adjustments are completed these values can be entered in the array. If no adjustments for hydraulically connected stream nodes are made, enter one card with I = 1, J = 1, KX(I,J) = 0., LAST = 99
XS	(I2,3I3,I2,2I5, F6.1,3F6.1)	I,J,NCX(I,J),NCY(I,J) NN(I,J),SURF(I,J), Z(I,J), PHI(I,J,1), (APHI(I,J,K),K = 1, NST)	XS = Total number of nodes within or at the model boundaries. (NCX(I,J) = 0) Enter one card per node point. Enter one, two or three values for APHI(I,J) dependent on values of NST

[1/] round off the ratio to upper whole number.