SYSTEMATIC DESIGN OF LEGAL REGULATIONS FOR OPTIMAL SURFACE-GROUNDWATER USAGE - PHASE 2

by

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ABSTRACT

The design of regulations for integrated management of surface and ground waters is a complex and crucial problem in the state of Colorado and in other regions of the United States with a dry climate and a paucity of water resources. The difficulty arises from the multitude of conflicts between the desires of the various water users, the existing body of laws, the elusive character of a dynamic free economy, the hidden nature of the groundwater resource and the whims of the weather.

The investigation was pursued simultaneously in three directions covering the three major aspects of the problem, involving respectively the disciplines of hydrology, economics and law. For this reason the body of the report is made up of three major parts.

Analysis of the problem indicated the need to develop a new approach to hydrologic modeling of the interactions between a stream and an alluvial aquifer. A new approach which conveniently provides the aquifer response to pumping from wells, the aquifer response to seepage flow from the river and the river response to pumping from wells is briefly presented. Following this approach, several computer programs, operating sequentially, have been developed. Tests of the programs reported here, show that the developed technique provides comparable accuracy to more standard approaches. The benefits of the new approach are realized when the hydrologic model is coupled with an economic objective and

with legal constraints.

Several computer programs were developed. One of these computer programs (the most important and complex one) is fully documented in this report. Illustrative applications of these programs to solution of management problems involving legal constraints and economic considerations are provided. A more detailed manual for use of the 'DELTA' and other programs is planned in the future under a new contract with O.W.R.T. which started July 1, 1975.

The legal analysis pursued in this project has been along two lines. The first, during Phase I of the program, was a general overview of the legal problems and solutions to surface and ground water management, examining the Colorado situation and contrasting the approach taken in Colorado with that of New Mexico and California.

During the second phase, a concentrated effort has been placed upon an analysis of the legal and administrative approaches and problems of integrated surface and ground water management called for under the 1969 Water Administration and Adjudication Act and study of organizational and operational conditions relevant to the problem of conjunctive use on the South Platte River.

The components of the analysis of the law include a discussion on the constitutional and legislative setting of the prior appropriation doctrine in Colorado, up to the current statutory enactments. It is essential to understand the concept of property rights in water to appreciate the complexity of the conjunctive use problem. The protection of water rights permeates all solutions --

physical and economic -- and thus is at both the heart of the problem and the apex of the considerations for alternative solutions.

The legal and administrative framework for implementation of Colorado water laws is unique and critical to an understanding of the conjunctive use problem. Among the offices and personnel are the State Engineer's Office, water court, water judge, referee and clerk. The private and quasi-public entities for water distribution and control are examined for their roles in integrated water management. Very important to Colorado users are the alternative organizational approaches, i.e. river basin authorities, conservancy districts and the newly emerging, private water user entities dedicated to flow augmentation and use of alternative sources of supply. For example, Ground Water Appropriators -- the South Platte, Inc. (GASP), has become a viable entity instrumental in attempting to optimize the ground water resources in that region.

Major attention is focused upon the specific legislative enactments concerning integrated ground and surface water use, the judicial decisions on the rights and obligations of the state and the water users, and the attempts of the State Engineer's Office to resolve the dilemma through rules and regulations.

Presently, the problem of administering the conjunctive use of ground and surface waters in the South Platte River has been resolved by stipulation of facts and regulating procedures between the state authorities and water users. See Appendix IIIA. In addition to this form of solution, water users of both water supply sources have found the "augmentation plan" approach adopted by the Colorado General Assembly to be workable and relatively satisfactory.

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RESEARCH OBJECTIVES

In this second phase of research on the "Systematic Design of Legal Regulations for Optimal Surface-Groundwater Usage" the original objectives, as stated in the Summary of Proposed Work (Notice of Research Project, September 1972), were the following:

> "The overall objective of the research is to design rules for the conjunctive use of surface and groundwater which satisfy the law and which maximize the beneficial use of the waters. To achieve this overall objective, a major objective is a realistic analysis of the simultaneous behavior of river flow and groundwater movement. An important specification for this analysis is that it be immediately and without modification compatible and usable in any economic regional analysis. The maximization of the human regional economic objective will determine a worthy set of rules for management within present law. Another objective is to find out if limited changes in the law might not result in large beneficial gains.
>
> The need for an objective design of rules of operations in surface-groundwater management is pressing in the state of Colorado. The state engineer is the legal authority for enforcement of the water laws for the state. In a recent (October 12, 1971) Supreme Court decision his rules and regulations for the summer of 1969 have been declared valid within the confines of Colorado Water Law doctrines, but the Court explicitly recognized that these rules and regulations were designed with limited physical data and only under then known conditions. Justice Groves anticipates that technological advances will warrant and lead to modifications in the existing rules."

Thus for the time being the Court's acceptance of the proposed rules and regulations eliminates the immediate need for design of new regulations for optimal beneficial use of the waters. On the other hand it created a new problem for the State Engineer - that of evaluation of the so-called *augmentation* plans. For this reason major attention of the project has been devoted to the development of the tools, rather than on particular (and belated) applications of them, leaving this task naturally to the mission-oriented agencies.

With this purpose in mind it becomes even more imperative to demonstrate to the concerned agencies the usefulness of the tools and to provide them with self-sufficient manuals on how to use them. These two new objectives of demonstration and documentation became major objectives during the second year of the project.

This report provides or summarizes the physical, economic and legal tools and parameters of operation in the conjunctive use of ground and surface waters within a specified area of the South Platte.

ACHIEVEMENTS OF CONTRACT

It is not desirable to repeat in this completion report all the results obtained over the past two years and the detailed procedures by which they were obtained. These results and procedures can (or will) be found in two dissertations (Johnson, 1975; Rodriguez, 1976), two theses (Conklin, 1974; Boudreaux, 1976), one book (Radosevich, Hamburg and Swick, 1975), three chapters in two books (Morel-Seytoux, 1975a,b; Radosevich, Allardice, Nobe and Kirkwood, 1976), several published papers (Morel-Seytoux and Daly, 1975; Morel-Seytoux, 1975 c,d,e; Radosevich and Daines, 1975), several published Bulletins (Conklin, 1975; Johnson, 1975) and several submitted papers (Morel-Seytoux, 1976).

Rather a brief review of the methods of attack and a sample of results will be given. On the other hand, a variety of illustrative examples of the type of problems the tools can solve are given. In addition a fairly complete guide to the use of the tools is provided. In short, material suitable for publication in the scientific literature is presented here to a minimum. The report deals with the nuts and bolts of the computer user's trade which are not generally considered suitable for publication in the scientific literature but are nevertheless the key to broad and successful use of the techniques. It also describes in detail the legal conditions and issues of ground and surface water integration in Colorado.

PART I

HYDROLOGIC ASPECTS OF CONJUNCTIVE SURFACE-GROUNDWATER MANAGEMENT

A. INTRODUCTION

In this part of the report the basic terminology and concepts behind hydrologic modeling of a stream-aquifer system are reviewed. The method of solution is also presented briefly.

A conceptual system of computer programs was devised to solve a variety of problems, stage by stage. This conceptual system, more complete than required by the project, is discussed, though only a few stages were actually implemented. The developed programs have been tested fairly thoroughly for accuracy, limitations and cost. As a result some approaches have been modified and the resulting improvements are documented. For the most basic stage of the system the detailed input data format is provided. A complete example of the data preparation for an illustrative case is discussed for the convenience of the eventual user. The use of the programs for solution of typical management problems is illustrated on small size problems.

B. NEED FOR STUDY

A brief statement of the need for this study is included in the Notice of Research Project of September 1972 quoted in the previous section on RESEARCH OBJECTIVES. More complete discussions were presented in the proposals to OWRT dated October, 1971 and September, 1972 and in the Phase I Final Report (Morel-Seytoux et al., 1973). In one section of its 1973 Report, entitled "Improving Ground Water Management", the National Water Commission has formulated twenty recommendations for future action in the area of ground-water use. These recommendations provide a framework for future ground-water use practices based upon sound principles of economics, conservation and land development. The section "considers the principal problems of ground-water *law, management and administration.* They are (1) integrating management of surface water and ground water; (2) depletion of ground-water aquifers at rates exceeding recharge (often referred to as the *mining* of ground water); and (3) impairment of ground-water quality". The current project was concerned with these very problems of law, management and administration particularly with regard to item (1): integrating management of surface and ground waters.

The recent ASCE Manual on Ground Water Management (1972) includes two interesting statements, namely on one hand: "The manager of a groundwater supply need be warned that adversary litigation in this field of law has proved to be among the most protracted, costly, and unproductive of any of the many fruitless endeavors known to civilized gamesmanship." (ASCE, (1972), p. 57) but also the apparently contrary statement: "It is almost instinctive to assume that lawyers, water rights, and litigation are troublesome constraints to be overcome in the formulation of a rational water management plan. This initial response may well obscure the potential utility and flexibility of litigation as a *tool* for water management." (ASCE (1972), p. 61). What these two statements emphasize is the basic need to reconcile the engineering logic with that of law. More rigor is required in the engineering, or planning, or management studies so that what is *obvious* to the technologists becomes *evidence* to the courts, and the social/economic needs and state responsibility to the public is interfaced with design of technological innovations.

In a letter to Professor Morel-Seytoux (see Appendix IA) Dr. Danielson, Deputy State Engineer of Colorado, lists *twelve* management problems of interest to his office on which the programs developed by this project could be used! Most of these twelve management problems can indeed be solved with the programs. In Appendix C answers to problems 8 and 9 listed by Dr. Danielson are provided for an illustrative situation.

C. BASIC GROUND WATER TERMINOLOGY

The basic saturated flow equation (using the Dupuit assumption and a few other traditional assumptions) describing the evolution of an isotropic water-table aquifer is the Boussinesq equation:

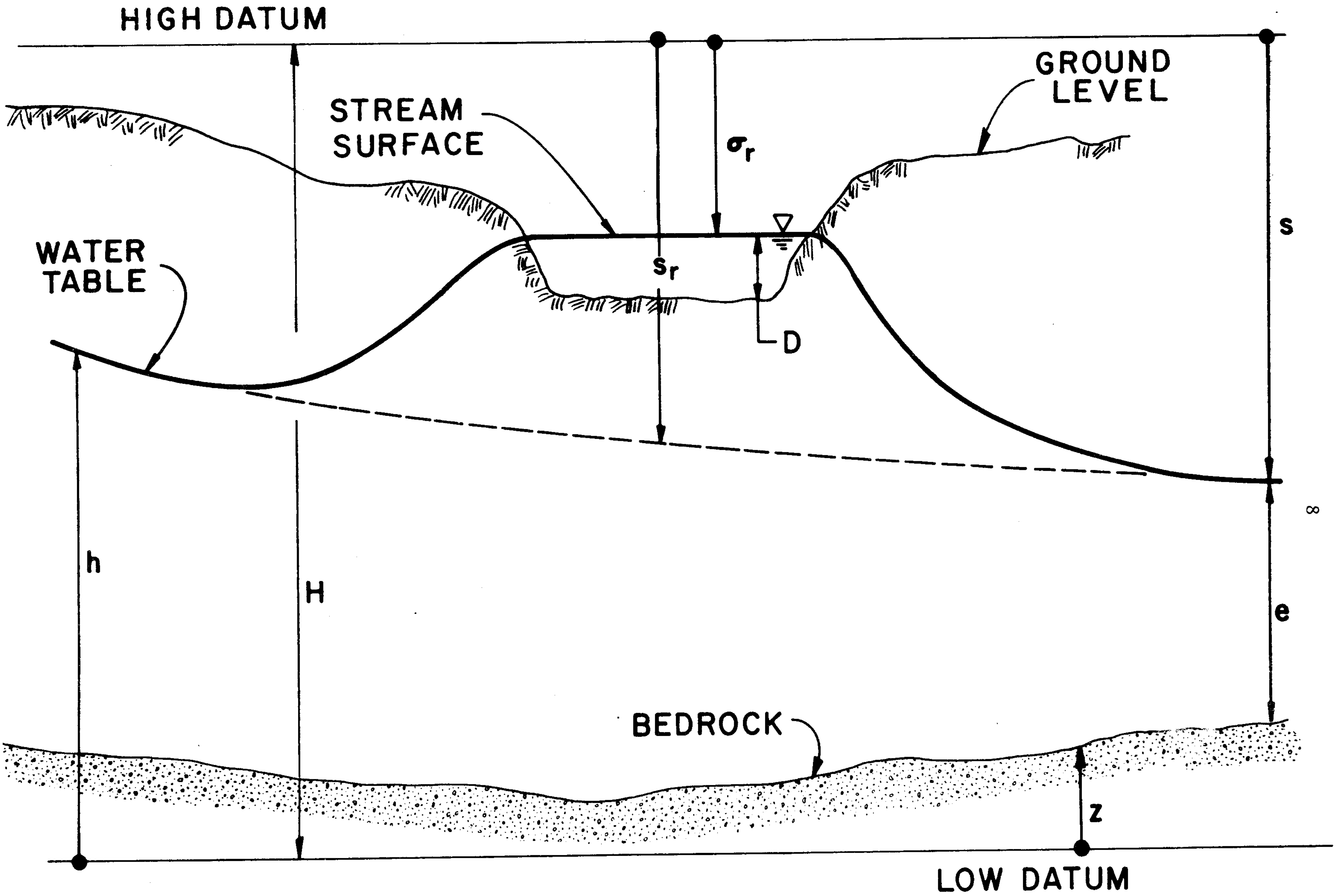
$$\phi \frac{\partial h}{\partial t} - \frac{\partial}{\partial x} (Ke \frac{\partial h}{\partial x}) - \frac{\partial}{\partial y} (Ke \frac{\partial h}{\partial y}) = - Q_w \delta_w - Q_r \delta_r \qquad (1)$$

where ϕ is the drainable (or effective) porosity, K is the saturated hydraulic conductivity, e is the saturated thickness, h is the water-table level measured positive upward from a (low) horizontal datum, Q_w is the instantaneous pumping volume of well w (chosen algebraically positive if it is an actual withdrawal rate), δ_w is a Dirac delta function singular at the point of coordinates ξ_w, η_w and τ (where ξ_w and η_w are the x,y coordinates of well w, τ is time), Q_r is the aquifer instantaneous discharge volume to the river reach and δ_r is a delta function singular along the r^{th} reach of the river. Let s be the drawdown measured positive downward from a (high) horizontal datum located at distance H above the datum for the water-table elevation, and z be the elevation of the impervious bottom of the aquifer above the water-table elevation (or head) datum (see Figure 1).

Eq. (1) can be rewritten in terms of s as:

$$\phi \frac{\partial s}{\partial t} - \frac{\partial}{\partial x} (T \frac{\partial s}{\partial x}) - \frac{\partial}{\partial y} (T \frac{\partial s}{\partial y}) = Q_w \delta_w + Q_r \delta_r \qquad (2)$$

where it has been assumed that the drawdowns are too small to cause a significant change on the transmissivity T. At first Eq.(2) will



VERTICAL CROSS SECTION OF SYSTEM

be studied for the case when there is *no stream intersecting the aquifer.*

1. Solution for an Undeveloped Infinite Homogeneous Aquifer (without stream)

a. The Traditional *Simulation* Approach

Assuming a homogeneous aquifer of infinite extent and no previous development then it is well known (Carslaw and Jaeger (1959), pp. 258-261) that the drawdown at point w at time t due to pumping at well p at a rate $Q(\tau)$ (L^3T^{-1}) is:

$$s_{wp}(t) = \int_0^t \frac{Q_p(\tau)\, e^{-\frac{\phi R_{wp}^2}{4T(t-\tau)}}}{4\pi T} \frac{d\tau}{t-\tau} \tag{3}$$

where R_{wp} is the distance between point w and well p (see Figure 2). If the pumping rates are constant within the basic time period (say the week) but vary from week to week, Eq.(3) can be written in the form:

$$s_{wp}(n) = \sum_{\nu=1}^{n} Q_p(\nu) \int_{\nu-1}^{\nu} \frac{e^{-\frac{\phi R_{wp}^2}{4T(\nu-\tau)}}}{4\pi T(\nu-\tau)}\, d\tau \tag{4}$$

Eq.(4) having been programmed for the digital computer, given *numerical* values of the pumping rates the computer calculates numerical values of drawdowns at point w via Eq.(4). If the effect of different pumping patterns, say 50 of them, on the drawdowns is to be investigated the operation is repeated 50 times. Note that, *each time*, the

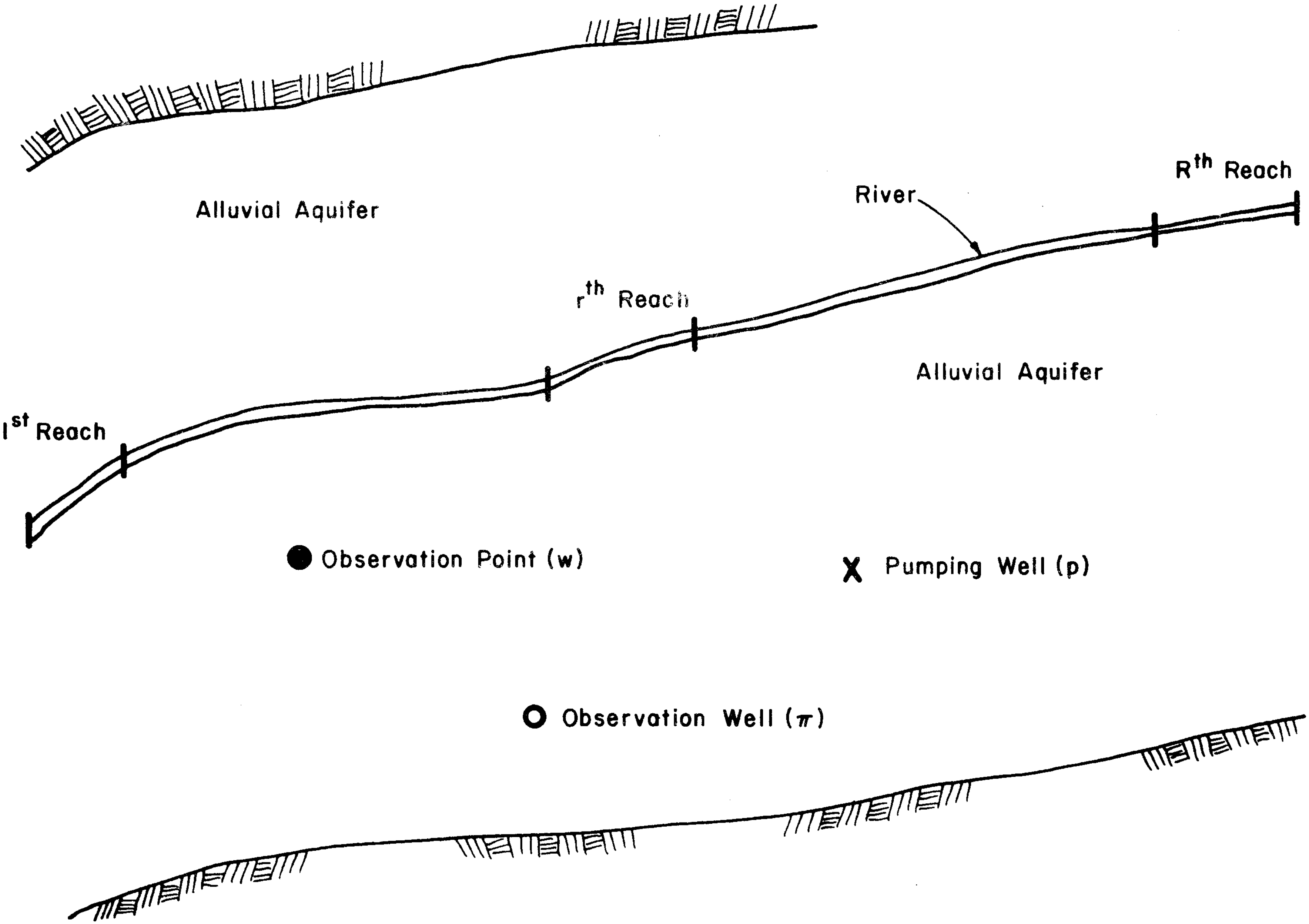

Alluvial Aquifer
River
R^th Reach
r^th Reach
1^st Reach
Alluvial Aquifer
● Observation Point (w)
X Pumping Well (p)
○ Observation Well (π)

Figure 2

integrals appearing in Eq.(4) are evaluated again.

b. The *Discrete Kernel* Approach

Let the *pumping* kernel be defined as:

$$k_{wp}(u) = \frac{e^{\frac{-\phi R_{wp}^{2}}{4Tu}}}{4\pi Tu} \qquad (5)$$

then Eq.(4) can be rewritten in the form:

$$s_{wp}(t) = \int_{o}^{t} Q_p(\tau)\ k_{wp}(t-\tau)\ d\tau \qquad (6)$$

or in discrete form:

$$s_{wp}(n) = \sum_{\nu=1}^{n} \delta_{wp}(n-\nu+1)\ Q_p(\nu) \qquad (7)$$

where the *discrete kernel* coefficients are defined as:

$$\delta_{wp}(\nu) = \int_{o}^{1} k_{wp}(\nu-\tau)\ d\tau \qquad (8)$$

Once the $\delta_{wp}(\nu)$ coefficients have been calculated and *saved*, then the *generation* of 50 sets of drawdowns corresponding to 50 different pumping patterns is easily obtained numerically from Eq.(7). Essentially 50 answers will be obtained practically for the price of one as compared with the traditional simulation approach.

Thus the response of the water table to any strategy of pumping at well p for any week is easily obtained by a simple algebraic relation. For a battery of P wells the solution (method of superposition) for the resulting drawdown is:

$$s_w(n) = \sum_{p=1}^{P} \sum_{\nu=1}^{n} \delta_{wp}(n-\nu+1)\; Q_p(\nu) \qquad (9)$$

The advantage of Eq.(9) over the repeated solution of:

$$\phi \frac{\partial s}{\partial t} - T\left(\frac{\partial s^2}{\partial x^2} + \frac{\partial s^2}{\partial y^2}\right) = \sum_{p=1}^{P} Q_p(\tau)\; \delta_p \qquad (10)$$

by a finite difference solution for all possible patterns of values for pumping, is obvious. In the case discussed above, the advantage is compounded by the fact that the Green function (another name for the kernel) was known analytically with little effort.

2. Solution for a General Aquifer (still without stream)

Since most aquifers are neither homogeneous nor of infinite extent, the analytical expression for the *pumping kernel* given by Eq.(5) is of no great value. What is important, however, is the fact that the classical theory of partial differential equations (Garabedian, 1964) has proven the existence of the kernel (Green) functions for heterogeneous aquifers of limited extent. That is in general the solution for the drawdown s_{wp} is still given by Eq.(6). Naturally $k_{wp}(\ )$ is no longer given by Eq.(5), cannot be obtained analytically but must be secured by some numerical procedure. With the proper kernel Eqs.(6), (7) (8) and (9) *remain valid.* A computer program was developed to secure directly the *discrete kernel* coefficients defined by Eq.(7). Appendix IB (published in a scientific journal) provides a fairly complete description of the theory behind the calculation of the *discrete kernels.* The reader (eventual user) need not study Appendix IB or the paper (Morel-Seytoux and Daly, 1975). For the eventual user it is important to know that: (1) a computer program to calculate the 'δ' ("*deltas*") was developed, (2) the program is operational, (3) the program was tested and found accurate, (4) deficiencies in the finite-difference scheme to calculate the *deltas* were found, (5) a method to correct for these deficiencies was developed and was successful, (6) the program *does have limitations*, (7) a detailed input description for use of the **DELTA** program is provided in this report and it describes the limitations of the program and (8) a complete example of preparation of input data for a sample situation is also provided.

The advantage of the methodology developed by the project over other approaches results from the following two facts:

(a) A finite difference model is used *only* to *generate* basic response functions to specialized excitations (e.g. pumping from a single well at a unit rate for the first period of time and no pumping thereafter) in an aquifer *without* any stream interaction. Once these basic response functions have been calculated for a particular aquifer and saved, simulation of the aquifer behavior to any pumping pattern is obtained without ever making use any longer of the (costly) numerical (e.g. finite difference) model, and

(b) Because the finite difference model is used only to generate the response functions (or influence coefficients) smaller grid sizes and time increments can be used to calculate *accurately* the influence coefficients than is usually feasible when performing a large number of simulation runs under many varied pumping patterns. Also with this procedure the accuracy of the calculations for an actual simulation *remains* that with which the influence coefficients were obtained. In typical simulation approaches the accuracy of the finite-difference model is usually tested against an analytical solution using small time and space increments. When the simulator is used on an actual aquifer, *vastly* different time and space increments are used and the accuracy of the results is to a large degree *unknown*.

3. Solution for a General Aquifer *With One Reach*

When a stream intersects the aquifer (see Figure 2) Eq.(2) still characterizes the evolution of the aquifer if it is understood that the sink terms (on the right-hand side) do not solely represent pumping wells but also discharge from the aquifer to the river in reach r (or recharge if Q_r is negative, i.e., a loss of surface water) or even recharge (negative Q_p) from an irrigation plot. In other words a reach can be viewed (at least mathematically) as a special well (pumping or injecting). It follows that the solution for drawdown due to a battery of P pumping wells and of a reach is of the form:

$$s_w(t) = \sum_{p=1}^{P} \int_0^t Q_p(\tau)\, k_{wp}(t-\tau)d\tau + \int_0^t Q_r(\tau)k_{wr}(t-\tau)d\tau \qquad (11)$$

Equation (11) involves two kernel functions, the pumping well kernel function (which is analytically known for the homogeneous aquifer of infinite extent and given by Eq.(5)) and the pervious reach kernel $k_{wr}(\ )$. For a homogeneous aquifer of infinite extent this kernel can also be obtained analytically (Morel-Seytoux, 1975) with the result that $k_{wr}(\ )$ has the same form as $k_{wp}(\ )$ given by Eq.(5) for $w \neq r$ and the special form for $w = r$:

$$k_{rr}(t-\tau) = \frac{1}{\phi ab}\left\{ \mathrm{erf}\ \frac{a}{2}\left[\frac{\phi}{4T(t-\tau)}\right]^{1/2} \right\} \mathrm{erf}\left\{\frac{b}{2}\left[\frac{\phi}{4T(t-\tau)}\right]^{1/2}\right\} \qquad (12)$$

where a is the length of the reach, b its width and erf( ) is the error function. Again since most aquifers are neither homogeneous nor of infinite extent, the analytical expression itself is of no great value. What is important, however, is the fact that the classical theory of partial differential equations (Garabedian, 1964) has proven the existence of the kernel functions for heterogeneous aquifers of limited extent. That is, Eq.(11) is *still valid under these more realistic conditions.*

Ignoring for the time being the fact that the kernel functions are not known (in the general case of heterogeneous aquifer of finite extent) it is still not possible to exploit

Eq.(11) because unlike the pumping rates $Q_p(\tau)$, the $Q_r(\tau)$ are not susceptible to man's control. In the case of pumping wells, the Q_p are decision variables, possibly limited by pumping capacities, but essentially controllable. In the case of discharge to the river (or recharge from the river) Q_r is a response to a decision. This response is related to the aquifer drawdown in reach r, $s_r(t)$ and to the water depth in reach r, $D_r(t)$. Explicitly, one can write:

$$Q_r(t) = \Gamma_r\left[H - \zeta_r - s_r(t) - D_r(t)\right] \tag{13}$$

where Γ_r is a coefficient of transmissivity characteristic of the reach, H is the distance between the two datum (see Fig.(1), ζ_r is the riverbed elevation above the (low) datum, and D_r is the river depth. It is convenient to define the drawdown to the river surface, namely (see Fig.(1):

$$\sigma_r(t) = H - \zeta_r - D_r(t) \tag{14}$$

In the design of regulations on pumping rates or in the evaluation of augmentation plans it is of great interest to the State Engineer (or planning authority, etc..) to know how much the return flow from the aquifer to the river will be reduced (or even how much water will be diverted by seepage from the river into the aquifer) due to the action of pumping from wells during the irrigation season. On the other hand the knowledge of drawdowns in the aquifer is of no special interest.

One therefore would like to predict return flow to the river without having to calculate drawdowns throughout the aquifer. This is possible. One can utilize Eq.(11) to calculate $s_r(t)$ and eliminate $s_r(t)$ from Eq.(13) with the result that $Q_r(t)$ is given as the solution of the equation:

$$Q_r(t) + \Gamma_r \int_0^t Q_r(\tau)k_{rr}(t-\tau)d\tau = \Gamma_r \left[\sigma_r(t) - \sum_{p=1}^{P} \int_0^t Q_p(\tau)k_{rp}(t-\tau)d\tau\right] \quad (15)$$

It is an integral equation. The exact analytical solution of this integral equation (which is linear of the Volterra type of the 2nd kind) is not feasible but its numerical solution is straightforward. Due to the linear character of the equation the solution for $Q_r(t)$ can be obtained by superposition of the solutions for the two situations $\sigma_r \neq 0$, $Q_p = 0$ and $\sigma_r = 0$, $Q_p \neq 0$. For example the State Engineer wants (primarily) to know the seepage from the river induced by pumping and not caused by other factors such as propagation of a flood. He is thus mostly interested in the solution of Eq.(15) when $\sigma_r(t) = 0$.

If for accuracy's sake the river is divided into several reaches, Eq.(15) generalizes to a system of R linear integral equations:

$$Q_r(t) + \Gamma_r \sum_{p=1}^{R} \int_0^t Q_p(\tau)k_{rp}(t-\tau)d\tau = \Gamma_r \left[\sigma_r(t) - \sum_{p=1}^{P} \int_0^t Q_p(\tau)k_{rp}(t-\tau)d\tau\right] \quad (16)$$

where R is the number of reaches. Equation (16) is a system of R integral equations to be solved simultaneously. Because the system is linear the solution of the discretized form of Eq.(16) (for $\sigma_r(t) = 0$), namely:

$$Q_r(n) + \Gamma_r \sum_{\rho=1}^{R} \sum_{\nu=1}^{n} \delta_{r\rho}(n-\nu+1)Q_r(\nu) + \Gamma_r \sum_{p=1}^{P} \sum_{\nu=1}^{n} \delta_{rp}(n-\nu+1)Q_p(\nu) = 0 \tag{17}$$

has necessarily a solution for $Q_r(n)$ of the form:

$$Q_r(n) = \sum_{p=1}^{P} \sum_{\nu=1}^{n} \varepsilon_{rp}(n-\nu+1)Q_p(\nu) \tag{18}$$

where $Q_r(n)$ is the return volume to the reach r during the n^{th} week, $Q_p(\nu)$ is the volume pumped at well p during the ν^{th} week, P is the total number of pumping wells, the δ_{wr} are defined in the same way the δ_{wp} were (see Eq.(8)) and the ε are coefficients. It is a tedious (see Appendix IB for details) but straightforward matter to substitute Eq.(18) into Eq.(17) to obtain a system of equations for the 'ε' ("epsilon") coefficients. The ε_{rp} coefficients are calculated recurrently as solutions of the system of equations:

$$\varepsilon_{rp}(n-\nu+1) + \Gamma_r \sum_{\rho=1}^{R} \sum_{m=\nu}^{n} \delta_{r\rho}(n-m+1)\varepsilon_{\rho p}(m-\nu+1) = -\Gamma_r \delta_{rp}(n-\nu+1) \tag{19}$$

Given the reach transmissivities, Γ_r , and the δ_{rp} coefficients (obtained from the 'delta' program) it is numerically a simple matter to calculate the *'epsilons'*. A computer program (the 'epsilon' program) was developed to calculate these coefficients. Once the δ_{rp} , δ_{rr} and ε_{rp} coefficients have been calculated it is a simple task to predict thereafter very simply from Eq.(18) the return flows to the river for any pumping pattern, i.e. for arbitrary values of the $Q_p(\nu)$ rates.

As was stated earlier a computer program to calculate the 'deltas' was developed. The meaning of these coefficients δ should be clear. If $s_{wp}(n)$ represents the drawdown at point w (anywhere in the aquifer) due to pumping at well p (anywhere in the aquifer) with pumped volumes $Q_p(1)$, $Q_p(2) \ldots, Q_p(\nu)$ etc. during weeks 1, 2, ... ν, etc. and with no stream interaction then:

$$s_{wp}(n) = \sum_{\nu=1}^{n} \delta_{wp}(n-\nu+1)\, Q_p(\nu) \qquad (20)$$

and $\delta_{wp}(n)$ represents the drawdown at the end of the n^{th} week (or time period) if a unit volume of water was withdrawn during the first week from well p and the well shut down indefinitely thereafter.

An additional advantage of the discrete kernel approach over the more traditional approaches is due to the fact that the interaction between the river and the aquifer at their interface is described *explicitly* by a finite difference form of Darcy's law. In most simulators these difference equations are *imbedded* within the set of difference equations which describe the aquifer. Here these equations are given a *special* treatment. They are separated from the aquifer difference equations. As a result the response of the stream-aquifer system is obtained from the knowledge of aquifer responses *without* stream and from the solution of a system of linear equations of smaller dimension (one order of magnitude smaller). For a river this advantage is not significant. However when a location of a canal (or recharge area) is considered in design stage, with the usual simulation approaches the response functions must be regenerated for each new location. With the kernel generation approach for the aquifer without stream, the basic influence coefficients are unchanged. It is only necessary to solve a new system of equations, again of smaller dimension (roughly N instead of N^2 in an $N \times N$ grid) for the composite stream-aquifer influence coefficients.

D. THE 'DELTA' PROGRAM

A computer program was developed to generate the "*discrete kernels*" $\delta_{wp}(n)$ and $\delta_{wr}(n)$. The subscript w refers to an (observation) point, which is any point in the grid system. The subscript p refers to a pumping well. The subscript r refers to a reach. The procedures to calculate the drawdown $\delta_{wp}(n)$ at the end of the n^{th} week (or time period) at point w due to pumping at well p and to calculate $\delta_{wr}(n)$ due to *pumping at reach* r are the same. In fact if both the p^{th} pumping well and the r^{th} reach are located in the same grid square only δ_{wp} is calculated since the δ_{wp} and δ_{wr} are then identical. Another distinction is made for observation wells. The discrete kernels $\delta_{w\pi}(n)$ are also calculated where the subscript π refers to an observation well. Some observation and pumping wells (or reaches) may coincide.

1. Accuracy and Cost of Program

After a computer program has been developed it is necessary to test that it is free of programming errors. One procedure (which is not full proof) is to compare the solution of the program with an analytical solution when such is available. For a homogeneous aquifer of infinite extent an analytical solution for drawdown due to pumping at a given well is known (see Eq.(3)). The computer program cannot model exactly this situation because the grid size must be of finite extent. However with a relatively extensive aquifer, the effect of the boundaries should not be felt for an observation well relatively close to the pumping well (located in the center of the grid) especially for early times following the impulse pumping at the central well.

Figure 3 displays the grid system used to perform the calculations. It is a grid with 45 columns and 31 rows (approximately one thousand grid points). For this grid the space interval Δx between grid points in the *finer* part of the grid system was 350m. The value of transmissivity used in the calculations was $T = 10{,}000\ m^2/\text{week}$ and the specific yield value was 0.2. Comparison of results of drawdowns at five observation points shown on Figure 3 and located at distances of 350, 1050, 1400, 2100 and 3150 meters respectively from the pumping well are shown on Figures 4 through 8. The agreement between the analytical and the finite difference results is satisfactory.

The agreement can be improved by reducing the grid size by a factor of 2 as is clear from Figures 4 through 8. The improvement however is minor but the cost of generating the $\delta_{wp}(n)$ for a *single well* for all the w points increased from \$3 to \$22. If one were interested only in a single well problem the change in cost is not significant. However if, as is more likely in actual problems of interest, there is a well at every grid point, the cost of generating the responses at all grid points from all wells jumps from \$3,000 (an already sizable number) to \$88,000! In practice it is unlikely that a grid size smaller than 350 meters will be used. Before pro-

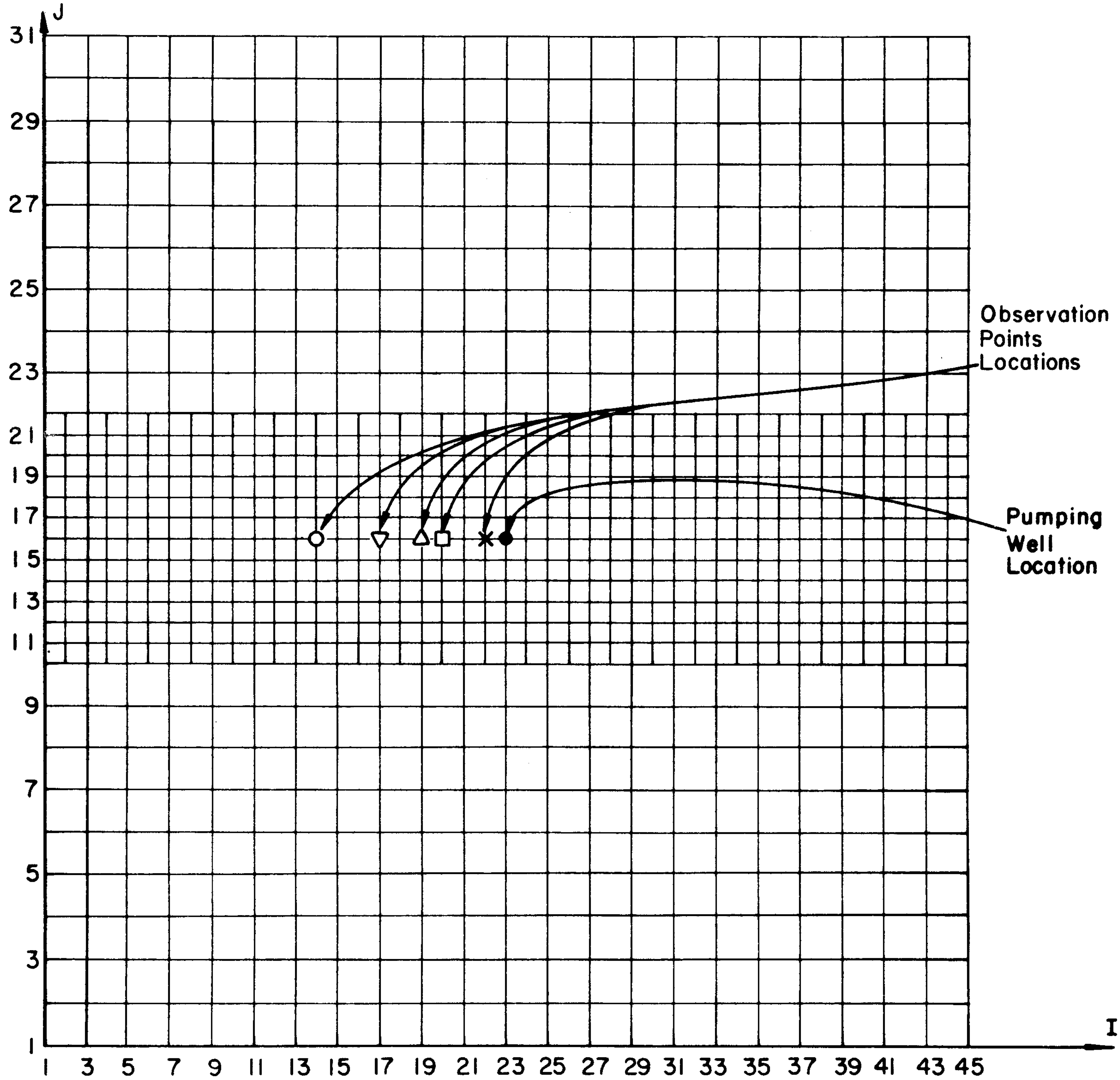

J
31
29
27
25
23
21
19
17
15
13
11
9
7
5
3
1
I
1 3 5 7 9 11 13 15 17 19 21 23 25 27 29 31 33 35 37 39 41 43 45
Observation Points Locations
Pumping Well Location

Figure 3

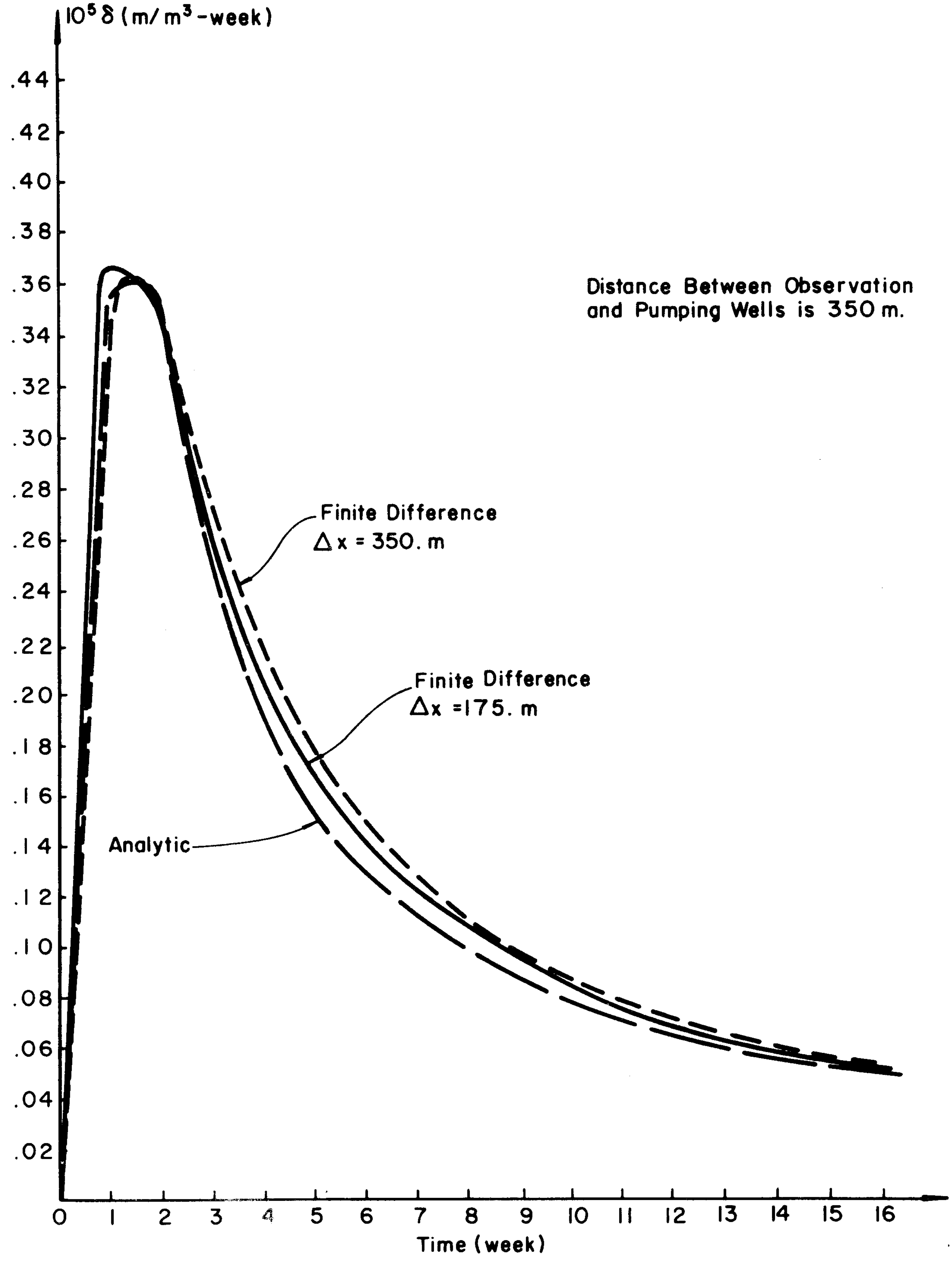

$10^5 \delta$ (m/m^3-week)
.44
.42
.40
.38
.36
.34
.32
.30
.28
.26
.24
.22
.20
.18
.16
.14
.12
.10
.08
.06
.04
.02
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Time (week)
Distance Between Observation and Pumping Wells is 350 m.
Finite Difference Δx = 350. m
Finite Difference Δx = 175. m
Analytic

Figure 4

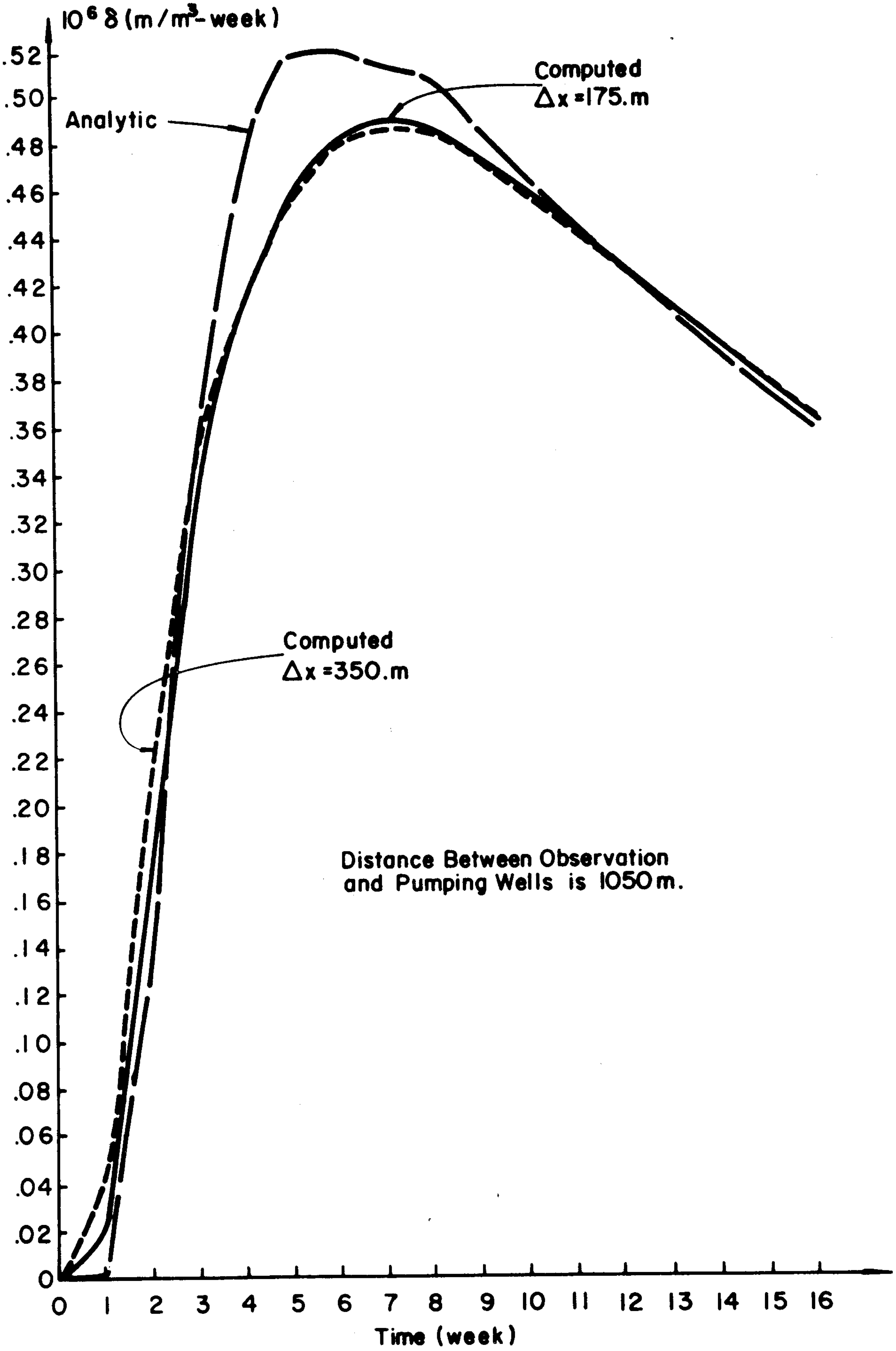

$10^6 \delta$ (m/m^3-week)
.52
.50
.48
.46
.44
.42
.40
.38
.36
.34
.32
.30
.28
.26
.24
.22
.20
.18
.16
.14
.12
.10
.08
.06
.04
.02
0
Analytic
Computed
Δx =175.m
Computed
Δx =350.m
Distance Between Observation
and Pumping Wells is 1050m.
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Time (week)

Figure 5

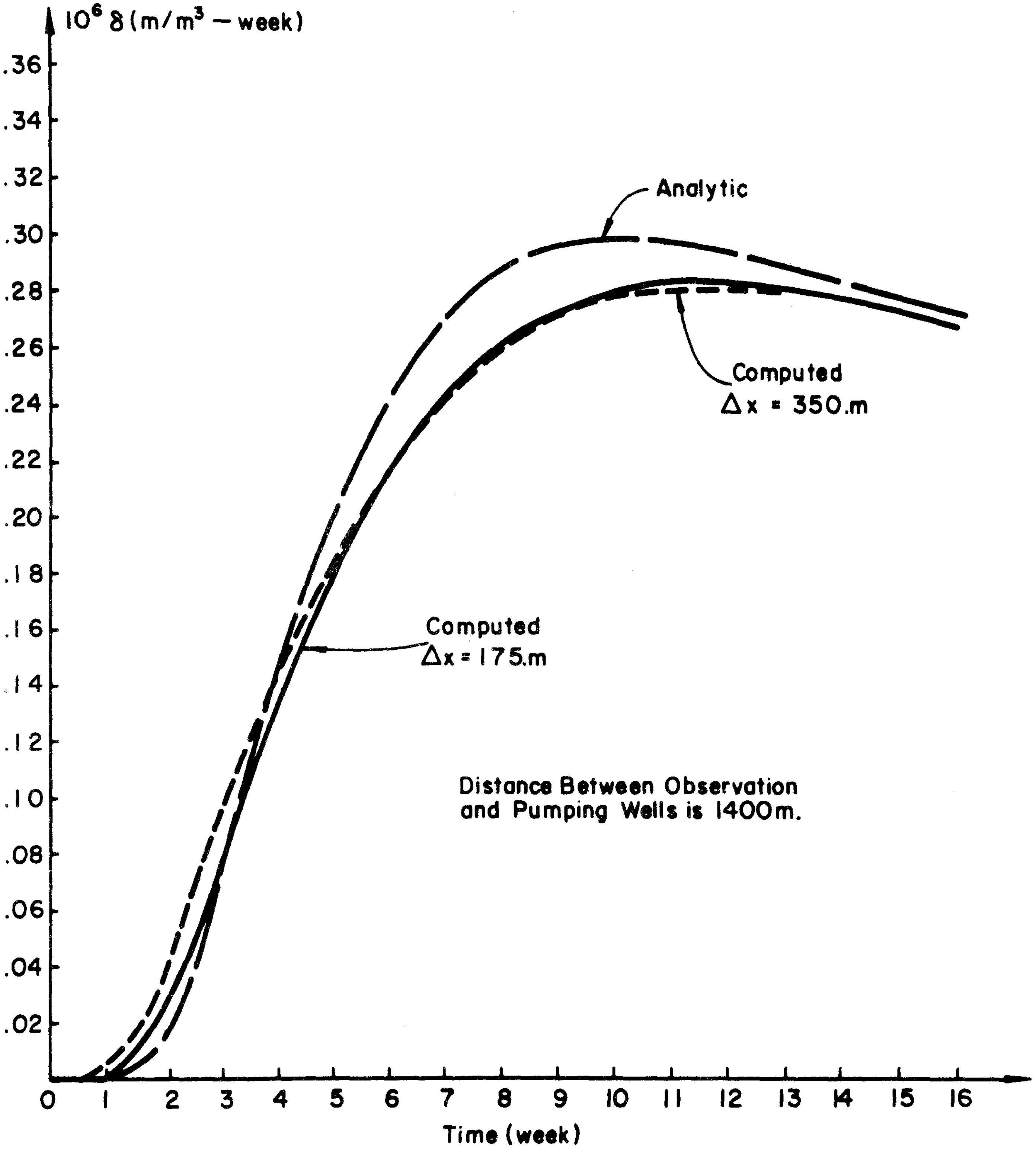

$10^6 \delta$ (m/m³ — week)
.36
.34
.32
.30
.28
.26
.24
.22
.20
.18
.16
.14
.12
.10
.08
.06
.04
.02
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Time (week)
Analytic
Computed
$\Delta x = 350.m$
Computed
$\Delta x = 175.m$
Distance Between Observation
and Pumping Wells is 1400m.

Figure 6

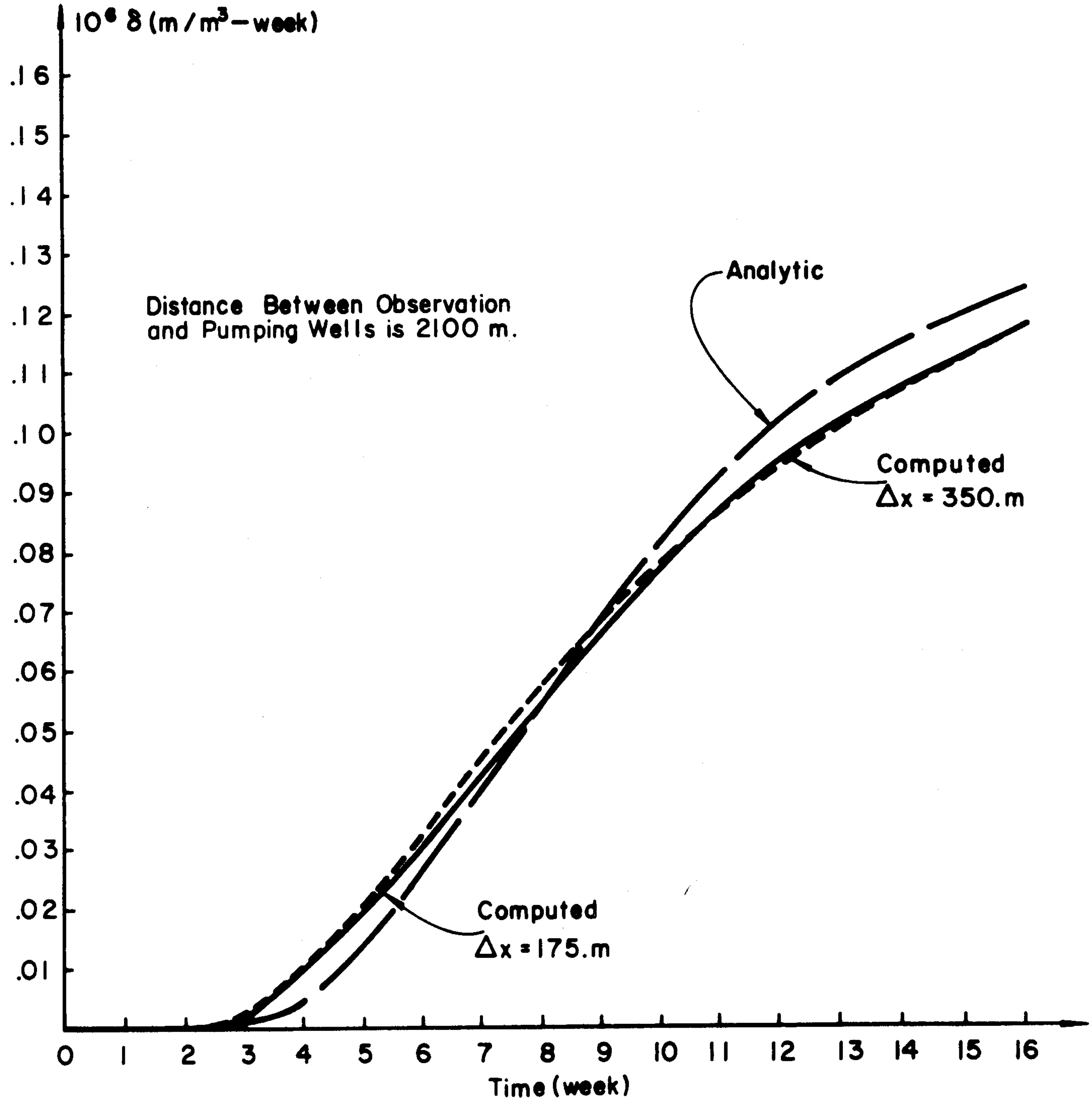

$10^6\ \delta$ (m / m^3 – week)
.16
.15
.14
.13
.12
.11
.10
.09
.08
.07
.06
.05
.04
.03
.02
.01
Distance Between Observation and Pumping Wells is 2100 m.
Analytic
Computed Δx = 350. m
Computed Δx = 175. m
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Time (week)

Figure 7

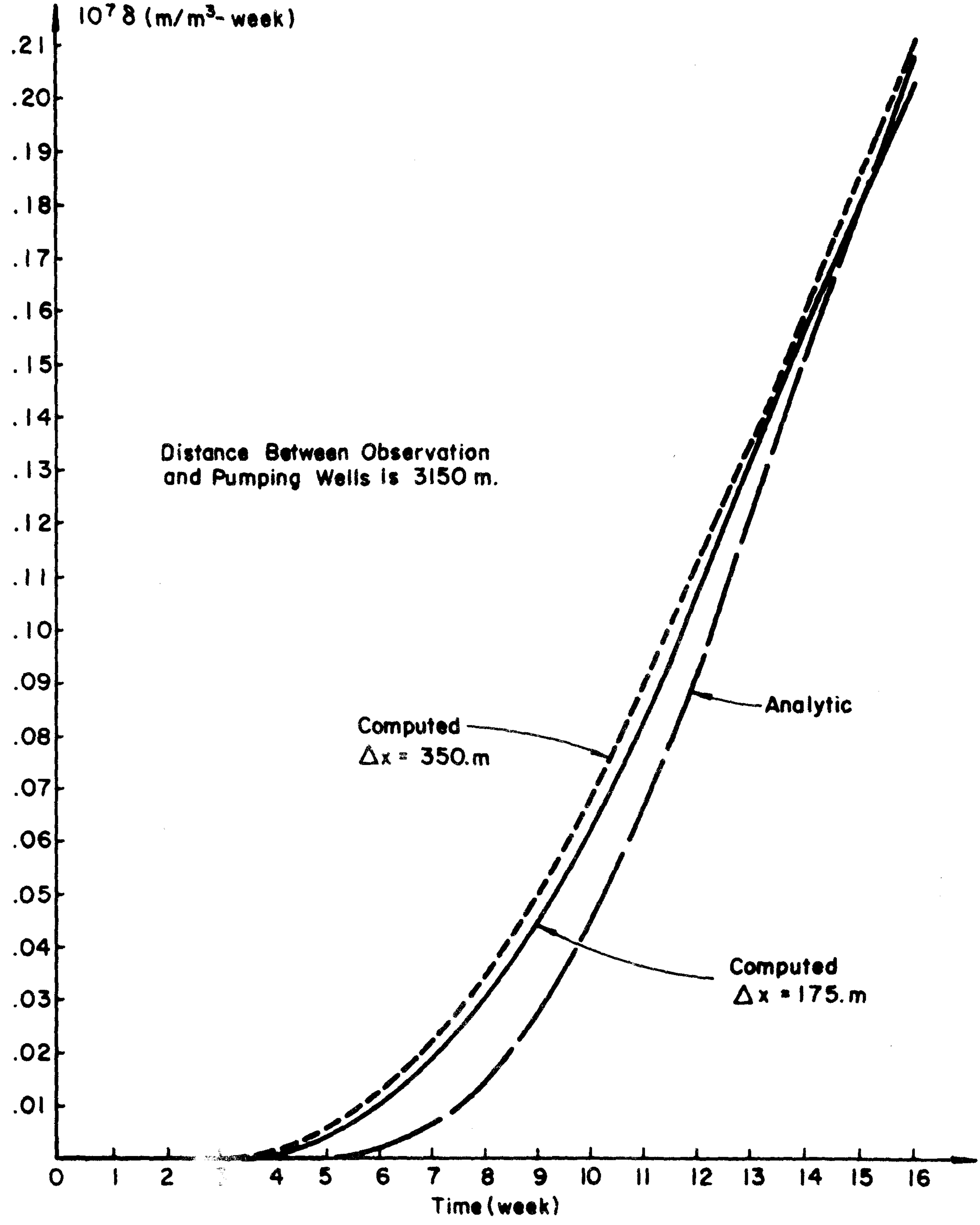

$10^7 \delta$ (m/m^3- week)
.21
.20
.19
.18
.17
.16
.15
.14
.13
.12
.11
.10
.09
.08
.07
.06
.05
.04
.03
.02
.01
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Time (week)
Distance Between Observation and Pumping Wells is 3150 m.
Computed Δx = 350. m
Analytic
Computed Δx = 175. m

Figure 8

ceeding with the calculations of all the $\delta_{wp}(n)$ for a complex aquifer system the user should probably perform this same check of accuracy for a representative value of transmissivity and a practical choice of grid size. The user can then decide on his trade-off point between accuracy and cost.

To check further the computer program a comparison of the results by the '*delta*' program with an analytical solution was performed for a heterogeneous case. The analytical solution corresponds to an indefinite continuous line sink of uniform strength located along the boundary of two homogeneous semi-infinite media with a sharp change of transmissivity at the interface. The geometry of the system is shown on Figure 9 with the selected grid for the computer finite difference solution. The overall network is 15,400 meters long and 10,500 meters wide. The analytical solution for the continuous and steady sink line is:

$$S_1(x,t) = \frac{2Q\sqrt{\frac{T_1}{\phi_1}\frac{T_2\,t}{\phi_2}}}{T_1\sqrt{\frac{T_2}{\phi_2}} + T_2\sqrt{\frac{T_1}{\phi_1}}}\ \text{ierfc}\left\{\frac{x}{2\sqrt{\frac{T_1\,t}{\phi_1}}}\right\} \tag{21}$$

$$S_2(x,t) = \frac{2Q\sqrt{\frac{T_1T_2\,t}{\phi_1\phi_2}}}{T_1\sqrt{\frac{T_2}{\phi_2}} + T_2\sqrt{\frac{T_1}{\phi_1}}}\ \text{ierfc}\left\{\frac{x}{2\sqrt{\frac{T_2\,t}{\phi_2}}}\right\} \tag{22}$$

where $S_i(x,t)$ is drawdown at abscissa x, shortest distance between the observation point and the sink line and at time t

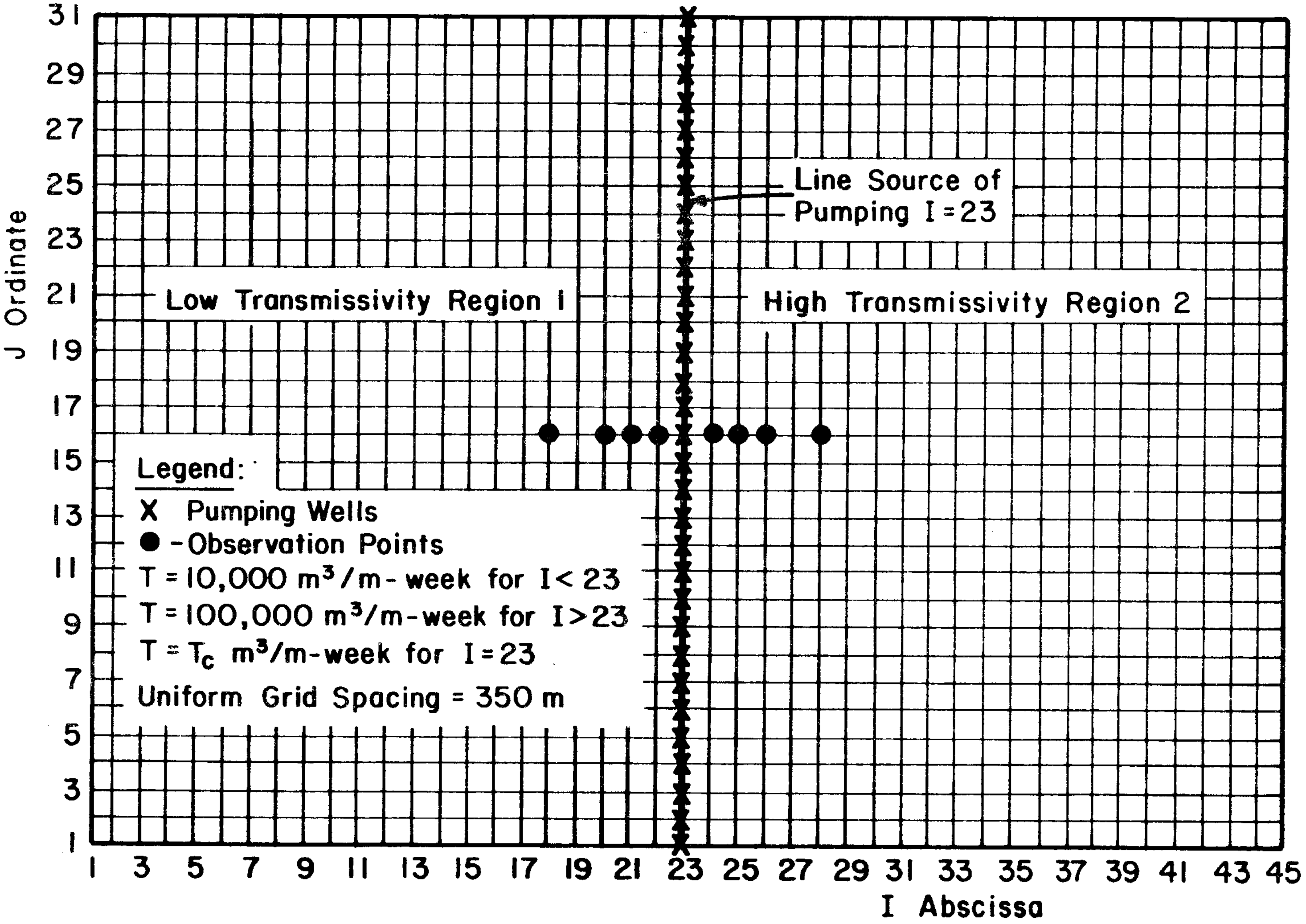

31
29
27
25
23
21
19
17
15
13
11
9
7
5
3
1
J Ordinate
1 3 5 7 9 11 13 15 17 19 21 23 25 27 29 31 33 35 37 39 41 43 45
I Abscissa
Line Source of Pumping I = 23
Low Transmissivity Region 1
High Transmissivity Region 2
Legend:
X Pumping Wells
● - Observation Points
T = 10,000 m³/m-week for I < 23
T = 100,000 m³/m-week for I > 23
T = Tc m³/m-week for I = 23
Uniform Grid Spacing = 350 m

Figure 9

in region i $(i = 1,2)$, Q is the pumping strength of the line of sinks i.e. pumping rate per unit length, and ierfc( ) is the integral complementary error function (Carslaw and Jaeger, 1959), which is tabulated (Jahnke and Emde, 1945). Eqs.(21) and (22) provide a continuous solution for a steady pumping. The drawdowns $s_i(x,n)$ due to a unit pumping volume during the first week (period) and *no pumping thereafter* are obtained (from the principle of superposition) by the relation:

$$s_i(x,n) = S_i(x,n) - S_i(x,n-1) \qquad \begin{array}{l} i = 1,2 \\ n = 1,2...N \end{array} \qquad (23)$$

with the choice for $Q = \frac{1}{\Delta y} = \frac{1}{\Delta x} = \frac{1}{350}$ m^3/m, so that the volume withdrawn by the sink line within a cell in the analytical solution and the volume withdrawn by the pumping well in the finite-difference solution are the same.

In the finite difference model a transmissivity value must be chosen for the grid points which are exactly on the interface between the two regions. How should that value, denoted T_c, be related to T_1 and T_2? Figure 10 shows that the results of the finite difference calculations for drawdown in the high transmissivity region depend very much on the choice of that central transmissivity value, T_c. Even with the better choices ($T_c = 30{,}000$ or $T_c = 40{,}000$) the fit is not good compared to the very good fit presented earlier for the case of a homogeneous medium. Figure 11 shows the results for the drawdowns in the low transmissivity region. In this case, the

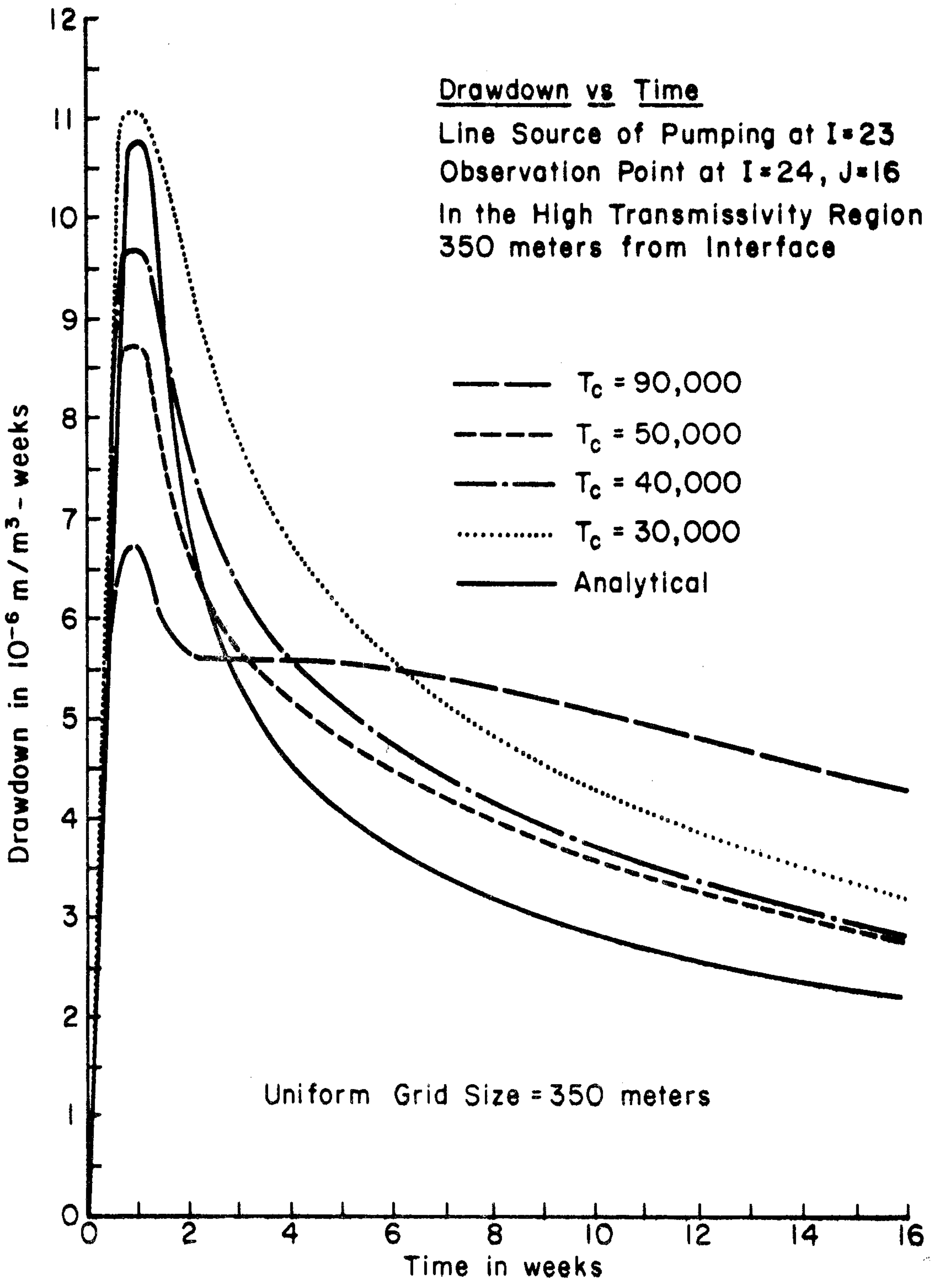

Drawdown vs Time
Line Source of Pumping at I=23
Observation Point at I=24, J=16
In the High Transmissivity Region
350 meters from Interface
T_c = 90,000
T_c = 50,000
T_c = 40,000
T_c = 30,000
Analytical
Uniform Grid Size = 350 meters
Drawdown in 10^{-6} m/m^3 - weeks
Time in weeks
0
1
2
3
4
5
6
7
8
9
10
11
12
0
2
4
6
8
10
12
14
16

Figure 10

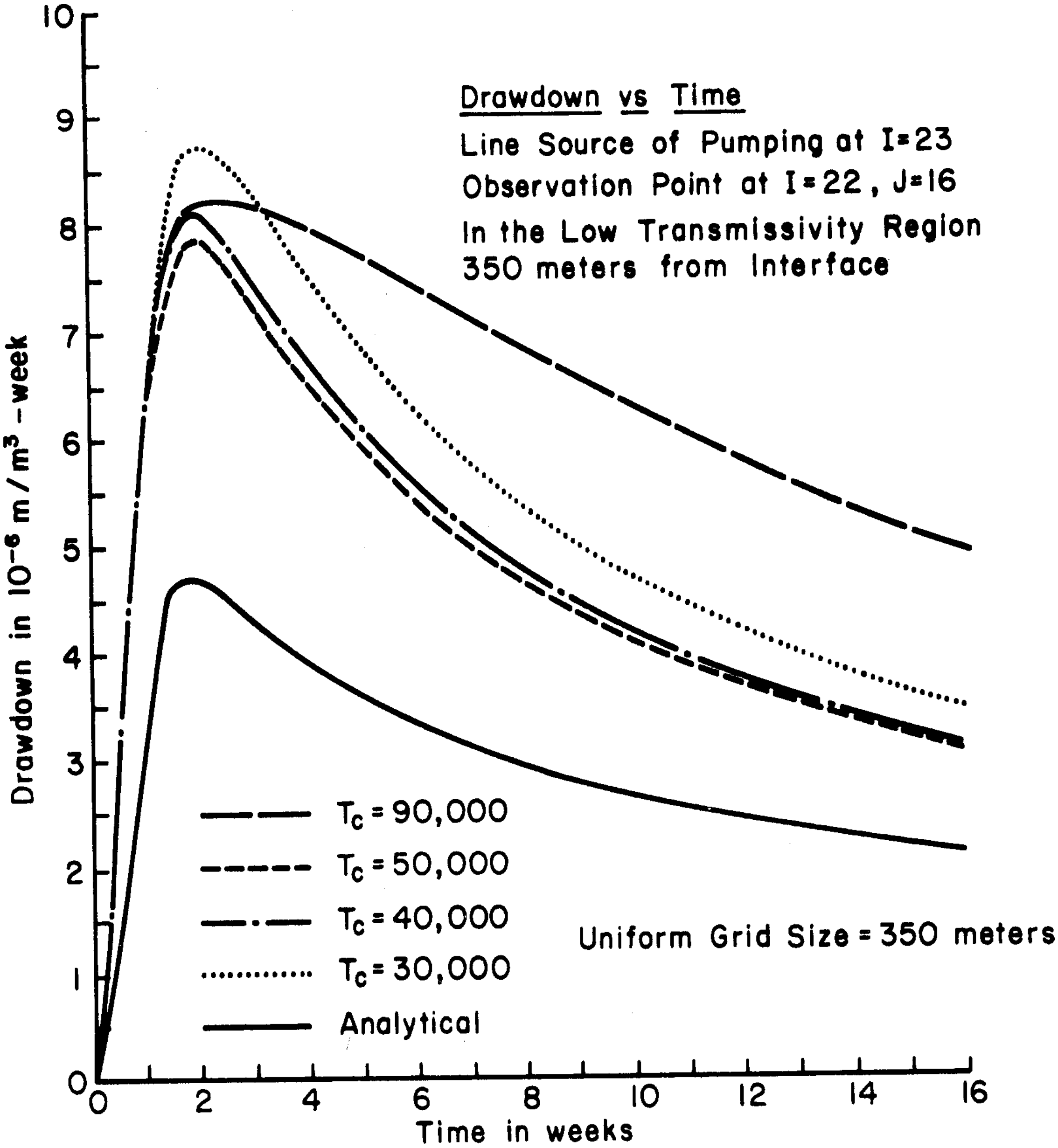

Drawdown vs Time
Line Source of Pumping at I=23
Observation Point at I=22, J=16
In the Low Transmissivity Region
350 meters from Interface
T_c = 90,000
T_c = 50,000
T_c = 40,000
T_c = 30,000
Analytical
Uniform Grid Size = 350 meters
Drawdown in 10^{-6} m / m^3 - week
Time in weeks
0
1
2
3
4
5
6
7
8
9
10
0
2
4
6
8
10
12
14
16

Figure 11

disagreement is striking and is not affected by the choice of value for T_c. Figures 12 and 13 display similar results for observation points located on either side of the sink line, 700 meters away from it.

Since the discrete set of wells and the continuous sink line are not exactly equivalent, the solutions for the homogeneous case were also compared. Figures 14, 15, 16 and 17 display a good agreement. It was concluded that the lack of fit in Figures 10, 11, 12 and 13 could not be blamed on the discrete representation in the finite difference model of the continuous sink line in the analytical solution.

In the original finite difference equations used in the Delta program, the flux (see Figure 18) across the boundary between two grid points was estimated as:

$$q = \frac{(T_1+T_2)}{2} \, \frac{(h_1-h_2)}{\Delta x} = \left(\frac{T_1+T_2}{2}\right)\frac{\Delta h}{\Delta x} \qquad (24)$$

If a sharp change in transmissivity occurred right at the interface between cell 1 and cell 2 then the application of Darcy's law yields the relations:

$$q = T_1\left(\frac{h_1-h_b}{\frac{\Delta x}{2}}\right) \qquad (25)$$

$$q = T_2\left(\frac{h_b-h_2}{\frac{\Delta x}{2}}\right) \qquad (26)$$

Elimination of h_b between these two equations yields:

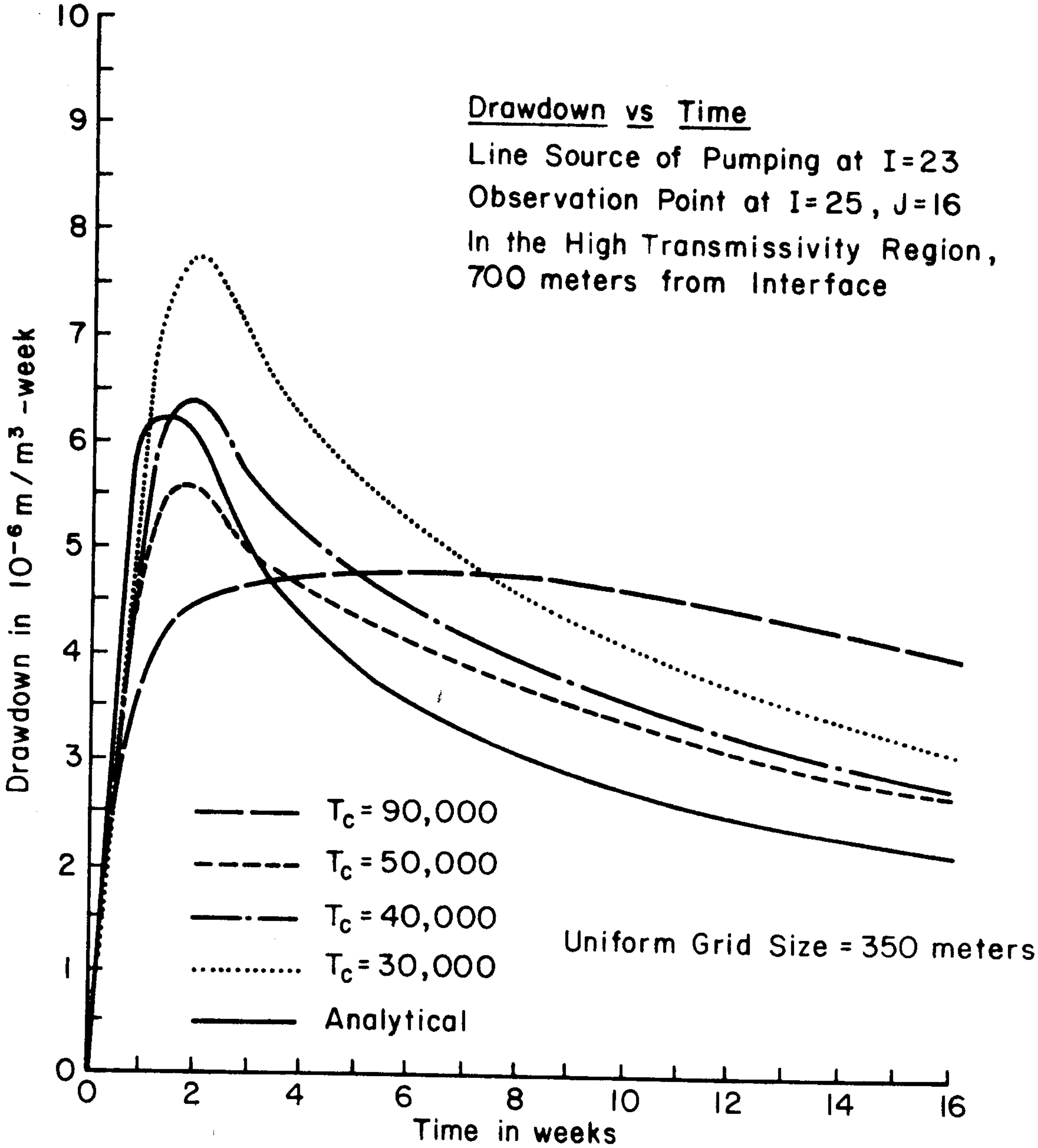

Drawdown vs Time
Line Source of Pumping at I=23
Observation Point at I=25, J=16
In the High Transmissivity Region,
700 meters from Interface
Drawdown in 10^{-6} m / m^3 -week
Time in weeks
0
1
2
3
4
5
6
7
8
9
10
0
2
4
6
8
10
12
14
16
T$_c$ = 90,000
T$_c$ = 50,000
T$_c$ = 40,000
T$_c$ = 30,000
Analytical
Uniform Grid Size = 350 meters

Figure 12

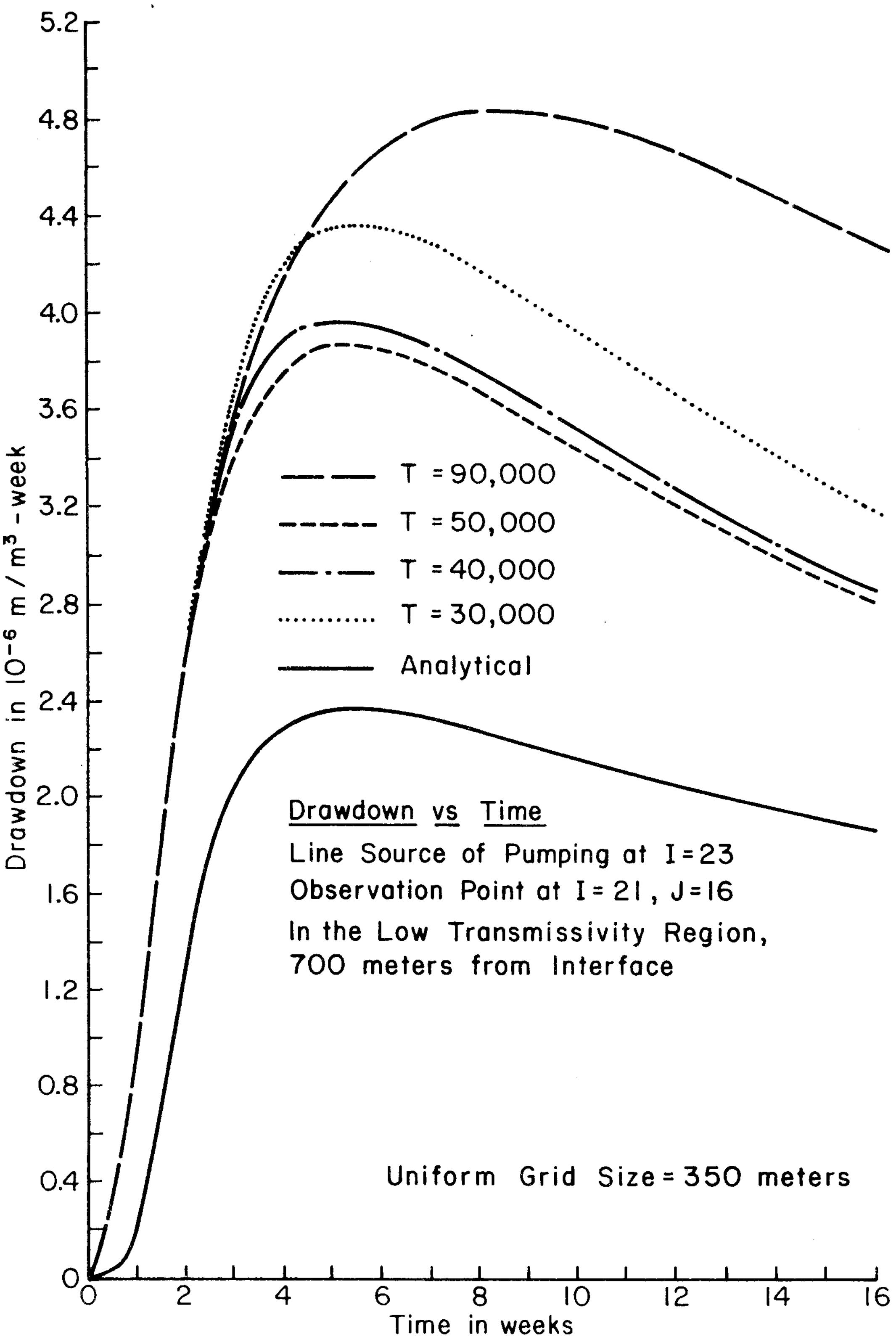

5.2
4.8
4.4
4.0
3.6
3.2
2.8
2.4
2.0
1.6
1.2
0.8
0.4
0
Drawdown in 10^{-6} m / m^3 -week
0
2
4
6
8
10
12
14
16
Time in weeks
T = 90,000
T = 50,000
T = 40,000
T = 30,000
Analytical
Drawdown vs Time
Line Source of Pumping at I = 23
Observation Point at I = 21, J = 16
In the Low Transmissivity Region,
700 meters from Interface
Uniform Grid Size = 350 meters

Figure 13

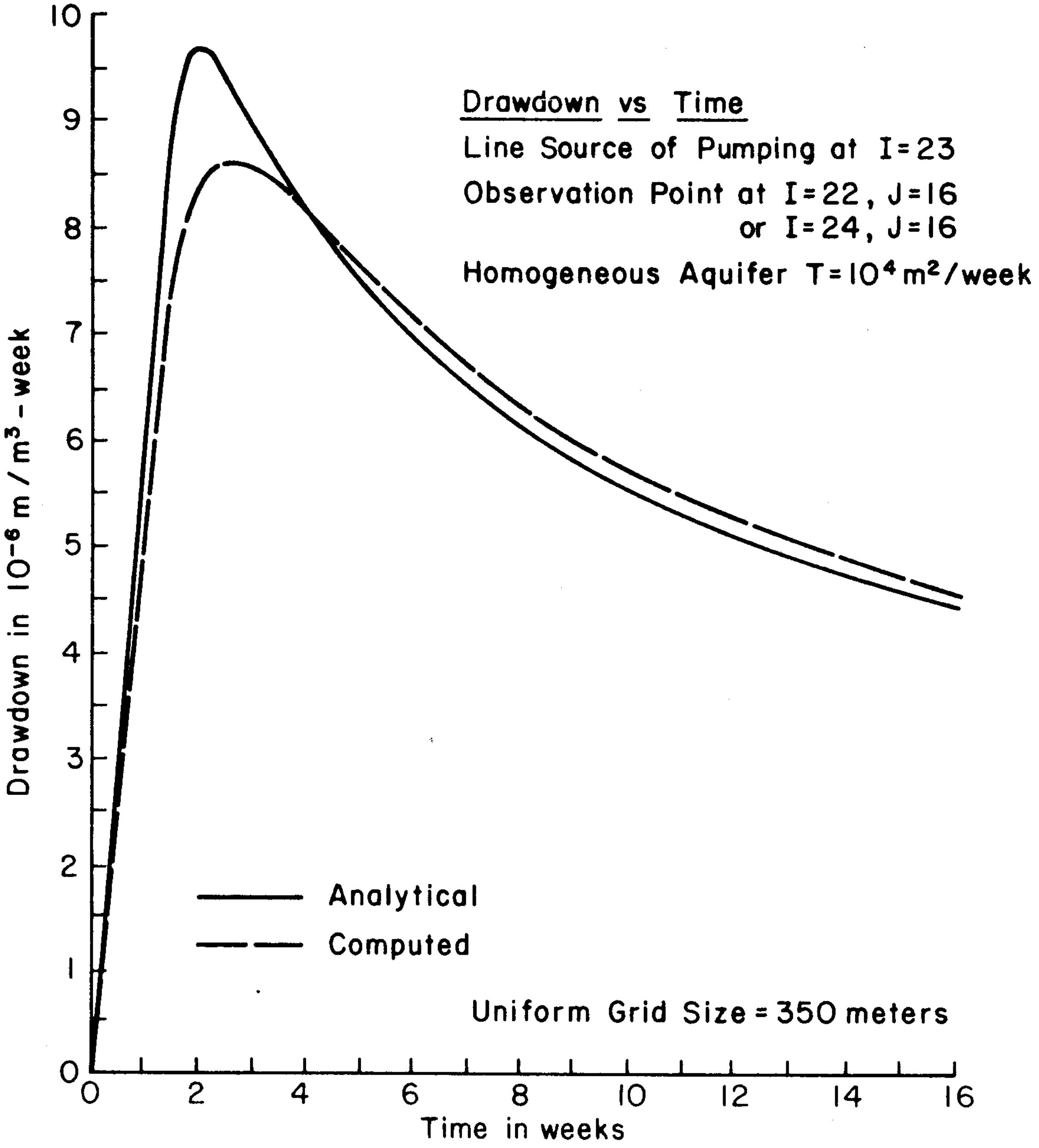

Drawdown vs Time
Line Source of Pumping at I=23
Observation Point at I=22, J=16
or I=24, J=16
Homogeneous Aquifer T=10⁴m²/week
Analytical
Computed
Uniform Grid Size=350 meters
Drawdown in 10⁻⁶ m / m³ - week
Time in weeks
0
1
2
3
4
5
6
7
8
9
10
0
2
4
6
8
10
12
14
16

Figure 14

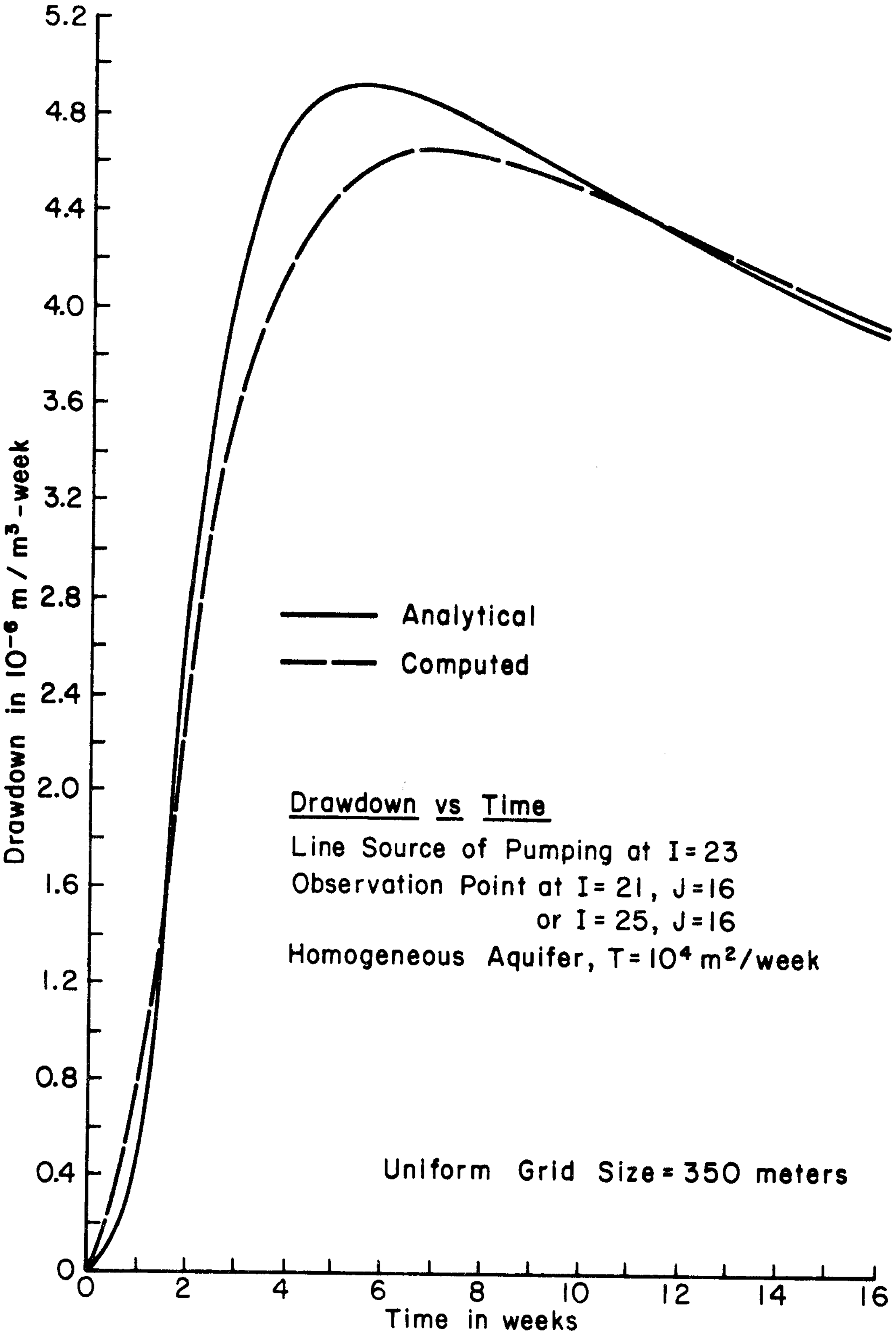
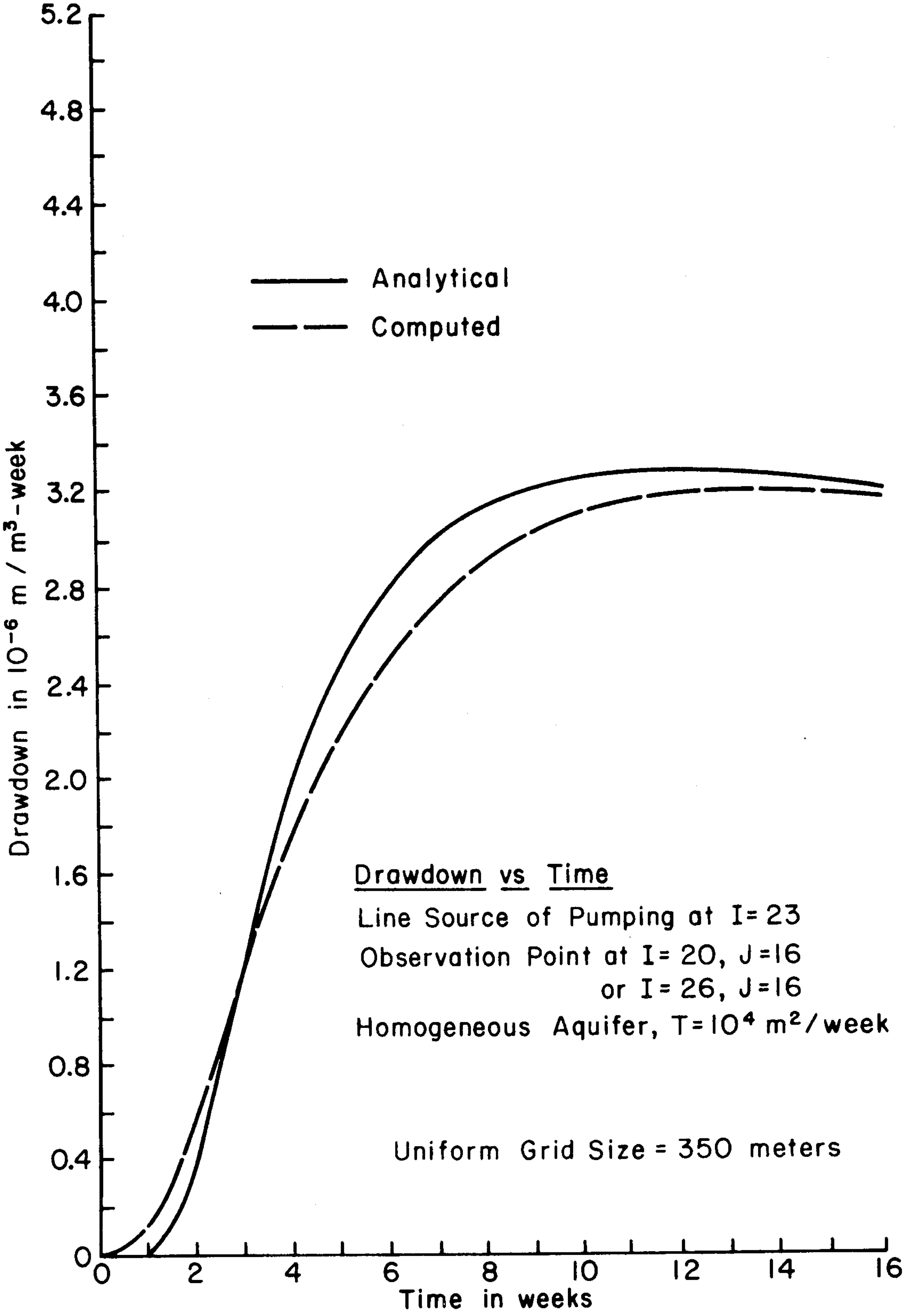

5.2
4.8
4.4
4.0
3.6
3.2
2.8
2.4
2.0
1.6
1.2
0.8
0.4
0
Drawdown in 10^{-6} m / m^3-week
0
2
4
6
8
10
12
14
16
Time in weeks
Analytical
Computed
Drawdown vs Time
Line Source of Pumping at I = 23
Observation Point at I = 21, J = 16
or I = 25, J = 16
Homogeneous Aquifer, T = 10^4 m^2/week
Uniform Grid Size = 350 meters

Figure 15



5.2
4.8
4.4
4.0
3.6
3.2
2.8
2.4
2.0
1.6
1.2
0.8
0.4
0
Drawdown in 10^{-6} m / m^3-week
Analytical
Computed
Drawdown vs Time
Line Source of Pumping at I= 23
Observation Point at I= 20, J=16
or I= 26, J=16
Homogeneous Aquifer, T= 10^4 m^2/week
Uniform Grid Size = 350 meters
0
2
4
6
8
10
12
14
16
Time in weeks

Figure 16

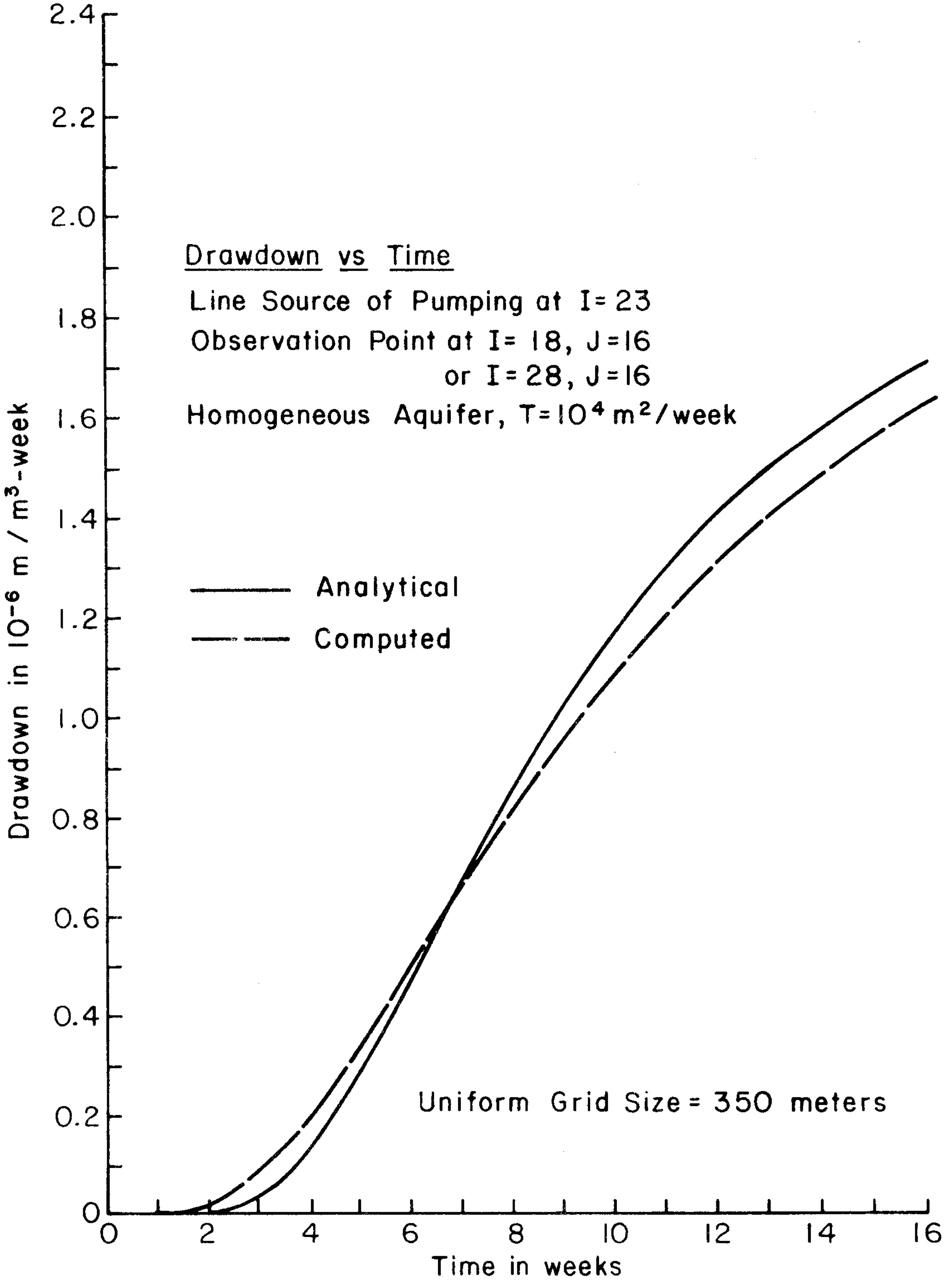

Drawdown vs Time
Line Source of Pumping at I= 23
Observation Point at I= 18, J=16
or I=28, J=16
Homogeneous Aquifer, T=10⁴ m²/week
Analytical
Computed
Uniform Grid Size = 350 meters
Drawdown in 10⁻⁶ m / m³-week
Time in weeks
2.4
2.2
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0
0
2
4
6
8
10
12
14
16

Figure 17

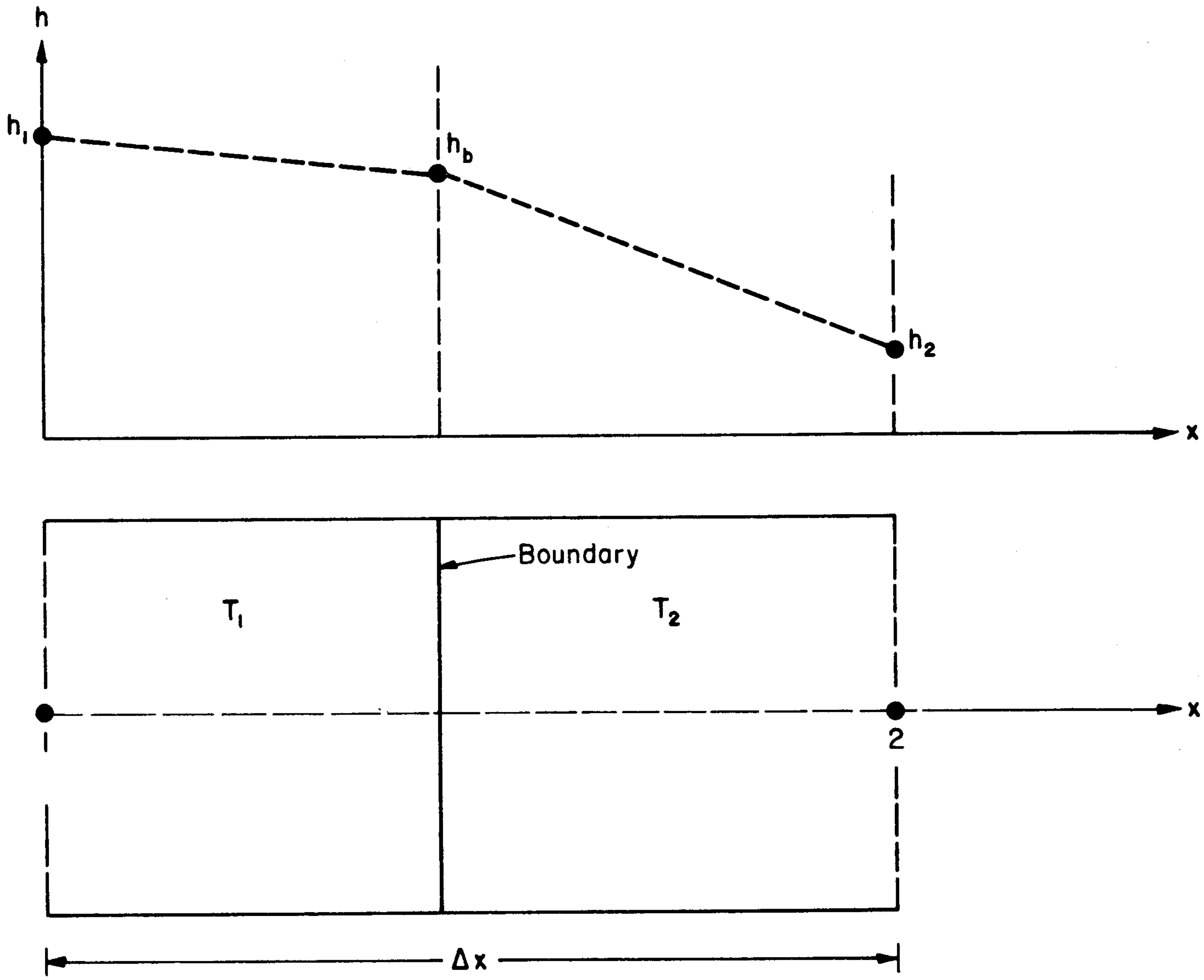


Figure 18

$$q = \frac{2T_1T_2}{T_1+T_2} \frac{(h_1-h_2)}{\Delta x} = \frac{2T_1T_2}{T_1+T_2} \frac{\Delta h}{\Delta x} \qquad (27)$$

Since this equation is exact, it shows that the flow across the interface between two media of different transmissivities for a given drop in head (Δh) is the same as the one that would take place in a homogeneous medium of transmissivity, $T_c = \frac{2T_1T_2}{T_1+T_2}$ or in other words:

$$\frac{2}{T_c} = \frac{1}{T_1} + \frac{1}{T_2} \qquad (28)$$

In other words the mean transmissivity at the boundary should be the harmonic mean rather than the arithmetic mean of the transmissivities on the two sides of the boundary. Once this modification of the finite difference equations was incorporated in the Delta program, the difficulty to a large degree disappears as is shown on Figures 19 and 20. The improvement in accuracy is clear. The relative error is still larger than the relative error displayed on Figures 14 and 15. The larger error is probably due to the fact that in the heterogeneous situation it is not possible to model the abrupt interface between the two regions of transmissivities T_1 and T_2 exactly.

2. Selection of a Grid

If the reader (eventual user) is satisfied that the existing 'DELTA' program is computing the $\delta_{wp}(n)$ coefficient with adequate accuracy and at a reasonable cost, he will now want to

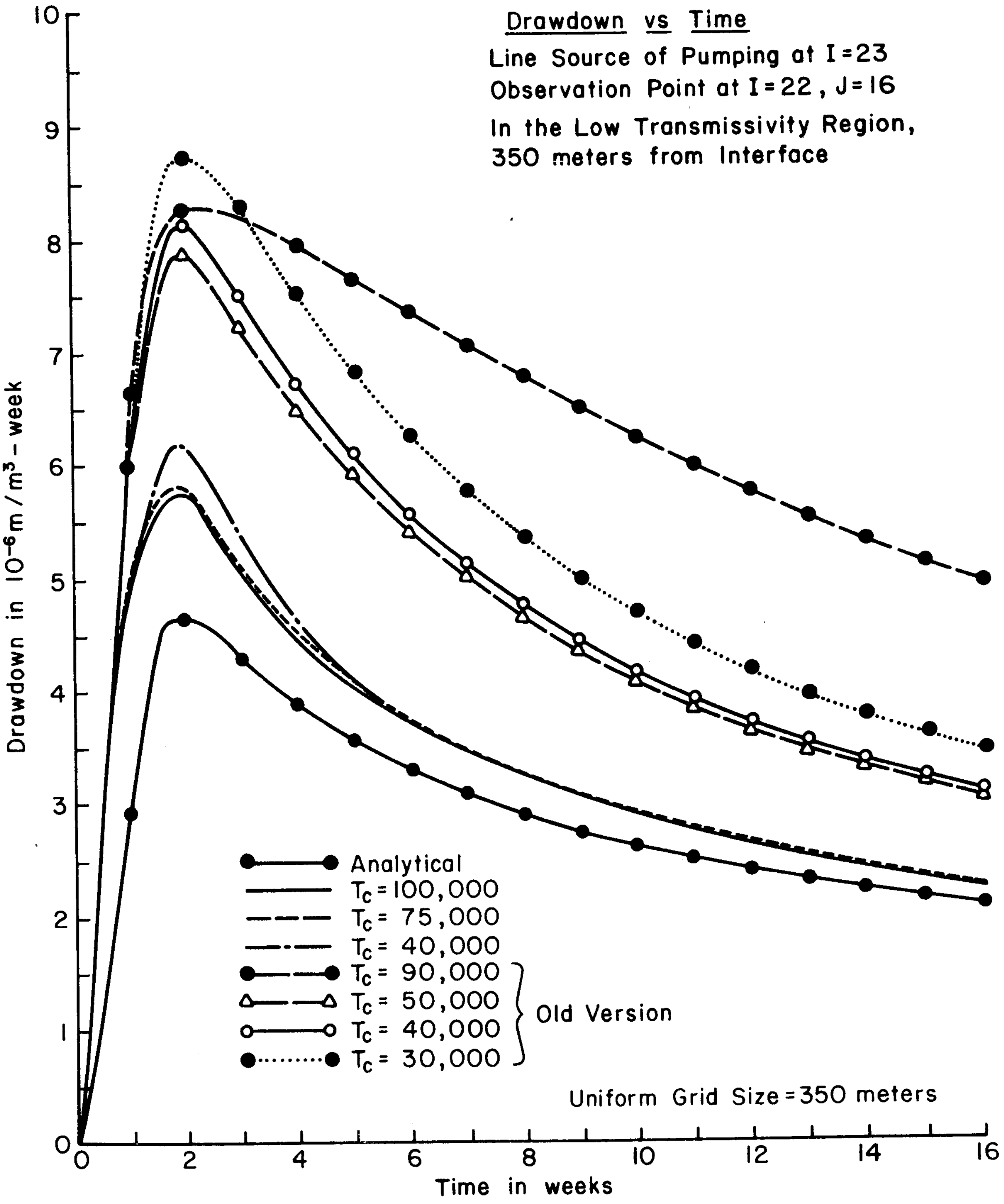

Drawdown vs Time
Line Source of Pumping at I=23
Observation Point at I=22, J=16
In the Low Transmissivity Region,
350 meters from Interface
Drawdown in 10⁻⁶m/m³-week
Time in weeks
0
1
2
3
4
5
6
7
8
9
10
0
2
4
6
8
10
12
14
16
Analytical
Tc=100,000
Tc= 75,000
Tc= 40,000
Tc= 90,000
Tc= 50,000
Tc= 40,000
Tc= 30,000
Old Version
Uniform Grid Size=350 meters

Figure 19

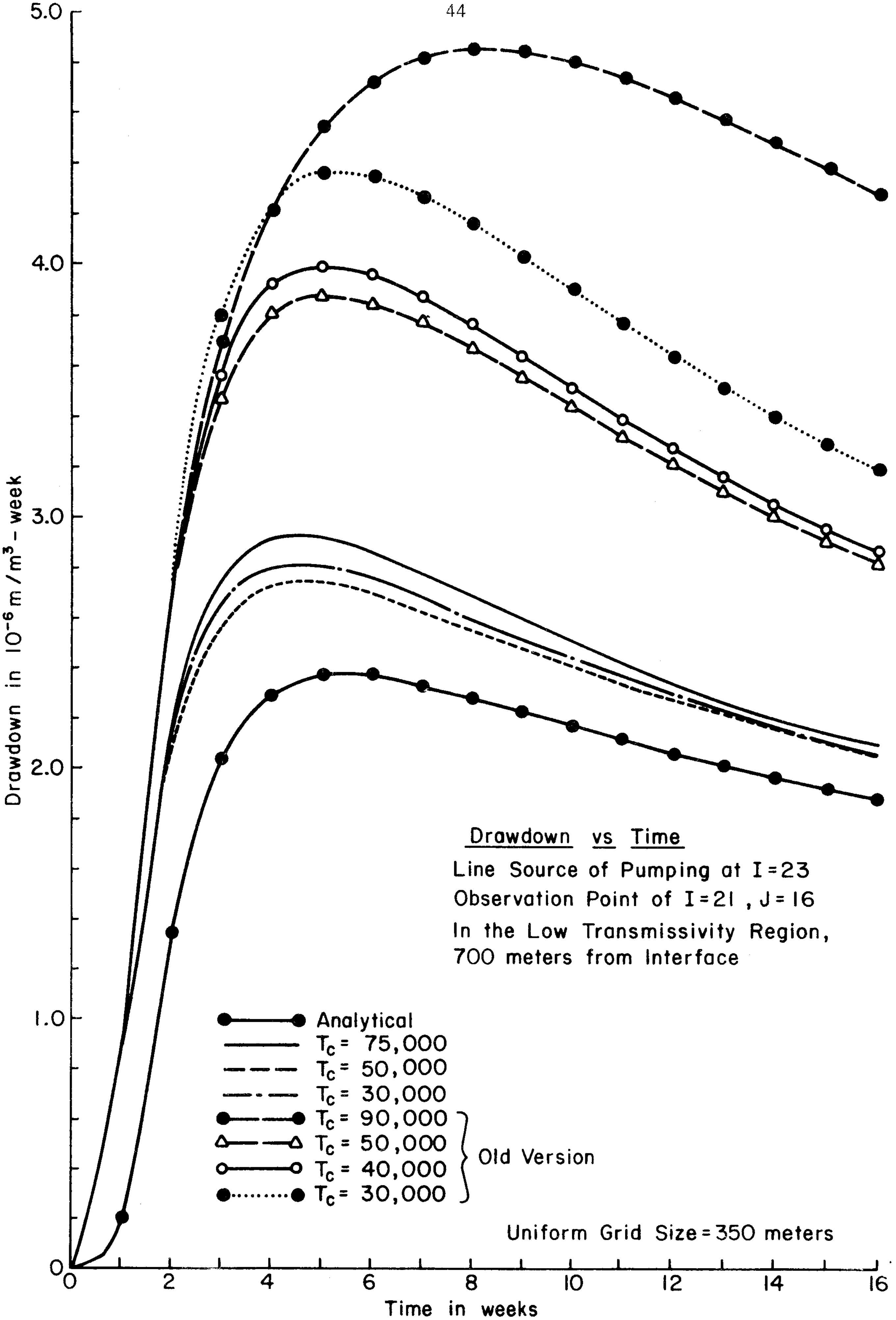

5.0
4.0
3.0
2.0
1.0
0
Drawdown in 10^{-6} m / m^3 - week
0
2
4
6
8
10
12
14
16
Time in weeks
Drawdown vs Time
Line Source of Pumping at I=23
Observation Point of I=21 , J=16
In the Low Transmissivity Region,
700 meters from Interface
Analytical
T_c = 75,000
T_c = 50,000
T_c = 30,000
T_c = 90,000
T_c = 50,000
T_c = 40,000
T_c = 30,000
Old Version
Uniform Grid Size = 350 meters

Figure 20

know how to use the 'DELTA' program. The first step is the selection of a grid to superimpose on the actual aquifer or stream-aquifer system.

Figure 21 shows an alluvial aquifer and its boundary as well as a meandering stream. On Figure 21 the aquifer boundary is shown. Also shown is the grid network. The intersects of horizontal and vertical grid lines are calculation points in the computer program. The calculated values represent average values for the square (or rectangle if the grid size changes) of influence of the grid point. Thus the grid influence boundary extends away in the horizontal and vertical direction one half grid space. As much as possible the actual aquifer boundary should be within the grid influence boundary as shown on Figure 21. The square of influence of grid point A is shown on Figure 21. Both an observation point and a well fall within this square of influence. Consequently the I, J coordinates of the observation point and of the well are exactly those of grid point A.

The river reaches, which are in fact continuous line sinks, are treated as discrete sinks (or sources). Every curvilinear segment of the river within a square (or rectangle) of influence of a grid point is treated as a separate reach, given a number and assumed to behave like a discrete sink (or source) located at the grid point. Thus the coordinates of reach 2 in Figure 22 are the same as those of grid point B. In Figure 22 all observation and well points have already been *relocated* at the proper grid points.

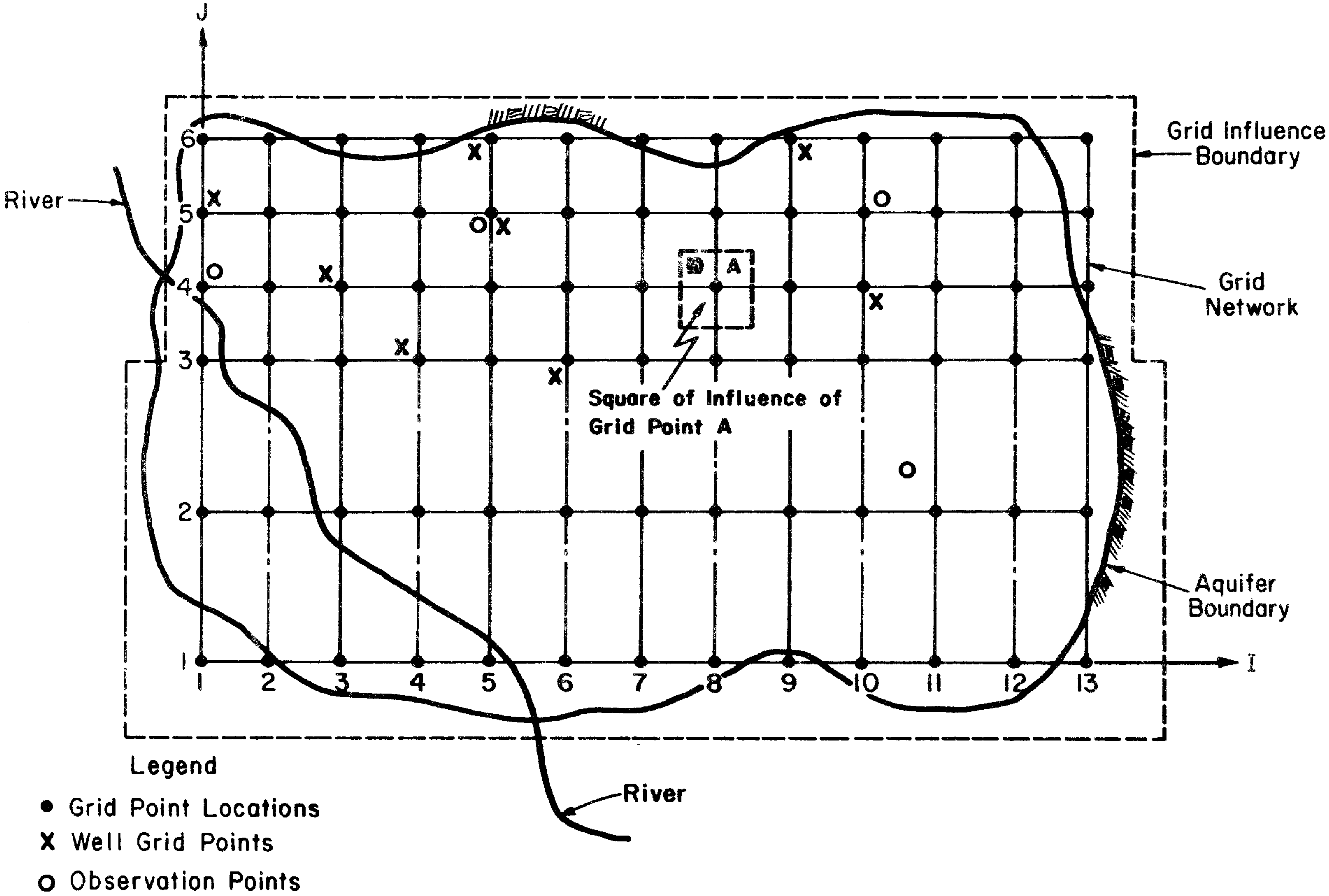

J
I
6
5
4
3
2
1
1
2
3
4
5
6
7
8
9
10
11
12
13
River
River
Grid Influence Boundary
Grid Network
Aquifer Boundary
A
Square of Influence of Grid Point A
Legend
● Grid Point Locations
X Well Grid Points
O Observation Points

Figure 21

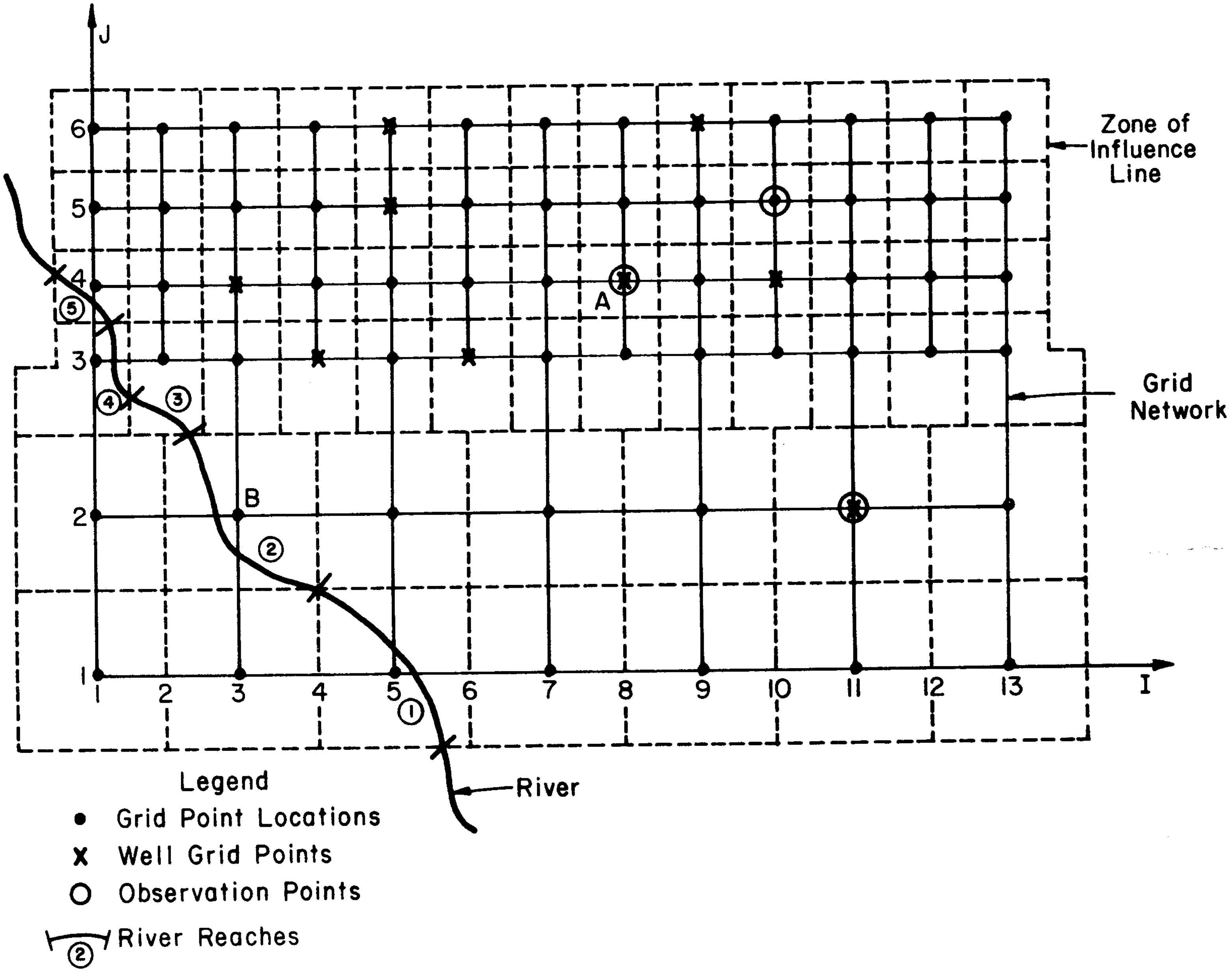

J
6
5
4
3
2
1
1
2
3
4
5
6
7
8
9
10
11
12
13
I
A
B
①
②
③
④
⑤
Zone of Influence Line
Grid Network
River
Legend
● Grid Point Locations
X Well Grid Points
O Observation Points
② River Reaches

Figure 22

The current DELTA program has a limitation for location of observation, reach or well points. None can be located nearer than 4 grid points away from the right boundary of the grid network. If wells are located close to the boundary (see Figure 23) then it is necessary to extend the grid network to the right with artificial grid points (the Z points on Figure 23). At the Z points the transmissivity values are <u>zero</u>. Thus on Figure 23 grid point A is a permissible well point. However grid point B is not, which is alright in this case since there is no well at point B. Point C is a permissible well point.

3. <u>Capability of the DELTA Program (Stage 1)</u>

Currently the program can be used as a '*generator*' of the discrete kernels, the δ_{wp}, δ_{wr}, $\delta_{w\pi}$. In addition it can be used, once the δ have been calculated, as a '*simulator*', that is drawdowns at various points and various times can be calculated for given pumping patterns. These two separate functions of '*generator*' and '*simulator*' can be utilized within the same run. It must be emphasized that the drawdowns thus calculated are the drawdowns that would take place if there were no reaches within the system. If indeed there is no river in the system or no hydraulic connection between the stream and the aquifer, the drawdowns are the real ones. Incidentally the DELTA program can be used to study a *confined aquifer*. It suffices to enter values of storage coefficient in place of effective porosity (specific yield). The resulting drawdowns are hydraulic head drawdowns rather than water table drawdowns.

If a stream does intersect the aquifer, the calculated drawdowns are not '*real*'. They only provide a reference level to assess later the relative effect of the stream-aquifer interaction on drawdowns in the aquifer. The real drawdowns are calculated in a following program (the second stage) called the 'EPSILON' program. Clearly for a true stream-aquifer system it is not meaningful to run the first stage without running the second stage later.

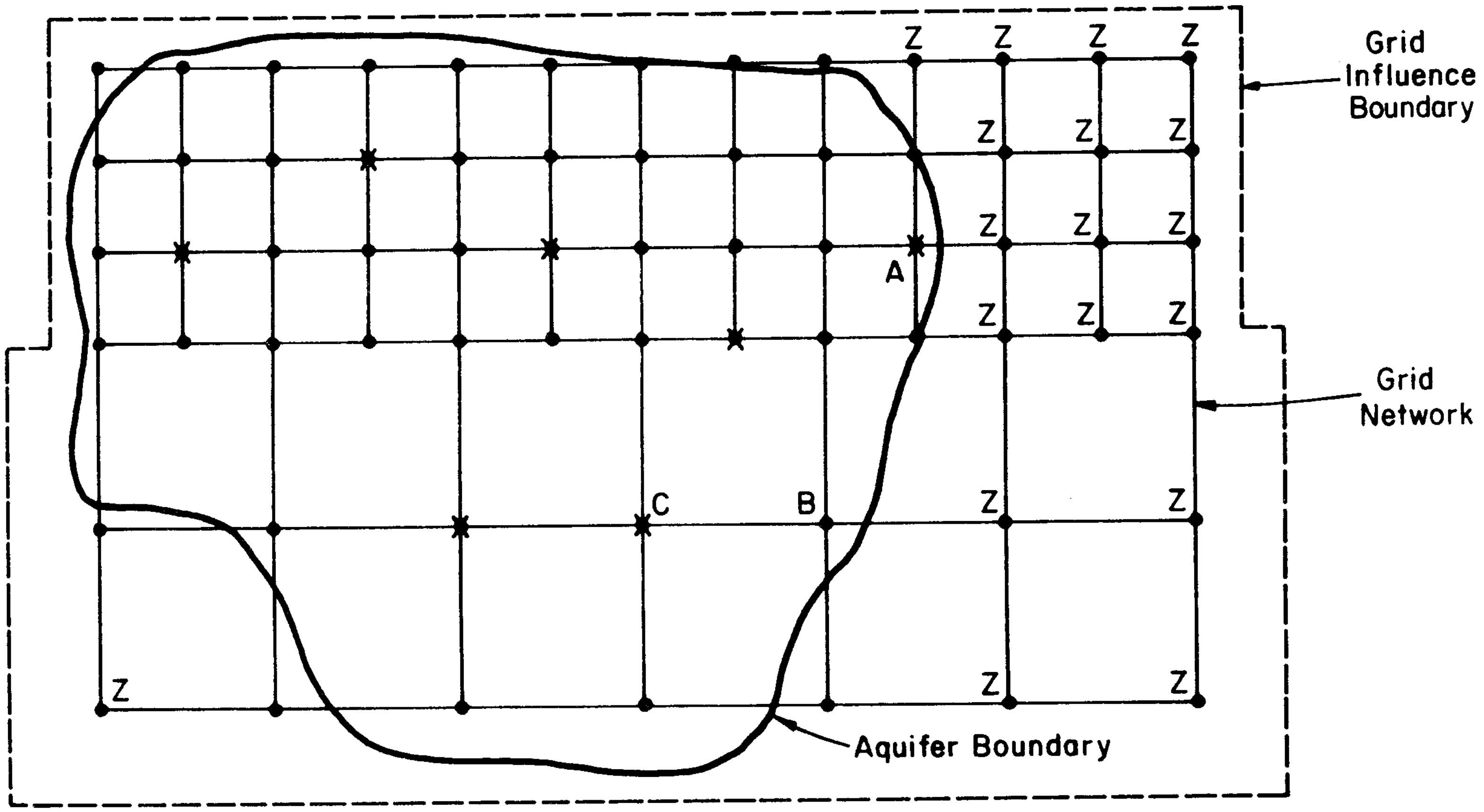
Much of the theory discussed previously was applicable to an undeveloped aquifer (i.e. zero initial drawdown). Few aquifers are left undeveloped and it is necessary to calculate the effect of initial drawdowns on the drawdown responses. Without proof we shall state that such drawdown at point w at time n (weeks) due to pumping at P wells given that initial drawdowns were known at Π observation wells (real or fictitious) is:

$$s_w(n) = \sum_{p=1}^{P} \sum_{\nu=1}^{n} \delta_{wp}(n-\nu+1)\; Q_p(\nu) + \sum_{\pi=1}^{\Pi} \theta_{w\pi}(n)\; s_\pi^\circ \qquad (29)$$

where s_π° is the observed initial drawdown at observation (well) point π and the $\theta_{w\pi}(n)$ are coefficients, functions of the $\delta_{w\pi}(\nu)$ coefficients, calculated by the DELTA program.

Basically the DELTA program performs the following steps:

1. It calculates (generates) the $\delta_{wp}(n)$, the $\delta_{wr}(n)$ (if there are reaches in the system), and the $\delta_{w\pi}(n)$ (if there are drawdown observation wells in the system).



Legend

● Grid Point Locations

X Well Grid Points

Z Grid Points Having Very Small (≅0) Transmissivities and Specific Yields

Figure 23

2. It calculates the $\theta_{w\pi}(n)$ coefficients.

3. It calculates the drawdowns due to either the influence of pumping wells (given a pumping pattern) or the influence of initial drawdowns (given initial drawdowns) or the combination of both effects.

4. Input Description for the DELTA Program

The Delta Program can model a grid system of varying grid sizes (distance between two grid points) and thus has a great deal of flexibility. Figure 24 shows how a grid network can be used to simulate a stream-aquifer system.

There are 12 types of input cards used by this program; however some are optional and may not be needed in all modeling situations. These cards are given below, in sequence of usage:

(1) Card A. Grid parameters.
(2) Card B. Grid parameters.
(3) Card J. Grid size change locations.
(4) Card O. Observation (π) points coordinates.
(5) Card C. Drawdown or elevation parameters.
(6) Card S. Initial drawdowns at "π" points.
(7) Card W. Well (p) points coordinates.
(8) Card N. Number of pumping rates for wells.
(9) Card Q. Pumping rates for wells
(10) Card R. Reach (r) points coordinates.
(11) Card P. Effective porosities (specific yields) of grid points.
(12) Card T. Transmissivities of grid points.

The following is a detailed description of the program input and its limitations. Figure 25 illustrates a typical grid network and the values of some of its parameters.

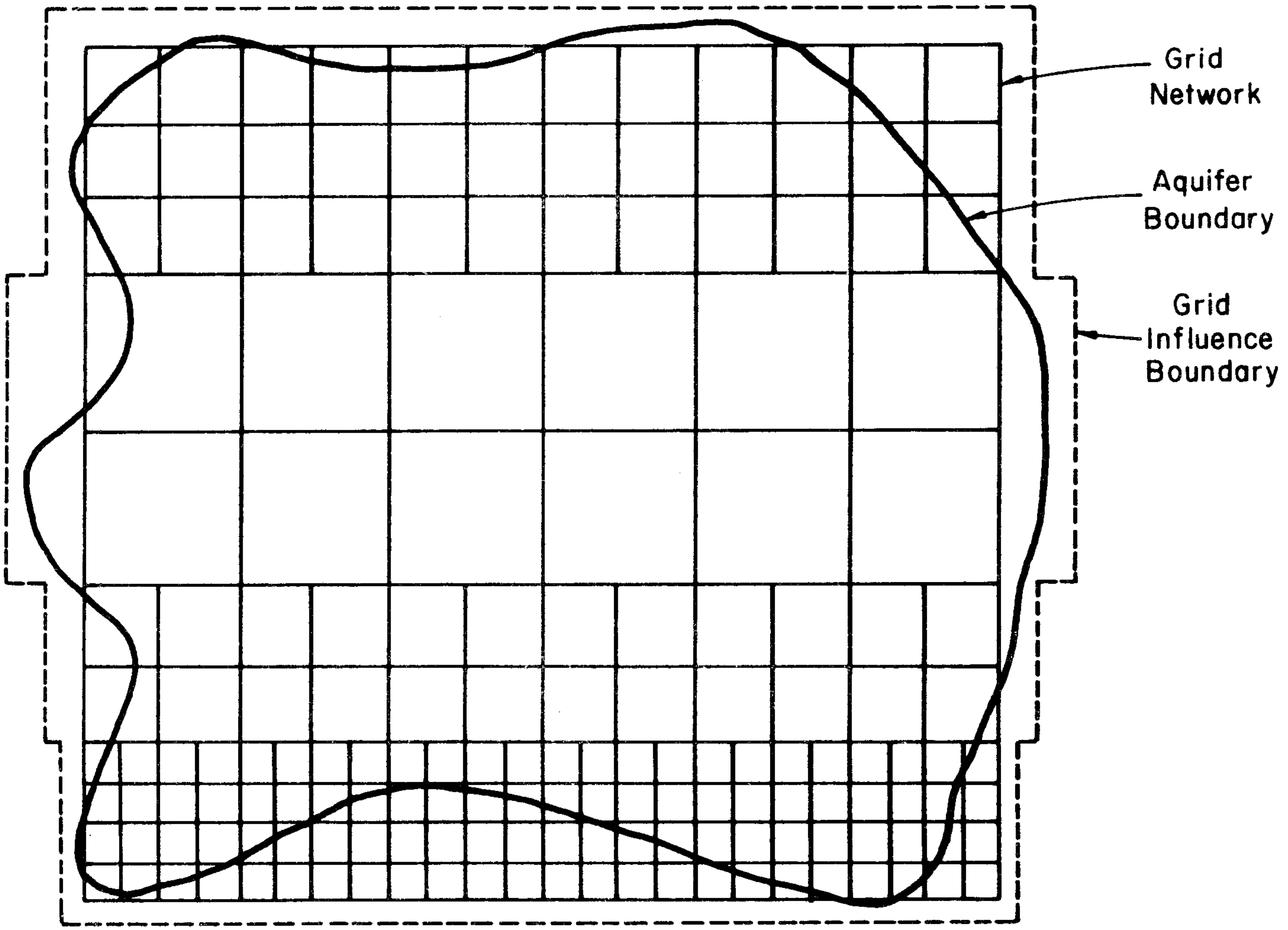


Figure 24

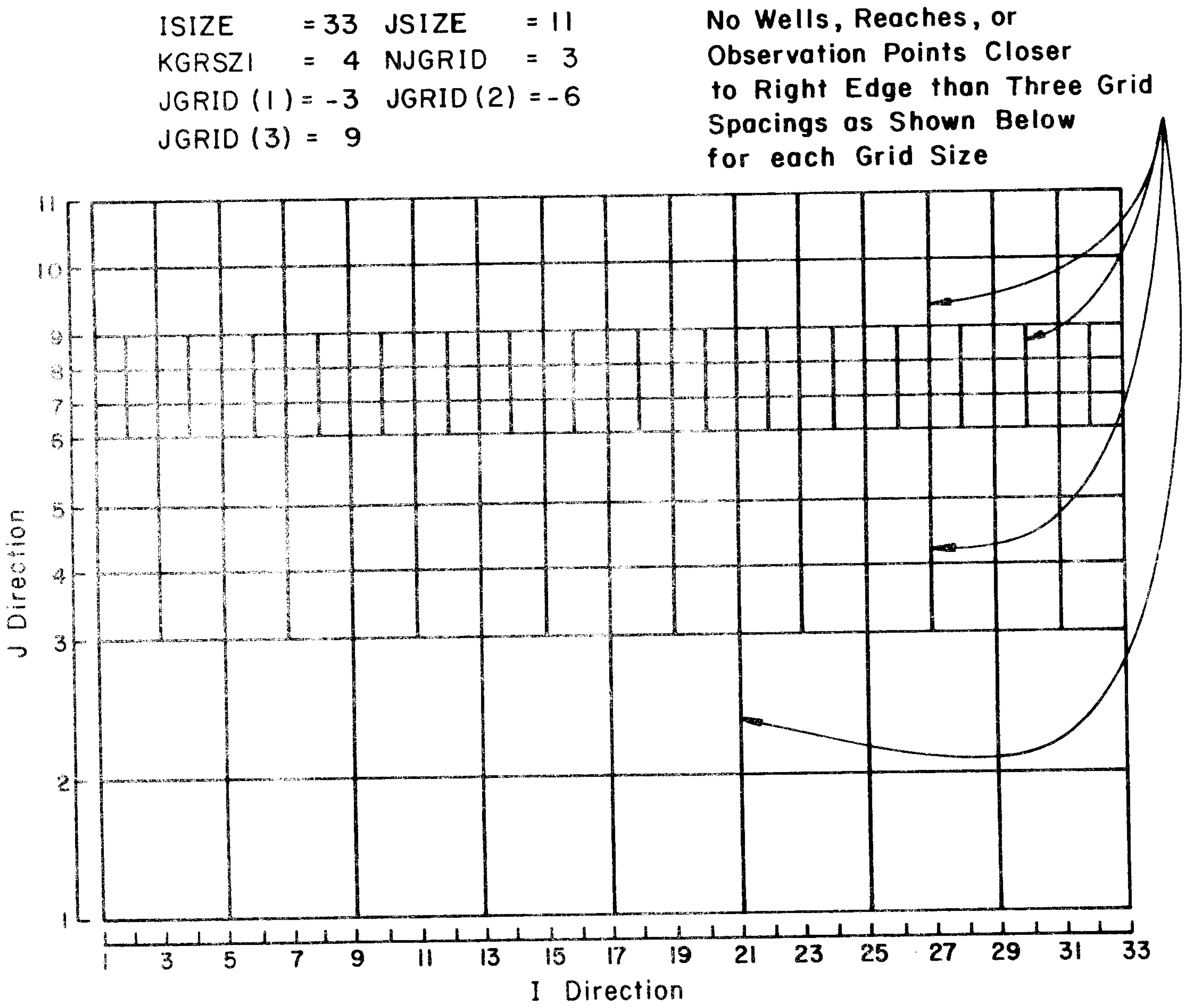

ISIZE = 33 JSIZE = 11
KGRSZI = 4 NJGRID = 3
JGRID (1) = -3 JGRID (2) = -6
JGRID (3) = 9
No Wells, Reaches, or Observation Points Closer to Right Edge than Three Grid Spacings as Shown Below for each Grid Size
J Direction
1
2
3
4
5
6
7
8
9
10
11
I Direction
1
3
5
7
9
11
13
15
17
19
21
23
25
27
29
31
33

Figure 25

INPUT DESCRIPTION FOR DELTA PROGRAM

SPECIAL NOTES

1. All fields are 8 spaces wide, except for the first field on each card which is only 6 spaces wide (columns 3-8). The first two columns on each card are for card identification only.

2. The following units may be used:

PARAMETER	STATE ENGINEER UNITS	RESEARCH UNITS
Discharge	Gallons/minute	Cubic meters/week
Transmissivity	Thousand gallons/day-foot	Cubic meters/week-meter
Distances	Feet	Meters
Drawdowns	Feet	Meters
Time	Days or weeks (as specified)	Days or weeks (as specified)

3. The following limitations have been placed on the selection of the grid network. See the accompanying sketch of a typical grid system (Figure 25).

   A. The value of "ISIZE" (the number of columns in the smallest grid size of the network) must be an odd number (i.e., 1,3,5,7...)

   B. No wells, reaches or observation points should be placed closer than 4 grid spacings from the right boundary of the grid network.

   C. If varying grid sizes are used in the grid network, a new grid size must be continued for at least two consecutive grid rows, before the grid size may be changed again.

   D. Large transmissivity gradients in the grid network should be avoided. A good "rule of thumb" to use for any set of three grid points is as follows: $|TS(3) - TS(1)| \leq (4)(TS(2))$

CARD "A"   Grid Parameters

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
1	3-8	ΔX_{min}	DXY	F6.0	+	Smallest grid size spacing (distance between two grid points) in grid network
2	8-16	Π	NPI	I8	0 or +	Number of grid points where initial drawdowns are known
3	17-24	P	NWELL	I8	0 or +	Number of pumping wells in grid network
4	25-32	R	NREACH	I8	0 or +	Number of stream reaches in grid network
5	33-40	n_{max}	NMAX	I8	+	Number of weeks of delta and theta calculations.
6	41-48	N	NWEEK	I8	+	Number of weeks of drawdown calculations (if the drawdown option is used in this program). NWEEK should be ≥ NMAX. If the drawdown option is not used, then NWEEK should be made equal to NMAX.
7	49-56	INDEX	IPRINT	I8	see below	Index defining type and time of output of delta and theta coefficients
					0	Delta and theta coefficients will be printed and written on tape at the end of each week.
					1	Delta and theta coefficients will not be printed. However they will be written on tape at the end of each week.

CARD "A" continued

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
8	57-64	INDEX	IPHI	I8	see below	Index defining method of reading specific yield values for the grid system
					0	A specific yield value will be read for each grid point in the grid system.
					1	A constant specific yield value (to be used at every grid point) will be read.
9	65-72	INDEX	ITS	I8	see below	Index defining method of reading transmissivity values for the grid system
					0	A transmissivity value will be read for each grid point in the grid system.
					1	A constant transmissivity value (to be used at every grid point) will be read.
10	73-80	INDEX	IUNITS	I8	see below	Index defining units of input and output
					0	Research units will be used.
					1	State Engineer units will be used.

CARD "B" Grid Parameters

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
1	3-8	I_{max}	ISIZE	I6	+	Total size of grid network in I direction (number of columns in the smallest grid size of the grid network). This must be an odd number.
2	8-16	J_{max}	JSIZE	I8	+	Total size of grid network in J direction (number of rows in the grid network). This may be either an odd or even number.
3	17-24	CH_{max}	NJGRID	I8	0 or +	The number of grid size changes in the grid network.
4	25-32	$\Delta X_1/\Delta X_{min}$	KGRSZ1	I8	+	The ratio of the grid size of the first row to the smallest grid size used in the grid network
5	33-40	INDEX	KDNO	I8	see below	Index defining whether condition numbers (measure of elimination and back substitution errors) will be calculated
					0	Condition numbers will not be calculated or printed.
					1	Condition numbers will be calculated and printed at the end of each week
6	41-48	K_t	AFAC	F8.0	+	Time step multiplying factor $\Delta t_{i+1} = (\Delta t_i) \times (K_t)$

CARD "B" continued

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
7	49-56	Δt_{min}	DT1	F8.0	+	Initial time step (in weeks) for first two weeks of delta calculations
8	57-64	Δt_{max}	D1MX	F8.0	+	Maximum time step (weeks) during first two weeks of delta calculations
9	65-72	INDEX	IDRAW	I8	see below	Index defining whether drawdowns due to pumping (no stream present) should be calculated and printed
					0	Drawdowns will not be calculated or printed.
					1	Drawdowns will be calculated and printed.
10	73-80	Δt_d	IDAYS	I8	+	Time interval (days) between drawdown printouts. This field should be blank if drawdowns are not to be calculated (IDRAW = 0)

CARD(S) "J" (optional)   Grid Size Change Locations

Do not use these cards if "NJGRID" (Card B, Field 3) has a value of zero.

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
1	3-8	CH_1	JGRID(1)	I6	+ or -	Row numbers (in increasing order) where grid size changes occur. Positive values indicate that the grid size is doubled. Negative values indicate that the grid size will be reduced to half. Use as many fields as necessary on a "J" card (maximum 10 fields per card). Additional "J" cards may be used if needed. The total number of fields used must equal "NJGRID."
2	9-16	CH_2	JGRID(2)	I8	+ or -	
3	17-24	CH_3	JGRID(3)	I8	+ or -	
"	"	"	"	"	"	
"	"	"	"	"	"	
10	73-80	CH_{10}	JGRID(10)	I8	+ or -	

CARD(S) "0" (optional) Observation Points Coordinates

Do not use these cards if "NPI" (Card A, Field 2) has a value of zero.

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
1	3-8	J_{π_1}	JPI(1)	I6	+	"J" (row) coordinate of initial observation point number 1
2	9-16	I_{π_1}	IPI(1)	I8	+	"I" (column) coordinate of initial observation point number 1
3	17-24	J_{π_2}	JPI(2)	I8	+	Continue with "J" and "I" coordinates for each initial observation point (observation point numbers in increasing order). Use as many fields on an "0" card as necessary. Additional "0" cards may be used if needed. The total number of fields used must equal twice the value of "NPI."
4	25-32	I_{π_2}	IPI(2)	I8	+	
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
9	65-72	J_{π_5}	JPI(5)	I8	+	
10	72-80	I_{π_5}	IPI(5)	I8	+	

CARD "C" (optional) System Parameters

Do not use use this card if "NPI" (Card A, Field 2) has a value of zero, or if "IDRAW" (Card B, Field 9) has a value of zero.

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
1	3-8	Index	ISTAGE	I6	see below	Index defining whether initial drawdowns or water table elevations will be entered at the observation points.
					0	Drawdowns will be entered.
					1	Elevations will be entered.
2	9-16	H	HIGHDM	F8.0	+	Elevation of the high datum in the system. This elevation should be greater than any expected water table or water surface elevation.
3	17-24	Index	NPRINT	I8	see below	Index defining the type of output that will be produced if the drawdown option is used.
					0	Print only drawdowns (as measured from the high datum)
					1	Print only water table elevations (as the response to pumping and natural redistribution)

CARD(S) "S" (optional)   Initial Drawdowns at Observation Points

Do not use these cards if "NPI" (Card A, Field 2) has a value of zero, or if "IDRAW" (Card B, Field 9) has a value of zero.

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
1	3 -8	$s^o_{\pi_1}$	SWO(1)	F6.0	0 or +	Initial drawdown or water table elevation* at observation point number 1.
2	9-16	$s^o_{\pi_2}$	SWO(2)	F8.0	0 or +	Initial drawdown or water table elevation* at observation point number 2.
3	17-24	$s^o_{\pi_3}$	SWO(3)	F8.0	0 or +	Continue with initial drawdowns or water table elevations* for each observation point (Observ. pt. numbers in increasing order). Use as many fields on an "S" card as necessary. Additional "S" cards may be used if needed. The total number of fields used must equal the value of "NPI."
4	25-32	$s^o_{\pi_4}$	SWO(4)	F8.0	0 or +	
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
9	65-72	$s^o_{\pi_9}$	SWO(9)	F8.0	0 or +	
10	73-80	$s^o_{\pi_{10}}$	SWO(10)	F8.0	0 or +	

* The variable "ISTAGE" (Card C, Field 1) specifies whether drawdowns or water table elevations are entered.

CARD(S) "W" (optional) Well Points Coordinates

Do not use these cards if "NWELL" (Card A, Field 3) has a value of zero.

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
1	3-8	J_{p_1}	JWELL(1)	I6	+	"J" (row) coordinate of pumping well number 1
2	9-16	I_{p_1}	IWELL(1)	I8	+	"I" (column) coordinate of pumping well number 1
3	17-24	J_{p_2}	JWELL(2)	I8	+	Continue with "J" and "I" coordinates for each well (well numbers in increasing order). Use as many fields on a "W" card as necessary. Additional "W" cards may be used if needed. The total number of fields used must equal twice the value of "NWELL."
4	25-32	I_{p_2}	IWELL(2)	I8	+	
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
9	65-72	J_{p_5}	JWELL(5)	I8	+	
10	73-80	I_{p_5}	IWELL(5)	I8	+	

CARD(S) "N" (optional) Number of Pumping Rates for Wells

Do not use these cards if "IDRAW" (Card B, Field 9) has a value of zero.

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
1	3-8	N_{p_1}	IPRC(1)	I6	+	Number of pumping rates to be read for well number 1
2	9-16	N_{p_2}	IPRC(2)	I8	+	Number of pumping rates to be read for well number 2
3	17-24	N_{p_3}	IPRC(3)	I8	+	Continue with the "number of pumping rates to be read" for each well (well numbers in increasing order). Use as many fields on an "N" card as necessary. Additional "N" cards may be used if needed. The total number of fields used must equal the value of "NWELL."
4	25-32	N_{p_4}	IPRC(4)	I8	+	
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
9	65-72	N_{p_9}	IPRC(9)	I8	+	
10	73-80	$N_{p_{10}}$	IPRC(10)	I8	+	

CARD(S) "Q" (optional) Pumping Rates for Wells

Do not use these cards if "IDRAW" (Card B, Field 9) has a value of zero.

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
1	3-8	Q_1	Q(1)	F6.0	+	First discharge rate for well #1
2	9-16	t_1	T(1)	F8.0	+	Time in days through which Q(1) is used
3	17-24	Q_2	Q(2)	F8.0	+	Second discharge rate for well #1
4	25-32	t_2	T(2)	F8.0	+	Time in days through which Q(2) is used
5	33-40	Q_3	Q(3)	F8.0	+	Continue with "discharge rates" and "time of termination" until the complete pumping pattern for well #1 has been described. Use as many fields on a "Q" card as necessary. Additional "Q" cards may be used if needed. The total number of fields used must equal twice the value of "N_{p1}."
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
9	65-72	Q_5	Q(5)	F8.0	+	
10	73-80	t_5	T(5)	F8.0	+	

Begin a new "Q" card for well #2 and enter pumping pattern as described above for well #1. Continue in this manner until the pumping patterns for all wells have been entered.

CARD(S) "R" (optional) Reach Points Coordinates

Do not use these cards if "NREACH" (Card A, Field 4) has a value of zero.

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
1	3-8	J_{r_1}	JRIV(1)	I6	+	"J" (row) coordinate of stream reach number 1
2	9-16	I_{r_1}	IRIV(1)	I8	+	"I" (column) coordinate of stream reach number 1
3	17-24	J_{r_2}	JRIV(2)	I8	+	Continue with "J" and "I" coordinates for each stream reach (stream reach numbers in increasing order). Use as many fields on an "R" card as necessary. Additional "R" cards may be used if needed. The total number of fields used must equal twice the value of "NREACH."
4	25-32	I_{r_2}	IRIV(2)	I8	+	
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
9	65-72	J_{r_5}	JRIV(5)	I8	+	
10	73-80	I_{r_5}	IRIV(5)	I8	+	

CARD(S) "P" Specific Yields of Grid Points

If "IPHI" (Card A, Field 7) has a value of one, only field 1 will be used on this card.

Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
1	3-8	ϕ_c or ϕ_1	PHICON or PHI(1)	F7.0	+ +	If "IPHI" equals one, enter constant specific yield value to be used at all grid points. Omit other fields on this card. Otherwise enter specific yield value for grid point #1 in first row
2	9-16	ϕ_2	PHI(2)	F8.0	+	Specific yield value for grid point #2 in first row (grid point numbers increase from left to right)
3	17-24	ϕ_3	PHI(3)	F8.0	+	Continue with specific yield values for each grid point in the first row. Use as many fields on a "P" card as necessary. Additional "P" cards may be used if needed. Begin a new "P" card for second row and enter ϕ values for each grid point as described above for the first row. Continue in this manner until ϕ values have been entered for all rows in the grid network.
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
9	65-72	ϕ_9	PHI(9)	F8.0	+	
10	73-80	ϕ_{10}	PHI(10)	F8.0	+	

CARD(S) "T" Transmissivities of Grid Points

If "ITS" (Card A, Field 8) has a value of one, only field 1 will be used on this card.

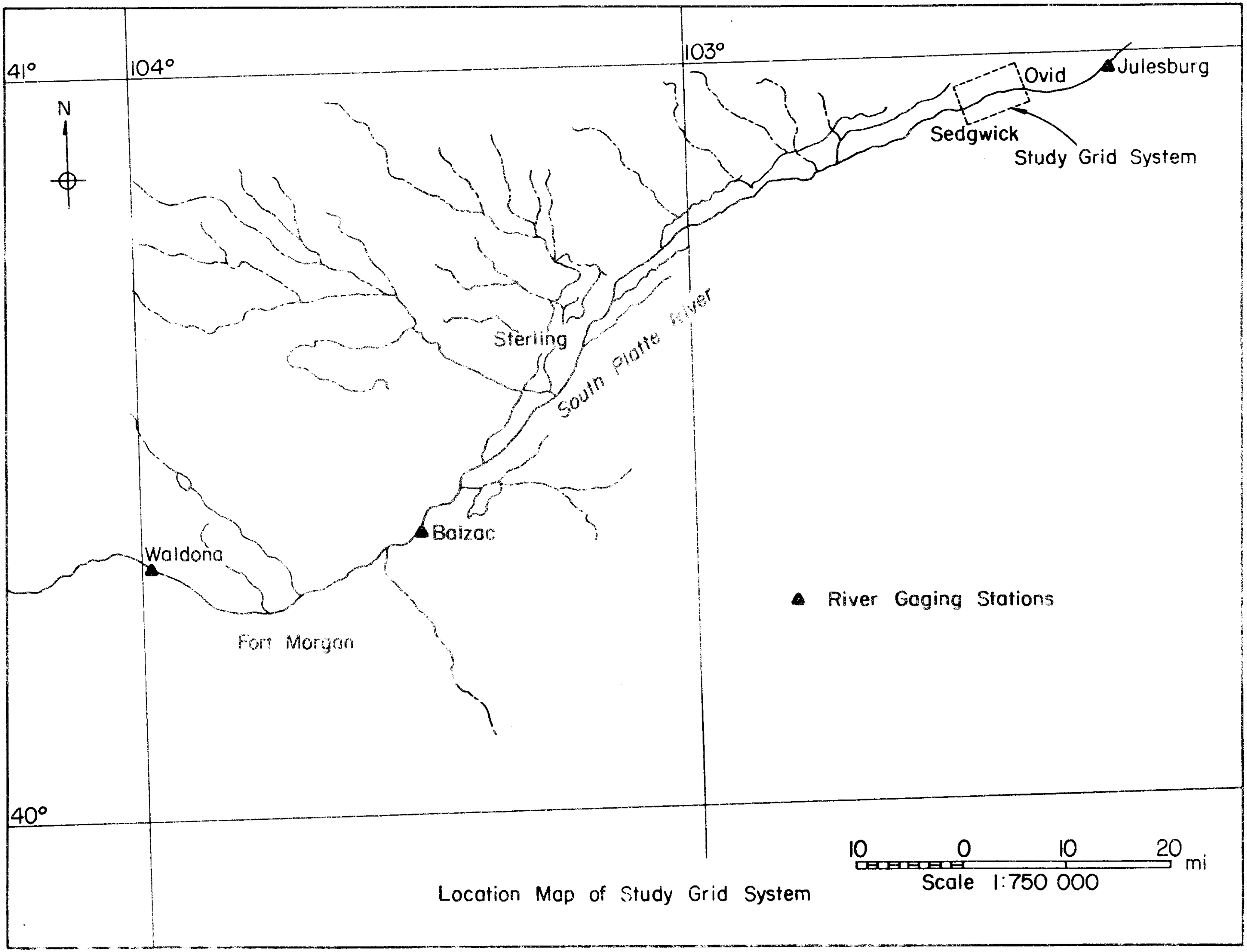
Field	Columns	Math Symbol	Fortran Symbol	Format	Value	Description
1	1-8	T_c or T_1	TSCON or TS(1)	F7.0	+ +	If "ITS" equals 1, enter constant transmissivity value to be used at all grid points. Do not use other fields on this card. Otherwise enter transmissivity value for grid point #1 in first row.
	9-16	T_2	TS(2)	F8.0	+	Transmissivity value for grid point #2 in first row (grid point numbers increase from left to right)
3	17-24	T_3	TS(3)	F8.0	+	Continue with transmissivity values for each grid point in the first row. Use as many fields on a "T" card as necessary. Additional "T" cards may be used if needed. Begin a new "T" card for second row and enter transmissivity values for each grid point as described above for first row. Continue in this manner until transmissivity values have been entered for all rows in the grid network.
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
"	"	"	"	"	"	
9	65-72	T_9	TS(9)	F8.0	+	
10	73-80	T_{10}	TS(10)	F8.0	+	

5. Illustrative Data Preparation Case

For the sake of illustration of how to prepare data for use with the 'DELTA' program a portion of the alluvial aquifer of the South Platte River in Colorado was studied. The general geographical location of the area, which was suggested by Dr. Qazi, Head, Planning and Investigations, is shown on Figure 26. Figures 27, 28 and 29 are larger scale maps of the Sedgwick-Ovid reach, providing respectively location of wells, aquifer transmissivities and saturated thicknesses.

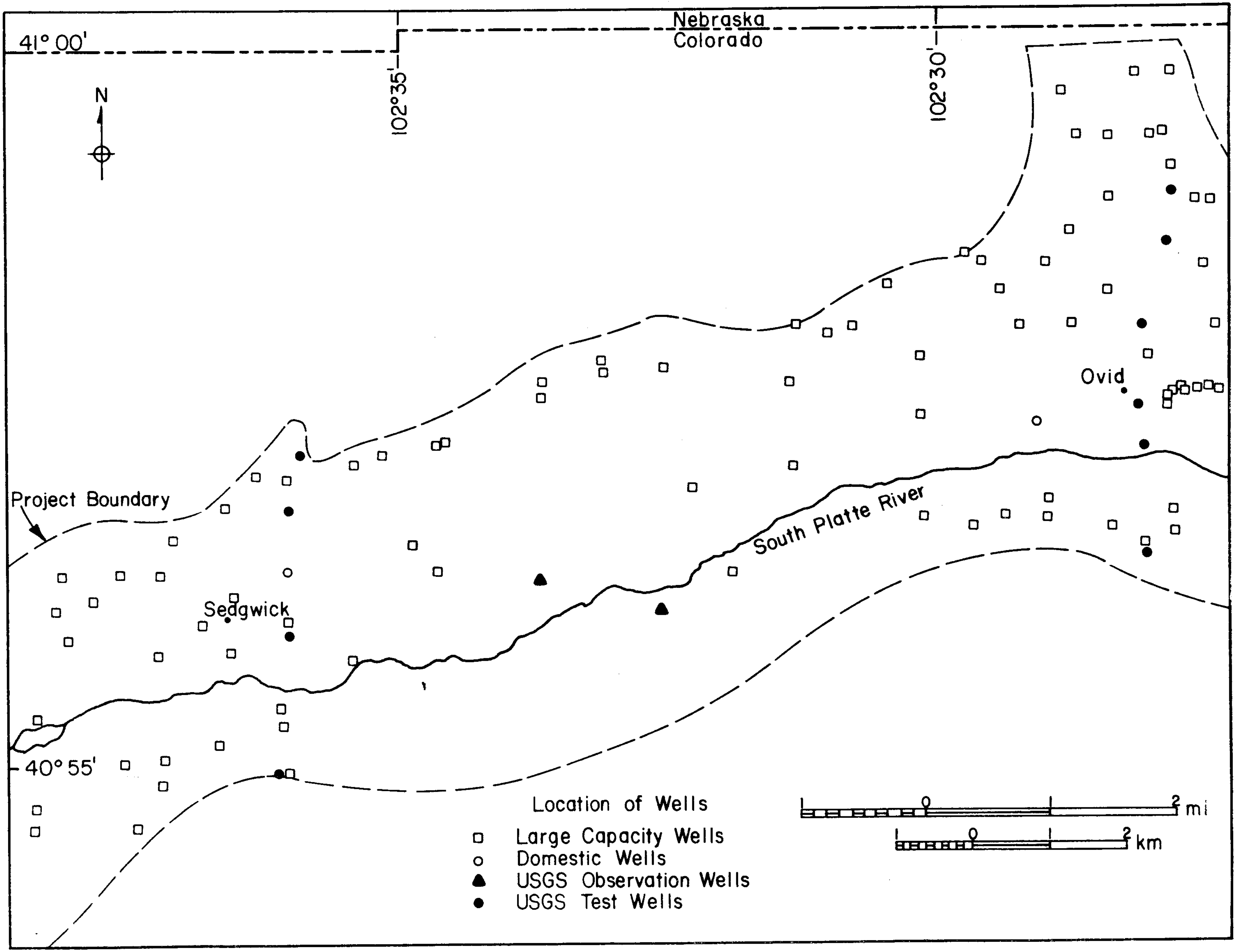
Figure 30 shows the selected grid (drawn on tracing paper so that it can be superimposed on any of the Figures 27, 28 and 29 to read transmissivity, saturated thickness values, etc...) superimposed on the map of saturated thickness.

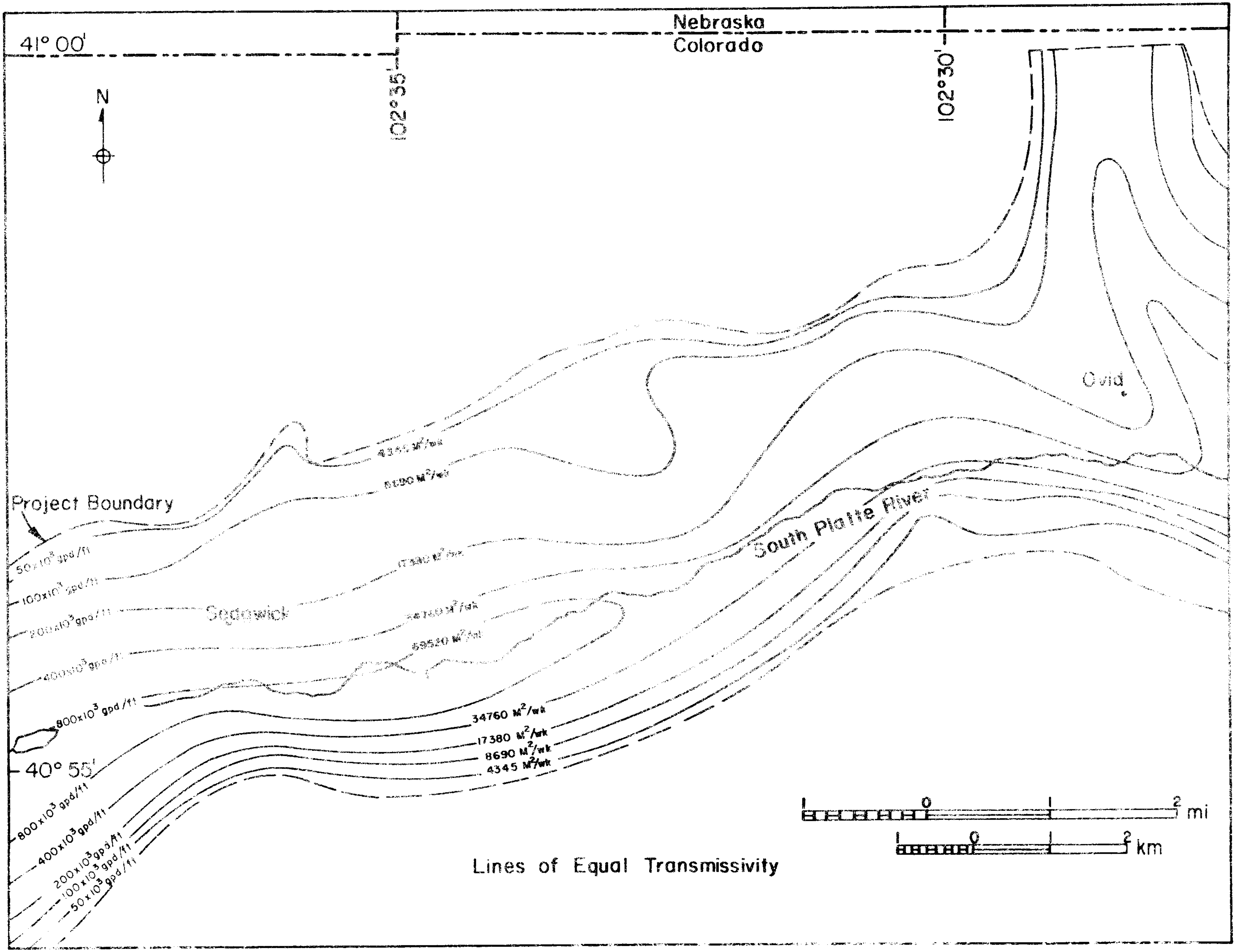
The values of the input data for the system are shown on FORTRAN coding sheets (Table 1). Punched cards in their proper order in the data deck are shown on Figure 31. Tables 2 to 5 display some of the computer printed output.



Location Map of Study Grid System

Figure 26


Nebraska
Colorado
41° 00'
102°35'
102°30'
N
Project Boundary
South Platte River
Sedgwick
Ovid
40° 55'
Location of Wells
□ Large Capacity Wells
○ Domestic Wells
▲ USGS Observation Wells
● USGS Test Wells
0
1
2
mi
0
1
2
km



Nebraska
Colorado
41° 00'
102° 35'
102° 30'
N
Ovid
Project Boundary
South Platte River
Sedgwick
40° 55'
50x10³ gpd/ft
100x10³ gpd/ft
200x10³ gpd/ft
400x10³ gpd/ft
800x10³ gpd/ft
4345 M²/wk
8690 M²/wk
17380 M²/wk
34760 M²/wk
69520 M²/wk
800x10³ gpd/ft
400x10³ gpd/ft
200x10³ gpd/ft
100x10³ gpd/ft
50x10³ gpd/ft
1 0 1 2 mi
1 0 1 2 km
Lines of Equal Transmissivity

Figure 28

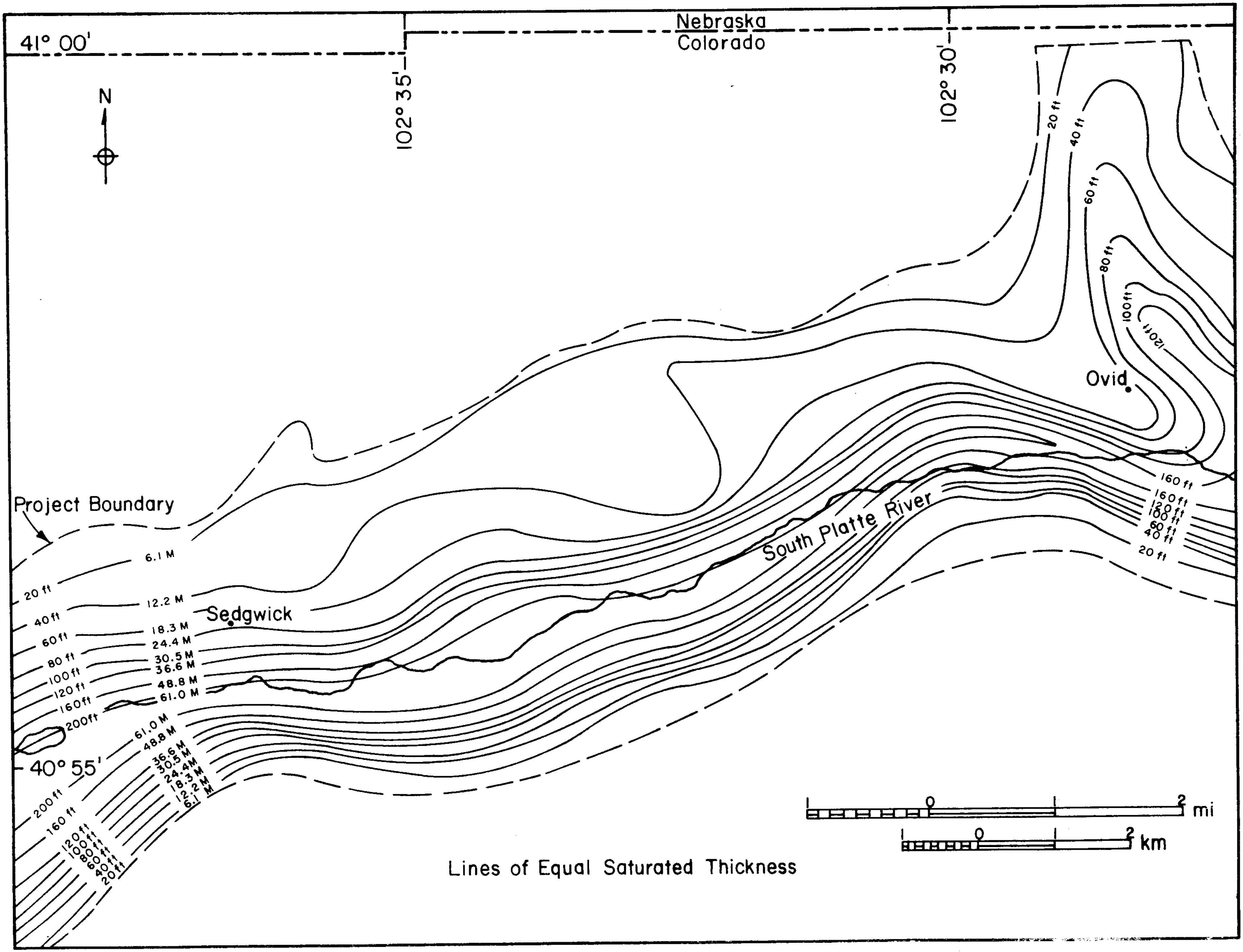
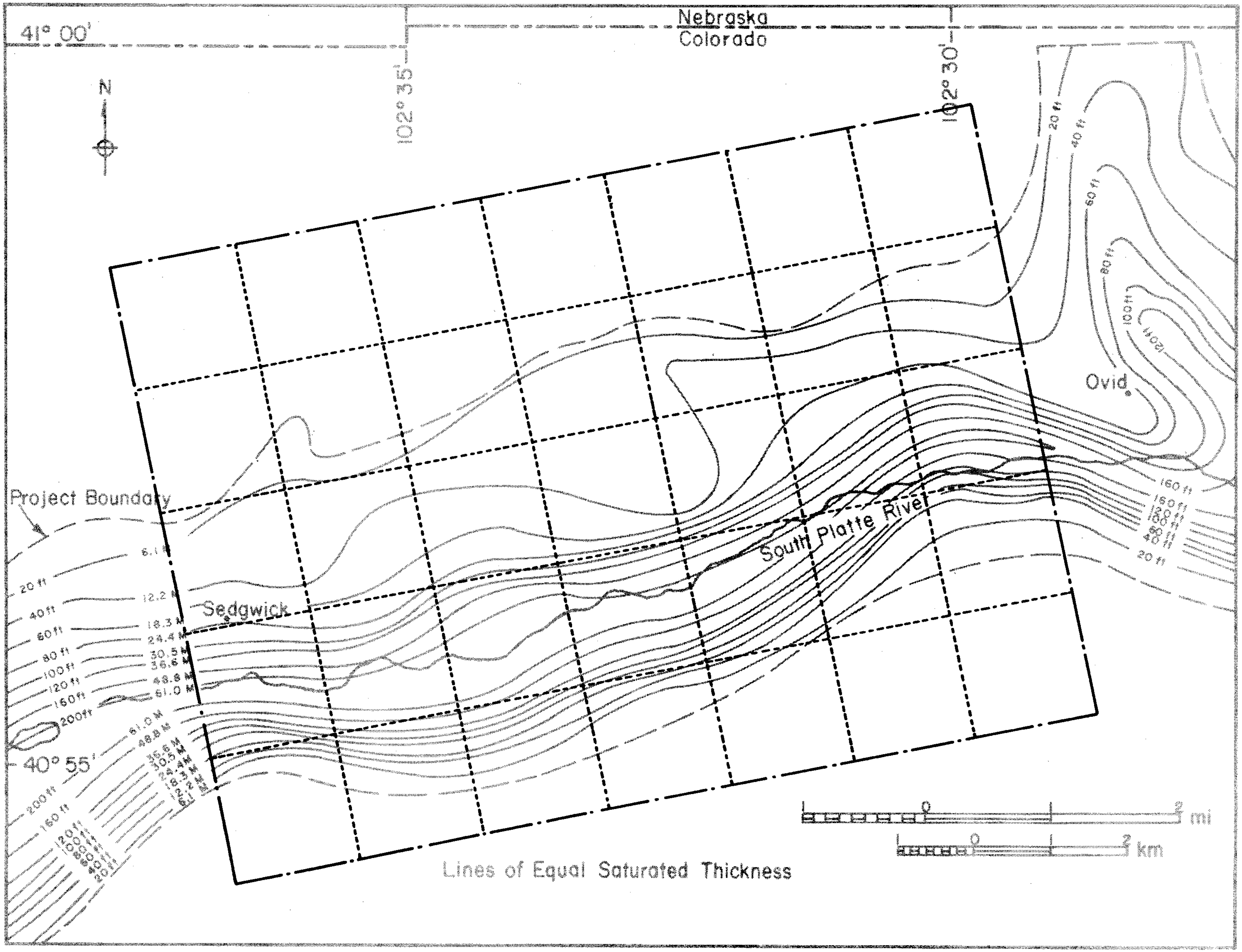

Nebraska
Colorado
41° 00'
102° 35'
102° 30'
N
20 ft
40 ft
60 ft
80 ft
100 ft
120 ft
Ovid
160 ft
160 ft
120 ft
100 ft
60 ft
40 ft
20 ft
South Platte River
Project Boundary
6.1 M
20 ft
12.2 M
Sedgwick
40 ft
18.3 M
60 ft
24.4 M
80 ft
30.5 M
100 ft
36.6 M
120 ft
48.8 M
160 ft
61.0 M
200 ft
61.0 M
48.8 M
36.6 M
30.5 M
24.4 M
18.3 M
12.2 M
6.1 M
40° 55'
200 ft
160 ft
120 ft
100 ft
80 ft
60 ft
40 ft
20 ft
0
2 mi
0
2 km
Lines of Equal Saturated Thickness

Figure 29



Lines of Equal Saturated Thickness

Figure 30

Table 1. INPUT DATA FOR SEDGWICK-OVID REACH ON FORTRAN CODING FORMS

FORTRAN CODING FORM

UNIVERSITY COMPUTER CENTER

COMPUTER CHARGE NO.

PAGE 1 OF 1

PROGRAM DELTA / DATA FOR SAMPLE PROBLEM IN SOUTH PLATTE

PROGRAMMER TISSA H. ILLANGASEKARE

DATE 9th SEPT 1975

STATEMENT NUMBER | CONT | FORTRAN STATEMENT | IDENTIFICATION SEQUENCE

```
A 1609.0      10       7       4       5       5       0       1       0       0

B      7       5       0       1       1     1.5    0.001     0.1       0

O      1       1       1       2       1       4       3       1       3       4
O      4       1       4       3       5       1       5       3       5       4

W      1       1       1       2       1       4       3       3       4       2
W      4       4       5       3

R      2       1       2       2       2       3       2       4

P    0.2

T  1000.  1500.0  4000.0  2000.0  3000.0  1100.0   200.0
T 67000. 67000.0 67000.0 35000.0 28000.0  9000.0  7500.0
T 18000. 18000.0 18000.0 11000.0 15500.0 29000.0 28000.0
T  5000.  5000.0  5000.0  5000.0  9000.0  9000.0  8000.0
T   200.   200.0   200.0  1500.0   500.0   200.0   200.0
```

```
R     2      1       2       3       3       3       3       4
   JRIV(1) IRIV(1) JRIV(2) IRIV(2) JRIV(3) IRIV(3) JRIV(4) IRIV(4)

W     4      4      5      3
 JWELL(6) IWELL(6) JWELL(7) IWELL(7)

W     1        1        1        2        1        4        3        3        4        2
 JWELL(1) IWELL(1) JWELL(2) IWELL(2) JWELL(3) IWELL(3) JWELL(4) IWELL(4) JWELL(5) IWELL(5)

0     4       1       4       3       5       1       5       3       5        4
  JPI(6)  IPI(6)  JPI(7)  IPI(7)  JPI(8)  IPI(8)  JPI(9)  IPI(9)  JPI(10)  IPI(10)

0     1       1       1       2       1       4       3       1       3       4
  JPI(1)  IPI(1)  JPI(2)  IPI(2)  JPI(3)  IPI(3)  JPI(4)  IPI(4)  JPI(5)  IPI(5)

B     7      5      0       1       1      1.5    0.001    0.1     0
  ISIZE JSIZE NJGRID KGRSZI  KDNO    AFAC    DT1     D1MX    IDRAW

A 1609.   10     7      4      5      5      0      1      0      0
  DXY     NPI   NWELL  NREACH NMAX   NWEEK  IPRINT IPHI   ITS    IUNITS
```

Figure 31

```
                        T   500.    200.0   200.0  1500.0   500.0   200.0   200.0

                          TS(29) TS(30) TS(31)  TS(32) TS(33) TS(34)  TS(35)

                      T . 5000.  5000.0  5000.0  5000.0  9000.0  9000.0  8000.0

                        TS(22) TS(23) TS(24)  TS(25) TS(26) TS(27)  TS(28)

                    T 18000. 18000.0 18000.0 11000.0 15500.0 29000.0 28000.0

                      TS(15) TS(16) TS(17) TS(18) TS(19)  TS(20) TS(21)

                  T 67000. 67000.0 67000.0 35000.0 28000.0  9000.0  7500.0

                    TS(8)  TS(9)   TS(10) TS(11) TS(12)  TS(13) TS(14)

                T  1000.  1500.0  4000.0  2000.0  3000.0  1100.0   200.0

                  TS(1)  TS(2)   TS(3)  TS(4)  TS(5)   TS(6)  TS(7)
              P    0.2

               PHICON
```

Figure 32

Table 2. PRINTED OUTPUT OF INPUT DATA

```
---------- THE FOLLOWING IS INPUT DATA ----------


RESEARCH UNITS HAVE BEEN SELECTED FOR INPUT AND OUTPUT

DISCHARGE          CUBIC METERS/WEEK
TRANSMISSIVITY     CUBIC METERS/WEEK-METER
DISTANCE           METERS
DRAWDOWNS          METERS
TIME               DAYS AND WEEKS


       DXY   NPI   NWELL   NREACH   NMAX   NWEEK   IPRINT   IPHI   ITS   IUNITS
    1609.0   10       7      4        5        5       0       1     0      0

      ISIZE  JSIZE  NJGRID  KGRSZ1  KDNO  AFAC      DT1       D1MX  IDRAW   IDAYS
        7       5        0       1       1     1.50     .001      .100     0      -0


INITIAL DRAWDOWN POINTS ARE LOCATED AT THE FOLLOWING J,I GRID POINTS
  1,  1       1,  2       1,  4       3,  1       3,  4       4,  1       4,  3       5,  1       5,  3
  5,  4


WELLS ARE LOCATED AT THE FOLLOWING J,I GRID POINTS
  1,  1       1,  2       1,  4       3,  3       4,  2       4,  4       5,  3


REACHES ARE LOCATED AT THE FOLLOWING J,I GRID POINTS
  2,  1       2,  2       2,  3       2,  4


THE GRID SYSTEM IS ASSUMED HOMOGENEOUS WITH RESPECT TO SPECIFIC YIELD (PHI)
THE CONSTANT VALUE OF PHI USED FOR ALL GRID POINTS IS PHI =  .200


ROW J =  1   TS VALUES
     1000.0     1500.0     4000.0     2000.0     3000.0     1100.0      200.0

ROW J =  2   TS VALUES
    67000.0    67000.0    67000.0    35000.0    28000.0     9000.0     7500.0

ROW J =  3   TS VALUES
    18000.0    18000.0    18000.0    11000.0    15500.0    29000.0    28000.0

ROW J =  4   TS VALUES
     5000.0     5000.0     5000.0     5000.0     9000.0     9000.0     8000.0

ROW J =  5   TS VALUES
      200.0      200.0      200.0     1500.0      500.0      200.0      200.0


---------- END OF INPUT DATA ----------
```

```
WELL NUMBER   6 LOCATED AT J,I =    4,    4


DELTAS AT THE END OF WEEK    1           CONDITION NUMBER(CN) =   1.0157   MPRE =   .14399E-13
-----------------------------

J VALUE = ROW  INDEX =  1
  .12195E-15  .47624E-14   .16477E-12   .20124E-11   .54599E-13   .22038E-15   .11622E-17

J VALUE = ROW  INDEX =  2
  .85444E-14  .29799E-12   .88648E-11   .17727E-09   .64743E-11   .79114E-13   .70310E-15

J VALUE = ROW  INDEX =  3
  .96621E-14  .10449E-11   .95217E-10   .67604E-08   .17517E-09   .42813E-11   .65793E-13

J VALUE = ROW  INDEX =  4
  .11178E-12  .36848E-10   .98242E-08   .18916E-05   .15533E-07   .10705E-09   .55110E-12

J VALUE = ROW  INDEX =  5
  .15763E-15  .60997E-13   .21822E-10   .64508E-08   .59833E-10   .31073E-12   .12125E-14


DELTAS AT THE END OF WEEK    2           CONDITION NUMBER(CN) =   1.0157   MPRE =   .20734E-13
-----------------------------

J VALUE = ROW  INDEX =  1
  .69913E-14  .16252E-12   .32118E-11   .21621E-10   .11074E-11   .82078E-14   .74672E-16

J VALUE = ROW  INDEX =  2
  .28068E-12  .56077E-11   .92397E-10   .97442E-09   .70660E-10   .16341E-11   .26138E-13

J VALUE = ROW  INDEX =  3
  .20501E-12  .11955E-10   .55843E-09   .18211E-07   .10138E-08   .47441E-10   .13530E-11

J VALUE = ROW  INDEX =  4
  .12749E-11  .21809E-09   .26981E-07   .18208E-05   .42086E-07   .62270E-09   .61844E-11

J VALUE = ROW  INDEX =  5
  .32662E-14  .70073E-12   .13098E-09   .17929E-07   .35497E-09   .35188E-11   .24756E-13
```

Table 3. PRINTED OUTPUT OF 'DELTA' COEFFICIENTS FOR WELL NUMBER 6 FOR WEEKS 1 AND 2

```
REACH NUMBER   2 LOCATED AT J,I =   2,    2


DELTAS AT THE END OF WEEK    1            CONDITION NUMBER(CN) =   1.0472   MPRE =   .29248E-13
----------------------------

J VALUE = ROW INDEX =  1
 .26[illegible]03E-08  .56282E-07   .31024E-08   .45454E-10   .55664E-12   .18716E-14   .41093E-17

J VALUE = ROW INDEX =  2
 .96498E-07   .14966E-05   .10421E-06   .34967E-08   .73592E-10   .55940E-12   .21914E-14

J VALUE = ROW INDEX =  3
 .32188E-08   .53427E-07   .34482E-08   .64671E-10   .11382E-11   .19535E-13   .23814E-15

J VALUE = ROW INDEX =  4
 .20173E-10   .39405E-09   .21853E-10   .29494E-12   .77371E-14   .15745E-15   .17830E-17

J VALUE = ROW INDEX =  5
 .27979E-13   .64080E-12   .30326E-13   .46580E-15   .15765E-16   .28190E-18   .26220E-20


DELTAS AT THE END OF WEEK    2            CONDITION NUMBER(CN) =   1.0472   MPRE =   .85546E-13
----------------------------

J VALUE = ROW INDEX =  1
 .12610E-07   .12743E-06   .14284E-07   .41198E-09   .11420E-10   .56404E-13   .20815E-15

J VALUE = ROW INDEX =  2
 .18955E-06   .95367E-06   .19386E-06   .15038E-07   .63352E-09   .91022E-11   .63694E-13

J VALUE = ROW INDEX =  3
 .15444E-07   .11744E-06   .16137E-07   .59704E-09   .19055E-10   .58982E-12   .12812E-13

J VALUE = ROW'INDEX =  4
 .19872E-09   .20221E-08   .20932E-09   .52418E-11   .23633E-12   .84403E-14   .16824E-15

J VALUE = ROW INDEX =  5
 .51612E-12   .65980E-11   .54670E-12   .14921E-13   .83490E-15   .25649E-16   .41314E-18
```

Table 4. PRINTED OUTPUT OF 'DELTA' COEFFICIENTS FOR REACH NUMBER 2 FOR WEEKS 1 AND 2

```
OBSERVATION POINT NUMBER     4 LOCATED AT J.I =     3.     1

THETAS AT THE END OF WEEK    1
------------------------------

J VALUE = ROW INDEX =   1
 .74923E-02  .70731E-03   .53765E-04   .10094E-05   .20757E-07   .78253E-10   .23318E-12

J VALUE = ROW INDEX =   2
 .15176E+00  .17943E-01   .14649E-02   .60681E-04   .16553E-05   .16933E-07   .90137E-10

J VALUE = ROW INDEX =   3
 .90997E+00  .47196E-01   .15106E-02   .19292E-04   .17371E-06   .23652E-08   .13374E-10

J VALUE = ROW INDEX =   4
 .20602E-01  .87059E-03   .20702E-04   .16939E-06   .16893E-08   .76885E-11  -.42450E-12

J VALUE = ROW INDEX =   5
 .67350E-04  .26147E-05   .45263E-07   .34647E-09   .94762E-12  -.82853E-13  -.26517E-14

THETAS AT THE END OF WEEK    2
------------------------------

J VALUE = ROW INDEX =   1
 .18830E-01  .25532E-02   .27422E-03   .82652E-05   .26314E-06   .15684E-08   .77224E-11

J VALUE = ROW INDEX =   2
 .20253E+00  .36101E-01   .44509E-02   .29320E-03   .12744E-04   .21307E-06   .18558E-08

J VALUE = ROW INDEX =   3
 .83487E+00  .70547E-01   .36937E-02   .84098E-04   .15231E-05   .45681E-07   .12084E-08

J VALUE = ROW INDEX =   4
 .35465E-01  .22270E-02   .85496E-04   .13437E-05   .33076E-07   .96767E-09   .20751E-10

J VALUE = ROW INDEX =   5
 .21677E-03  .11013E-04   .32714E-06   .56769E-08   .15889E-09   .38314E-11   .63514E-13
```

Table 5. PRINTED OUTPUT OF 'THETA' COEFFICIENTS FOR OBSERVATION POINT NUMBER 4 FOR WEEKS 1 AND 2

E. THE 'EPSILON' PROGRAM

Once the various $\delta(\ )$ (and $\theta(\ )$ coefficients) have been calculated and saved on cards or tape, one can proceed to calculate the ε coefficients defined in Eq.(18) from the system of Eqs.(19). A computer program was developed to calculate these $\varepsilon_{rp}(n)$ coefficients. There are altogether $R \times P \times N$ of these. The calculation of the $\varepsilon_{rp}(n)$ coefficients is straightforward but many other coefficients and quantities are also calculated as a result of interest expressed by the staff of the Sate Engineer's office. As a consequence the program is not ready for documentation as it is still in a state of flux.

F. ILLUSTRATIVE SIMPLE MANAGEMENT PROBLEMS

Once the *epsilon* coefficients have been calculated many management problems which involve interaction between stream and aquifer can be studied. Several papers have been written on the subject (Morel-Seytoux, 1975a; Morel-Seytoux, 1975c; Morel-Seytoux, 1975d; Morel-Seytoux, 1975e). Appendix C is a reproduction of one of these papers. In this paper a (fictitious) regulatory agency tries to develop a strategy of pumping quota to maintain quality standards in the river, protect the downstream senior water rights while minimizing the detrimental effects for a new residential development depending on the alluvial aquifer for water supply.

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APPENDIX IA

Letter from Dr. Jeris A. Danielson
dated October 31, 1974

JOHN D. VANDERHOOF
Governor

C. J. KUIPER
State Engineer

DIVISION OF WATER RESOURCES
Department of Natural Resources
300 Columbine Building
1845 Sherman Street
Denver, Colorado 80203

October 31, 1974

Dr. Hubert Morel-Seytoux
Department of Civil Engineering
Colorado State University
Fort Collins, Colorado 80521

Dear Dr. Morel-Seytoux:

In accordance with our discussion on October 29, 1974 concerning management problems and objectives which this office might have that are susceptible to solution through model techniques, the following objectives are presented for your consideration:

1) Determine optimal drawdown in an unconfined aquifer or optimal head decline in an artesian aquifer to maximize economic return from agricultural use.

2) Determine those areas adjacent to a stream where a given percentage of water recharged to the alluvium is available for pumping at a given time.

3) Determine the best conjunctive use of surface flows, surface storage, and groundwater for a given set of conditions to maximize the effective use of all waters available in a stream-aquifer system.

4) Determine maximum allowable head decline that can be permitted in a river reach without suffering a long-term degradation of groundwater yields.

5) Determine those areas where a given amount of recharge will result in drainage problems or damaging high water-table levels.

6) Determine the optimal area in a stream reach where recharge can be applied such that the maximum amount

of recharged water is available to a given grouping of wells in the following irrigation season.

7) Determine the expected losses of a reservoir run of a given amount and duration in a stream reach and identify what losses are recoverable and at what time assuming given river conditions.

8) Determine what amount of groundwater pumping can be permitted so as to not cause a depletion of stream flows above a given percentage during a given period of time.

9) Determine the amount of stream depletion that is repaired if all wells made a given percentage of their extractions available from surface water sources to meet senior surface right demands.

10) Develop a method for accurately estimating groundwater withdrawal by wells on a seasonal basis by using manageable samples.

11) Determine the impact on groundwater availability by ceasing historic winter irrigation practices and storing that water previously used in upstream reservoirs for release and use during the irrigation period.

12) Determine the impact on a stream hydrograph with respect to volume and time of changing a historic flood irrigation practice on a seasonal basis to a year-round industrial use assuming consumptive use is held constant.

I find your work most interesting and applicable to the problems that the Division of Water Resources faces, and would encourage you and your group to continue evaluation and development of your present model.

Very truly yours,

Jeris A. Danielson

Dr. Jeris A. Danielson
Deputy State Engineer

JAD:mvg

APPENDIX IB

"A DISCRETE KERNEL GENERATOR FOR STREAM-AQUIFER STUDIES"

A DISCRETE KERNEL GENERATOR FOR STREAM-AQUIFER STUDIES

by

H. J. Morel-Seytoux[1]

and

C. J. Daly[2]

July 1974

Revised November 1974

Accepted for Publication

To appear in April 1975 issue

of Water Resources Research Journal

---

[1] Professor of Civil Engineering, Colorado State University, Fort Collins, Colorado 80523

[2] Graduate Research Assistant, Colorado State University, Fort Collins, Colorado 80523

CEP74-75HJM-CJD1a

ABSTRACT

A finite difference model of an aquifer behavior without stream interaction was developed as a first stage component of a management model of a stream-aquifer system. The model is not built as a usual simulator but as a discrete impulse response generator. Once the basic response coefficients have been generated the finite difference model is no longer necessary to simulate the behavior of the aquifer. Any aquifer response (e.g. return flow to a given reach for a given week) is expressed as an explicit function of the pumping rates. A complete description of the "discrete kernel generator" is provided including the basic equations, truncation error propagation, accuracy and run costs.

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Table

A DISCRETE KERNEL GENERATOR FOR STREAM-AQUIFER STUDIES

INTRODUCTION

Some progress toward the solution of problems of management of groundwater and surface waters resources can be achieved through the use of mathematical models. Many hydrologic models of a stream-aquifer system have been developed in the past (e.g. Glover, 1960, Eshett and Bittinger, 1965). These were pioneering efforts and are very commendable. However as the titles of the following studies imply (Bittinger, 1967), the models were designed to predict the hydrologic behavior of the system in response to a particular set of numerical values of the excitations (e.g. pumping rates at a given well over several time periods) rather than provide a functional relation between the response and the excitation (or forcing function). In addition the responses to alternative strategies of management were evaluated in physical terms (e.g. volumes of water shortages) rather than in economic ones (e.g. losses in economic benefits). Today most hydrologists (or water resources engineers) would agree with the argument made by the economists (e.g. Young, 1970) that the ranking of project alternatives cannot and should not be made solely on the basis of volumes or flow rates of water.

Bredehoeft and Young (in 1970 and 1972) in a significant departure from previous works have combined in a single study a physically realistic hydrologic model of the stream-aquifer system with a realistic dscription of the economic behavior of the water users. Though they utilized Mathematical Programming to obtain the water user's optimal reaction to the hydrologic behavior of the system, their overall management plan remained suboptimal. It was adequate nevertheless to pass judgment on the relative values of several discrete alternative plans.

A definite evolution in the modeling of stream-aquifer system is apparent. At first the hydrologic model was viewed as an end in itself. Now it is viewed primarily as a necessary intermediate component of a more complex system. Since the role of the hydrologic model has changed, should not its design also be modified?

DESIGN OF THE HYDROLOGIC MODEL

First, we must ask the question: what information is the model expected to provide? For any pattern of well pumping the hydrologic model to be useful to the water user (e.g. a typical farmer) in the planning and operating stages must provide the drawdown in all the pumping wells for costs are related to the lifting heights. To be useful to the regulatory agency (e.g., the State Engineer of Colorado) in defining his strategy, to satisfy the senior legal rights while minimizing the detrimental effects to the junior water rights holders, the model must provide the stream losses to the aquifer as a response of pumping of wells and the drawdowns in the observation wells.

Immdiately it is apparent that the usual numerical (e.g. finite difference) models are somewhat inefficient for this type of dual service. For accuracy's sake, drawdowns must be calculated at many grid points where the information has no economic interest (e.g., no wells). In addition it is even impossible at such points to verify that the calculated value is correct. If it is necessary, nevertheless, to use a finite-difference model the calculations of these intermediate values must be reduced to a minimum.

Maddock (1972) points out that an efficient method to obtain the information only where needed is to use the Green function (Garabedian, 1964) of the boundary value problem. When the boundary conditions change (i.e., pumping rates) the drawdowns in the wells can be calculated from the knowledge of the Green function. Before proceding further with the analysis, the basic definitions needed for aquifer modeling are reviewed.

1. Basic Definitions and Equations

The basic saturated flow equation (using the Dupuit assumption and a few other traditional assumptions) describing the evolution of an isotropic water table aquifer is the Boussinesq equation:

$$\phi \frac{\partial s}{\partial t} - \frac{\partial}{\partial x} \left(T \frac{\partial s}{\partial x}\right) - \frac{\partial}{\partial y} \left(T \frac{\partial s}{\partial y}\right) = Q_w \delta_w + Q_r \delta_r \qquad (1)$$

where ϕ is the drainable (or effective) porosity, s is the drawdown measured positive downward from a (high) horizontal datum located at distance H above the datum for the water table elevation, t is time, x and y are the horizontal cartesian coordinates, T is the transmissivity, Q_w is the instantaneous pumping volume of well w (chosen algebraically positive if it is an actual withdrawal rate), δ_w is a Dirac delta function singular at the point of coordinates ξ_w, η_w and τ (where ξ_w and η_w are the x,y coordinates of well w, τ is time), Q_r is the aquifer instantaneous discharge volume to the river per unit reach of river and δ_r is a delta function singular along the r^{th} reach of the river. The various definitions are graphically illustrated on Figure 1. At first, Eq. (1) will be studied for the case when there is no stream intersecting the aquifer.

Assuming a homogeneous aquifer of infinite extent and no previous development then it is well known (Carslaw and Jaeger, 1969, pp. 258-261) that the drawdown at well w at time t due to pumping at well p at a rate $Q(\tau)$ (L^3T^{-1}) is:

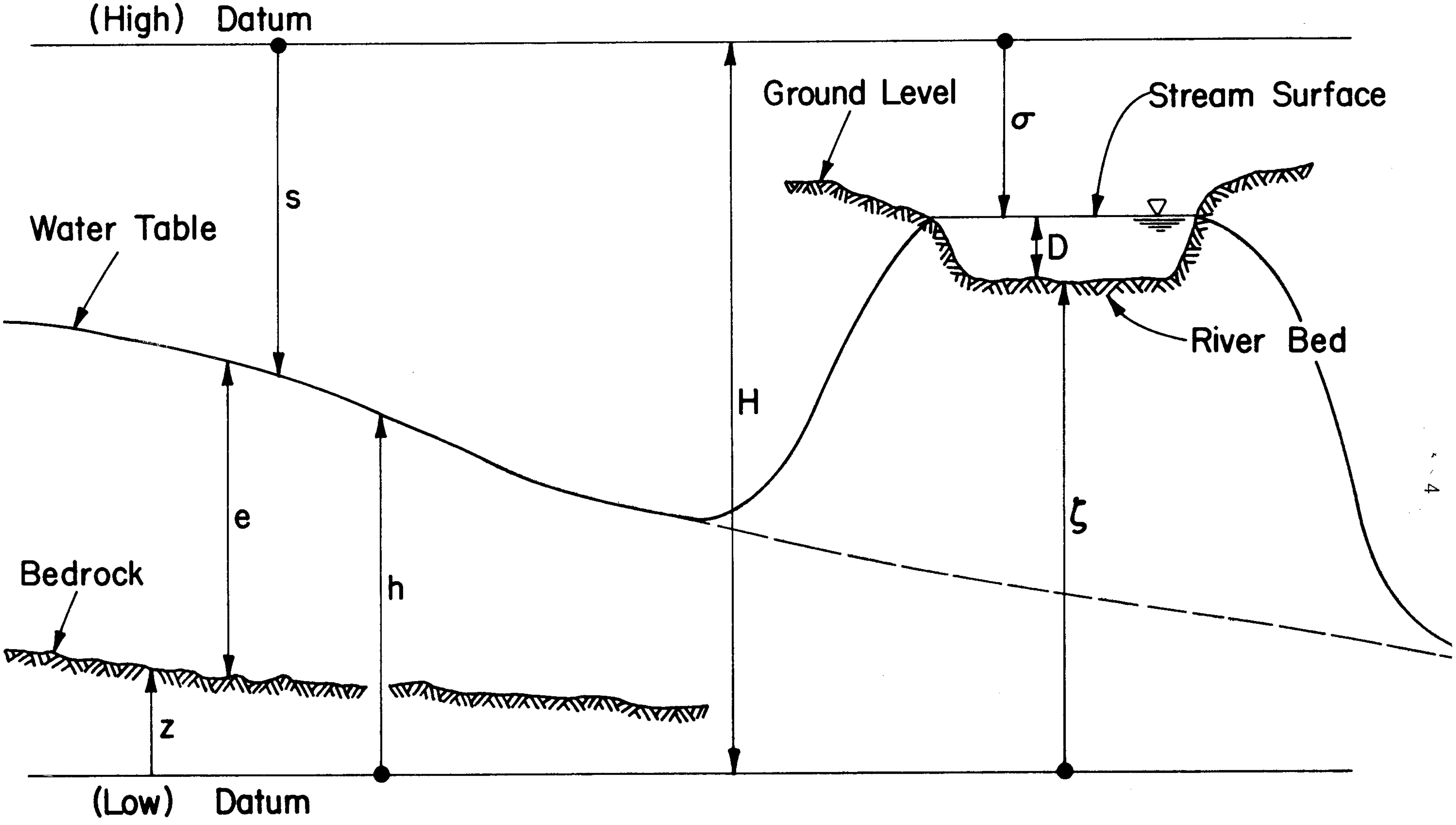

(High) Datum
s
Water Table
H
Ground Level
σ
Stream Surface
D
River Bed
e
h
ζ
Bedrock
z
(Low) Datum

Figure 1.

$$s_{wp}(t) = \int_0^t \frac{Q_p(\tau)\, e^{-\frac{\phi R_{wp}^2}{4T(t-\tau)}}}{4\pi T} \frac{dt}{t-\tau} \tag{2}$$

where R_{wp} is the distance between well w and well p. If the pumping rates are constant within the basic time period (say the week) but vary from week to week, it is easy to derive (think of the unit hydrograph theory; Dooge, 1973) the equation (Morel-Seytoux et al., 1973):

$$s_{w.}(n) = \sum_{p=1}^{P} \sum_{\nu=1}^{n} \delta_{wp}(\nu)\, Q_p(n-\nu+1) \tag{3}$$

where $s_{w.}(n)$ is the drawdown at well w at the end of the n^{th} week with P wells pumping, and where:

$$\delta_{wp}(\nu) = \frac{1}{4\pi T} \int_0^1 e^{-\frac{\phi R_{wp}^2}{4T(\nu-\tau)}} \frac{d\tau}{\nu-\tau} \tag{4}$$

When the aquifer is intersected by a stream the solution is of the same form because a reach can be viewed (at least mathematically) as a special well (pumping or injecting), namely:

$$s_{w.}(n) = \sum_{p=1}^{P} \int_0^t Q_p(\tau)\, k_{wp}(t-\tau)dt + \int_0^t Q_r(\tau)\, k_{wr}(t-\tau)d\tau \tag{5}$$

Eq. (5) involves two kernel functions, the pumping well kernel function (which is analytically known for the homogeneous aquifer of infinite extent, i.e. the integrand in Eq. (2)) and the pervious reach kernel, k_{wr} ( ). For a homogeneous aquifer of infinite extent this kernel can also be obtained analytically (Morel-Seytoux, 1974). Since most aquifers are neither homogeneous nor of infinite extent, the analytical expression itself is of no great value. What is important however is the fact that the classical theory of partial differential equations (Garabedian, 1964), the modern theory of Linear Systems (Dooge, 1973) and the derivation of Maddock (1972) have proven the existence of the kernel functions for heterogeneous aquifers of limited extent. That is Eq. (5) is still valid under these more realistic conditions.

Ignoring for the time being the fact that the kernel functions are not known, it is still not possible to exploit Eq. (5) because to the contrary of the pumping rates, $Q_p(\tau)$, the $Q_r(\tau)$ are not susceptible to man's control. The rate of seepage to or recharge from the aquifer depends on the state of the aquifer. It is necessary to relate $Q_r(\ )$ to drawdowns in the aquifer.

Bouwer (1969) has shown that Q_r is proportional to a difference in the drawdowns to the stream surface level and to the aquifer water table a few stream widths away from the stream. The coefficient of proportionality depends on the streambed characteristics and shape of the stream cross-section (Morel-Seytoux, 1964; Bouwer, 1969). Symbolically:

$$Q_r = \Gamma_r\ (\sigma_r - s_r) \tag{6}$$

If, as in the case of the State Engineer, one is interested in the flows at the stream-aquifer interface as affected by pumping from wells, and not in drawdowns in the aquifer, one can utilize Eq. (5) to calculate s_r and eliminate s_r from Eq. (5) by using Eq. (6) with the result:

$$Q_r(t) + \Gamma_r \int_0^t Q_r(\tau)\ k_{rr}(t-\tau)\ d\tau = \Gamma_r \left[\sigma_r(t) - \sum_{p=1}^{P} \int_0^t Q_p(\tau) k_{rp}(t-\tau)\ d\tau\right] \tag{7}$$

Eq. (7) is an integral equation for the unknown function of time $Q_r(t)$. In the case of several pervious reaches, Eq. (7) generalizes in the form:

$$Q_r(t) + \Gamma_r \sum_{\rho=1}^{R} \int_{o}^{t} Q_\rho(\tau)\ k_{r\rho}\ (t-\tau)d\tau = \Gamma_r \left[\sigma_r(t) - \sum_{p=1}^{P} \int_{o}^{t} Q_p(\tau)k_{rp}(t-\tau)\ d\tau\right] \tag{8}$$

where R is the number of reaches. Eqs. (8) is a system of R integral equations to be solved simultaneously. Due to the linear character of the system the solution for the $Q_\rho(t)$ $(\rho = 1 \ldots r \ldots R)$ can be obtained by superposition of the solutions for the two situations $\sigma_r = 0$ but $Q_p \neq 0$ and $\sigma_r \neq 0$ but $Q_p = 0$. For example the State Engineer wants to know the seepage from the river induced by pumping and not caused by other factors such as propagation of a flood. He is thus directly interested in the solution of Eq. (8) when $\sigma_r(t) \equiv 0$. The system of Eqs. (8) can be discretized with the result:

$$Q_r(n) + \Gamma_r \sum_{\rho=1}^{R} \sum_{\nu=1}^{n} \delta_{r\rho}(n-\nu+1)Q_\rho(\nu) + \Gamma_r \sum_{p=1}^{P} \sum_{\nu=1}^{n} \delta_{rp}(n-\nu+1)Q_p(\nu) = 0 \tag{9}$$

Because the system of Eqs. (9) is linear, the solution for $Q_r(n)$ is known a priori to be of the form:

$$Q_r(n) = \sum_{p=1}^{P} \sum_{\nu=1}^{n} \varepsilon^{n}_{rp}\ (\nu)\ Q_p(\nu) \tag{10}$$

where $Q_r(n)$ is the return volume to the reach r during the nth week, $Q_p(\nu)$ is the volume pumped at well p during the νth week, P is the total number of pumping wells and the ε are coefficients.

Eq. (10) does not include the return flow due to other causes than pumping from wells, such as for example the natural fluctuations of the river stage. Eq. (10) thus provides precisely the information needed by the regulatory agency which has the responsibility of managing the stream-aquifer system.

To determine the unknown coefficients $\varepsilon^n_{rp}(\nu)$ it suffices to substitute the solution (Eq. 10) in Eqs. (9) and equate to zero the coefficients of the $Q_p(\nu)$. This calculation is straightforward and is given in Appendix B. (More general results are given in Morel-Seytoux, 1974 for the case $\sigma \neq 0$).

Thus the coefficients $\varepsilon^n_{rp}(\nu)$ can be obtained recurrently by simple linear equations once the $\delta_{rp}(\ )$ and $\delta_{ri}(\ )$ coefficients are known. The meaning of these latter coefficients is fairly straightforward. If $s_{wp}(n)$ represents the drawdown at point w (anywhere in the aquifer) due to pumping at well p (anywhere in the aquifer) with pumped volumes $Q_p(1)$, $Q_p(w)$..., $Q_p(\nu)$ etc. during weeks 1, 2, ..., ν, etc., then:

$$s_{wp}(n) = \sum_{\nu=1}^{n} \delta_{wp}\ (n - \nu + 1)\ Q_p(\nu) \tag{11}$$

and δ_{wp} represents the drawdown at the end of the n^{th} week (or time period) if a unit volume of water was withdrawn during the first week from well p and the well shut down indefinitely thereafter.

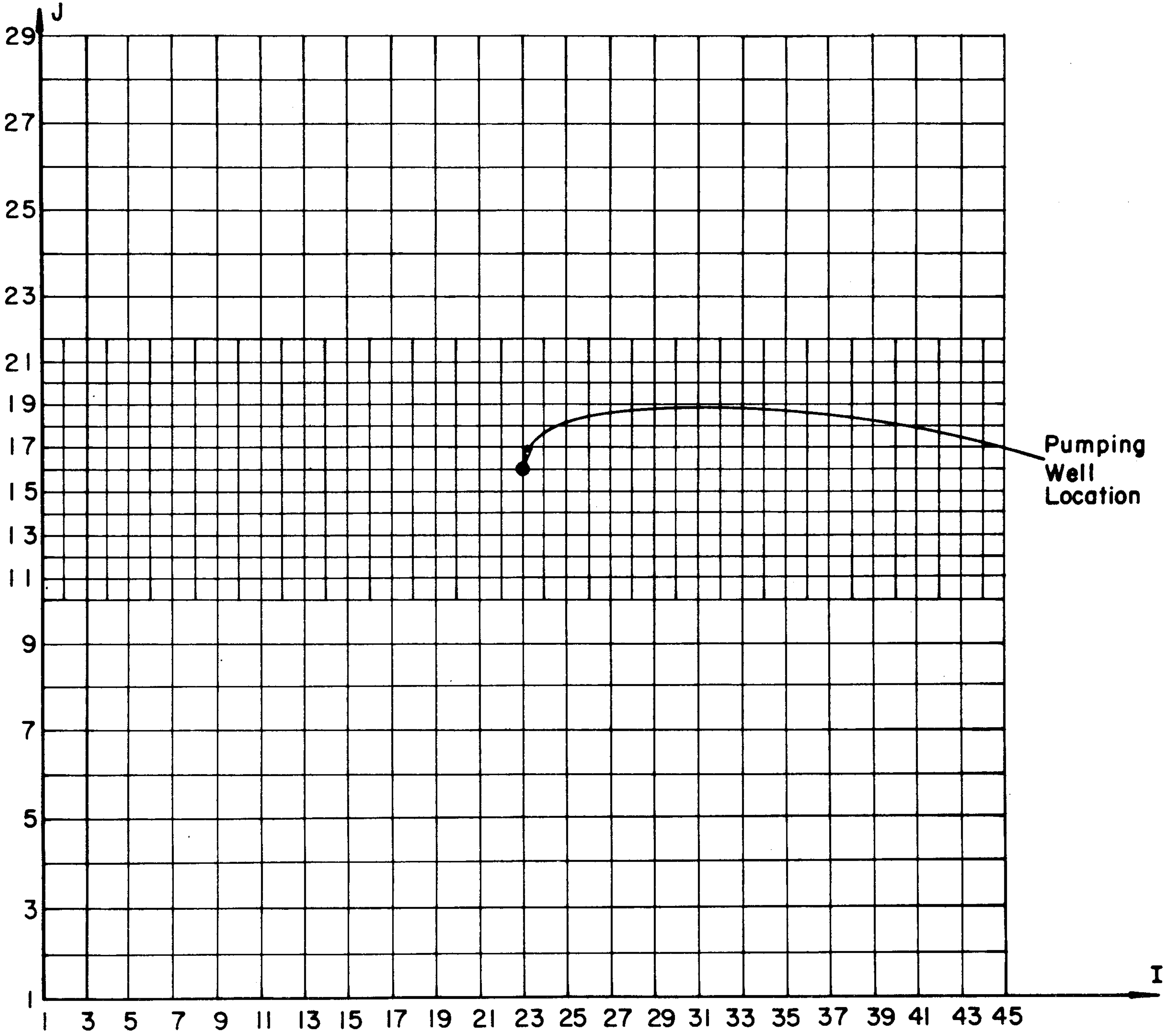
Speaking like the mathemetician we can say that the problem of conjunctive surface-groundwater management is solved once the δ_{wp} coefficients have been obtained! Unfortunately this is the costly part. It is the main purpose of this paper to document the cost, accuracy and feasibility of the Discrete Kernel Generator approach to solve actual groundwater management problems.

2. Numerical Determination of the δ Influence Coefficients

Many computer hydrologic models have already been developed to describe a stream-aquifer interaction (Pinder and Bredehoeft, 1968; Jenkins, 1968; Moulder and Jenkins, 1969; Bredehoeft and Young, 1970; Prickett and Lonnquist, 1971; Maddock, 1974).

The majority of these models, however, were designed to simulate repeatedly the response of the aquifer or the river to given patterns of pumping rather than generate once for all the ε coefficients. The model of Maddock (1974) is the exception. However in his model the ε influence coefficients are calculated by a finite difference model and they would have to be recalculated if for example a different location of a canal or of a recharge pond is considered. The approach proposed here, though not vastly different, is somewhat more flexible. A finite difference model is used to calculate the δ influence coefficients, that is, the coefficients characterizing the response to pumping without any stream interaction. Once the δ coefficients are obtained the ε coefficients are calculated from a simple linear algebraic system of equations. If the position of the canals or recharge ponds is modified, only the ε need be recalculated, not the δ coefficients.

The δ coefficients are generated by a finite difference model based on the well-known alternating directions implicit method (Douglas and Rachford, 1956; Pinder and Bredehoeft, 1968). For this reason the finite difference forms of the linear Boussinesq equation are given in Appendix A. The grid network consists of square meshes but the mesh size can change by a factor of 2 only. All grids are uniform in the x direction A typical possible grid layout is illustrated on Figure 2. With this grid flexibility it is possible to have a fine mesh generally


J
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5
7
9
11
13
15
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31
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35
37
39
41
43
45
Pumping
Well
Location

Figure 2.

parallel and close to the river, where many wells are located, and a coarse grid far from it.

3. Propagation of Errors in the Finite Difference Model

At a given time step the solution for drawdown depends on the previous solutions. As a result errors in the solution tend to propagate in time and grow. The solution of a system of linear equations by the Gaussian elimination procedure will reflect the errors in the estimation of the matrix of coefficients A, and the errors in the right-hand side vector. With a time-invariant linear system the errors in the coefficients of A are negligible. However errors in the estimation of the time-variant right-hand side vector, $\underline{b}$, tend to grow. Assuming negligible error in A it has been shown (Forsythe and Moller, 1967, p. 20) that an upper bound for the relative error in the solution of $A\underline{x} = \underline{b}$ (where the under bar indicates a vector) is given by:

$$\frac{||\delta x||}{||\underline{x}||} \leq C_A \frac{||\delta \underline{b}||}{||\underline{b}||} \tag{12}$$

where $|| \;\; ||$ indicates a norm and C_A is the condition number of A defined as $||A|| \cdot ||A^{-1}||$. The condition number depends on the chosen norm. With the infinity norm then by definition:

$$||A||_\infty = \text{maximum of } \sum_j |a_{ij}| \text{ over all rows i} \tag{13}$$

and:

$$||\delta \underline{x}||_\infty = \text{maximum element of } \delta \underline{x} \tag{14}$$

The error in x_i is denoted δx_i and the error in b_i is denoted δb_i. Because the alternating directions implicit (A.D.I.) method requires an intermediate solution at the half time step and starting pumping from one well at a unit rate at $t = \Delta t/2$ with the error e_o (CDC 6400 precision = 10^{-14}), the relative error at the end of the n^{th} week is:

$$\frac{\|\delta \underline{x}\|}{\|\underline{x}\|} = C_A^{\frac{2n}{\Delta t}} e_o \tag{15}$$

The condition number depends on the matrix coefficients and therefore depends on Δx, Δt and the aquifer characteristics. Ideally C_A should be as close to 1 as possible and Δx and Δt chosen accordingly. If A is diagonally dominant and symmetric, one obtains:

$$C_A = \frac{\max\limits_i(\sum\limits_j |a_{ij}|)}{\max\limits_i\{|a_{ii}| - \sum\limits_{j,j\neq i} |a_{ij}|\}} \tag{16}$$

For a homogeneous aquifer, a uniform grid and time-step size, using A.D.I. Eq. (16) yields

$$C_A = 1 + \frac{4T\Delta t}{\phi(\Delta x)^2} \tag{17}$$

where T is the aquifer transmissivity and ϕ is the drainable porosity. Correlatively the relative error on $\underline{x}$ at the end of the n^{th} week is:

$$e(n) = e_o\left(1 + \frac{4T\Delta t}{\phi(\Delta x)^2}\right)^{\frac{2n}{\Delta t}} \tag{18}$$

It is interesting to note that for a fixed Δx, reducing Δt reduces C_A but increases the exponent in Eq. (8). In the limit when Δt tends to zero we obtain:

$$e(n) = e_o\, e^{\frac{8Tn}{\phi(\Delta x)^2}} \tag{19}$$

For accuracy sake Δx should be as small as possible. However Eq. (9) shows clearly that when Δx becomes too small the propagated error will grow at a fast exponential rate. Table 1 illustrates the error growth as a function of the week index and for different time-step and grid sizes. Table 1 shows that for a fixed (relatively large Δx, $\Delta x = 500$ m) a reduction in Δt makes the error worse. Eq. (9) therefore provides an upper bound for the maximum error for all Δt as a function of n and Δx. Eq. (9) is especially useful if the time-step is varied during the computations. For example, if the relative error at the end of 16 weeks is to be less than 1 percent regardless of how small the time step may be, then Δx must be greater than 480 meters. However this estimate is much too conservative. In the actual computations an initial time step of 0.001 week is used and increased by the factor $r = 3/2$ during the first week when the well is pumping at a unit rate until Δt reaches the value 0.1. Then it is kept at that value for the rest of the first week. Starting at the beginning of the second week (when the well is permanently shut down) roughly the same pattern is followed until the time step reaches the value of 1 week. Thereafter the time step is 1 week. Under these conditions a better estimate for $e(n)$ is given approximately by the relation:

$$e(n) = e_o \prod_{i=1}^{12} [1 + 10^{-3} a (1.5)^{i-1}]^4 \prod_{i=13}^{18} [1 + 10^{-3} a (1.5)^{i-1}]^2 \cdot$$

$$(1 + 10^{-3} a)^{14} (1 + a)^{2(n-4)} \text{ for } n > 4 \qquad (20)$$

where $a = \frac{4T}{\phi (\Delta x)^2}$. For $\phi = 0.2$, $T = 10{,}000$ and $\Delta x = 350$ a takes the value 1.63. For this value of a Eq. (20) gives for

	Δt = 1 week		Δt = 0.1 week	
week index n	Δx = 50 m	Δx = 500 m	Δx = 50 m	Δx = 500 m
1	6.56×10^{-11}	3.24×10^{-14}	1.22×10^{5}	4.66×10^{-14}
2	4.30×10^{-7}	1.05×10^{-13}	1.48×10^{24}	2.17×10^{-13}
3	2.82×10^{-4}	3.40×10^{-13}	1.80×10^{43}	1.01×10^{-12}
5	1.22×10^{5}	3.57×10^{-12}	2.66×10^{81}	2.20×10^{-11}
8	3.43×10^{16}	1.21×10^{-10}	4.77×10^{138}	2.23×10^{-9}
10	1.48×10^{24}	1.27×10^{-9}	7.06×10^{176}	4.84×10^{-8}
16	1.18×10^{47}	1.47×10^{-6}	2.28×10^{291}	4.96×10^{-4}

Table 1. Table of relative errors at the end of n weeks as a function of time-step and grid sizes. T = 10,000 m^2/week and $\phi = 0.2$

e(11), e(12), e(13) and e(14) the values 0.0002, 0.0016, 0.011 and 0.08 respectively.

4. Accuracy of the Finite Difference Model

A good finite difference operator is one that converges to the partial differential operator of the original equation as Δx and Δt tend to zero, at least in theory. With such operator the solution of the finite difference approximation approaches the solution of the original equation in the limit. Thus in theory the solution gets better and better as Δx and Δt tend to be zero. However as discussed in the preceding section this result is only valid if the solution of the linear system of equations can be obtained with perfect accuracy. In practice perfect accuracy is not possible and even with machine accuracy of the order of 10^{-14}, errors will grow very rapidly if Δx or Δt are too small (see Table 1). To test the accuracy of the finite difference model as a function of Δx and Δt, the results of the finite difference model are compared with the known analytic solution for the single pumping well in a homogeneous aquifer of finite extent and no initial drawdown. The grid used for the generation of the influence coefficients δ is shown on Figure 2. The infinite aquifer is modeled by a finite rectangular aquifer of length 15,400 meters and width 16,800 meters. The grid size around the pumping well is 350 m. The number of grid points is 999. Figures 3 and 4 show a comparison of the δ coefficients as a function of the week index as calculated by the δ-generator and the analytical solution, for points located at distances from the pumping well equal to 350 and 1400 meters respectively. The relative error is small. The values of the δ coefficients are also given in Table 2.

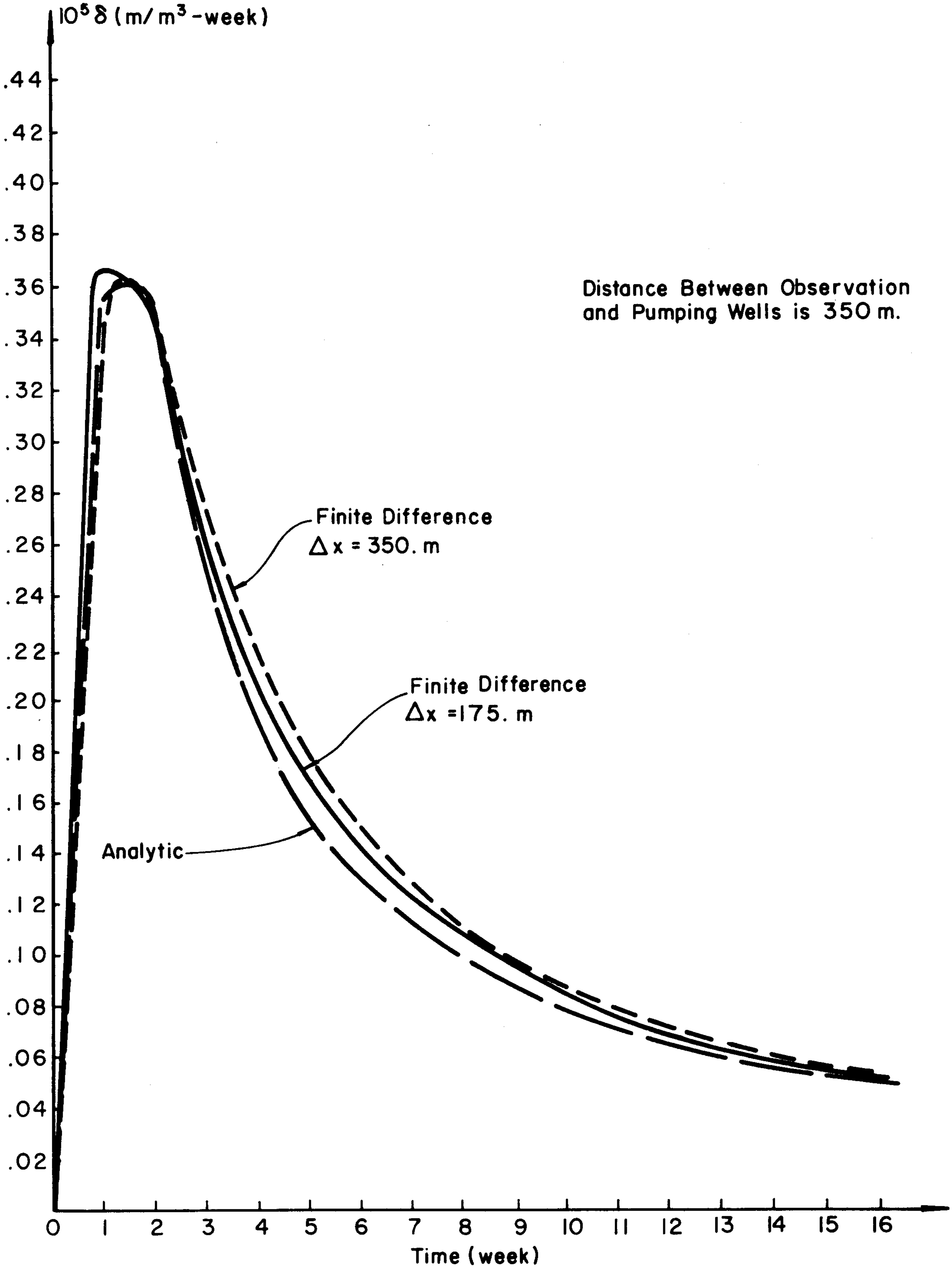
$10^5 \delta$ (m/m^3-week)
.44
.42
.40
.38
.36
.34
.32
.30
.28
.26
.24
.22
.20
.18
.16
.14
.12
.10
.08
.06
.04
.02
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Time (week)
Distance Between Observation and Pumping Wells is 350 m.
Finite Difference Δx = 350. m
Finite Difference Δx =175. m
Analytic

Figure 3.

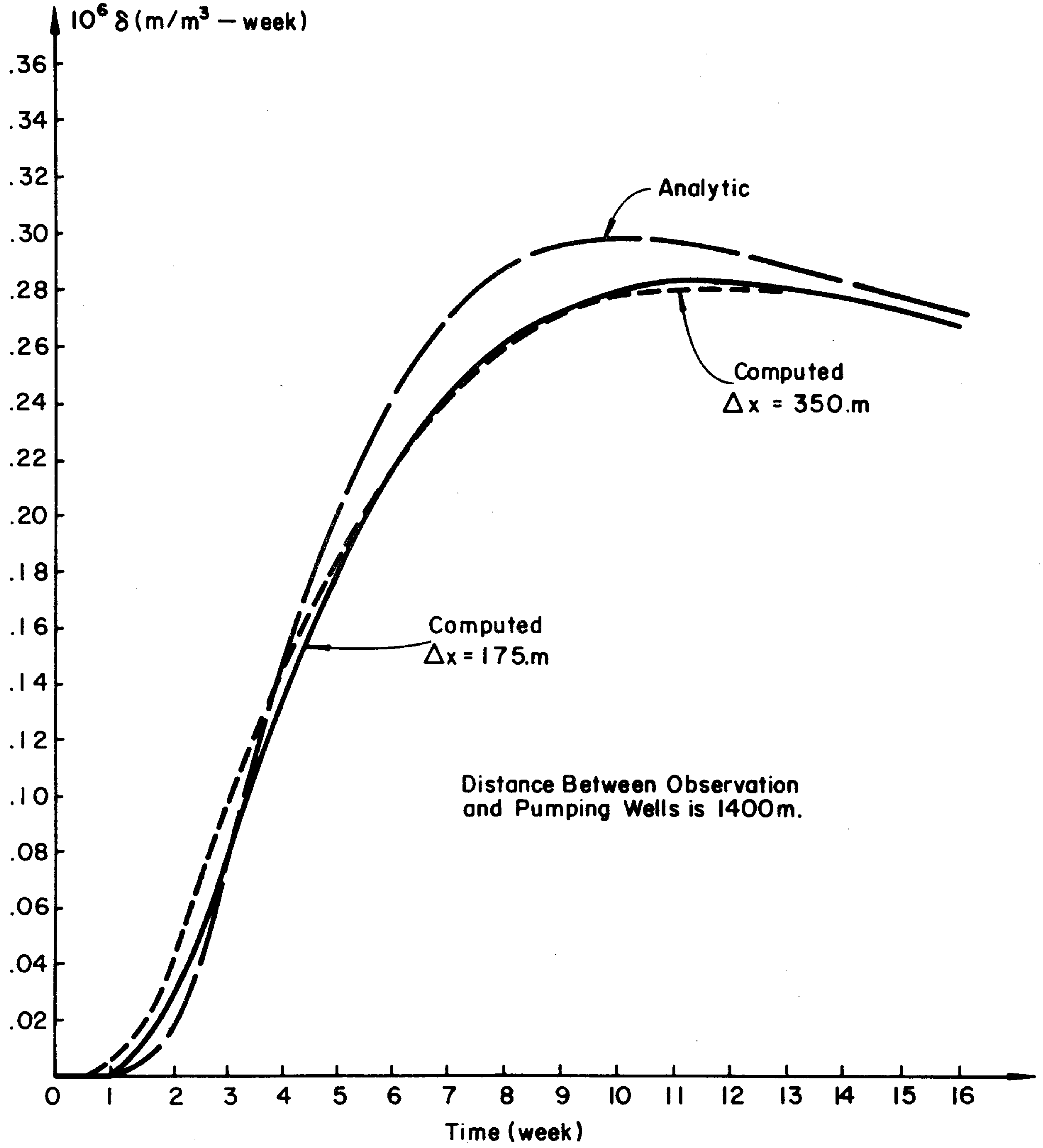
$10^6 \delta$ (m/m^3 — week)
.36
.34
.32
.30
.28
.26
.24
.22
.20
.18
.16
.14
.12
.10
.08
.06
.04
.02
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
Time (week)
Analytic
Computed
Δx = 350.m
Computed
Δx = 175.m
Distance Between Observation
and Pumping Wells is 1400m.

Figure 4.

distance from pumping well Week	350 m			1050 m			1400 m			2100 m			3150 m		
	Analytic	$\Delta x = 350$	$\Delta x = 175$	Analytic	$\Delta x = 350$ m	$\Delta x = 175$ m	Analytic	$\Delta x = 350$ m	$\Delta x = 175$ m	Analytic	$\Delta x = 350$ m	$\Delta x = 175$m	Analytic	$\Delta x = 350$ m	$\Delta x = 175$ m
1	$.353\ 10^{-5}$	$.332\ 10^{-5}$	$.367\ 10^{-5}$	$.503\ 10^{-8}$	$.459\ 10^{-7}$	$.207\ 10^{-7}$	$.412\ 10^{-10}$	$.438\ 10^{-8}$	$.111\ 10^{-8}$	$.917\ 10^{-16}$	$.302\ 10^{-10}$	$.200\ 10^{-11}$	$.447\ 10^{-28}$	$106\ 10^{-13}$	$.720\ 10^{-16}$
2	$.360\ 10^{-5}$	$.353\ 10^{-5}$	$.350\ 10^{-5}$	$.137\ 10^{-6}$	$.208\ 10^{-6}$	$.163\ 10^{-6}$	$.102\ 10^{-7}$	$.380\ 10^{-7}$	$.232\ 10^{-7}$	$.108\ 10^{-10}$	$.930\ 10^{-9}$	$.302\ 10^{-9}$	$.521\ 10^{-17}$	$.222\ 10^{-11}$	$.245\ 10^{-12}$
3	$.251\ 10^{-5}$	$.273\ 10^{-5}$	$.263\ 10^{-5}$	$.346\ 10^{-6}$	$.344\ 10^{-6}$	$.323\ 10^{-6}$	$.639\ 10^{-7}$	$.913\ 10^{-7}$	$.756\ 10^{-7}$	$.609\ 10^{-9}$	$.460\ 10^{-8}$	$.264\ 10^{-8}$	$.299\ 10^{-13}$	$.313\ 10^{-10}$	$.902\ 10^{-11}$
4	$.192\ 10^{-5}$	$.218\ 10^{-5}$	$.209\ 10^{-5}$	$.468\ 10^{-6}$	$.412\ 10^{-6}$	$.407\ 10^{-6}$	$.132\ 10^{-6}$	$.139\ 10^{-6}$	$.130\ 10^{-6}$	$.441\ 10^{-8}$	$.122\ 10^{-7}$	$.938\ 10^{-8}$	$.242\ 10^{-11}$	$.217\ 10^{-9}$	$.120\ 10^{-9}$
5	$.155\ 10^{-5}$	$.178\ 10^{-5}$	$.171\ 10^{-5}$	$.512\ 10^{-6}$	$.455\ 10^{-6}$	$.458\ 10^{-6}$	$.200\ 10^{-6}$	$.182\ 10^{-6}$	$.177\ 10^{-6}$	$.133\ 10^{-7}$	$.220\ 10^{-7}$	$.189\ 10^{-7}$	$.336\ 10^{-10}$	$.611\ 10^{-9}$	$.403\ 10^{-9}$
6	$.130\ 10^{-5}$	$.149\ 10^{-5}$	$.144\ 10^{-5}$	$.530\ 10^{-6}$	$.478\ 10^{-6}$	$.483\ 10^{-6}$	$.243\ 10^{-6}$	$.216\ 10^{-6}$	$.214\ 10^{-6}$	$.263\ 10^{-7}$	$.330\ 10^{-7}$	$.301\ 10^{-7}$	$.186\ 10^{-9}$	$.127\ 10^{-8}$	$.933\ 10^{-9}$
7	$.112\ 10^{-5}$	$.128\ 10^{-5}$	$.123\ 10^{-5}$	$.524\ 10^{-6}$	$.486\ 10^{-6}$	$.490\ 10^{-6}$	$.271\ 10^{-6}$	$.241\ 10^{-6}$	$.242\ 10^{-6}$	$.412\ 10^{-7}$	$.446\ 10^{-7}$	$.421\ 10^{-7}$	$.610\ 10^{-9}$	$.223\ 10^{-8}$	$.176\ 10^{-8}$
8	$.979\ 10^{-6}$	$.111\ 10^{-5}$	$.108\ 10^{-5}$	$.509\ 10^{-6}$	$.483\ 10^{-6}$	$.487\ 10^{-6}$	$.287\ 10^{-6}$	$.259\ 10^{-6}$	$.261\ 10^{-6}$	$.560\ 10^{-7}$	$.561\ 10^{-7}$	$.542\ 10^{-7}$	$.144\ 10^{-8}$	$.350\ 10^{-8}$	$.293\ 10^{-8}$
9	$.872\ 10^{-6}$	$.981\ 10^{-6}$	$.953\ 10^{-6}$	$.489\ 10^{-6}$	$.474\ 10^{-6}$	$.476\ 10^{-6}$	$.295\ 10^{-6}$	$.271\ 10^{-6}$	$.273\ 10^{-6}$	$.699\ 10^{-7}$	$.671\ 10^{-7}$	$.658\ 10^{-7}$	$.275\ 10^{-8}$	$.507\ 10^{-8}$	$.442\ 10^{-8}$
10	$.786\ 10^{-6}$	$.877\ 10^{-6}$	$.854\ 10^{-6}$	$.469\ 10^{-6}$	$.461\ 10^{-6}$	$.462\ 10^{-6}$	$.298\ 10^{-6}$	$.278\ 10^{-6}$	$.280\ 10^{-6}$	$.822\ 10^{-7}$	$.771\ 10^{-7}$	$.764\ 10^{-7}$	$.454\ 10^{-8}$	$.691\ 10^{-8}$	$.622\ 10^{-8}$
11	$.715\ 10^{-6}$	$.791\ 10^{-6}$	$.774\ 10^{-6}$	$.448\ 10^{-6}$	$.445\ 10^{-6}$	$.446\ 10^{-6}$	$.298\ 10^{-6}$	$.281\ 10^{-6}$	$.284\ 10^{-6}$	$.927\ 10^{-7}$	$.862\ 10^{-7}$	$.860\ 10^{-7}$	$.674\ 10^{-8}$	$.898\ 10^{-8}$	$.828\ 10^{-8}$
12	$.656\ 10^{-6}$	$.720\ 10^{-6}$	$.695\ 10^{-6}$	$.429\ 10^{-6}$	$.429\ 10^{-6}$	$.423\ 10^{-6}$	$.295\ 10^{-6}$	$.282\ 10^{-6}$	$.280\ 10^{-6}$	$.102\ 10^{-6}$	$.941\ 10^{-7}$	$.931\ 10^{-7}$	$.927\ 10^{-8}$	$.112\ 10^{-7}$	$.104\ 10^{-7}$
13	$.606\ 10^{-6}$	$.661\ 10^{-6}$	$.649\ 10^{-6}$	$.410\ 10^{-6}$	$.412\ 10^{-6}$	$.412\ 10^{-6}$	$.291\ 10^{-6}$	$.280\ 10^{-6}$	$.282\ 10^{-6}$	$.109\ 10^{-6}$	$.101\ 10^{-6}$	$.102\ 10^{-6}$	$.120\ 10^{-7}$	$.136\ 10^{-7}$	$.130\ 10^{-7}$
14	$.564\ 10^{-6}$	$.610\ 10^{-6}$	$.600\ 10^{-6}$	$.392\ 10^{-6}$	$.396\ 10^{-6}$	$.396\ 10^{-6}$	$.285\ 10^{-6}$	$.278\ 10^{-6}$	$.279\ 10^{-6}$	$.115\ 10^{-6}$	$.107\ 10^{-6}$	$.108\ 10^{-6}$	$.149\ 10^{-7}$	$.161\ 10^{-7}$	$.155\ 10^{-7}$
15	$.526\ 10^{-6}$	$.566\ 10^{-6}$	$.558\ 10^{-6}$	$.375\ 10^{-6}$	$.380\ 10^{-6}$	$.380\ 10^{-6}$	$.279\ 10^{-6}$	$.272\ 10^{-6}$	$.274\ 10^{-6}$	$.120\ 10^{-6}$	$.112\ 10^{-6}$	$.113\ 10^{-6}$	$.179\ 10^{-7}$	$.186\ 10^{-7}$	$.181\ 10^{-7}$
16	$.494\ 10^{-6}$	$.527\ 10^{-6}$	$.521\ 10^{-6}$	$.360\ 10^{-6}$	$.364\ 10^{-6}$	$.365\ 10^{-6}$	$.273\ 10^{-6}$	$.267\ 10^{-6}$	$.269\ 10^{-6}$	$.124\ 10^{-6}$	$.116\ 10^{-6}$	$.117\ 10^{-6}$	$.209\ 10^{-7}$	$.211\ 10^{-7}$	$.208\ 10^{-7}$

Table 2. Values of the δ coefficients (in meters/m^3-week) at different distances from the pumping well as a function of the week index, obtained by analytic procedures and by finite difference for two grid sizes.

Aquifer parameters are: $\phi = 0.2$ and $T = 10{,}000$ m^2/week.

5. Cost of the Finite Difference Model

As discussed in the preceding sections the error growth resulting from the solution of the linear system of equations puts a limit on how small the grid size may be. For a grid size of 350 meters according to Eq. (12) the error due to this cause could be large, especially for the last time period (the 16th period). However for the 12th week the relative truncation error should be only about 0.1 percent. It is clear from Figure 4, for example, that the relative deviation from the analytic solution is in excess of 0.1 percent, in fact of the order of five percent. Thus this deviation cannot be blamed on the truncation error but rather on the coarseness of the grid size. Indeed with a finer mesh ($\Delta x = 175$ m) the accuracy improved. The results are also shown on Figures 3 and 4.

The improvement in accuracy is clear but minor. On the other hand the computer costs increased by a factor of 7.5. It appears therefore that the major practical limitation to a small grid size will be cost. Indeed this is not a new discovery as Bredehoeft and Young repeatedly warned the eventual user. Said they: "Owing to limitations on computing resources, formal search procedures for determining the optimum were not employed, nor were the increments in pumping capacity between the various runs as small as might be desirable," (Young and Bredehoeft, 1972, p. 549); "Since computational resources did not permit a systematic sampling of the response surface..., no global maximum can be claimed," (Bredehoeft and Young, 1970, p. 7); "The limited resources available to the project precluded any detailed field studies of the hydrologic, legal and economic relationships necessary to represent a specific area accurately." (Bredehoeft and Young, 1970, p. 12.) "Therefore it was decided that the additional precision was not worth the extra costs, and all subsequent studies were made employing the 5-year interval" (Bredehoeft and Young, 1970, p. 15) and "Because of limitations of time and financial resources, it is infeasible to examine the sensitivity of the solutions to variables other than those already described." (Bredehoeft and Young, 1970, p. 19). The warning is clear. The hydrologic model must be very efficient computer-wise.

More precisely the breakdown of computer times and costs for the two grid sizes considered (Δx = 350 or 175 meters) is shown on Table 3. The principal computer costs are incurred for core storage at $60/hour and central processor time at $290/hour.

6. Cost Estimate for the Study of an Actual Reach. Conclusion

For a 50 mile long, 20 mile wide reach using a one mile by one mile grid a thousand grid points would be necessary.

Service	Grid Size = 350 m (999 grid points)	Grid Size = 175 m (3801 grid points)
Core Storage	42,600 bytes$_8$	122,500 bytes$_8$
Compilation Time	18.8 sec	18.8 sec
Central Processor Time	30.0 sec	113.1 sec
Cost Adjustment Factor (A function of Core Storage)	.7	1.4
TOTAL COST	$2.90	$21.83
TOTAL COST COMPARISON FACTOR	1.	7.5

Table 3. Cost distribution of runs as a function of grid size using a CDC 6400

If at least one well is present in every mesh then 16,000 δ values must be generated at a cost (see Table 3) of approximately \$3,000.00. After these costs have been incurred the ε coefficients must be calculated: 320,000 of them. The cost of determining these ε coefficients is not known precisely yet but preliminary studies indicate that it is less than that of calculating the δ coefficients. Ultimately the determination of optimal weekly pumping quota will require the solution of a Linear Programming problem with 16,000 variables and many legal constraints (16 times as many as there are surface diversion points). Since the optimization must be repeated every week in an irrigation season and every year, the initial fixed cost of determining the δ and ε coefficients distributed over every week of a 50 year horizon amounts to approximately \$4.00. Clearly the major cost involved in designing optimal rules of operation for conjunctive surface-groundwater management protecting the senior legal rights will come from the L.P. solution costs.

ACKNOWLEDGMENTS

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It is planned to apply the concepts described here on an actual alluvial aquifer in cooperation with Dr. Danielson, Deputy State Engineer, Colorado and Dr. Qazi, Chief of Planning and Investigations. Various algorithm approaches to the solution of the large scale L.P. management problem are under study.

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Appendix A

Finite Difference Form of the Boussinesq equation using the A.D.I. method.

Following Douglas and Rachford (1955) the finite difference form of the equation:

$$\phi \frac{\partial s}{\partial t} - \frac{\partial}{\partial x} (T \frac{\partial s}{\partial x}) - \frac{\partial}{\partial y} (T \frac{\partial s}{\partial y}) = Q \qquad \text{(A1)}$$

for a point A (see Figure 8) of coordinates (i,j) located in a region where the mesh size is uniform, is:

$$\begin{aligned} &[\frac{\phi_{ij}}{\Delta t} + 2 \frac{T_{ij}}{(\Delta x)^2}] s^*_{ij} - [\frac{\Delta T_{xij}}{2\Delta x} + \frac{T_{ij}}{(\Delta x)^2}] s^*_{i+1,j} \\ &+ [\frac{\Delta T_{xij}}{2\Delta x} - \frac{T_{ij}}{(\Delta x)^2}] s^*_{i-1,j} = [\frac{\phi_{ij}}{\Delta t} - \frac{2T_{ij}}{(\Delta y)^2}] s_{ijn} \\ &+ [\frac{\Delta T_{yij}}{2\Delta y} + \frac{T_{ij}}{(\Delta y)^2}] s_{i,j+1,n} - [\frac{\Delta T_{yij}}{2\Delta y} - \frac{T_{ij}}{(\Delta y)^2}] s_{i,j-1,n} \\ &+ Q\, \delta_{ijn} \qquad \text{(A2)} \end{aligned}$$

The solution of Eq. (A2) along lines $j = \text{constant}$ yields values of s^* at time index $n + \frac{1}{2}$. The index n refers to the old time and $n + 1$ refers to the new time. The values of s^* are calculated at all grid points from Eqs. (A2) and substituted in the equation:

$$\begin{aligned} &[\frac{\phi_{ij}}{\Delta t} + \frac{2T_{ij}}{(\Delta y)^2}] s_{i,j,n+1} + [\frac{-T_{ij}}{(\Delta y)^2} + \frac{\Delta T_{yij}}{2\Delta y}] s_{i,j-1,n+1} \\ &- [\frac{T_{ij}}{(\Delta y)^2} + \frac{\Delta T_{yij}}{2\Delta y}] s_{i,j+1,n+1} = \frac{\phi_{ij}}{\Delta t} s^*_{ij} + \frac{2T_{ij}}{(\Delta y)^2} s_{ijn} + \\ &[- \frac{T_{ij}}{(\Delta y)^2} + \frac{\Delta Ty_{ij}}{2\Delta y}] s_{i,j-1,n} - [\frac{T_{ij}}{(\Delta y)^2} + \frac{\Delta T_{yij}}{2\Delta y}] s_{i,j+1,n} \\ &\qquad \text{(A3)} \end{aligned}$$

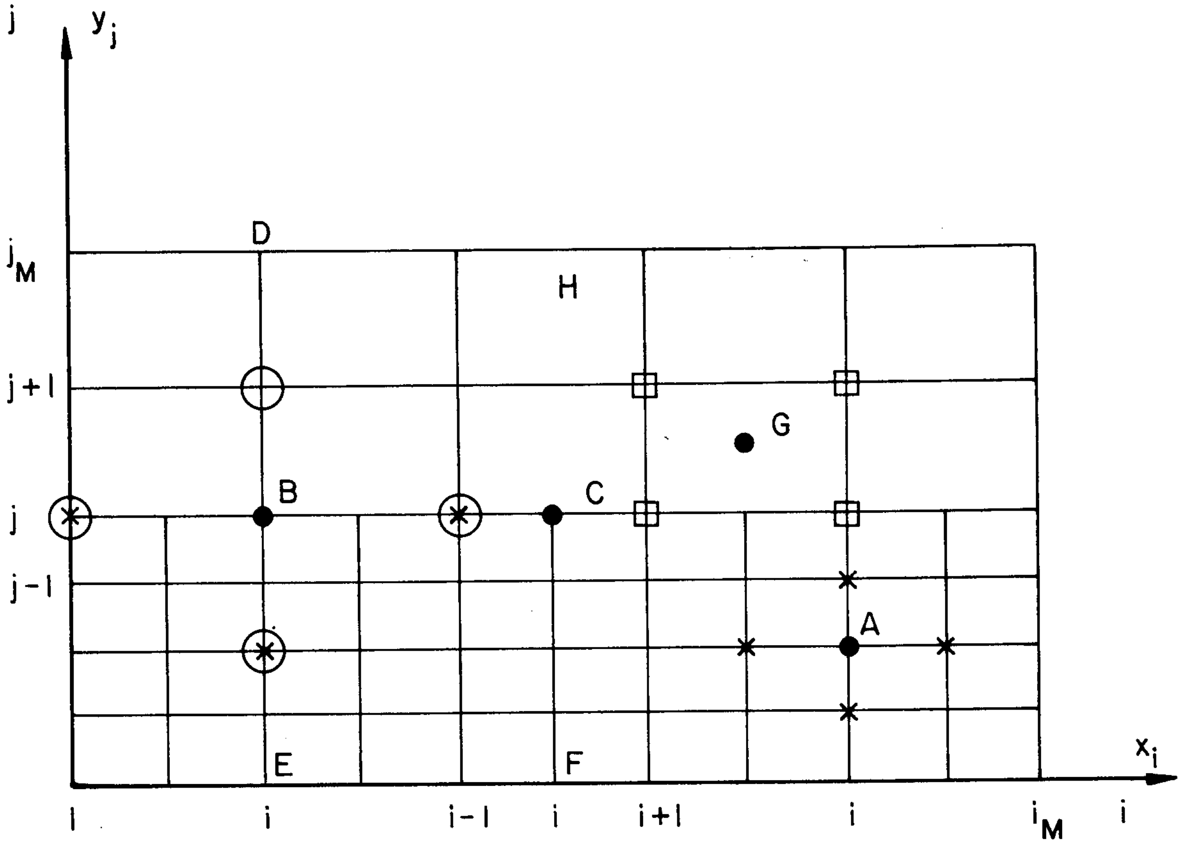

Figure 5.

Eqs. (A2) and (A3) apply for a rectangular grid ($\Delta x \neq \Delta y$). However in the developed computer program it is assumed that $\Delta x = \Delta y$.

The solution of Eqs. (A3) along the lines i = constant for the unknowns $s_{i,j,n+1}$ yield the drawdowns at the grid points of coordinates i,j at time index $n+1$. In the above equations Δt represents the time step. It may vary from time step to time step. Q is the pumping rate at point (i,j) during the period interval Δt. δ_{ijn} equals 1 if i,j are the coordinates of a pumping well, pumping during the time interval (t_n, t_{n+1}) and is zero otherwise. By definition

$$\Delta T_{xij} = \frac{T_{i+1,j} - T_{i-1,j}}{2\Delta x} \tag{A4}$$

and

$$\Delta T_{yij} = \frac{T_{i,j+1} - T_{i,j-1}}{2\Delta y} \tag{A5}$$

Equation (A4) applies even at the interface between two different mesh sizes regions with Δx equal to the smaller mesh size. Equation (A4) does not apply on the edges of the domain ($i = 1$ and i = maximum value of index $i = i_M$). At the edges we have:

$$\Delta T_{x,1,j} = \frac{T_{2j} - T_{1j}}{\Delta x} \tag{A6}$$

and

$$\Delta T_{x,i_M,j} = \frac{T_{i_M,j} - T_{i_{M-1},j}}{\Delta x} \tag{A7}$$

Likewise in the vertical direction we have:

$$\Delta T_{y,i,1} = \frac{T_{i,2} - T_{i,1}}{\Delta y} \tag{A8}$$

and

$$\Delta T_{y,i,j} = \frac{T_{i}{}^{k}{}_{j_M} - T_{i,j_M-1}}{\Delta y} \tag{A9}$$

In a region of changing grid size Eq. (A3) does not apply and the solution of Eq. (A2) requires modification. Consider the point B of coordinate (i,j) on Figure 8 located on the interface between two regions of different grid size. Eq. (A2) will apply with the interpretation that the subscript j-1 refers to an ordinate $y_j - \max(\Delta x)$. In the case of point B shown on Figure 8 the s* values in the finite difference equation are the ones evaluated at the points shown by ⓧ. In the case of point C no Eq. (A2) is used but rather s_c^* is evaluated as the average of the two neighboring values of s* on the j line.

When proceeding with the vertical direction Eq. (A3) does not apply at all if the point (i,j) is at the interface of a changing mesh. If as shown on Figure 8 the mesh size increases as j increases the proper finite difference equation is:

$$\left[\frac{\phi_{ij}}{\Delta t} + \frac{T_{ij}}{(\Delta y)^2} - \frac{\Delta T_{yij}}{\Delta y}\right] s_{i,j,n+1} - \frac{T_{ij}}{3(\Delta y)^2} s_{i,j+1,n+1}$$

$$+ \left[-\frac{2}{3}\frac{T_{ij}}{(\Delta y)^2} + \frac{\Delta T_{yij}}{\Delta y}\right] s_{i,j-1,n+1} = \frac{\phi_{ij}}{\Delta t} s^*_{ij}$$

$$+ \left[\frac{T_{ij}}{(\Delta y)^2} - \frac{\Delta T_{yij}}{\Delta y}\right] s_{i,j,n} + \left[\frac{-2}{3}\frac{T_{ij}}{(\Delta y)^2} + \frac{\Delta T_{yij}}{\Delta y}\right] s_{i,j-1,n}$$

$$- \frac{T_{ij}}{3(\Delta y)^2} s_{i,j+1,n} \tag{A10}$$

where Δy ($= \Delta x$) is the smaller of the two grid sizes. In this case the definition of ΔT_{yij} is:

$$\Delta T_{yij} = \frac{T_{i,j+1} - T_{i,j-2}}{4\,\Delta y} \tag{A11}$$

where $\Delta y (= \Delta x)$ is the smaller of the two mesh sizes, for a point such as B. For points such as C, ΔT_{yij} is calculated as the average of ΔT_{yij} at its nearest two neighbors along the line j = constant. A similar equation is obtained if the mesh size decreases as j increases, namely:

$$\left[\frac{\phi_{ij}}{\Delta t} + \frac{T_{ij}}{(\Delta y)^2} + \frac{\Delta T_{yij}}{\Delta y}\right] s_{i,j,n+1} - \frac{T_{ij}}{3(\Delta y)^2} s_{i,j-1,n+1}$$

$$- \left[\frac{2}{3} \frac{T_{ij}}{(\Delta y)^2} + \frac{\Delta T_{yij}}{\Delta y}\right] s_{i,j+1,n+1} = \frac{\phi_{ij}}{\Delta t} s^*_{ij} + \left[\frac{T_{ij}}{(\Delta y)^2} + \frac{\Delta T_{yij}}{\Delta y}\right] s_{i,j,n}$$

$$- \frac{T_{ij}}{3(\Delta y)^2} s_{i,j-1,n} - \left[\frac{2}{3} \frac{T_{ij}}{(\Delta y)^2} + \frac{\Delta T_{yij}}{\Delta y}\right] s_{i,j+1,n} \qquad \text{(A12)}$$

Equations (A10) or (A12) apply only on vertical lines such as EBD which extend from boundary to boundary. In the case of line FCH which terminates at point C Eq. (A10) can be used only after solutions along the lines such as EBD have been obtained. Then $s_{i,j+1,n+1}$ (value of s at point G) is estimated as the average of the four neighboring points (shown as □ on Figure 8). This establishes the necessary boundary condition to be met along lines such as FCH.

Appendix B

Derivation of the $\varepsilon^{n}_{rp}(\nu)$ coefficients

The discrete form of the system of Eqs. (9) is:

$$Q_r(n) + \Gamma_r \sum_{\rho=1}^{R} \sum_{m=1}^{n} \delta_{r\rho}(n-m+1)\, Q_\rho(m) + \Gamma_r \sum_{p=1}^{P} \sum_{\nu=1}^{n} \delta_{rp}(n-\nu+1) Q_p(\nu) = 0 \tag{B1}$$

The solution is given by Eq. (10) which can be rewritten, playing on indices, as

$$Q_\rho(m) = \sum_{p=1}^{P} \sum_{\nu=1}^{m} \varepsilon^{m}_{\rho p}(\nu)\, Q_p(\nu) \tag{B2}$$

Substitution of Eq. (10) and (B2) in (B1) yields:

$$\sum_{p=1}^{P} \sum_{\nu=1}^{n} \varepsilon^{n}_{rp}(\nu) Q_p(\nu) + \Gamma_r \sum_{p=1}^{P} \sum_{\rho=1}^{R} \sum_{m=1}^{n} \sum_{\nu=1}^{m} \delta_{r\rho}(n-m+1)\, \varepsilon^{m}_{\rho p}(\nu)\, Q_p(\nu)$$

$$+ \Gamma_r \sum_{p=1}^{P} \sum_{\nu=1}^{n} \delta_{rp}(n-\nu+1)\, Q_p(\nu) = 0 \tag{B3}$$

The coefficient of a particular $Q_p(\nu)$ i.e. where p and ν are fixed values (not summation indices from 1 to P and 1 to n) in the first term is simply $\varepsilon^{n}_{rp}(\nu)$. In the third term similarly it is simply $\Gamma_r\ \delta_{rp}(n-\nu+1)$. The second term requires a little more attention. The variable $Q_p(\nu)$ for a particular ν value will not appear in the summation $\sum_{\nu=1}^{m}$, where ν is a running index unless m is $\geq$ the particular ν value. Thus the coefficient of $Q_p(\nu)$ in the second term is $\Gamma_r \sum_{\rho=1}^{R} \sum_{m=\nu}^{n} \delta_{r\rho}(n-m+1)\ \varepsilon^{m}_{\rho p}(\nu)$.

Finally the equations determining $\varepsilon^{n}_{rp}(\nu)$ are:

$$\varepsilon^{n}_{rp}(\nu) + \Gamma_r \sum_{\rho=1}^{R} \sum_{m=\nu}^{n} \delta_{r\rho}(n-m+1)\ \varepsilon^{m}_{\rho p}(\nu) + \Gamma_r\ \delta_{rp}(n-\nu+1) = 0 \tag{B4}$$

for $r = 1, ..R$; $n = 1, 2...N$; $p = 1..., P$; $\nu = 1,2...n$ leading apparently to RPN(N-1) equations to solve simultaneously. N is the total number of weeks of interest. In fact what is required is the successive solution of P independent systems of R unknowns to be repeated N times. The procedure is to solve Eqs. (B4) for $n = 1$, $\nu = 1$, $p = 1$ for all possible values of $r(r = 1, ...R)$. One then obtains all the coefficients $\varepsilon_{r1}^{1}(1)$ for $r = 1, 2...R$. One repeats for $p = 2$, etc... and then obtains all the $\varepsilon_{rp}^{1}(1)$ for $r = 1, ...R$; $p = 1, ...P$.

Then one repeats the procedure for $n = 2$, $\nu = 1$, $p = 1$ for all r ($r=1, ...R$) etc... One then obtains all the $\varepsilon_{rp}^{2}(1)$ for all r and p. There is no need to calculate the $\varepsilon_{rp}^{2}(2)$ because the system is a time-invariant one and consequently it follows that generally:

$$\varepsilon_{rp}^{n}(\nu) = \varepsilon_{rp}^{n-\nu+m}(m) \text{ with } 1 \leq m;\ \nu \leq n \tag{B5}$$

Thus $\varepsilon_{rp}^{2}(2) = \varepsilon_{rp}^{1}(1)$ and all the $\varepsilon_{rp}^{2}(2)$ for all r and p values are known without calculation.

For the case $n = 3$ it is only necessary to calcualte the $\varepsilon_{rp}^{3}(1)$ because from (B5) it follows that $\varepsilon_{rp}^{3}(2) = \varepsilon_{rp}^{2}(1)$ and the latter are already known, and $\varepsilon_{rp}^{3}(3) = \varepsilon_{rp}^{2}(2) = \varepsilon_{rp}^{1}(1)$ and the latter are already known. In short all elements of the (NxN) $\varepsilon_{rp}^{n}(\nu)$ matrix for fixed r and p on any diagonal are equal.

These results become obvious if one wrote the ε coefficients as $\varepsilon_{rp}(n-\nu+1)$ instead of $\varepsilon_{rp}^{n}(\nu)$. This alternative notation has the disadvantage of making the already long equations such as (B3) or (B4) even longer.

APPENDIX IC

"OPTIMAL OPERATION OF SURFACE AND GROUND WATERS FOR POLLUTION DILUTION"

OPTIMAL OPERATION OF SURFACE AND GROUNDWATERS FOR POLLUTION DILUTION

by

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International Association for Hydraulic
Research

OPTIMAL OPERATION OF SURFACE AND GROUND WATERS FOR POLLUTION DILUTION

(Category D.c. River Basin Management)

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Synopsis

The applicability of a new approach to hydrologic modeling is demonstrated on a highly simplified case, for the purpose of illustration. In this highly simplified illustration the Environmental Quality regulatory agency is faced with the decision of imposing weekly quota on volume of water withdrawn from a well serving a new subdivision, in order to preserve both the downstream water quality standards and the senior water rights of a farmer. Decisions are made at the beginning of the irrigation season and then updated every week. The minimum needed storage capacity to supply the subdivision while satisfying the downstream legal and quality constraints is calculated each week. From the distribution of these various minimum capacities a design capacity is then selected.

Résumé

On démontre sur un cas simple la valeur d'un modèle hydrologique de type nouveau pour des problèmes de gestion. Dans le cas étudié le Directeur de l'Agence de Bassin doit imposer des quota hebdomadaires aux débits prélevés sur la nappe aquifère pour des besoins domestiques afin de satisfaire des droits d'eau prioritaires et maintenir une qualité adéquate en aval. La valeur des quotas est décidée au début de la saison, puis réévaluée d'une semaine à l'autre. Chaque semaine on calcule la capacité minimum de la réserve nécessaire pour l'alimentation domestique qui satisfait l'ensemble des contraintes du problème. Cette distribution de capacités minimales permet de choisir rationellement une valeur donnée pour la reserve.

INTRODUCTION

Though sometimes discredited, in-stream treatment and stream dilution of pollution remain the cheapest means of achieving minimum environmental quality standards. In areas which are both water-short (such as the South-Western part of the United States) and yet utilizing intensively their meager water resources, maintaining minimum flows for environmental quality is a difficult management problem. The problem is particularly difficult in period of droughts and is further complicated by the body of laws within which the regulatory agency must operate. A realistic management model of a stream-aquifer system cannot ignore the legal constraints any more than the physical constraints. Nor can such a model ignore the economic factors and the hydrologic uncertainties.

Whereas in principle the existing hydrologic models reported in the literature describing quantitatively the stream-aquifer interactions, could be linked to an operational decision-making model, the required number of simulation runs and their cost may discourage the regulatory agencies from using them. It is imperative that efficient hydrologic models be developed. One such model has been developed and described in the literature (Morel-Seytoux, 1975; Morel-Seytoux and Daly, 1975). Briefly the model combines the classical finite-difference method with the efficient systematic generation of solutions by the Green's function approach. The optimal rules of operation are deduced from a well structured Mathematical Programming formulation for which efficient solution algorithms exist.

The applicability of this new approach to hydrologic modeling is demonstrated on a highly simplified case, for which an analytical solution exists, for the purpose of illustration. In the highly simplified illustration the Environmental Quality regulatory agency is faced with the decision of imposing weekly quota on volume of water withdrawn from a well serving a new subdivision, in order to preserve both the downstream water quality standards and the senior water rights of a farmer. Whereas the legal and physical constraints on the problem are easily formulated and unchanging, it is not clear a priori what objective function should be optimized.

ILLUSTRATIVE (FICTITIOUS) EXAMPLE

Downstream from a reach in hydraulic connection with a (former river bed) aquifer a farmer is entitled since 1865 to divert a flow of 300 m^3/week to irrigate his fields. In 1974 a new residential area has just been finished and in December 1974 the Developer has petitioned for the right to drill a well to supply the residential area. In January 1975 a decree is granted. The Director of the Water Quality Control Agency is concerned with the impact of the withdrawals from the aquifer on the stream runoff and particularly on the B.O.D. (biochemical oxygen demand) load just below the municipal discharge (Figure 1). Engineering studies performed for the State Agency have shown that B.O.D. loads in excess of 1.5 ppm will produce environmental hazards further downstream, in violation of the recent State Water Quality Act of 1972. In view of the proximity of the well to the stream (100 meters) the Director decides to investigate further the situation.

From the State Engineer's office he is told that the aquifer is remarkably homogeneous and extensive of known transmissivity, T=10,000 m^2/week (courtesy U.S. Geological Survey) and that the seepage transmissivity of the reach is Γ = 4,000 m^2/week. "What would be the effect of pumping from the well on the runoff just above the municipal discharge point?" asks the Director. Utilizing

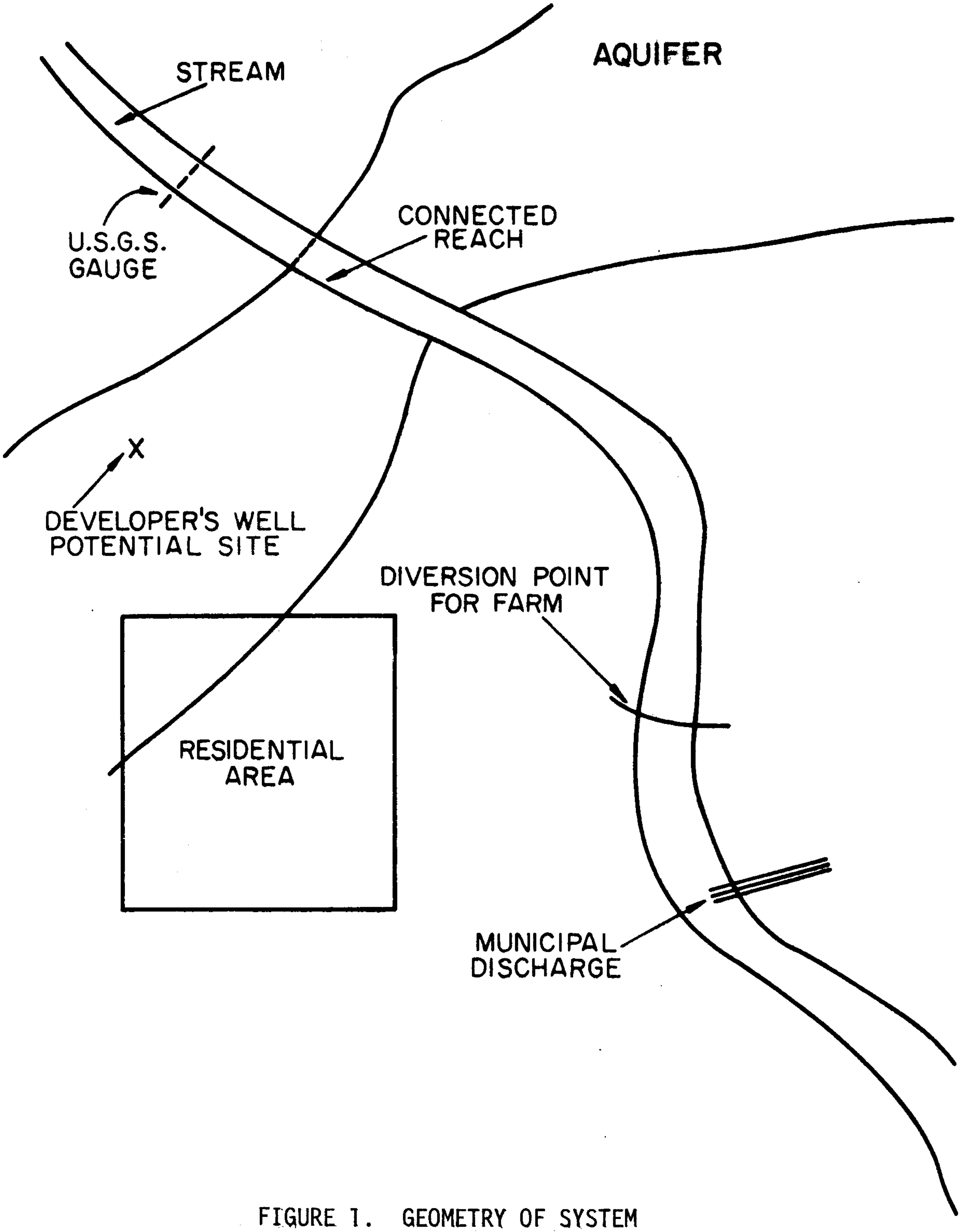

FIGURE 1. GEOMETRY OF SYSTEM

the Water Resources Division's new computer program to solve stream-aquifer problems, the answer is quickly given. The runoff just above the municipal outlet during week n, R_d (n), is:

$$R_d\ (n) = R_e\ (n) - W_r - \sum_{\nu=1}^{n} \varepsilon\ (n-\nu+1)\ Q\ (\nu) \tag{1}$$

for n = 1,2... 16, there being 16 weeks in the irrigation season. In Eq. (1) R_e (n) is the expected flow for week n (shown in column 3 in Table 1) at the U.S.G.S. gauge, W_r is the farmer's water right (300 m^3/week in this case), B (n) is the municipal discharge (given in Table 1), the ε are positive coefficients (tabulated in Morel-Seytoux, 1975) and Q (n) is the pumping rate.

The recommended B.O.D. maximum concentration, C_s (=1.5 ppm in this case), will not be exceeded provided the inequality

$$\frac{C_R\ R_d\ (n) + C_b\ B(n)}{R_d(n) + B(n)} \leqslant C_s \tag{2}$$

is satisfied, where C_R is the upstream B.O.D. concentration (in this case essentially zero) and C_b is the municipal discharge B.O.D. concentration (in this case 20 ppm). Eq. (2) takes the final form, after substitution of the expression of R_d (n) from Eq. (1) into Eq. (2) and rearranging:

$$\sum_{\nu=1}^{n} \varepsilon(n-\nu+1)\ Q(\nu) \leqslant R_e\ (n) - 12.33\ B(n) - W_r \tag{3}$$

1. Director's minimum storage strategy

Taking his cue from the State Engineer's experience (Morel-Seytoux, 1975) the Director decides to select the weekly pumping rates, Q(n), by solving a Linear Programming (L.P.) problem, namely:

$$\text{Min}\ \left\{S\right\} \tag{4}$$

with respect to the Q (ν), ν = 1,2... 16 and S (storage), subject to the non-negativity conditions:

$$Q\ (\nu) \geqslant o,\ \nu = 1,2...\ 16;\quad S \geqslant o \tag{5}$$

the quality constraints, defined by Eq. (3), the farmer's rights constraint, namely:

$$\sum_{\nu=1}^{n} \varepsilon(n-\nu+1)\ Q\ (\nu) \leqslant R_e\ (n) - W_r \qquad n = 1,2...\ 16 \tag{6}$$

the demand constraint:

$$\sum_{\nu=1}^{n} Q\ (\nu) \geqslant 200n \qquad n = 1,2...\ 16 \tag{7}$$

$B(\nu)$ (m^3)	Remaining Weeks ν	Conditional Expected Runoffs (m^3)							
		16	15	14	13	12	11	13'	11'
30	1	1000	--	--	--	--	--	--	--
32	2	1100	1025	--	--	--	--	--	--
34	3	1200	1120	1400	--	--	--	--	--
36	4	1150	1080	1340	1100	--	--	1000	--
38	5	1000	955	1180	970	1025	--	900	--
40	6	900	860	1020	880	925	950	840	840
40	7	850	823	950	840	870	880	810	810
38	8	820	800	910	815	835	840	785	785
37	9	800	785	875	785	805	810	765	765
36	10	780	780	850	770	785	790	750	750
35	11	765	755	825	755	770	775	735	735
34	12	750	740	800	740	755	755	725	725
33	13	740	735	785	735	745	745	720	720
32	14	730	726	770	725	735	735	715	715
31	15	725	723	755	720	725	725	710	710
30	16	720	715	740	715	720	720	707	707
Linear Programming Cost in $			0.66	0.58	0.51	0.44	0.43	0.34	0.30
Central Processing Time (seconds)			5.4	4.7	4.0	3.4	3.3	2.8	2.3
Input-Output Time (seconds)			1.1	1.1	1.0	1.0	1.0	1.0	1.0

Table 1. Conditional Expected River Flows and Municipal Discharge.

(the subdivision weekly demand is 200 m^3) and the feasibility constraint:

$$\sum_{\nu=1}^{n} Q(\nu) - 200\,n \leqslant S \qquad n = 1,2\text{--}\ 16 \tag{8}$$

The solution to this problem leads to the pumping schedule shown as line 1 of Table 2. All constraints are satisfied and no storage is required on the part of the Developer. Everybody is happy! Apparently the Director's fears were not justified.

2. Director's stochastic storage strategy

Reporting the happy ending of his investigation to the State Engineer, the Director (a political appointee, but learning quickly) is startled by the somewhat ironical remark, "Yes, if flows turn out as expected," he hears at the other end of the line. He quickly has sequences of runoffs generated for future flows given the flow realizations for the first week, then for the first two weeks, etc... by an expert on stochastic processes and data generation. He then repeats all the calculations for the pumping schedule for the remaining 15, then 14, 13, etc... weeks. The results are displayed in Table 2.

The Director discovers that storage will be needed and that the extent of storage needed will depend on the variations in the runoff. Based on the mean minimum needed storage (37 m^3) and the standard deviation (18 m^3) he recommends to the Developer that he build a storage of capacity at least 66 m^3 if he does not want to fail to supply the subdivision demand more than 5% of the time or 78 m^3 if he wants to reduce this risk to 1%.

3. Director's Augmentation Strategy

It dawns on the Director that at times it may not be possible to satisfy the constraints even if the well was shut down. For example, the L.P. problem associated with a different sequence of (lower) runoffs generated for the remaining 13 or 11 weeks (given in Table 1 in columns 13' and 11') has no solution. The demand constraints cannot be satisfied. Even though the Developer may have a large storage capacity,he is not allowed to pump enough to fill it for subsequent needs of the subdivision. Instead of imposing low quota on pumping in anticipation of possible (but not certain) quality violations,would it not be better on the contrary to allow the Developer to keep its reserve full but require from him immediate release back to the river of the exact amount of water needed to maintain the stream quality standards?

The L.P. problem takes the new form:

$$\text{Min} \left[S \right] \tag{9}$$

subject to the usual non-negativity constraints, the new demand constraints:

$$\sum_{\nu=1}^{n} Q(\nu) - r(\nu) \geqslant 200n \qquad n = 1,2,\ldots\ 16 \tag{10}$$

the new feasibility constraints:

$$\sum_{\nu=1}^{n} \left[Q(\nu) - r\ (\nu) \right] - 200n \leqslant S \qquad n = 1,2,\ldots\ 16 \tag{11}$$

Remaining Weeks	ν	PUMPING SCHEDULE																Minimum Storage
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	
16		200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	200	0
15			244	200	200	200	200	200	200	200	200	200	156	244	200	200	200	44
14				156	200	200	200	200	200	200	200	200	200	200	200	200	200	44
13					245	200	200	200	200	200	200	200	155	245	200	200	200	45
12						155	200	200	200	200	200	200	200	200	200	200	200	45
11							200	200	200	200	200	200	200	200	200	200	200	45

Table 2. Pumping Schedules

the new quality constraints:

$$C_s \left[\sum_{\nu=1}^{n} \varepsilon(n-\nu+1)\ Q\ (\nu) - r(n) \right] \leqslant C_s\ (R_e\ (n) - W_r) - (C_b - C_s)\ B(n) \qquad (12)$$

for $n = 1,2\ldots 16$

and the additional non-negativity conditions:

$$r\ (n) \geqslant o \qquad n = 1,2\ldots 16 \qquad (13)$$

where $r\ (n)$ is a release back to the river from the Developer's storage to meet the downstream quality standards. The solution to this problem always exists and the required storage capacity is zero, because it is always feasible to pump the required release at the time it is needed. The reason for this happy solution results from the fact that $\varepsilon\ (1)$ is less than one (in this case $\varepsilon\ (1) = 0.06$) and significantly more water is drawn from the aquifer by pumping than is lost by interception of return flow or by seepage from the river. The maximum needed release was 21 m^3, during the 11th week, slightly more than 10% of the water demand. Instead of storage, the Developer needs to install a greater pumping capacity.

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PART II

ECONOMIC ASPECTS OF CONJUNCTIVE SURFACE-GROUNDWATER MANAGEMENT

A. Introduction

The distinctive properties of the water resource and the conditions of its availability, distribution and consumption call for special institutions (organizational, administrative and property) to control its allocation [Gaffney, 1969; Castle and Stoevener, 1970]. The most significant of these is the mobile, flowing nature of water, which makes it particularly difficult to establish and maintain the property rights which are the basis for allocation and exchange in a market economy.

Groundwater, in particular, presents a well-recognized example of a natural resource which the unregulated market economy fails to allocate so as to achieve the maximum net value of production [Ciriacy-Wantrup, 1963]. In the next section, the following theses are developed. First, many of the significant problems arising in connection with groundwater are what economists call resource allocation problems and can be properly explained and understood within theoretical economic framework. Second, these problems can be characterized as a failure of the institutions to keep pace with the growing utilization of the resources, such that the allocative institutions have become obsolete under present demands on the resource. Given these two propositions, an examination of the conceptual bases for institutional inadequacy is therefore an appropriate place to begin in formulation and evaluation of alternative allocative mechanisms.

Market Failure and Groundwater Allocation

Groundwater resources are typically utilized by a number of independent pumpers withdrawing from a common pool. Since water in unconfined aquifers ordinarily moves in response to withdrawals, the actions of any one pumper affects the conditions of production experienced by other users. Users are thus interdependent, and "external" or spillover effects may occur. The existence of substantial external effects violates at least one of the conditions required to justify the market mechanism as an optimal solution to the problem of resource allocation. In the presence of substantial external effects, the costs and gains as calculated by each of the many individual units fail to reflect their total impact on society. When "private" costs are not as large as "social" costs, the individual's incentive is to engage in more of the productive activity than is socially desirable, and a misallocation of resources is said to exist.

The type of external effect of interest here is called a "technological diseconomy"; technological because the impact is registered through a technological or physical link between production processes and "diseconomy" because the effect imposes a cost rather than a benefit upon recipient units. Thus, under conditions of heavy exploitation aquifer management becomes an issue of community concern because of technical conditions which prevent the market mechanism from properly functioning in the allocation of groundwater. In the case under study, in the South Platte Valley in Colorado, the problem takes the form of the groundwater pumpers utilizing, in effect, water to which surface water users downstream claim a right. [Other examples of such detrimental external effects include salt water intrusion, subsidence of overlying land surface and increasing pumping costs due to lowered water tables (Young, 1970).] The case at interest here is characteristic of river basins in arid or semi-arid regions, where surface water supplies are fully appropriated. Withdrawal of water from aquifers intimately associated with the stream may disrupt patterns of groundwater flow to and from the river, thus jeopardizing the water supply of holders of junior surface water rights.

Problems of this sort arise largely because groundwater is treated in practice as an unrestricted open access resource; that is, a resource which any overlying land owner can use for whatever purpose he might chose, without charge or hindrance. Fugitive resources, such as air and water, were traditionally managed in this fashion due to physical characteristics which make their administration relatively costly [See Haveman, 1972]. However, changing economic and technological conditions are making such institutions obsolete, a point which applies particularly to groundwater management. Groundwater has been administered as an open access resource by the public (its nominal owner) because the cost of enforcing property rights in fugitive resources under conditions of low utilization relative to supply easily may be greater than the benefits of such a policy. By its nature, groundwater flows are difficult and expensive to observe and measure. As long as withdrawals in a basin are so small that negligible effects are imposed on the group of groundwater users as a whole, it made little sense to develop elaborate systems for groundwater regulation. However, the technological changes and economic growth factors have encouraged rapid use of groundwater in the past two decades. It is becoming clear that the century-old tradition of ignoring the external consequences of groundwater withdrawal cannot persit indefinitely. Benefits from regulation have risen, in the sense that there are external costs which can be mitigated by more elaborate control mechanisms. Similarly, technological advances in fields relating to administration of groundwater resources, including monitoring and measuring devices, mathematical modeling and digital computers, suggest that costs of management may be declining. These shifts in cost and benefit suggest the possibility that some system of community management is not only feasible but optimal.

Groundwater regulation has followed a classical historical pattern consistent with the hypotheses outlined above. As detailed in the legal section of this report, groundwater use was ignored in early water statutes in Colorado. As interdependency effects became apparent, the State adopted regulations defining beneficial use and eventually placing limitations on individual use. However, there remains a strong resistance (based both on economic pressures and ideological convictions) to regulation when free access has become an established tradition.

A number of alternative solutions to market failure are suggested by economic theory (Hirshleifer, et al., 1960). These include taxation of withdrawals (to equate private and social costs), firming up property rights in water (Ciriacy-Wantrup, 1963) and centralized control. Taxation, in spite of its appeal to economists, has been adopted only as a vehicle for collecting revenue, not as a resource allocation tool. The second approach, firming up property rights is the approach followed by the present legislative trend in Colorado, in that groundwater is being brought under the appropriative doctrine, with modifications for the special characteristics of groundwater (i.e., the "doctrine of futile call"). It is further proposed that this approach be supplemented by a quasi-market system, in which the junior groundwater rightholder be able to purchase surface rights so as to permit him to use the well as an alternative source of diversion. The last general method, centralized control, also has not been tested in practice, although a variation has been shown to be desirable on narrow economic efficiency grounds in a highly simplified setting (Young and Bredehoeft, 1972).

Economic Analysis and the South Platte Optimization Model

The conceptual economic basis for selecting alternative approaches to solving conjunctive management problems has been described above. In addition to that contribution, potential additional economic aspects of the study include (a) specification of an objective function, and (b) predicting resource allocation behavior of water users in response to alternative regulatory practices. The latter task, in turn, requires detailed knowledge of the scale and technology of water user production activities and resource constraints which influence their production responses. During this study, work has been undertaken mainly on the latter of these tasks, predicting irrigator behavior.

Three specific issues have been addressed. First, empirical data regarding water users in the study area was collected, in order to properly specify parameters of the water allocation model. Secondly, an effort was made to provide theoretical and empirical refinements to existing knowledge of crop response to various quantities and timing of irrigation water. Finally, since uncertainty in both "natural" and economic factors is thought to modify profit maximizing behavior by private water users, an effort was made to formulate an allocation model which hypothesizes behavior other than pure profit maximization by water users. Each of these three facets of the research is summarized briefly below.

B. Economics of Irrigation Water Use in the Lower South Platte Valley[1]

In order to provide data for an irrigation water allocation model, a study of costs and returns to irrigated farms was conducted in a selected reach of the Lower South Platte in Northeastern Colorado. The study area extended from Fort Morgan to the Nebraska border, a distance of some sixty miles. Farm descriptions and estimates of costs and returns were based on a survey of 89 farm operators, approximately 10 percent of those farming over the valley fill aquifer. Principal crops presently grown include corn grain, corn silage, alfalfa, sugar beets and edible dry beans.

Irrigation was developed in this area in the 1870's and 1880's with the construction of ditches to distribute water out of the river. Reservoir construction in the early 1900's provided storage capacity so that irrigation water supply became less dependent on river flow during dry summer months. With an extended drought in the early 1950's, farmers began to install wells for the purpose of supplementing surface water supplies. Well development continued as more farmers realized the advantages of the "water insurance" provided by wells.

Among the 89 sample farms, 78 had wells to supplement ditch water while eight farms were entirely dependent on well water. Only three of the sample farms had no well. The sample farms were divided into three size groups based on irrigated acreage: Group I, 50-200 acres; Group II, 201-400 acres; and Group III, over 400 acres. Average farm size was about 160 acres in Group I, about 300 acres in Group II and about 525 acres in Group III.

1/ Summarized from L. R. Conklin "Farm and Irrigation Management, South Platte Valley, Colorado," Thesis for M.S. degree, Colorado State University, 1974.

Two "farm types" were identified in each size group, one a "feed crop farm" producing corn grain, corn silage and alfalfa, and the other a "cash crop farm" producing corn grain, alfalfa, sugar beets and pinto beans. Costs and returns were computed for each farm type on the basis of their typical machinery inventory and the price conditions in effect during the 1973 growing season. (While many farms had livestock enterprises in addition to their crop production operations, the costs and returns to livestock were not examined in this study.)

Farm business investment is shown by type of assets in Table 1. For both types of representative farms, equipment investment per acre declines markedly as farm size increases.

TABLE 1: Average Value of Investment by Farm Size and Type, 1973, South Platte Valley, Colorado

	FEED CROP FARMS			CASH CROP FARMS		
	Size I (160 Ac.)	Size II (300 Ac.)	Size III (525 Ac.)	Size I (160 Ac.)	Size II (300 Ac.)	Size III (525 Ac.)
Tractors	$16,000	$14,375	$24,810	$14,210	$19,375	$24,810
Field Equipment	12,787	13,970	22,990	19,882	21,752	30,965
Irrigation Equipment	4,236	5,133	10,268	6,220	7,047	12,323
Trucks	7,475	7,475	9,200	7,475	7,475	12,075
Misc. Equipment	2,805	2,805	2,805	2,805	2,805	2,805
Land[a]	128,000	240,000	420,000	128,000	240,000	420,000
Total	171,303	283,758	490,073	178,592	298,454	502,978
Irrigation Equipment ($/Ac.)	26	17	20	39	23	23
Other Equipment ($/Ac.)	244	129	114	277	171	136
Total Equipment ($/Ac.)	270	146	134	316	194	159

[a] Land is valued at $800 per acre.

Table 2 shows the costs and returns for each farm type. Gross sales include crop sales and the market value of crops fed to livestock. Gross expenses include operating costs (seed, fertilizer, labor, water, etc.) and cash overhead costs (depreciation, taxes, insurance, building maintenance). The difference between gross sales and gross expenses is the return to management and investment. All of the required annual payments on borrowed money must be paid out of this amount before any remainder is available for family income. All labor is charged as a cash operating expense at the prevailing wage rate. For labor performed by the operator and his family (rather than hired workers), the labor charge is not an out-of-pocket expense and can be considered an addition to family income.

TABLE 2. Costs and Returns by Farm Size and Type, 1973, South Platte Valley, Colorado

	FEED CROP FARMS			CASH CROP FARMS		
	Size I	Size II	Size III	Size I	Size II	Size III
Gross Sales	$47,661	$89,382	$146,200	$55,178	$97,415	$173,700
Gross Expenses						
Operating Costs	10,469	20,396	38,814	13,473	23,749	42,921
Cash Overhead Costs	7,736	9,717	16,613	8,796	11,887	18,855
Total Costs	18,205	30,113	55,427	22,269	35,636	61,776
Return to Management & Investment	29,456	59,269	90,773	32,909	61,779	111,924
Management Charge (10% of gross sales)	4,766	8,938	14,620	5,518	9,742	17,370
Return to Investment	24,690	50,331	76,153	27,391	52,037	94,554
Percent Return on Average Investment	14.4	17.7	15.5	15.3	17.4	18.8

The returns to management and investment must have a charge for management deducted out in order to show the returns to investment. This management charge was computed at 10 percent of gross sales.

Part of the purpose of the analysis of farm size and types was to determine if such factors would have a significant influence on productivity of (and ability to pay for) irrigation water. Most farm management studies in Colorado and elsewhere have shown that larger farms commonly achieve higher returns on investment for two reasons. The fixed or overhead costs rise less than proportionately as farm size and volume of output are increased (also referred to as the spreading of overhead costs). Furthermore, farm operations can be performed more efficiently on larger farms due to the specialization of labor and management and the use of higher capacity machinery.

In this study, returns to investment do increase somewhat with farm size among the cash crop farms. However, among the feed crop farms, returns peak with the middle sized operation. This peak is explained by the combination of lower corn yields on the largest farms (about 10 bushels per acre lower) and the predominance of corn on the feed crop farms. The lower yields are probably due to a loss of timeliness in tillage and planting (in order to get the work done, it is necessary to begin earlier and finish later than would be optimal) and a greater dependence on hired labor. The relationship between corn yields and farm size may change as more of the larger farmers replace their present equipment with wider tillage machinery and more efficient corn planters.

For crops other than corn, yields were found to be roughly the same for all size groups.

Thus, while there are some differences in returns to investment by both farm size and type, these differences are not judged to be significant enough to warrant a water allocation model disaggregated into several size and/or type categories.

C. Modeling Irrigation Water Allocation Under Uncertainty[1/]

Decision making is the central coordinating concept of any organization, whether it is a family farm business, a giant industrial complex or a public agency. The small farmer may be the least prepared to make effective decisions, yet in the agricultural sector, prices, productivity, human relations, and other factors out of his control are constantly changing yet must be dealt with.

The model of the firm in classic economic theory rests on the assumption that the decision maker knows the relevant production functions, demand functions and factor costs with complete certainty. However, many decisions must be made in the absence of complete knowledge of the relation between alternative courses of action and outcomes.

Many hypotheses have been offered to explain decision making under "risk", where probabilities are known. Beginning with Bernoulli, there have been many variations on the approach of maximizing expected value. However, in spite of the voluminous literature, no real concensus has emerged.

For the purpose of modeling decision making in agriculture, it appears necessary to drop the assumption of known probabilities and turn to the theory of decision making under uncertainty. The von Neuman-Morgenstern game theoretic approach spurred a considerable literature. However, applied research has uncovered a number of weaknesses and new approaches continue to be developed. Modern

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[1/] Summarized from Johnson and Young, 1975.

decision models for planning under uncertainty can be divided into three main types. The most common approach utilizes some form of Bayesian model together with a hypothesis of risk aversion and posits an expected utility maximization approach. A second approach is based on a lexicographic utility function which ranks a hierarchy of objectives. This model assumes the entrepreneur has a number of goals and a multi-dimensional utility function is required to reflect these goals. A lexicographic function is one which ranks goals in an array of most preferred to least preferred. The objective is to maximize the most preferred goal subject to the other goal(s) being at "satisfactory" levels.

A third approach uses the "focus loss-focus gain" concepts of Shackle. The essence of the concept is the assumption that the decision maker wishes to maximize expected gain so long as the possibility of attaining a ruinous level of income is so small it can be ignored. This approach has been formalized by Boussard and Petit (1967). This decision criterion requires that a "negligible possibility of ruin" be defined in both a practical and mathematical sense. Once so defined, it is possible to constrain the maximization problem to a point that such a risk is not taken.

This latter approach has been undertaken in one of our ground water allocation studies. It is assumed that a farm production plan would not be selected which would risk reducing income below some minimum which would cover unavoidable expenses, where these include a minimum amount for family consumption, out of pocket production expenses, half of equipment expenses (including loan repayments) insurance and other general expenses.

The "focus of loss" estimate for each crop was obtained by interviewing people familiar with farming in the selected area, chiefly extension workers, as well as farmers themselves. Minimum consumption income was specified by obtaining relevant information from local banks and other farm credit sources.

This approach was studied on a high mountain valley in Alamosa County in southern Colorado. The short growing season limits crop alternatives to small grains, potatoes and forages. Where better soils are available, only potatoes and malting barley are the main crop opportunities available to cash crop farmers. Table 3 illustrates an example water allocation model utilizing the focus loss concept for such a sub-region. Table 4 compares predicted acreages from that actually observed in the area in 1974.

A profit maximization model would have allocated all land and water to potato production, while the focus-loss model achieves a fairly accurate prediction of actual farmer behavior.

It is concluded that some hypotheses concerning irrigator choices under uncertainty can feasibly be incorporated into ground water planning models.

Table 3

TABLEAU FOR WATER ALLOCATION MODEL, SUBAREA III, ALAMOSA COUNTY, 1974

Constraint Type	Crop Activity		Water Activity		STBO	LOSS	Constraint Level
	(Barley) X_1	(Potatoes) X_2	(Ditch) X_3	(Well) X_4	X_5	X_6	
Max. Acreage Available	1.0	1.0					$\leq$ 39,360.0
Water Balance[a]	5.0	3.6	-1.0	-1.0			$\leq$ 0.0
Max. Ditch Water[a]			1.0				$\leq$ 24,750.0
Max. Well Water[a]				1.0			$\leq$ 145,794.0
Focus Loss Constraints	35.0					-0.5	$\leq$ 0.0
		100.0				-0.5	$\leq$ 0.0
Capital (FUND)[b]	126.0	290.0			-1.0		1,782,000.0
Minimum Income (MINI)[b]	200.0	450.0	-1.5[c]	-2.1[c]	-1.0	-1.0	=2,310,000.0
Objective Function[b]	200.0	450.0	-1.5	-2.1	- .04		Maximize

[a] Water is in units of acre-feet.

[b] Capital, income, and objective function are expressed in dollars.

[c] Water prices are in dollars per acre-foot.

Table 4

COMPARISON OF OUTPUT VALUES FROM THE PLANNING MODEL, SUBAREA III, 1974

Crop	Predicted Planted Acreage	Actual Planted Acreage	Differences Acres	%
Barley	21,496	22,187	691	4
Potatoes	17,863	16,693	1,170	7

D. Irrigation Response Functions[1/]

The resource allocation model to be adopted for this study requires estimates of crop responsiveness to alternative levels of water supply, i.e., an irrigation response function. Much of the existing work on irrigation response in the study area, as elsewhere, has been based on a model in which yield is hypothesized to be responsive to annual water applications, regardless of the timing of water inputs. However, it is well known by agronomists that a more precise measurement of crop response can be achieved by formulating a model of response as related to soil moisture status. The occasion of a several month visit by Dr. Dan Yaron of the Faculty of Agriculture, Hebrew University of Jerusalem, Israel, to Colorado State University provided an opportunity to refine available crop response functions. Professor Yaron holds advanced degrees in agronomy and agricultural economics, and is an acknowledged authority on crop response.

The concept of "stress day" or "critical day" was defined as one in which the soil moisture in the root zone was depleted below a certain level (55% of the available soil moisture, AMS). The number of critical days for the crop season and for specific growth stages was used an an explanatory varible in the response function.

Irrigation experiments on field corn conducted at Colorado State University in 1972 under the direction of Dr. Robert Danielson provided data for the analysis. These data were reported in an M. S. thesis by Twyford (1973). There were eleven treatments varying with respect to the number of applications of irrigation water and their timing. Each irrigation applied 2 inches of water.

1/ The following paragraphs summarizes the work, reported in Neghassi, Yaron, and Young (1975).

A computer program was prepared to estimate available soil moisture on a daily basis, depending upon climatic factors, rainfall, applied irrigation water and predicted drainage from the root zone. Evapotranspiration was predicted by a modified Jensen-Haise procedure.

The resulting estimates of stress days (for growth stages and the season as a whole) combined with experimental crop yields were fit to various non-linear equation forms, including exponential and Mitscherlich types. A non-linear least squares algorithum was used (Marquandt, 1963). An adequate explanation of observed variation (76%) was obtained with the exponential functions. The Mitscherlich type did not converge to a solution.

The model giving the best fit to the data was:

$$Y = Ymax\ B_1^{X_1}\ B_2^{X_2}$$

where

Y : Predicted grain yield (Kg/Ha)

$Ymax$: Maximum yield (kg/Ha)

X_1: Number of days of soil moisture depletion in excess of 55% between June 22 and October 2

X_2: Number of days irrigation is delayed after beginning of the critical silking period

The estimated parameters were:

$B_1 = 0.995$

$B_2 = 0.973$

$Ymax = 10,960$

Note that, since B_1 and B_2 are less than 1.0, an increase in X_1 or X_2 leads to reduced yield. The coefficient B_1 can be interpreted to the effect that one stress day reduces expected yield one half of one percent, while B_2 indicates that each day's delay in irrigation during silking reduces yields 2.7 percent.

It is concluded that their approach is suitable for simulation of crop response so long as accurate measurements of soil water status can be made. It is hoped that experiments on other crops in the area will be performed so that response parameters can be utilized in a similar fashion.

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PART III

LEGAL ASPECTS OF CONJUNCTIVE SURFACE-GROUNDWATER MANAGEMENT IN COLORADO*

A. INTRODUCTION

Colorado has experienced the past decade of conflict and attempts of resolution through litigation and legislation over use of surface and tributary ground waters. Basically, the law requires two objectives: (1) protection of senior rights and (2) optimum use of the state's water resources. But in the early 1950's, the law was deficient or unable to resolve two general areas of conflict: (1) determination of priorities between surface and well water users and (2) determination of priorities and rights between well water users.

The paradoxes between the objectives and inability to satisfactorily resolve the "priorities" problem led to a decade of dispute, litigation, legislation and negotiation. This portion of the report goes into the details of the conjunctive water use problem in Colorado and, more specifically, the South Platte River Basin.

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*Part III is the result of the efforts of George E. Radosevich, Assistant Professor of Environmental Law and Economics, Department of Economics, Colorado State University, and research assistance from Donald Freemeyer and Craig Kirkwood, law students at the University of Colorado in Boulder and the University of Wyoming in Laramie, respectively. In addition, legislative aspects of the project have been documented in a separate report with the title, "Colorado Water Laws: A Compilation of Statutes, Regulations, Compacts and Selected Cases," by George Radosevich, Donald H. Hamburg and Loren L. Swick, Center for Economic Education, Department of Economics, Colorado State University, Fort Collins, Colorado 80523.

B. BACKGROUND

Water has meant prosperity to certain areas of Colorado and segments of the state's economy. Applied to the arid lands, it has yielded a thriving agricultural economy. It has enabled towns and cities to grow and to create a pleasant environment for their inhabitants and is essential to the industrial growth of the state. Where water has not been available there has been little economic growth.

The importance of water to Colorado and the need for its maximum utilization were recognized by the Colorado Supreme Court in Colorado Springs v. Bender [1] and clearly enunciated in Fellhauer v. People.[2] The court acknowledged that historically vested rights must be protected in resolving the questions of ground and surface water use; however, ". . . there shall be maximum utilization of water in this state."[3] Warning was given that the " . . . right to water does not give the right to waste it."[4] The implied meaning of this phrase is that historically vested rights could not claim water to the detriment of growth and maximum utilization.

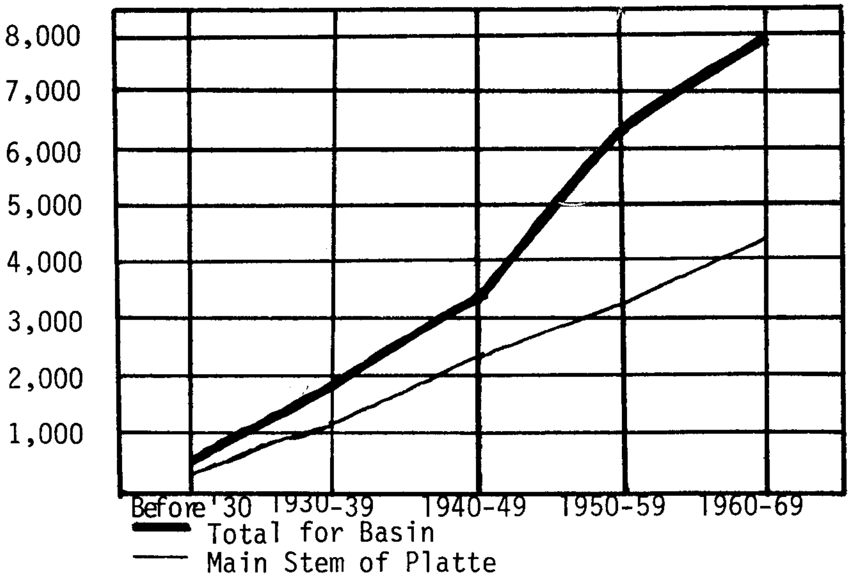
Historically, Colorado water law dealt with only surface water rights. The Irrigation Act of 1879 did establish public administration of water but did not deal with groundwater rights.[5] Not until 1953 did the state legislature deal with ground water.[6] However, this act " . . . amounted to no more than a requirement for filing well logs and the authorization of certain studies on the effect of withdrawal in given areas . . ."[7] In 1957 an act was passed which ostensibly would require obtaining drilling permits prior to drilling. The act also brought all groundwater within the state under the administration of the state engineer.[8] However, this law was given little effect and was followed by another act in 1965 (Groundwater Administration Act of 1965).[9] The authority of the state engineer to regulate wells under this act was questioned in the Fellhauer case and found void for lack of appropriate rules and regulations.[10] The legislature has again responded with the 1969 Water Rights Determination and Administration Act, which seeks to integrate the use of ground and surface waters and make tributary groundwaters fully subject to the appropriation doctrine.

While this legislative activity was taking place there was a boom of well-drilling activity along the irrigable river valleys of the state. Two historical events exist for this activity which became one of the most hotly litigated topics of the late 1960's and early 1970's. The first was the drought in 1931. Farmers were required to dig wells as an alternate source of supply when surface flows no longer met their needs. The second event was the Rural Electrification Association (REA), which enabled farmers to install electric motors to their pumping systems.

In 1959 there were an estimated 8,900 wells on the Eastern slope and San Luis Valley of Colorado.[11] By 1969, in the South Platte River Basin, where managing the surface and ground water diversions was emerging as a major problem, there were nearly 8,000 wells, of which approximately 4,400 were on the main stem (See Figure III-1). The increase in well installation was considerably higher away from the main stem, but the total diversion of groundwater within the basin increased significantly while surface diversions began tapering off (See Figure III-2).

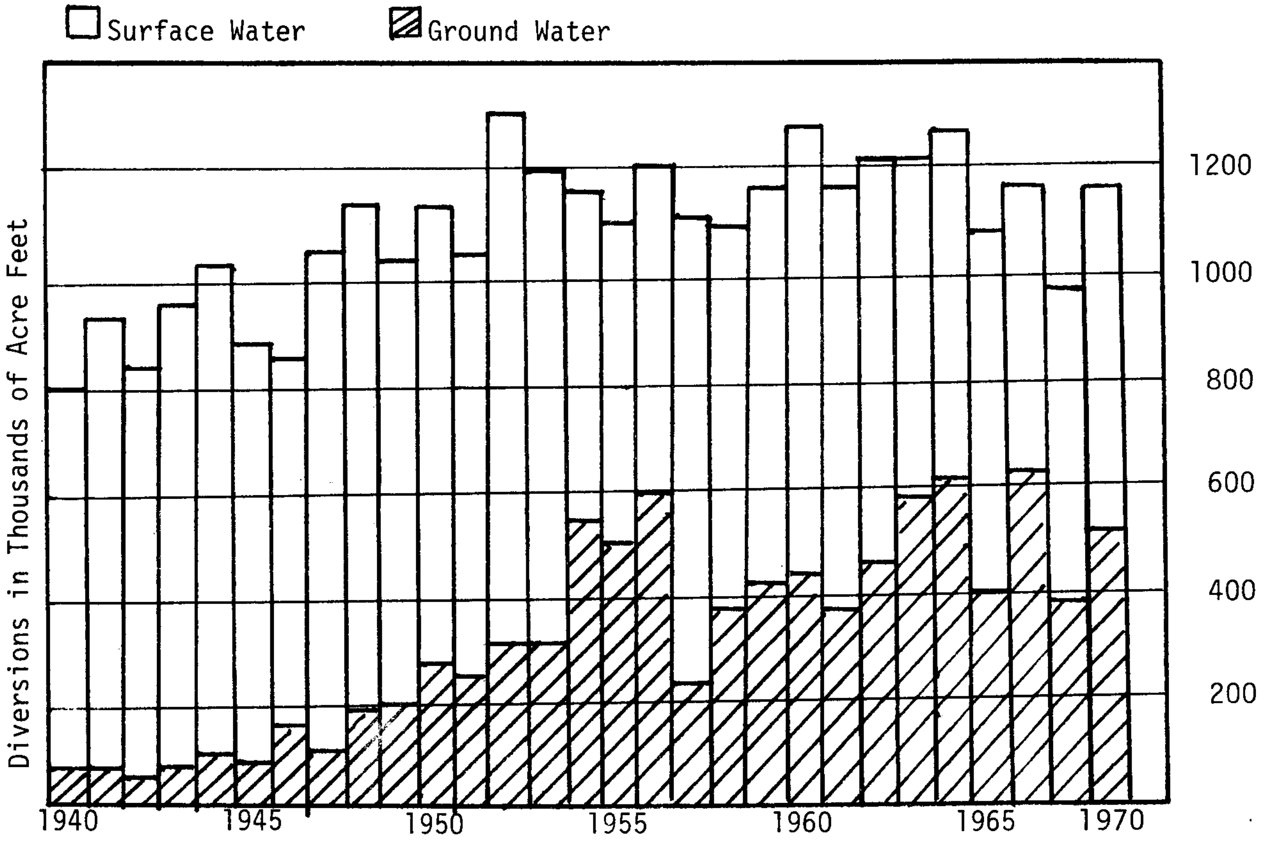
The vast number of wells competed with the surface rights for the available water and diminished the streamflow substantially and measurably.[12]

FIGURE III-1
NUMBER OF IRRIGATION WELLS
South Platte Basin



Sourses:Total Basin Information - Colo. State Engineer
Main Stem Information - U.S.G.S., Dept. of Int.

FIGURE III-2
TOTAL WATER DIVERSIONS
South Platte Basin



Source: U.S. Geological Survey, Dept. of Interior

Figures from "Colorado's Future: A Full Water Use Pōlicy",The Water Integration Committee, Sterling Colorado, 1971

The well owners, however, felt their rights " . . . had become 'vested' even though they had never been adjudicated."[14]

The positions of the well owners and surface rights holders both have merit. The surface rights were first in time, there was a considerable expenditure in the construction of the canals, reservoirs, dikes, etc., necessary to transport the water, and there has been reliance on the seniority of these rights. Well owners, however, can point out that the alluvium is a huge storage reservoir which can be refilled during times of excess flow, if this excess is not used then it passes unused out of the state, the underground flow needs no hydrostatic support contrary to surface waters, and there are no losses due to evaporation with ground waters.[15]

The legal problem then takes on heavy economic and technological overtones. Maximum utilization of all tributary waters within the state is desired. However, the seniority of the surface rights is to be respected and ground water is to be brought under the appropriation system as if it were surface water. The question, then, is whether these different goals can be accomplished under the 1969 Act.

C. THE CONSTITUTIONAL SETTING

The basic Colorado constitutional provisions concerning water are found in Article XVI, Section 5, declaring the water of streams to be public property, stated as follows:

> The water of every natural stream, not heretofore appropriated, within the State of Colorado, is hereby declared to be the property of the public, and the same is dedicated to the use of the people of the state, subject to appropriation as hereinafter provided.

The right to divert the unappropriated waters of these streams for a beneficial use is never to be denied. Also, in time of need there shall be a priority of uses. The priority is: domestic, agricultural, and industrial uses.[16] Section 7[17] guarantees that the right-of-way across public, private and corporate lands for conveyance of water shall not be denied upon payment of just compensation or permission by the owner. This section in effect allows a private eminent domain proceeding. Section 8[18] allows the county commissioners to levy taxes for the use of water.

In the case of Coffin v. Left Hand Ditch Co.[19] it was argued that prior to the constitution there were riparian rights for those owning land along a stream. This contention was completely rejected with the court saying the appropriation doctrine had always been in effect in Colorado.[20] In People ex rel. Park Res. Co. v. Hinderlider the court noted that the appropriation doctrine antedated the Colorado Constitution.[21] The United States Supreme Court also recognized that Colorado had adopted the appropriation doctrine in Kansas v. Colorado.[22] In rejecting a claim of riparian rights in land patented from the United States the state's choice of water law was upheld as being a valid state right.[23] In Stockman v. Leddy[24] the waters of the natural streams in the state were held to be non-navigable and the property of the state. Pursuant to Colo. Const. Art. XVI, Sec. 5, the state was free from interference by any other sovereign (Federal Government) in choosing the type of water law it wanted.

It should be noted that the "right to appropriate" as set forth in Article XVI, Sec. 6, is not absolute. In Fundingsland v. Colo. Ground

Water Commission[25] a permit to drill a well was denied by the Colorado Ground Water Commission. The court upheld the commission on the grounds that allowing the application would over-appropriate the water in the designated ground water basin and would harm senior appropriators. At first glance, this might appear to be a restriction on Article XVI, Sec. 6; however, the section is self-limiting in that it guarantees the right to appropriate only unappropriated water. This case dealt with a "designated ground water basin" and could possibly be distinguished in any attempt to apply it to tributary water. However, this would be doubtful as it upholds the principle that the prior appropriators can't be unduly harmed by another who wants to appropriate.

By its language Sec. 6, Article XVI of the constitution sets a list of priority among users. The order of preference is: (1) Domestic use, (2) Agricultural use, and (3) Manufacturing use. This preference order is to be effective in case there is a shortage of water in the stream.

Trelease[26] has noted that a true preference would exist if a junior preferred user could take the supply of s senior non-preferred user without compensation. This is not the case in Colorado: "Although, in the constitution of Colorado, the principle of priority in time of appropriating water is declared, this principle is made applicable as between persons who use the water for the same purpose . . ."[27] with the priority provisions to take place in times of short supply. "Despite the failure of this section to provide for compensation to the holder of the inferior right, . . ."[28] it has been held that the taking superior right must compensate the inferior right.[29] A "preference . . . does mean a power to condemn a prior water appropriation for a purpose made inferior by the constitution."[30] This contention seems borne out in Pine Martin Mining v. Empire Mining Company which contains dicta saying that a quasi-private organization can use the eminent domain proceedings in securing water from an inferior right.[31] Trelease also believes that a preference in Colorado amounts to a right to condemn inferior uses in times of need.[32] The preferences have also been construed to allow and limit cities to make appropriations for future growth.[33]

D. STATUTORY LAW

Substantive Provisions

The current law governing tributary water within the state was passed in 1969 and was known as the "Water Rights Determination and Administration Act of 1969." The policy statement of the Act acknowledges the interrelationship of ground and surface waters: "it shall be the policy of this state to integrate the appropriation, use and administration of underground water tributary to a stream with the use of surface water to maximize the beneficial use of all the waters of this state."[34]

The justification for attaining this maximization optimum is also stated. The legislature recognizes that the present and future wellfare of the state and its people warrants the maximization of beneficial use.[35] To attain this goal the following principles apply:

1) All previously vested rights and uses protected by law, including an appropriation from a well, shall be protected.[36]

This principle, if strictly applied, would continue the seniority system and wherever a stream is overappropriated would, theoretically, shut down all junior rights (wells). This could be all the wells on the stream. Based on this language alone it is questionable that an unadjudicated well is protected.

In 1971 the legislature allowed the adjudication date on a well to relate back to its actual appropriation date.[37] The language of both provisions would allow a well appropriation to be protected if the well is adjudicated pursuant to the above statute even though not previously adjudicated. In times of shortage the wells would still probably be the first to be shut down but, those wells which are adjudicated would be senior to the unadjudicated ones, and the Act also provides other means for the use of wells even though there might be a call on the river, i.e., augmentation plans,[38] and alternate means of diversions.[39]

2) The present use of wells, either independently or in conjunction with surface rights shall be given the fullest possible recognition. However, this principle will be limited by existing vested rights. Each diverter must establish a reasonable means of diversion and he can't command the whole flow to take his appropriation.[40]

This principle in part clarifies some of the questions alluded to under 1 above. The first part of this principle recognizes the previous and present use of wells and sanctions their continued use subject to existing vested rights. The requirement for a reasonable diversion and not allowing one to command the whole flow for his appropriation seems to be a codification of Colorado Springs v. Bender.[41]

3) Use of a well may be an alternate or supplemental source for a surface decree.[42]

Again this principle is limited by previous usage and vested rights. There may be reluctance to use the wells as an alternate point of diversion as the Act implies that the well can't be used independently from the surface right.

4) No junior appropriator can be limited unless this reduction would result in an increased water supply available to the senior appropriator.

This principle recognized the "futile call" concept whereby the downstream senior appropriator can't put a call on the water being used by an upstream junior if it will not reach him. This again is in line with the overall concept of maximization of beneficial use. This principle was also set forth in the Fellhauer decision.

The scope of the Act and the intent of the legislature is also illustrated by some of the definitions used. Under CRS 37-92-103 pains are taken to insure jurisdiction over all tributary waters within the state. Subsection 13 defining 'waters of the state' includes all tributary surface and ground water within the state. 'Underground water' means that water contained in the alluvial aquifer which is hydrologically connected to the surface or underground flow within the total aquifer.[43] The waters referred to in the two above definitions exclude 'designated ground waters.'[44] Other definitions of interest under CRS 37-92-103 include:

> (3) "Appropriation" means the application of a certain portion of the waters of the state to a beneficial use.
>
> (4) "Beneficial use" is the use of that amount of water that is reasonable and appropriate under reasonably efficient practices to accomplish without waste the purpose for which the appropriation is lawfully made and, without limiting the generality of the foregoing, includes the impoundment of water for recreational purposes, including fishery or wildlife. For the benefit and enjoyment of present and future generations, "beneficial use" shall also include the appropriation by the state of Colorado in the manner prescribed by law of such minimum flows between specific points or levels for and on natural streams and lakes as are required to preserve the natural environment to a reasonable degree.
>
> (7) "Diversion" or "divert" means removing water from its natural course or location, or controlling water in its natural course or location, by means of a ditch, canal, flume, reservoir, bypass, pipeline, conduit, well, pump, or other structure or device.

Administrative Framework

The 1969 Act made substantial changes in the administration of water in the state. The main changes are the establishment of divisions within the state and the creation of the water judge, water referee, and the water clerk. The overall responsibility for the administration and distribution of waters of the state falls to the state engineer with the administration and distribution within the divisions left to the division engineer.[45]

In order to make the administration of water in the state conform to the natural situation seven water divisions were created. Generally, there is a division which approximates the drainages of the following rivers: South Platte, Arkansas, Rio Grande, Gunnison, Colorado, White, and San Juan.[46] A division engineer appointed by the state engineer and approved by the director of natural resources administers the water within each division. The division engineer must be a registered professional engineer in addition to any qualifications which may be set by the state engineer.[47] The division engineer must reside within his division and maintain an office at a prescribed location.[48] He may also establish field offices to be manned by a water commissioner.[49] His overall direction and guidance comes from the state engineer.[50]

The state engineer, as mentioned previously, has the overall responsibility for the administration and distribution of waters of the state.[51] He also directs the division engineers and other employees responsible to them.[52] In performance of his duties the division engineers are to be guided by the Colorado Constitution, statutes, and written instructions by the state engineer. There is also a legislative intent to not allow ground water withdrawal if this would deprive senior surface rights of water which would have been available had the withdrawal not been made. However, if this water would not be available to the senior right then the junior well deverter can use it in spite of the call.[53] This again expresses the desire for maximum use of ground and surface waters and the "futile call" doctrine.

Any rules and regulations which the state engineer draws up are to be guided by certain principles. The principles are:

1. The geologic characteristics of the aquifers are different and must be so recognized. Aquifers of the same type in the same water division must be governed by the same rules.[54]

2. The particular characteristics of the aquifer shall be considered.[55]

3. The priority system will be considered as well as quantities and the time of year demands are made.[56]

4. The possibility of one owner owning both ground and surface rights shall be recognized.[57]

5. The objective of any rule or regulation shall be optimum use of the resources tempered only by the priority system.[58]

6. As knowledge expands and circumstances change about each aquifer the rules and regulations can be changed.[59]

7. Notification of the rules and regulations or any changes thereof shall be by newspaper publication once per county at least 60 days prior to the effective date of the item. In addition copies of the items shall be sent to all on the mailing list in the division and they shall be available free of charge to the owner of a water right at the office of the water clerk.[60]

8. Protest procedures are the same as for the protesting of a ruling by a referee as found in Colo. Rev. Stat. 37-92-304, 1973.[61]

9. Protests must be filed by the end of the month following the month of publication.[62]

10. In addition to the above guidelines the rules and regulations are also subject to the right of the engineer to limit water rights as found

in CRS 37-92-502 (2), 1973.[63] The original article 37-92-501 (1973) provided only that the Colorado Constitution and applicable statutes would govern the state engineer in his governing of water of the state. The present section is in part a response to Fellhauer which dealt in part with the lack of rules and regulations in the regulation of wells.

The state engineer may issue orders to owners and users of water concerning rules and regulations in addition to other situations.[64] The engineer may order the total or partial discontinuance of a diversion if the amount diverted is not necessary for application to a beneficial use or, if material injury is or would result to the senior, when a senior calls for water being used by a junior appropriator. Factors determining material injury are:

1. The current and prospective volumes of waters in the stream or tributary thereto,
2. Distance and type of stream bed between diversions,
3. The velocities of water, surface and underground,
4. Probable duration of available flow, and
5. The predictable return flow.

Each diversion shall be evaluated and administered independently to determine effects upon senior users.[65] If a discontinuance by a junior would not make available additional flow to the senior then it shall be rescinded.[66] Also, if the senior has a well as an alternate means of diversion both it and his surface right must be used before he can place a call on the river.

The engineer must order the release of any illegally stored water and insure its proper delivery.[67]

The water involved in a plan of augmentation shall be administered by the engineer of the division involved if it stays within the division. If more than one division is involved then the state engineer must administer the waters movement however he may act through the division engineers. The state engineer may issue such orders as are necessary to achieve the overall goal of maximum economic benefit and use.[68]

The state engineer and division engineers may order the installation of volumetric measuring devices by the owner or user of a water right and to make periodic reports thereon.[69]

The state engineer, division engineers and their assistants and staffs may enter private property for the purposes of administering the requirements placed upon them.[70]

The state engineer, division engineers, and their assistants may order the removal from a stream of any unnecessary dam or obstruction.[71]

If an order issued pursuant to CRS 37-92-502 is not complied with the state engineer and the particular division engineer, through the attorney general, may apply to the appropriate water judge for an injunction.[72] If the state is upheld in this proceeding then the enjoined party shall pay the costs including a reasonable attorneys fee.[73] In considering the request for an injunction the judge must generally determine if the requesting senior would benefit from its release, would be or is now being harmed by its retention.[74]

Intervention in such a proceeding by any party is allowable if done timely and will not unduly delay the proceeding.[75] Failure to comply with an injunction subjects the violater to contempt proceedings.[76] These proceedings and penalties shall be in addition to whatever else is provided by law.[77]

If failure to comply with an order issued pursuant to 37-92-502 results in injury to another party. Then the violater is subject to triple damages plus reasonable attorneys fees.[78]

In the distribution of water the priorities of water rights under this and preceding acts govern. The only exception being where under section 1, 37-92-502(2), 1973 the engineer can limit the right due to the application to nonbeneficial uses.[79] This provision also allows the use of a well to satisfy a surface right.[80] If the well draws from the same stream system as the surface right then the owner may have it declared an alternate point of diversion.[81] Until the mandatory date for adjudicating wells passed any well could be so used, however it could still be regulated.[82]

Special Water Courts

One of the unique features of the 1969 Act was the creation of the posts of Water Judge, Water Clerk and Water Referee. As their titles imply these judges have as their primary job the legal administration of water under the 1969 Act.

The water clerk is an associate clerk of the district court and is appointed in the same manner as a district court clerk. The job may be full or part time.[83] The offices of the water clerks are to be colocated with the office of the clerk of the district court in the counties specified in section 1, 37-92-204(1)(b), 1973 (Weld, Pueblo, Alamosa, Montrose, Garfield, Routt, and LaPlata). Basically, the water clerk is responsible for keeping all records of past actions affecting water rights, proceedings and rulings of the water judge, and other duties prescribed by the water judge and supreme court.[84]

The position of water judge was established in each division. This is a collective position for all the district courts situated entirely or partly within the division. As the title implies they have exclusive jurisdiction over water matters within the division. However, the matters they may hear entail only questions under this act and other laws which specify jurisdiction to the water judge.[85]

The water judges were originally appointed within 10 days after June 7, 1969 and since then by January 10 of each year. The judges have a one year appointment which can be renewed by the supreme court. They must be selected from among the district court judges within the division. Vacancies occurring during the year are also filled by the supreme court. During this period the judges shall hear all pending and new water matters which arise in the division. The water judge has normal duties as a district court judge, however any water matter takes precedence. Should the need arise the supreme court can appoint additional water judges within a division. These judges may even come from outside the division.[86]

The water judge will normally sit in the district court where the water clerk is located; however, should the parties request otherwise the judge may, in his discretion, conduct the proceedings in other locations.[87]

The water referee was created to assist the water judge in the administration of the water law. The referee, if needed, is appointed by the water judge of the division involved.[88] The water judge may appoint additional referees as needed.[89] The water judge may elect to perform the functions of the water referee himself.[90] The basic qualification of the referee is that he be expert in water law matters.[91]

Procedures Followed on Actions Affecting Water Rights

Formally a person desiring to get a legal determination on any act which will affect a water right will start at the water clerks' office by picking up the proper form for the type of request he may have.[92] Basically, with these forms he must show a justification for his request.[93]
If the request is for a water right or a conditional water right then some specifics are required. Items such as: "the legal description of the diversion or proposed diversion, a description of the source of the water, the date of initiation of the appropriation or proposed appropriation, the amount of water claimed, and the use or proposed use of the water."[94] For a change of water right "a description of the water right or conditional water right for which the change is sought, the amount and priority of the water right or conditional water right, and a description of the proposed change of water right" should be included. A plan of augmentation requires a complete statement of the plan. Should the application involve the construction of a well the state engineers permit, denial, or failure to act for a six month period must be attached.[95]

The water clerk upon receipt of the application includes it in the resume of all requested actions which is prepared each month.[96] By the end of the month these actions are published in a newspaper of general circulation in each county.[97] Notification is also accomplished by the end of the month by use of the mailing list of the water clerk.[98] The newspaper notification procedure may also be augmented by radio and TV coverage if necessary.[99]

Should a party wish to oppose an application by the other party he may do so by filing a statement of opposition[100] by the last day of the second month following the month in which the application was filed.[101] The water clerk must forward a copy of the opposing statement to the state engineer and division engineer.[102]

The applications and statements of opposition or copies thereof filed pursuant to section 1, 37-92-302 are to be promptly referred to the referee by the water judge.[103] The referee then conducts an investigation of the matters contained in the application and statement of opposition.[104] He "shall consult with the appropriate division engineer and may consult with the state engineer, the Colorado Water Conservation Board and other state agencies."[105] A copy of this consultation is then made available to the consulted agency or party and a copy filed in the proceedings.[106]

"Within sixty days from the last day on which statements of opposition may be filed . . ." on the application the referee must make his ruling on the matter, unless this time is extended by the water judge or the referee re-refers the matter to the water judge.[107] A significant aspect of this section is the flexibility allowed the referee. He may wholly or partially approve or reject the application even if no statements of opposition were filed.[108] The ruling must be filed with the water clerk and is subject judicial review. Notification of the results must be given to the applicant(s), those filing statements of opposition, the state engineer, and to the division engineer.[109]

If there has been statements of opposition filed the referee may re-refer the matter to the water judge. This must be done within a month following the last month in which statements of opposition could have been filed.[110]

If a party wishes to protest the ruling of a referee he must do so in writing to the water clerk within twenty days after the referee's ruling. The protest must clearly set out the factual and legal grounds for the protest. A copy of this protest must be sent to the applicant or applicants and to a party or parties who have filed statements of opposition.[111]

On specified dates throughout the year the water judges will hear matters which are the subjects of protests or have been re-referred by a referee.[112] Should a party request the place of hearing shall be in a county where the point or points of diversion, water rights, or conditional water right(s) are located. If more than one county is involved then the hearing shall take place in the county where the majority of the right or rights in question are located.[113]

Proceedings before a water judge do have some special features. Though they are governed by the Colorado Rules of Civil Procedure no pleadings are required and the judge is not bound by the referee's ruling. The division engineer must appear to offer pertinent information and can be examined by any party. The division engineer shall be represented by the attorney general, at the engineer's request.[114] The burden of proof falls on the applicants in spite of the referees ruling. The hearing is also open to all persons interested and they may represent themselves or be represented by counsel.[115] Finally, service of applications, protests, or statements of opposition, or other documents is not necessary for jurisdictional purposes.[116]

The water judge has complete discretion with respect to a referee's rulings which are before him by re-referral or protest. Should he modify a ruling he may grant a different priority date than that granted by the referee. He may also specify his own terms or conditions with respect to a change of water right or plan of augmentation. He must, however, fully dispose of any matter re-referred by a referee. He must also confirm and approve by judgment and decree a ruling by the referee the subject of which had no protest filed against it. He may however reverse or reverse and remand any ruling which he deems to be contrary to law.[117]

In matters concerning a chance of water right or plan for augmentation the water judge may attach the condition that the matter of injury to the vested rights of others is subject to reconsideration for two years after the decision. He may also add any conditions he feels necessary to protect the rights and interests of other parties involved.[118]

The water judge also has flexibility in his judgments and decrees, the only criterion being that they be promptly filed. He may confine the judgment and decree to one matter or include several. In any event he must give the name or names of the applicants, the location of the point or points of diversion, place or places of storage, means of diversion, type or types of use, the amount and priority, and any other information necessary. If the application was for a determination of water rights or conditional water rights the date of the filing of the application shall be stated in judgment and decree.[119]

Copies of the judgments and decrees are to be sent to the state engineer and division engineer. They must then adjust their records and distribute the water accordingly.[120]

An important aspect of the '69' Act concerns appellate review of the water judge's decision. Review of the judgment and decree is allowed, ". . . but no appellate review shall be allowed with respect to that part of the judgment or decree which confirms a ruling with respect to which no protest was filed."[121]

Harsh as the appellate review procedure may be there is opportunity to file a protest or correct errors after a decision has been rendered by the water judge. The water judge may have clerical errors corrected on his own initiative or on the petition of <u>any</u> person. A substantive error may be corrected by the water judge on the petition of <u>any</u> person whose rights have been adversely affected. There must be a satisfactory showing to the water judge that the person failed to file a protest because of mistake, inadvertence or excusable neglect. There is a three year time limit for these actions and any decision by the water judge is subject to appellate review.[122]

There is also another procedure with which a person can challenge a referee's ruling. After the granting of an application in whole or in part a person has thirty days in which he can assert the effect of the ruling will cause him injury. This initial procedure is <u>ex parte</u>. In effect the petitioner presents facts about the granted application which he believes will cause him harm. If the water judge believes this contention has merit then he can order the applicant to show cause at a hearing why no damage would result from the ruling by the referee. Should the judge find that material injury would result then he may stay the ruling by the referee until a judicial review can take place.[123]

The standards for rulings by the referee and decisions of the water judge approximate the 'old material injury test' by use of the concept of injurious effect.[124] Specifically, in section 1, 37-92-305(3), 1973 a change of water right or plan for augmentation must be approved if it will not injuriously affect the other water rights holders. Should there be an injurious affect the applicants can propose modifications in their requests to eliminate the injury. To eliminate injury such things as a limitation on use,[125] relinquishment of some rights to prevent an enlarged historic use or diminished return flow,[126] time limitations on the use of the right,[127] and any other conditions necessary to protect the right.[128] Another feature of the act is the allowance of a particular means or point of diversion to serve as the source for more than one right.[129]

The referee or water judge may also use the findings of the state engineer on a well application as evidence if the well is included in the application in question. If the well permit was justifiably denied then the judge should also deny the request. However, if the permit was granted he may grant a conditional decree. If the court grants a conditional or final decree the state engineer must issue the well permit.[130]

E. LITIGATION AND REGULATION*

The courts have stated in numerous cases that "subject to prior appropriations, underground waters supplying a natural stream are open to appropriation like surface waters, because they belong to the river."[131] The courts have also noted that the burden of proof in claiming that groundwater is not tributary lies on the party making the assertion, and that it must be proved by "clear and satisfactory evidence."[132]

In trying to define what the relationship is between ground and surface water, Hutchins makes the following statement:

> [T]he fact that surface streams 'lose' water into the ground at some times and places and 'gain' water therefrom at others has long been recognized not only by ground water hydrologists and engineers but also by attorneys, judges, and legislators as well. Nevertheless, integration of surface and ground water doctrines and rights of use has not always kept pace with comprehension of physical conditions. Rival claimants to waters of surface streams have usually litigated their relative rights as between themselves, without intervention of owners of wells who depend on ground water that feeds the stream or that escapes from it, and the reverse holds true with respect to most adjudications of ground water rights. Lack of correlation has more serious results in such cases than where separate adjudications are made of rights on a surface stream and on its main tributaries, because the character of the surface water rights is the same--appropriative, or appropriative and riparian, depending on the jurisdiction. But in some states surface stream rights may be solely appropriative and ground water rights may be based on land ownership--even the rule of absolute ownership in overlying land. Repeated court decisions may have welded this rule into a rule of property, which may be difficult to overturn when many more rights become vested and more knowledge as to physical interrelationships is available.[133]

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* The following three sections are included in Radosevich, G. E. et al., Evolution of Colorado Water Law, 1976.

Groundwaters in the alluvium underlying the drainage basin of a river and hydraulically connected with its surface streams are a part of the river system, and removals either from the surface portion of the system or the underground portion of it decrease the water available in the whole system.[134] Groundwater diversions, junior in time and in right to surface appropriators, result in a reduction of surface supplies of water which otherwise are available to senior surface appropriators. Therefore, regulation of diversions by means of wells is needed to lessen the material injury to senior appropriators. This is not to say that groundwater diversions are to be discontinued. Indeed, groundwater constitutes a slowly moving body of water, much of which is below the influence of plant transpiration and evaporation and is available for diversion and application to beneficial use, subject to conditions necessary to protect senior rights.[135]

One of the earliest cases concerning tributary groundwater was that of McClellon v. Hardle.[136] The plaintiff in this case was the owner of 400 acres of land in Weld County. In 1886 he had filed the necessary papers to secure a water right and constructed diversion works to irrigate his land from Love Tree Creek. The defendants subsequently sunk a well near the creek and put in a pump. The stream was one that "at times and places, flows above the ground . . . and at other times and places, below the surface as a subterranean current. The surface water and underflow of said stream are connected and coexist.[137] The court held that the defendants had not invaded the rights of the prior appropriator but also held that it is an invasion of the rights of a prior appropriator to divert water from a stream--surface or subterranean--by means of dams, wells, or pumps, whereby the flow of water is diminished. However, in this case the court felt that the evidence was vague and indefinite and did approve the claim for damages.

One of the more significant cases involving the recognition of groundwater in the appropriation system in Colorado was that of Medano Ditch Co. v. Adams. In this decision, the courts held that:

> Underground currents of water which flow in well-defined channels the course of which can be distinctly traced, are governed by the same rules of law as streams flowing upon the surface. The channels and existence of such streams, though not visible, are 'defined' and 'known' within the meaning of the law when their course and flow are determinable by reasonable inference.[138]

Since the mid-1960s, regulation of groundwater pumping to protect senior surface water rights has been a major issue in the Arkansas and South Platte basins. Administrative control efforts were followed by repeated litigation, legislation and negotiation.

Fellhauer v. People

The initial and perhaps most significant case concerning the authority of the State Engineer is Fellhauer v. People.[139] Therefore, the circumstances surrounding the case will be briefly reviewed. On June 24, 1966, there was insufficient water in the Arkansas River to fill the adjudicated rights of downstream users having priority dates as early as 1887. The State Engineer responded by notifying the defendant to halt pumping until further notice. The defendant's well in this case was located approximately 30 to 35 miles south of Pueblo, Colorado, and was approximately 400 feet from the bank of the river. The well was drilled to a depth of about 35 feet. The defendant refused to comply with the order. The lower court then issued a preliminary restraining order, halting the defendant from pumping water. The lower court subsequently issued a permanent injunction.

The State Engineer had acted in this instance under the authority given him by House Bill 1066, passed in 1965.[140] This bill stated that:

> The State Engineer or his duly authorized representative shall execute and administer the laws of the state relative to the distribution of the surface waters of the state including the underground waters tributary thereto in accordance with the right of priority of appropriation, and he shall adopt such rules and regulations and issue such orders as are necessary for the performance of the foregoing duties.

The Supreme Court stated that it was within the power of the State Assembly to "delegate to the water officials the power to protect the stream against unreasonable injury by junior wells when lower senior appropriators are not receiving, but are in need of and asking for, their decreed rights." The court also stated that, in determining the effect that a well might have on a surface flow, the following factors must be considered: (1) distance of the well from the stream, (2) transmissibility of the aquifer, (3) depth of the well, (4) time and volume of pumping, and (5) return flow characteristics. It was the conclusion of the court that "a well in or at the bank of the stream may have substantially the same effect as a surface diversion at that point."

Concerning the actions of the State Engineer in this case, the court found that he had acted "without any written rules or regulations and without any prescribed guidelines." The court also considered the fact that "of the thirteen water districts under his jurisdiction, he acted only in two. In these, he ordered 39 wells to cease pumping." These 39 were out of a total of 1600 to 1900 wells that were pumping more than 100 gallons per minute. The court concluded from the testimony given that:

> In his attempted enforcement of the 1965 act, he [the State Engineer] proceeded discriminatorily in violation

> of the equal protection clause of the Fourteenth Amendment of the U. S. Constitution and of the due process clause in Article II, Section 25 of the Colorado Constitution.

The court proceeded to set forth three requirements that must be met in regulating the wells, as contemplated by the 1965 act, in order to be valid and constitutional. These requirements are:

1) the regulation must be under and in compliance with reasonable rules, regulations, standards, and a plan established by the State Engineer prior to the issuance of the regulative orders;

2) reasonable lessening of material injury to senior rights must be accomplished by the regulation of wells; and

3) if, by placing conditions upon the use of a well or upon its owner, some or all of its water can be placed to a beneficial use by the owner without material injury to senior users, such conditions should be made.

The court asserted that the first requirement mentioned:

> . . . will prevent arbitrary and discriminatory action of the Division Engineer . . ., of erroneously making his guidelines on agreement with certain senior users; of attempting to protect the economy of the valley without plan; and of discriminating unreasonably between wells.

The court was of the opinion in setting forth the third requirement that there must be another consideration besides the vested rights of the users. The court stated that:

> It is implicit in these constitutional provisions (Article XVI, Section 6) that, along with vested rights, there shall be maximum utilization of the water of this state.

The court finally came to the decision that, due to "arbitrary and capricious conduct on the part of the Division Engineer," the injunction of the lower court was to be dissolved.

Based on the pronouncement of the court in the Fellhaure case, the State Engineer's Office established rules and regulations that were intended to comply with the requirements of the court. These rules and regulations were intended to go into effect on August 8, 1969. In general, the rules were as follows:

1. In administering the waters of the state, such factors as weather conditions, present and prospective water supply conditions, records of the State Engineer's Office, past experience, and any other factors were to be considered. No curtailment of any diversion will result unless it shall result in a reasonable lessening of material injury to senior appropriators.
2. All rights to divert groundwater that have not been adjudicated according to Colorado law, shall be administered as if they all had the same date of priority and the same right to divert until such time as these rights have been adjudicated.
3. Any groundwater appropriator affected by the rules may use a part or all of the water the well or wells will produce without injury to any herein established regulation, provided that the Division Engineer may approve a proposed written plan submitted by the appropriator or appropriators, whereby the amount of the depletion from the stream from said well or wells during the irrigation season will be restored to the stream by replacement or exchange from sources other than groundwater at the time and in the amount that the depletion tables place, so that prior vested rights are not damaged.

4. An appropriator may elect to treat any well or wells as alternate points of diversion for part or all of any decreed surface right upon submission and approval of a written plan to the Division Engineer.

Well Owners v. Kuiper

As a result of these rules and regulations a case known as Well Owners' Conservation Association v. Kuiper was filed.[141] Therein the court held that "The rules and regulations as promulgated by the State Engineer and to be implemented as of 8 August 1969 were discriminatory, arbitrary and capricious." Also the rules were held by the court to be "unreasonable, vague and unenforceable," and "the regulations themselves . . . unconstitutional and vague." The court was also of the opinion that the term regulations means "shutting" down the wells, and that the manner which was proposed "would not produce water in the stream to satisfy any call in time of need."

The court was of the opinion that:

> The State Engineer may and should charge to the surface decree the amount of water diverted by wells and applied to the same lands as are served by a surface decree, prior to the recognition of any purported call under the surface decree.

The court also ordered that the State Engineer should give consideration to what is known as a futile call and should administer the water in such a manner:

> . . . that an appropriator will not be permitted to command the whole flow of the stream, merely to facilitate his taking a fraction thereof and that there will be no reductions of any lawful diversions because of the priority system, unless such reductions would increase the amount of water available to and required by the water rights of a senior appropriator.

Finally, the court ordered a permanent injunction against the State Engineer, and he was "enjoined and restrained from enforcing or attempting to enforce the rules and regulations. . . ."

Kupier v. Well Owners

The case previously reviewed was challenged in the case of Kuiper v. Well Owners' Conservation Association.[142] This court was of the opinion that the findings of the trial court "gave no probation effect to the scientific evidence presented by the State Engineer." In fact, this court went on to say that "the evidence presented was insufficient to support the findings and decree." The court, after reviewing the record of the trial court, was of the opinion that:

> The rule followed [by the trial court] was that the State Engineer had the burden of proof of the validity of his regulations. On the contrary, his regulations are presumed to be valid until shown otherwise by a preponderance of the evidence.[143]

The court challenged each of the 24 points presented by the trial court; a summary of these major points follows:[144]

1. Concerning the trial court's opinion that the regulations were so "unreasonable and vague as to be violative of the Fourteenth Amendment . . .," this court held that they were not unconstitutional. The court stated that "regulations and statutes are presumed to be constitutional until shown otherwise."
2. Concerning the point that the regulations were contrary to the 1969 act, this court held that;

   > It would be an impossibility for the State Engineer in 1969 to promulgate regulations which would realize the maximum use of all of the surface or ground water of the Platte. All that

> can be expected is that he exercise his best judgement, using information then available to attempt to reach the goal of maximal use, of course, without being arbitrary or capricious.

3. Concerning the point that the State Engineer failed to follow the rules established by Fellhauer, the court responded that the engineer was, in fact, "attempting to follow the mandates of both Fellhauer and the statute."
4. Referring to the charge made by the trial court that the regulations were (a) discriminatory, arbitrary, and capricious; and (b) unreasonable, vague, and unenforceable, the court made the following finding: concerning the first point (a), the court was of the opinion that the regulations were not "fatally vague;" on the second point (b), the court stated that it had been "unable to find anything in the record indicating that the regulations are unenforceable."

The next point of the trial court challenge concerned the view of the trial court that "the regulations do not promote the continuance of existing uses." The Supreme Court said that "if the regulation of wells which are inferior in priority will reasonably contribute to the satisfaction of earlier priorities, the owners of the wells cannot be heard to say that they have a right to continue the use thereof."

The Supreme Court reviewed three of the points of the trial court together: (1) shutting down wells will not cause water to reach the stream to satisfy any call in time of need; (2) the regulation promotes and encourages futile calls; and (3) waste results from shutting off the wells.

Concerning the first point, the court said it could "see no logical distinction between the result of an intervening storm in the case of a call on the surface right and the case of a call on a well." Secondly, the court could find no "evidence to support the findings that the regulations promote and encourage futile calls." Finally, there was no evidence available to the court that indicated waste would result from shutting off wells.

Concerning the charge by the trial court that the rules established by the engineer were "neither sound nor flexible for the integrated use of all the wastes of the state," the Supreme Court was of the opinion that the rules established "should be considered here solely with respect to their application on the Platte River," and not as a uniform regulation for every river basin in the state.

Referring to the charge by the trial court that the "regulations permit appropriators to command the whole flow of the stream," the Supreme Court said that since the regulations halt pumping only 3/7 of the time, they do not allow a certain group to "command the whole flow of the stream."

The trial court had been quite concerned with the portions of the regulations that failed to require of an appropriator before he could place a call on the river that the wells he owned be charged first against his surface water decree. The Supreme Court said the 1969 act did not require "and we know of no other requirement compelling an owner of a surface decree to first apply his well water to that decree before making the call upon junior appropriators, be they surface or underground."

The Supreme Court made reference to several other points mentioned by the trial court. The Supreme Court saw fit to reverse the judgment of the trial court on every point. The court concluded that some of the points mentioned in the case may be proven erroneous by further research, but "further research and testing will not only result in correction of past mistakes, but also will lead us closer to the goal of minimal waste of water."

F. 1972 RULES AND REGULATIONS

Based on the Fellhauer v. People and Kuiper v. Well Owners' Conservation Association cases, the State Engineer instituted a new set of regulations which became effective on May 15, 1972. The regulations provide:

1. These proposed rules and regulations shall affect all underground water as defined in §37-91-103(4), Colorado Revised Statutes, 1963, amended as follows:

    > 'Underground water,' as applied in this act for the purpose of defining the waters of a natural stream, means that water in the unconsolidated alluvial aquifer of sand, gravel, and other sedimentary materials, and all other water hydraulically connected thereto which can influence the rate or direction of movement of the water in that alluvial aquifer or natural stream. Such 'underground water' is considered different from 'designated groundwater' as defined in 37-90-103 (3), except water withdrawn from wells exempted under §37-92-602, Colorado Revised Statutes, 1963, as amended.

2. All rights to appropriate underground water for which an application for determination of the amount and priority thereof has not been filed with the Water Clerk prior to July 1, 1972, are junior to all claims for underground water filed with the Water Clerk prior to that date. Such junior rights may not be allowed to divert any water if curtailment of any water diversion is necessary to satisfy prior vested rights.

3. Any underground water appropriator or appropriators affected by these rules and regulations may use a part or all of the water the well or wells will produce without regard to any regulation if the

Diversion Engineer approves a written plan submitted by the appropriator or appropriators whereby the amount of the depletion from the stream by said well or wells wil be returned to th stream so that prior vested rights are not damaged.

4. An appropriator may elect to treat any well or wells as alternate points of diversion for part or all of any decreed surface right upon submission and approval of a written plan to the Diversion Engineer.
5. There shall be kept on file in the Office of the State Engineer and Division Engineer of Irrigation Division No. 1 maps of the South Platte River, its tributaries, and the aquifers associated therewith, depicting the administrative zones and reaches thereof. Underground water affected by these rules and regulations shall be divided into the following four zones and regulated accordingly:
   a. Zone A: underground water located such that the time of initial effect of the appropriation thereof occurs from zero to ten days after appropriation commences. Regulation of wells in Zone A shall commence on the date of a written demand or five days prior to the date of anticipated demand by a senior vested right.
   b. Zone B: underground water located such that the time of initial effect of appropriation thereof occurs from ten to thirty days after appropriation commences. Regulation of wells in Zone B shall commence on the date of a written demand or 20 days prior to the date of anticipated demand by a senior vested right.

c. Zone C: underground water located such that the time of initial effect of appropriation thereof occurs from 30 to 75 days after appropriation commences. Regulation of wells in Zone C shall commence on the date of a written demand of 52 days prior to the date of anticipated demand by a senior vested right.

d. Zone D: underground water located such that the time of initial effect of appropriation thereof occurs 75 or more days after appropriation commences. Regulation of wells in Zone D shall commence on the date of a written demand or 112 days prior to the date of anticipated demand by a senior vested right.

The South Platte River, its tributaries, and the aquifers associated therewith shall be further divided into the following reaches or segments of the river:

Reach I: from the headgate of the Fort Morgan Canal upstream to the headwaters. In this reach the anticipated date of first demand is April 30.

Reach II: from the point where the South Platte River crosses the west boundary of Washington County to the headgate of the Fort Morgan Canal. In this reach the anticipated date of first demand is May 30.

Reach III (A): from the headgate of the Harmony Ditch to the point where the South Platte River crosses the west boundary of Washington County. In this reach the anticipated date of first demand is May 30.

Reach III (B): downstream from the Harmony Ditch to the state line. In this reach the anticipated date of first demand is June 8.

Wells in the above-designated zones and reaches shall be regulated during that period of the year in which their pumping will affect the surface flow of the river or stream from the anticipated date of first demand in that reach until October 1st of that irrigation season. The State Engineer and the respective Division Engineer shall estimate the amount of the shortage resulting to surface appropriators from well operations. Groundwater appropriations being regulated at any particular time, except those set forth in Rule 2 hereof, will be regulated on a basis not to exceed 3/7 of the time. The Division Engineer shall administer this rule so that the operator of the well or wells may have a cycle of operation to make most efficient use of water available, provided that other appropriators are not adversely affected.

The anticipated date of first demand referred to above for each of the reaches of the river may be set at a later date by the Division Engineer, with agreement by the State Engineer, if present and anticipated hydrologic conditions so justify.

The general policy statement in support of these regulations--entitled Proposed Rules and Regulations Governing the Use, Control, and Protection of Surface and Ground Water Rights Located in the South Platte River and its Tributaries--is as follows:

1. The relative rate of movement of ground water as compared to surface water is considered herein. Every effort is made to utilize the water found in the alluvium which water is hydraulically connected to the surface channel of the streams of the state.

2. Water right and uses heretofore vested in any person by virtue of previous or existing laws shall be protected.
3. The existing use of ground water, either independently or in conjunction with surface rights, shall be recognized to the fullest extent possible, subject to the preservation of other existing vested rights.
4. The use of ground water may be considered as an alternate or partial source of supply for surface decrees heretofore entered, taking into consideration both previous usage and the necessity to protect the vested rights of others.
5. Cooperative agreements previously made which result in maximum utilization of both surface and subsurface water supply without injury to vested rights shall be recognized to the fullest extent possible in implementing these rules and regulations.

Many of the water users in the South Platte region continued to reject the enforcement of the 1972 Rules and Regulations while numerous groundwater pumpers organized to form augmentation plans described below.

The extent of the State Engineer's authority not only to regulate existing wells, but to prevent new wells in tributary areas was still not settled until 1973 in Hall v. Kuiper.[145]

The State Engineer denied applications to drill two water wells under Colorado Revised Statutes, §37-90-102 _et seq_. A review hearing was held and the applications were again denied. The administrative action was appealed to the district court where the State Engineer's action was upheld. Upon appeal to the Supreme Court, the decision was affirmed.

In this case, the top of the water table was 20 feet below the surface of the land. The flow of water was to the Cache la Poudre River, which in turn flows to the South Platte River. The State Engineer felt that the wells would diminish the flow to the Poudre and Platte, both of which were already overappropriated. Thus, vested rights would be injured. The court held that:

1) even though part of the water used for irrigation would return to the rivers, there would still be harm done to vested users;

2) the fact that no surface appropriator could show injury was immaterial. The fact that there was no water available for appropriation is enough to forbid new wells;

3) surplus water in the river at flood times is not enough grounds to maintain that extra water is available. There must be a surplus year round; and

4) the constitutional provision of Article XVI, Section 6, saying that "the right to divert unappropriated waters of any natural stream to beneficial use shall never be denied" was inapplicable, because there was no unappropriated water available.

The case clearly recognized the authority and responsibility of the state through its administrative organization to not only control divisions of water and administer the water rights, but also to protect existing private and public rights in the resource.

By this time, the conflict and litigation within the South Platte Valley over tributary water use and regulation had made the numerous parties and the Court sensitive to all relevant interests and issues. It became obvious that unless an amicable agreement was reached, strict enforcement of the prior appropriation doctrine was the only solution - to the detriment of groundwater users (many of whom primarily rely upon surface rights) and to the economy of the state, as a result of reduced agricultural output and partial nonuse of groundwater resources of the Valley.

To avoid this drastic action, the parties reached an agreement by way of stipulation to the facts and conclusions of law surrounding the regulation of wells in the South Platte River and tributaries. The stipulation was signed on 15 March 1974 and the District Court for Water Division No. 1 rendered judgment on the matters that day. (See Appendix III-A.)

Concurrently, on March 13, 1974, the State Engineer issued a set of "Amended Rules and Regulations" applicable only to underground water of the South Platte River drainage basin, pursuant to a Water Court decision. This decision and the regulations are the most recent word in the area of conjunctive use on the South Platte, but there may be more litigation in this area. The amended rules are set forth as follows:[146]

> RULE 1. Except as specifically noted below, these Rules and Regulations shall apply to all underground water of the South Platte River and its tributaries as defined in Colorado Revised Statutes Annotated, 1963, §37-91-103(4) (Supplement 1969), and reproduced below, as follows:
>
> > (4) 'Underground water' as applied in this act for the purpose of defining the waters of a natural stream, means that water in the unconsolidated alluvial aquifer of sand, gravel, and other sedimentary materials, and all other waters hydraulically connected thereto which can influence the rate of direction of movement of the water in that alluvial aquifer or natural stream. Such

'underground water' is considered different from 'designated ground water' as defined in 37-90-103(3).

These Rules and Regulations shall not apply to water withdrawn from wells, such as domestic and livestock wekks, which are exempted from administration under Colorado Revised Statutes Annotated, 1963, §37-92-602 (Supplement 1972), and these Rules and Regulations shall not apply to water withdrawn from wells which are exempted from administration by Court decree or statute.

RULE 2. (a) Ground water diversions will be continuously curtailed according to the following schedule to provide for a reasonable lessening of material injury to senior appropriators:

1. During the Calendar Year 1974, five-sevenths of the time;
2. During the Calendar Year 1975, six-sevenths (6/7) of the time; and
3. During the Calendar Year 1976, and thereafter, total curtailment.

Pumping shall be permitted on every Monday and Tuesday of each week in 1974 and on every Monday of each week in 1975. The Division Engineer shall administer this rule so that the operator of a well, or wells, may have a cycle of operation to make more efficient use of the water available; provided that senior appropriators are not materially injured thereby.

RULE 2. (b) Ground water diversions shall be curtailed as provided under part (a) hereof unless the ground water appropriator submits proof to the Division Engineer and upon th basis of that proof the Division Engineer shall find:

1. That the well is operating pursuant to a decreed plan of augmentation, that the well is operating pursuant to a decree as an

alternate point of diversion, or that a change in point of diversion to the well has been decreed for a surface water right; or

2. That the ground water appropriation can be operated under its priority without impairing the water supply to which a senior appropriator is entitled; or
3. That the water produced by a well does not come within the definition of underground water in RULE 1.

RULE 3. Any ground water appropriator affected by these Rules and Regulations may use a part or all of the water diverted without regard to curtailment described in RULE 2 (a) to the extent his ground water diversion is in compliance with a temporary augmentation plan approved by the Division Engineer in accordance with Colorado Revised Statutes Annotated, 1963, §37-92-307(4) and where there is a plan for augmentation filed in the Water Court in accordance with Colorado Revised Statutes Annotated, 1963, §37-92-302 (Supplement 1971). The Division Engineer will promptly approve or disapprove such temporary augmentation plans submitted to him. The guidelines for any such temporary augmentation plan will be expected to meet at least the following criteria:

1. That replacement water for stream depletion shall be made equal to five percent of the projected annual volume of a ground water diversion, and may be used by him at a rate of flow sufficient to compensate for any adverse effect of such ground water diversion on a lawful senior requirement, as evidenced

by a valid senior call, but at a rate not exceeding five percent of the capacity of the diversion structure.

2. Such capacity shall be determined by Court decree, if adjudicated, by application for a water right, if filed in the Water Court, by well permit, or by registration. If none of these means of determination is available, the capacity will be the maximum pumping or delivery rate, which must be substantiated by the appropriator.
3. The operation of the temporary augmentation plan shall not be used to allow ground water withdrawal which would deprive senior surface rights of the amount of water to which said surface rights would have been entitled in the absence of such ground water withdrawal, and ground water diversions shall not be curtailed nor required to replace water withdrawn, for the benefit of surface right priorities, even though such surface right priorities be senior in priority date, when, assuming the absence of ground water withdrawal by junior priorities, water would not have been available for diversion by such surface right under the priority system.

RULE 4. Whenever the Division Engineer is satisfied, upon the basis of competent evidence, that operation of a temporary plan of augmentation pursuant to RULE 3.1. will not meet the requirements of RULE 3.3. above, modification of the plan will be undertaken by reference to criteria as follows:

1. The stream depletion caused by a well will be calculated by the method shown in The Pumped Well by Robert E. Glover, Technical

Bulletin 100, Colorado State University, or by other accepted engineering formulae appropriately modified to reflect the pertinent physical conditions.

2. The transmissivity value will be obtained from the U. S. Geological Survey Open-File Reports, Hydrogeologic Characteristics of the Valley-Fill Aquifer in the South Platte River Valley, Colorado, 1972, or from updated editions, or from calculations using accepted engineering methods.
3. The specific yield or effective voids ratio generally descriptive of the material in the aquifer will be assumed to be twenty percent (20%), or a different value may be used when it can be substantiated generally or as to any particular area or situation.
4. The consumptive use for irrigation purposes will be assumed to be forty percent (40%) of the total quantity pumped for irrigation uses, subject to modification upon proof that a different consumptive use situation exists with respect to a particular diversion. For uses other than irrigation, the amount will be determined from actual conditions.

G. AUGMENTATION PLANS - A USERS SOLUTION

The legislature addressed the problem of integrating ground and surface water use and provided a solution through use of approved augmentation plans.[147] The plan for augmentation is defined as a "detailed program to increase the supply of water available for beneficial use by the development of new or alternative means or points of diversion, by a pooling of water resources, by water exchange projects, by providing substitute supplies of water, by development of new sources of water or by any other appropriate means."[148] This approach is an alternative to strict enforcement of the appropriation doctrine by the State Engineer. However, the burden is upon the water users to design the plan. These plans may be handled in one of two ways. The choice is up to the water user.

The first method is to file an application for approval of the plan with the Water Clerk of the appropriate water district (the district in which the plan is to be implemented).[149] Statements of opposition may be filed with the Water Clerk if this is done by the last day of the second month following the month in which the application is filed.[150] On a date set by statute[151] judgment may include the condition that the plan may be reconsidered on the question of injury to the vested rights of others during any hearing commencing in the two calendar years succeeding the calendar year in which the decision was rendered.[152] The decision may also contain any other provisions which the Water Judge deems proper in determining the rights and interests of the persons involved.[153] The decision may be appealed, or any part of it, on matters which have been the subject of protest before the Water Judge. If there is no protest filed on a matter, there is no appeal from the decision.

The second method is very similar to the first method, but is for approval of temporary augmentation plans. Using the second method the applicant must still file his application with the Water Clerk in the appropriate district. The provision for allowing statements of opposition remains the same. However, the applicant may--at this point it is his option--submit the proposed plan to the State Engineer for his approval.[155] The State Engineer shall approve the plan if it is determined "with reasonable assurance" that the proposed use will not injuriously affect the owner of, or persons entitled to use water under, a vested water right.[156] If it is determined that injury to vested water rights may occur if the plan is approved, the applicant is to be afforded an opportunity to provide protective terms or conditions. In addition to or in place of these terms, the State Engineer is authorized to impose protective terms and conditions on his own.[157]

Where the State Engineer approves a temporary plan for augmentation, the findings of the State Engineer shall be *prima facie* evidence of the proposal's inadequacy.[159]

There are advantages to choosing the second method. The State Engineer's finding is highly persuasive evidence in the Water Court. There is, of course, the danger that the finding will go against the applicant but this result can be easily avoided by properly checking the State Engineer's own records. If these records indicate that there is water available, there should be no problem for the applicant. Equally as important is the estoppel raised by the finding to a later attack by the State Engineer o the plan. Since the State Engineer Office is the only state entity able to attack an augmentation plan, this estoppel argument should give much security to the user. In either case, the plan must be adjudicated by the Water Court. The Water Judge may not refer the plan to a referee; his jurisdiction is exclusive.[160]

For the applicant, the burden of proof will be shifted to him to show that his proposed use will not be detrimental to existing surface users and that, in times of severe water shortage, his plan for augmenting surface flows will be sufficient to protect pre-existing surface rights. It is to be noted that this shifting of the burden of proof from the state to the applicant is not as burdensome as it might seem. Information which is sufficient to carry this burden is available from the State Engineer's Office. And the information would still have to be gathered to rebut a protest in a normal court proceeding. For practical purposes, the shift of the burden amounts only to a change in the order of presentation of the applicant's case.

The second point of importance for the applicant is the role of the State Engineer's finding on his application. The application will have to be ultimately approved by the Water Judge no matter which route is taken by the applicant. But, when the State Engineer has approved the plan subject to the court's adjudication, the finding of the State Engineer is *prima facie* evidence in support of the application and will have to be challenged by a competent evidence to be successfully protested.[161] As a practical matter, the finding of the State Engineer is nearly irreversible as courts are very reluctant to overturn an administrative finding. Of course, if the State Engineer rejects a proposal, that rejection carries equal weight and must be rebutted by the applicant by competent evidence.[162]

For the state, the significance of the second option relates mainly to water management. The burden of proof is shifted to the applicant under the second option. This has the effect of freeing the state from having to challenge each application to insure management of the resource. Under this option, the applicant must make his case with a preponderance of the evidence. From his case, the state can tell whether to challenge him and force a suit or to approve the application. This is a great advantage in terms of time saved in preparation of cases.

The management function of the state over water resources is enhanced, as well. Instead of the applicant going to a Water Court and getting a determination of a plan or a right without the input of the State Engineer, the State Engineer can now exercise his expertise before the plan is approved or a right granted. This gives a *prospective* planning outlook rather than putting the State Engineer in the position of trying to catch up after the courts have acted. Clearly, this prospective planning is superior for setting reasonable objectives.

H. ORGANIZING ACTIVITIES IN THE VALLEY

Water users in the troubled South Platte gradually began to take advantage of the augmentation plan scheme provided by the General Assembly. At present there are a half-dozen augmentation organizations in the Basin.[163]

The largest and oldest augmentation entity is GASP (Ground Water Appropriations of the South Platte, Inc.). GASP was organized to provide augmentation water to senior divertons so that its members could continue to pump groundwater when such pumping effects the stream flows. Membership in GASP is voluntary and assessment made by a fixed fee plus an annual charge for pumping each member's water to him. The annual fee graduates according to how much water a member has pumped--based on what he normally would pump during an irrigating season. The initiation fee is only paid once. The income is used to finance purchases of water. The fee is independent of the wells location in the valley.

In 1974, GASP members had approximately 2,750 wells from Denver to the state line. The augmentations supply to surface users comes from owned, leased, and assigned resevoirs and surface rights and wells stratigically located to provide certain senior rights their entitlements upon demand.

The other augmentation plan entities operate in a similar fashion with the exception of the Central Colorado Water Conservancy District - Sub-District. The plan of the sub-district makes it mandatory that all well owners in the Conservancy District area belong to the organization.

Assessments are based upon ad valorem valuations. There are approximately 1100 wells with the sub-district.

By the use of such entitites as GASP, junior appropriators can now be assured of a continuous supply, even in dry spells. Thus, the traditional appropriation doctrine is no longer the obstacle that it once was for junior users.

Some interesting plans have been put forth as constituting plans for augmentation. Among these are capture of water flowing off roads and removal of trees which use an excessive amount of water. Recognizing that legitimation of these plans would cause havoc with the priority system already established, the Colorado legislature reacted in 1975 by specifically excluding all plans which use the increased runoff as the source of supply but which do not add to the existing supply of tributary water.[164]

I. CONCLUSION

Management of tributory waters in Colorado, and particularly the South Platte has been a very difficult, time consuming, and expensive undertaking by the state. The Office of the State Engineer has been involved in continuous litigation since 1965, requiring deplacement of considerable manpower and finances to achieve the two objectives required by law: (1) protect existing rights, and (2) optimize the use of all waters of the state.

The experience has been a valuable one for Colorado, however. As a result of the enthusiastic positions of surface and goundwater users (in conflict or concert) and of the state officieals to do their job in protecting and pursuing the public and private rights, a scheme has evolved which appears to work satisfactorily to all parties concerned.

In addition, water appears to be used more efficiently. Mr. Wilkerson, Division Engineer of the South Platte, noted that normally on an average day during the crop season, 165 cubic feet per second would be delivered to water users in the Valley. Now this has been reduced to 85-90 c.f.s. as a result of coordination and cooperation in the use and management of the Basin's ground and surface waters.

In spite of the success or satisfaction achieved to date, however, one cannot ignore the words of Mr. Justice Groves, "There must be change and courts, legislators, the State Engineer, and users must recognize it. We recognize that future research and testing may prove erroneous some of the things that we found predominately shown in the record. By the same token, further research and testing will not only result in correction of past mistakes, but also will lead us closer to the goal of minimal waste of water."[165]

J. FOOTNOTES

1. Colorado Springs v. Bender, 148 Colo. 458, 366 P.2d 552 (1961).
2. Fellhauer v. People, 167 Colo. 320, 447 P.2d 986 (1969. Hereinafter cited as Fellhauer.
3. Ibid., p. 336.
4. Loc. cit.
5. Harrison and Sandstrom, "The Groundwater-Surface Water Conflict and Recent Colorado Water Legislation," 43 U. Colo. Law Review 1, p. 10 (1971).
6. Colo. Rev. Stat. §§148-18-1 et seq. (1953).
7. 43 U. Colo. Law Review, p. 9.
8. Farmer, "Colorado's Ground-Water Problem," 'Water and the Law,' Agri. Exp. Sta., Colo. State U. Bull. 505-S, p. 23 (1964).
9. 43 U. Colo. Law Review, p. 9.
10. Ibid., p. 10.
11. As expressed by Mr. Dugan Wilkerson, Division Engineer, Division No. 1, during an interview on Nov. 14, 1974.
12. Bittinger, "Colorado's Ground-Water Problems," 'Ground-Water in Colorado,' Agri. Exp. Sta., Colo. State U. Bull. 504-S, p. 17 (1967).
13. Supra. fn. 5, 43 U. Colo. Law Review, p. 21.
14. Ibid.
15. Ibid., p. 14.
16. Colo. Constitution Article XVI, Section 6.
17. Ibid., Section 7.
18. Ibid., Section 8.

19. 6 Colo. 443 (1882).

20. Ibid., pp. 446-447.

21. 98 Colo. 505, 57 P.2d 894 (1936).

22. 27 S. Ct. 655, 206 U.S. 46, 51 L. Ed. 956 (1907).

23. Snyder v. Colorado Gold Developing Company, 181 F 62, 104 C.A. 136 (1910).

24. 55 Colo. 24, 129 P 220 (1916).

25. 171 Colo. 487, 468 P.2d 835 (1970).

26. Trelease, "Preferences to the Use of Water," 27 Rocky Mtn. Law Review 133 (1955).

27. Hutchins, Ellis and Debraal, "Water Rights Laws in the Nineteen Western States," Vol. I, U.S. Dept. of Agri., Misc. Pub. No. 1206, p. 19, 1971.

28. Ibid., p. 19.

29. Town of Sterling v. The Pawnee Ditch Extension Company, 42 Colo. 421, 94 P 339.

30. Thomas, "Appropriation of Water for a Preferential Purpose," 22 Rocky Mtn. Law Review 422, 425 (1950).

31. 90 Colo. 529, 537, 11 P.2d 221 (1932).

32. Supra. fn. 24, p. 146.

33. Denver v. Sheriff, 105 Colo. 193, 96 P.2d 836 (1939).

34. C.R.S., 37-92-102(1), 1973.

35. C.R.S., 37-92-102(2), 1973.

36. C.R.S., 37-92-102(2)(a), 1973.

37. C.R.S., 37-92-306, 1973.

38. C.R.S., 37-92-302 (Augmentation plans are discussed in great detail in the following sections).

39. C.R.S., 37-92-301(3).
40. C.R.S., 37-92-102(2)(c), 1973.
41. 148 Colo. 458, 356 P.2d 552 (1961).
42. C.R.S., 37-92-301(3)(a),(b),
43. C.R.S., 37-92-103(11), 1973.
44. C.R.S., 37-90-103, 1973.
45. C.R.S., 37-92-301(1).
46. C.R.S., 37-92-201, 1973.
47. C.R.S., 37-92-202(1)(a), 1973.
48. C.R.S., 37-92-202(1)(b), 1973.
49. C.R.S., 37-92-202(3), 1973.
50. C.R.S., 37-92-202(2), 1973.
51. C.R.S., 37-92-301(1), 1973.
52. C.R.S., 37-80,102(1)(a), 1973.
53. C.R.S., 37-92-502(2), 1973.
54. C.R.S., 37-92-501(2)(b), 1973.
55. C.R.S., 37-92-501(2)(c), 1973.
56. C.R.S., 37-92-501(2)(d), 1973.
57. C.R.S., 37-92-501(2)(e), 1973.
58. C.R.S., 37-92-501(2)(f), 1973.
59. C.R.S., 37-92-501(2)(g), 1973.
60. C.R.S., 37-92-501(2)(h), 1973.
61. C.R.S., 37-92-501(2)(i), 1973.
62. C.R.S., 37-92-501(2)(j), 1973.
63. C.R.S., 37-92-501(2)(a), 1973.

64. C.R.S., 37-92-502, 1973.

65. C.R.S., 37-92-502(2), 1973.

66. C.R.S., 37-92-502(2), 1973.

67. C.R.S., 37-92-502(3), 1973.

68. C.R.S., 37-92-502(4), 1973.

69. C.R.S., 37-92-502(5), 1973.

70. C.R.S., 37-92-502(6), 1973.

71. C.R.S., 37-92-502(7), 1973.

72. C.R.S., 37-92-503(1)(a), 1973.

73. C.R.S., 37-92-503(1)(b), 1973.

74. C.R.S., 37-92-503(2), 1973.

75. C.R.S., 37-92-503(3), 1973.

76. C.R.S., 37-92-503(4), 1973.

77. C.R.S., 37-92-503(5), 1973.

78. C.R.S., 37-92-504, 1973.

79. C.R.S., 37-92-301(3)(a), 1973.

80. C.R.S., 37-92-301(3)(a), 1973.

81. C.R.S., 37-92-301(3)(b), 1973.

82. C.R.S., 37-92-301(3)(c), 1973.

83. C.R.S., 37-92-204(1)(a), 1973.

84. C.R.S., 37-92-204(2), 1973.

85. C.R.S., 37-92-203, 1973.

86. C.R.S., 37-92-203(2), 1973.

87. C.R.S., 37-92-203(3), 1973.

88. C.R.S., 37-92-203(4), 1973.

89. C.R.S., 37-92-203(5), 1973.
90. C.R.S., 37-92-203(5), 1973.
91. C.R.S., 37-92-203(6), 1973.
92. C.R.S., 37-92-302(2), 1973.
93. C.R.S., 37-92-302(1)(a), 1973.
94. C.R.S., 37-92-302(2), 1973.
95. C.R.S., 37-92-302(2), 1973.
96. C.R.S., 37-92-302(3)(a), 1973.
97. C.R.S., 37-92-302(3)(b), 1973.
98. C.R.S., 37-92-302(3)(c), 1973.
99. C.R.S., 37-92-302(3)(d), 1973.
100. C.R.S., 37-92-302(1)(b), 1973.
101. C.R.S., 37-92-302(1)(c), 1973.
102. C.R.S., 37-92-302(1)(d), 1973.
103. C.R.S., 37-92-203(7), 1973.
104. C.R.S., 37-92-302(4), 1973.
105. C.R.S., 37-92-302(4), 1973.
106. Ibid.
107. C.R.S., 37-92-303(1), 1973.
108. Ibid.
109. C.R.S., 37-92-303(1), 1973.
110. C.R.S., 37-92-303(2), 1973.
111. C.R.S., 37-92-304(2), 1973.
112. C.R.S., 37-92-304(1), 1973.
113. C.R.S., 37-92-304(4), 1973.
114. C.R.S., 37-92-304(3), 1973.

115. *Ibid.*, p. .

116. Ibid., p. .

117. C.R.S., 37-92-304(5), 1973.

118. C.R.S., 37-92-304(6), 1973.

119. C.R.S., 37-92-304(7), 1973.

120. C.R.S., 37-92-304(8), 1973.

121. C.R.S., 37-92-304(9), 1973.

122. C.R.S., 37-92-304(10), 1973.

123. C.R.S., 37-92-304(11), 1973.

124. 43 *U. Colo. Law Review*, p. 27 (1971).

125. C.R.S., 37-92-305(4)(b), 1973.

126. *Ibid.*, (4)(c).

127. *Ibid.*, (4)(d).

128. 125-127 305(4)(b-d), 1973.

129. C.R.S., 37-92-305(2), 1973.

130. C.R.S., 37-92-305(6), 1973.

131. *Faden v. Hubbell*, 93 Colo. 358, 28 P.2d 247 (1933); also, *Nevius v. Smith*, 86 Colo. 178, 279P44 (1929); and *La Jara Creamery and L. S. Association v. Hansen*, 35 Colo. 105, 83P644 (1905).

132. *Safranek v. Limon*, 123 Colo. 330, 228 P.2d 975; *Comrie v. Sweet*, 75 Colo. 199, 225P214 (1924); *Leadville Mine Development Company v. Anderson*, 91 Colo. 536, 17 P.2d 303 (1932); and *Dalpez v. Nix*, 96 Colo. 540, 45 P.2d 176 (1935).

133. Hutchins, "Ground Water Legislation," 30 *Rocky Mtn. Law Review* 426-427 (1958).

134. "Findings of Fact in Cases No. W-7209, W-7249, W-7289, W-7290, W-7295, W-7296, W-7298 of Water Division No. 1," Water Div. No. 1, Greeley, Colo., p. 2; also, Hall v. Kuiper, 510 P.2d 329 (1973).

135. Ibid., p. 3.

136. 3 Colo. App. 430, 33P280 (1893). Reviewed in Denver Law Review 47, pp. 308-309.

137. 3 Colo. App. 430, 33P280 (1893), p. 431.

138. 20 Colo. 317, 68P431; also, Safranek v. Town of Limon, 123 Colo. 330, 228 P.2d 975 (1951).

139. 167 Colo. 320, 447 P.2d 986 (1969).

140. C.R.S., 148-11-22 (1963), "Concerning Water, Providing for the Execution and Administration of the Water Laws of the State by the State Engineer, and Providing Penalties."

141. Water Division No. 1, Case W2 (1969).

142. Kuiper v. Well Owners' Conservation Association, 420 P.2d 268 (Colo., 1971).

143. Cases cited in support of this opinion are: Thompson v. Consolidated Gas Utilities Corporation, 300 U.S. 55, 57 S. Ct. 364, 81 L. Ed. 510 (1937); New York Foreign Freight Forwarders and Brokers Association, Incorporated v. Federal Maritime Commission, 337 f.2d 289 (1964), cert. denied, 380 U.S. 910, 85 S. Ct. 893; Perko v. United States, 204 f.2d 446 (1953), cert. denied, 346 U.S. 852, 74 S. Ct. 48; United States v. Obermeier, 186 f.2d 243 (1950), cert. denied, 340 U.S. 951, 71 S. Ct. 569; Carter v. Forrestal, 175 f.2d 364 (1949); and Boshe v. Comingore, 177 U.S. 459, 20 S. Ct. 701 (1900).

144. Opinion based on People ex rel. v. Letford, 102 Colo. 284, 79 P.2d 274 (1938); Flank Oil Company v. Tennessee Gas Transmission Company, 141 Colo. 554, 349 P.2d 1005 (1960); and Railroad Commission of Texas v. Rowen and Nichols Oil Company, 310 U.S. 573, 60 S. Ct. 1021, 84 L. Ed. 1368 (1940).

145. 510 P.2d 329 (Colo., 1973).

146. Promulgated pursuant to the decisions in Cases W-7209, W-7232, W-7249, W-7289, W-7290, W-7295, W-7296, W-7298 of Water Division No. 1 and in accordance with the decision in Fellhauer v. People (ibid.), which holds that the State Engineer cannot regulate wells in the absence of written rules and regulations and prescribed guidelines.

147. C.R.S., 37-92-302 and 37-92-307.

148. C.R.S., 37-92-103(a).

149. Ibid., 37-92-302(1)(a).

150. Ibid., 37-92-302(1)(b).

151. Ibid., 37-92-304(1).

152. Ibid., 37-92-304(6).

153. Ibid., 37-92-304(6).

154. Ibid., 37-92-304(9).

155. C.R.S., 37-92-307(2).

156. Ibid., 37-92-307(2).

157. Ibid., 37-92-307(2); the protective terms available to the State Engineer are set forth in C.R.S., 37-92-305(4).

158. C.R.S., 37-92-307(5).

159. Ibid., 37-92-307(6).

160. Ibid., 37-92-307(5).

161. C.R.S., 37-92-307(5).

162. *Ibid.*, 37-92-307(6).

163. Based upon an interview by Professor Radosevich with Mr. Dugan Wilkerson, Division Engineer, Div. No. 1 on 14 Nov. 1974;and examination of public records.

164. C.R.S., 37-92-103(9), 1975.

165. *Kuiper* *v*. *Well Owners' Conservation Association*, 176 Colo. 119, 490 P.2d 268 (1971).

APPENDIX III-A

WATER DIVISION NO. 1
STATE OF COLORADO

CIVIL ACTIONS NO. W-7209, W-7232, W-7249,
W-7289, W-7290, W-7295,
W-7296, W-7298

IN THE MATTER OF THE PROPOSED)
RULES AND REGULATIONS GOVERNING)
THE USE, CONTROL AND PROTECTION)
OF SURFACE AND GROUND WATER)
RIGHTS LOCATED IN THE SOUTH)
PLATTE RIVER AND ITS TRIBUTARIES)

STIPULATION

It is stipulated among the parties to these proceedings that no objection will be made by any of the parties signatory hereto, acting through their respective attorneys, to the entry of the attached "Findings of Fact, Conclusions of Law and Judgment"; nor will objection be made to the adoption of rules and regulations within the principles therefore, as contained in said findings, conclusions and judgment; nor to the "Amended Rules and Regulations of the State Engineer" attached hereto; and it is further stipulated that no further evidence either on direct or cross examination will be offered herein except as may be required pursuant to the "Findings of Fact, Conclusions of Law and Judgment" if entered by the Court in the terms attached hereto.

Each of the parties specifically reserves the right to raise constitutional questions in some other proceeding without in any way being prejudiced in, estopped, or precluded therefrom by virtue of this Stipulation or said Judgment.

Dated this 15th day of March, 1974

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STATE OF COLORADO

NOTICE

Pursuant to CRS 1963, 148-21-34(h), [37-92-501(h), CRS 1973], As Amended, YOU ARE HEREBY NOTIFIED that the following is the Findings of Fact, Conclusions of Law and Judgment, and the Amended Rules and Regulations of the State Engineer

approved March 15, 1974

Case No. W-7209, W-7232, W-7249,
W-7289, W-7290, W-7295,
W-7296, W-7298

IN THE MATTER OF THE PROPOSED)
RULES AND REGULATIONS GOVERNING)
THE USE, CONTROL AND PROTECTION)
OF SURFACE AND GROUND WATER)
RIGHTS LOCATED IN THE SOUTH)
PLATTE RIVER AND ITS TRIBUTARIES)

FINDINGS OF FACT,
CONCLUSIONS OF LAW
AND JUDGMENT

All references to statutes herein refer, without specific designation to the Colorado Revised Statutes.

FINDINGS OF FACT

1. These proceedings concern Rules and Regulations adopted by C. J. Kuiper, State Engineer of Colorado on the 16th day of November, 1972 to become effective February 19, 1973. The Rules and Regulations apply to the waters of the South Platte River and its tributaries.

2. Evidence was presented to the Water Court June 4 through 7 and June 11 through 14, October 29 through 31 and November 1, 5 and 6 of 1973. Of the parties bound by these proceedings, a fairly representative cross section has been active through numerous competent counsel supported by well informed engineering advisors. At a time when no party to these proceedings was foreclosed from placing further evidence before the Court, the active parties submitted suggestions for a final judgment herein and have stipulated and agreed, under the supervision of the Court, to these Findings of Fact, Conclusions of Law and Judgment.

3. All protests were consolidated for trial and relevant objections to the consolidation noted and reserved. In the interest of justice and to simplify proceedings under these protests, all objections to the consolidation of these protests were overruled and the protests were consolidated for trial.

4. During the pendency of the proceedings before this Court various parties made various motions. The Court reserved ruling upon certain motions and the admissibility of certain matters of evidence to permit making a complete record in this complex and highly technical proceeding.

5. Ground waters in the alluvium underlying the drainage basin of the South Platte River and hydraulically connected with its surface streams are a part of the river system, and removals either from the surface portion of the system or the underground portion of it, decrease water available in the whole system. A historical background is necessary to an understanding of the derivation of the final determination herein. Until some thirty years ago, only limited diversions were made of the ground waters and nearly all diversions were made from the surface waters of the Platte River system. Until 1965, there was practically no administration by the State Engineer's office of groundwater diverions while surface water diversions were generally administered according to priority. The Office of the State Engineer, in regulating diversions of various appropriators, endeavored to curtail or shut down junior diversions to the extent necessary to provide a water supply needed for beneficial use by senior appropriators. To facilitate this work, the State Engineer had access to records of surface stream flows at various strategic places in the Platte River system. From experience gained in administration, the State Engineer operated according to practices which were the equivalent of regulations, which were well understood in his office, and, whether written or not, were acquiesced in by appropriators of water in general.

6. It has been the long practice of the State Engineer in administering appropriations by diversion from surface streams to take into account the time it takes for water to flow along surface streams. When surface stream flows are diminished so that curtailment of upstream diversions becomes necessary to provide water for downstream senior appropriators, the timing and amount of curtailment is ordered on the basis of the well known velocities of flow in the various surface streams involved.

7. The evidence shows that in recent years the Office of the State Engineer has become increasingly familiar with the characteristics of flow of the ground water part of the South Platte River system. His office has undertaken extensive studies of that ground water flow which is at such a slow rate that administration of ground water is more intricate and requires greater skill and expertise for proper administration.

8. There is evidence that ground water diversions, junior in time and in right to surface appropriators, have resulted in reduction of surface supplies of water which might otherwise have been available to senior surface appropriators. Sufficient facts exist to support the conclusion that a reasonable lessening of material injury to senior appropriators will be accomplished by the proper regulations of diversions by means of wells. The extent that diversions by means of wells shall be regulated to accomplish this reasonable lessening is provided for herein.

9. There are periods of many years when there is an overabundance of water in the surface portion of the South Platte River system and that over-abundance, together with return flows from beneficial uses, charge and recharge the ground water aquifer of the Platte River. The ground water of the Platte River constitutes a slowly moving body of water, much of which is below the influence of plant transpiration and evaporation. Much of said ground water is susceptible of diversion and application to beneficial use upon imposition of conditions necessary to protect senior rights.

has developed a set of measurements of the physical characteristics of the ground water aquifer to calculate when diversions from the ground water aquifer by junior appropriators are or may be expected to be injurious to senior appropriators.

11. The time of impact of ground water diversions on the surface stream varies according to varying conditions including the distance of ground water diversion from the surface of the stream, the volume and duration of the diversion, and the elevation of the water in the ground water aquifer at the time the diversion is made. Ordinarily, river conditions are such that provision can be made by the ground water appropriator to provide to seniors the amount of any deprivation due to ground water diversions. Because of the time lag between a ground water diversion and its impact on surface water users, conditions may arise such that a potential injury to surface diverts may not actually occur, but the burden of assuring that there will be no injury to the senior appropriator must fall on the junior appropriator.

12. The evidence shows that the method described in the treatise by Robert E. Glover entitled "The Pumped Well", Technical Bulletin 100, Colorado State University, is one of the generally accepted methods of calculating any depletion needed to be replaced in order to avoid injury to a senior exercising a valid call. The evidence also showed, that because the method (which is sometimes referred to as the "Glover formula") is based on certain assumed factual idealizations, expert judgment must be exercised in its application to account for certain variations from these limiting assumptions. Other methods may be more accurate for solution of the problem in a particular case.

13. The proceedings herein show that this Court has jurisdiction of all water users in Water Division I and, whether present or not, all such water users are bound by the actions of the Court herein. The evidence shows that the factual determinations relied upon herein are the subject of some uncertainty, and that judgments required to be made by the Division and State Engineers in the enforcement and application of these Amended Rules and Regulations could potentially adversely affect the rights of parties hereto. It is necessary, however, to proceed with regulation on the best basis currently possible. Due to the anticipated complexity of the application of the Amended Rules and Regulations to particular fact situations, Jurisdiction should be retained.

CONCLUSIONS OF LAW

14. By Section 148-11-22(1), the legislature provided that the State Engineer, in the distribution of water according to priority, "shall adopt such rules and regulations and issue such orders as are necessary for the performance of***" his duties in distributing water. In Fellhauer vs. People, 167 Colo. 320, 447 P.2d 986 (1968), the Supreme Court held that the State Engineer could not regulate wells in the absence of written rules and regulations and prescribed guidelines. In 1971, by an amendment to Section 148-21-34, [37-92-501, CRS 1973], the legislature made its intention clear in this regard by repealing 148-11-22(3) and repealing and amending 148-11-22(1)

and (2) as set forth in 148-21-34, 148-21-35 and 148-21-36 [37-92-501, 37-92-502, 37-92-503, CRS 1973 respectively], in the 1969 Water Adjudication and Administration Act. The mandatory word "shall" was removed and now the last sentence of 148-21-34(1) [37-92-501] provides "the State Engineer may adopt rules and regulations to assist in, but not as a prerequisite to, the performance of the foregoing duties." Section 148-21-34, 35 and 36 [37-92-501, 502 and 503, CRS 1973] when read together now indicate that such a proceeding as this, pursuant to a protest filed in this Court, is not for the purpose of suspending the obligations of the Office of the State Engineer to "order the total or partial discontinuance of any diversion***" to the extent the water being diverted is required by persons entitled to use water under water rights having senior priorities***" 148-21-35(2) [37-92-502, CRS 1973] but to assure that rules and regulations be consonant with the basic requirement for implementing the priority system among all appropriators.

15. The State Engineer has the continuing obligation to administer the water supply which is under his jurisdiction and to issue appropriate orders to effectuate such administration whether or not he has adopted rules and regulations to assist him in the performance of his duties. The "Amended Rules and Regulations" attached hereto are in full force and effect from and after the signing of this decree because stipulated to herein, without prejudice to a further determination with respect thereto if required pursuant to protest hereafter filed following their publication as required by law. Administration of the water of the South Platte River pursuant to the Amended Rules and Regulations attached to this decree will be in accordance with the order of this Court dated August 11, 1972 in Case No. W-6958.

16. The legislature has made special provision for integrating ground and surface water use by 148-21-23 [37-92-307, CRS 1973]. In apparent recognition that augmentation plan approval before the Courts may take a considerable time, the legislature specifically provided by 148-21-(3) (148-21-23(4) in 1971 Session Laws) [37-92-307, CRS 1973] that "until the determinations shall have been made under subsection (2) ***the state engineer and division engineers shall develop temporary augmentation plans*** to allow continuance of existing uses and to assure maximum beneficial utilization of the waters of this state." Unless water users filed augmentation proceedings in the Water Court, the State Engineer may not hereafter authorize temporary plans of augmentation.

17. The Protestants contend that the "Proposed Rules and Regulations" dated November 16, 1972, which are the subject of this proceeding, are not proper as a matter of law; however, as a result of this stipulation to amend the Rules and Regulations it is not necessary to decide this issue.

NOW THEREFORE IT IS HEREBY ORDERED, ADJUDGED AND DECREED AS FOLLOWS:

18. The separate protests to the rules and regulations of the State Engineer have been consolidated for trial, and the protections accorded by the Rules of Civil Procedure in the consolidated action are preserved for each party.

19. All requests for rulings by the Court, other than objections to evidence, which were not otherwise formally rules upon are hereby denied.

20. All objections to evidence not otherwise formally ruled upon are hereby denied, and all evidence submitted herein except as formally excluded is admitted.

21. The Amended Rules and Regulations of the State Engineer attached hereto have been agreed to by virtue of the stipulation of the parties participating in this proceeding and are hereby approved. Said Amended Rules and Regulations are effective herewith and shall remain in effect unless modified or amended in accordance with law. The said Amended Rules and Regulations shall be published as provided by statute, but shall remain in effect during the period of said publication and during the pendency of any protest.

22. Plans for augmentation involving ground water diversions from the South Platte River and its tributaries hereafter filed before this Water Court should utilize the facts and determinations developed in these proceedings to facilitate the administration of water in Water Division One. The method sometimes called the "Glover formula," as described in the treatise by Robert E. Glover and entitled The Pumped Well, Technical Bulletin 100, Colorado State University, may be used for the purpose of calculating replacement water necessary to make up for depletions caused by diversions of ground water to comport with current practices in the Office of the State Engineer. However, some another appropriate method may be used. Such plans should also provide for meeting the other requirements of this decree.

23. To avoid a deprivation of water to some senior appropriator, ground water appropriator, shall make replacement water available for delivery as reasonably required by the Division Engineer, in a quantity, during a period, and at a place so as to prevent a deprivation of water to a senior appropriator caused by such ground water diversion. The Division Engineer shall use valid senior water calls as the normal criteria for requiring such replacements. In applying the terms of this paragraph, it is expected that the Division Engineer will be mindful of all applicable law without overlooking that part of 148-21-34 [37-92-501, CRS 1973] which reads:

> (1) "It is the legislative intent that the operation of this section shall not be used to allow ground water withdrawal which would deprive senior surface rights of the amount of water to which said surface rights would have been entitled in the absence of such ground water withdrawal, and that ground water diversions shall not be curtailed nor required to replace water withdrawn, for the benefit of surface right priorities, even though such surface right priorities be senior in priority date, when assuming the absence of ground water withdrawal by junior priorities, water would not have been available for diversion by such surface right under the priority system."

24. This Court shall retain continuing jurisdiction under these consolidated cases for the purpose of providing an immediate hearing to review the validity of a call, or requirement for providing replacement water, the approval or disapproval of temporary augmentation

plans, findings of the Division Engineer pursuant to Rule 2(b) of the Amended Rules and Regulations stipulated to herein, or any other matter contained within the said Amended Rules and Regulations.

25. The Amended Rules and Regulations of the State Engineer, stipulated to by the parties hereto and attached to this decree, shall be published as provided by law, and all persons affected by any amendment contained in the Amended Rules and Regulations stipulated to herein other than any party bound by the stipulation herein shall have their statutory right to protest.

26. This order does not consitute an injunctive order, but this proceeding may be used, after appropriate notice, as the basis for securing any appropriate injunctive order. No damage occurring prior to issuance of such an injunction shall be the basis for damages, costs of attorneys fees referred to in '63 C.R.S. 148-21-37 [37-92-504, CRS 1973].

27. Since this is an action in rem, all who could have participated are bound by this order, judgment and decree.

DONE IN OPEN COURT this 15th day of March, 1974.

/s/ Donald A. Carpenter
Honorable Donald A. Carpenter
Water Judge
Water Division I

IN THE MATTER OF THE RULES AND)
REGULATIONS GOVERNING THE USE,)
CONTROL, AND PROTECTION OF)
SURFACE AND GROUND WATER RIGHTS)
LOCATED IN THE SOUTH PLATTE)
RIVER AND ITS TRIBUTARIES)

AMENDED RULES AND REGULATIONS
OF THE
STATE ENGINEER

Pursuant to authority vested in the Office of the State Engineer, the State Engineer hereby,

FINDS, that on November 16, 1972 the State Engineer ordered that Rules and Regulations for the South Platte River were to become effective on February 19, 1973. As a result of protests filed to those Rules and Regulations and upon the basis of subsequent proceedings in the Water Court for Water Division I, those Rules and Regulations are hereby amended and changed to read as reproduced below.

The said Amended Rules and Regulations are adopted and shall become effective as of the 16th day of March, A.D., 1974, and shall remain in full force and effect unless changed or amended as provided for by law.

"AMENDED RULES AND REGULATIONS"

RULE 1. Except as specifically noted below, these Rules and Regulations shall apply to all underground water of the South Platte River and its tributaries as defined in Colo. Rev. Stat. Ann. 1963, Sec. 148-21-3(4) (Supp. 1969) 37-92-103, CRS 1973 , and reproduced below, as follows:

> (4) "Underground water" as applied in this act for the purpose of defining the waters of a natural stream, means that water in the unconsolidated alluvial aquifer of sand, gravel, and other sedimentary materials, and all other waters hydraulically connected thereto which can influence the rate of direction of movement of the water in that alluvial aquifer or natural stream. Such "underground water" is considered different from "designated ground water" as defined in 148-18-2(3) 37-90-103, CRS 1973 .

These Rules and Regulations shall not apply to water withdrawn from wells, such as domestic and livestock wells, which are exempted from administration under Colo. Rev. Stat. Ann. 1963, Section 148-21-45 (Supp. 1972) 37-92-602, CRS 1973 , and these Rules and Regulations shall not apply to water withdrawn from wells which are exempted from administration by Court decree or statute.

RULE 2. (a) Ground water diversions will be continuously curtailed according to the following schedule to provide for a reasonable lessening of material injury to senior appropriators:

(1) During the Calendar Year 1974, five-sevenths (5/7) of the time;

(2) During the Calendar Year 1975, six-sevenths (6/7) of the time; and

total curtailment.

Pumping shall be permitted on every Monday and Tuesday of each week in 1974 and on every Monday of each week in 1975. The Division Engineer shall administer this rule so that the operator of a well, or wells, may have a cycle of operation to make more efficient use of the water available; provided, that senior appropriators are not materially injured thereby.

(b) Ground water diversions shall be curtailed as provided under part (a) hereof unless the ground water appropriator submits proof to the Division Engineer and upon the basis of that proof the Division Engineer shall find:

(1) That the well is operating pursuant to a decreed plan of augmentation, that the well is operating pursuant to a decree as an alternate point of diversion, or that a change in point of diversion to the well has been decreed for a surface water right; or

(2) That the ground water appropriation can be operated under its priority without impairing the water supply to which a senior appropriator is entitled, or

(3) That the water produced by a well does not come within the definition of underground water in RULE 1.

RULE 3. Any ground water appropriator affected by these Rules and Regulations may use a part or all of the water diverted without regard to curtailment described in RULE 2(a) to the extent his ground water diversion is in compliance with a temporary augmentation plan approved by the Division Engineer in accordance with <u>Colo</u>. <u>Rev</u>. <u>Stat</u>. <u>Ann</u>. 1963, Sec. 148-21-23(4) 37-92-307, CRS 1973 and where there is a plan for augmentation filed in the Water Court in Accordance with <u>Colo</u>. <u>Rev</u>. <u>Stat</u>. <u>Ann</u>. 1963, Sec. 148-21-18 (Supp. 1971) 37-92-302, CRS 1973 . The Division Engineer will promptly approve or disapprove such temporary augmentation plans submitted to him. The guidelines for any such temporary augmentation plan will be expected to meet at least the following criteria:

(1) That replacement water for stream depletion shall be made available to the Division Engineer in an amount equal to 5 percent of the projected annual volume of a ground water diversion, and may be used by him at a rate of flow sufficient to compensate for any adverse effect of such ground water diversion on a lawful senior requirement, as evidenced by a valid senior call, but at a rate not exceeding 5% of the capacity of the diversion structure.

(2) Such capacity shall be determined by Court decree, if adjudicated, by application for a water right, if filed in the Water Court, by well permit, or by registration. If none of these means of determination

or delivery rate, which must be substantiated by the appropriator.

(3) The operation of the temporary augmentation plan shall not be used to allow ground water withdrawal which would deprive senior surface rights of the amount of water to which said surface rights would have been entitled in the absence of such ground water withdrawal, and ground water diversions shall not be curtailed nor required to replace water withdrawn, for the benefit of surface right priorities, even though such surface right priorities be senior in priority date, when, assuming the absence of ground water withdrawal by junior priorities, water would not have been available for diversion by such surface right under the priority system.

RULE 4. Whenever the Division Engineer is satisfied, upon the basis of competent evidence, that operation of a temporary plan of augmentation pursuant to RULE 3(1) will not meet the requirements of RULE 3(3) above, modification of the plan will be undertaken by reference to criteria as follows:

(1) The stream depletion caused by a well will be calculated by the method shown in The Pumped Well by Robert E. Glover, Technical Bulletin 100, Colorado State University or by other accepted engineering formulae appropriately modified to reflect the pertinent physical conditions.

(2) The transmissivity value will be obtained from the U. S. Geological Survey Open-File Reports, Hydrogeologic Characteristics of the Valley-Fill Aquifer in the South Platte River Valley, Colorado, 1972, or from updated editions, or from calculations using accepted engineering methods.

(3) The specific yield or effective voids ratio generally descriptive of the material in the aquifer will be assumed to be twenty percent (20%), or a different value may be used when it can be substantiated generally or as to any particular area or situation.

(4) The consumptive use for irrigation purposes will be assumed to be forty percent (40%) of the total quantity pumped for irrigation uses, subject to modification upon proof that a different consumptive use situation exists with respect to a particular diversion. For uses other than irrigation, the amount will be determined from the actual conditions.

Dated this 15th day of March, 1974.

/s/ C. J. Kuiper
C. J. Kuiper, State Engineer