"AN ANALYSIS of the CHIPUXET GROUND - WATER RESERVOIR by DIGITAL and ELECTRICAL MODELS"

RHODE ISLAND WATER RESOURCES CENTER
CHIPUXET GROUND-WATER RESERVOIR

An analysis of the Chipuxet Ground-Water Reservoir by digital and electrical models

by: William E. Kelly
  Director
  Rhode Island Water Resources Center
  University of Rhode Island

Prepared by the Rhode Island Water Resources Center,
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LIST OF SYMBOLS USED IN THIS REPORT

A  - coefficient matrix
A_s - area of streambed
C  - capacitance
K_s - streambed hydraulic conductivity
K_x,K_y,K_z - hydraulic conductivity
M_s - thickness of streambed
Q  - discharge per unit area
Q_s - streambed leakage
R  - resistance
S  - storage coefficient
S_s - specific storage
S_y - specific yield
T  - transmissivity
T_x,T_y - transmissivity in the x and y direction respectively
V  - voltage
W  - recharge per unit area
W_0 - recharge at node zero
D  - variable matrix
h  - total head
h_i  - total head at node i
h_a - aquifer head beneath stream
h_s - stream head
i  - gradient
m  - aquifer thickness
t  - time
x,y,z - space variables
INTRODUCTION

In 1973 the Rhode Island Water Resources Board began a cooperative study with the Department of Civil and Environmental Engineering to develop analog and digital models of the Chipuxet ground-water reservoir. From 1974 through 1976 the Office of Water Resources Research supported design, construction and testing of an analog model of the Chipuxet ground-water reservoir through a grant under the Allotment Program of the Rhode Island Water Resources Center. This two year grant (A-056-RI) provided funds for the purchase of specialized equipment for operating resistance capacitance analog models.

On October 30, 1974 a public demonstration of the Chipuxet analog model was made at a meeting organized by the Rhode Island Water Resources Board and held at the University of Rhode Island's Memorial Student Union. On May 12, 1975 results of simulations of the actual pumping schemes proposed for the Chipuxet ground-water reservoir by the RI Water Resources Board were presented at a regular meeting of the Board.

Since the beginning of the cooperative program in 1973, there have been significant advances in ground water modeling techniques. Resistance-capacitance analog models which were common then (Walton, 1970) are now rare in practice. Part of the reason, is the increasing accessibility of large digital computers, and the associated graphical output devices. Coupled with this, are generally lower computing costs, faster execution times, and increased storage capacities. Increased storage capacity means that
problems that could not be readily run nine years ago, are now run routinely.

This report is based largely on theses by D. Geisser (1975) and W. Beckman (1978). Geisser developed both an analog and a digital model of the Chipuxet ground-water reservoir. Geisser tested the five pumping schemes proposed by the RI Water Resource Board assuming steady-state equilibrium. Beckman modified the digital computer program used by Geisser so that aquifer behavior on a monthly or even weekly basis could be simulated. Beckman simulated proposed pumping schemes I and II on a monthly basis and compared these results with Geisser's. In the remainder of this report we will discuss the development and application of ground-water models to be used for predicting the effects of proposed future pumping patterns on water levels and streamflows in the Chipuxet ground-water reservoir.

THEORETICAL BACKGROUND

The brief introduction to modeling which follows is intended only to demonstrate that there is nothing mysterious about computer models; they must be completely described by the modeler using data which best represents actual field conditions.

The equation describing ground-water flow can be written

\[
K_x \frac{\partial^2 h}{\partial x^2} + K_y \frac{\partial^2 h}{\partial y^2} + K_z \frac{\partial^2 h}{\partial z^2} = S_s \frac{\partial h}{\partial t}
\]  (1)
where: \( K_x, K_y, K_z \) are hydraulic conductivities; \( h \) is total head; \( t \) is time; \( x, y, \) and \( z \) are space variables; and \( S_s \) is specific storage. In aquifer solutions the assumption is generally made that head does not vary with depth \( (z) \) so that equation 1 becomes,

\[
K_x \frac{\partial^2 h}{\partial x^2} + K_y \frac{\partial^2 h}{\partial y^2} = S_s \frac{\partial h}{\partial t}
\]

(2)

for a unit thickness.

To account for flow through the entire aquifer thickness, equation 2 is written in terms of transmissivity \((T)\) which is hydraulic conductivity \((K)\) times aquifer thickness \((m)\). Also \( T_x \) is normally taken equal to \( T_y \) although \( T \) may vary within the aquifer--be a function of \( x \) and \( y \). Equation 2 then can be written

\[
T \frac{\partial^2 h}{\partial x^2} + T \frac{\partial^2 h}{\partial y^2} = m S_s \frac{\partial h}{\partial t}
\]

(3)

Compare equation 3 with equation 4 for the flow of electricity,

\[
\frac{1}{R} \frac{\partial^2 V}{\partial x^2} + \frac{1}{R} \frac{\partial^2 V}{\partial y^2} = C \frac{\partial V}{\partial t}
\]

(4)

where \( R \) is resistance and \( C \) is capacitance and the basis for using electrical networks for modeling ground-water flow is apparent.

When flow is steady, head does not change with time and equation 3 can be written

\[
T \frac{\partial^2 h}{\partial x^2} + T \frac{\partial^2 h}{\partial y^2} + W - Q = 0
\]

(5)
where input (aquifer recharge, underflow) and output (well withdrawals, evapotranspiration, underflow) terms have been added. We can also write equation 5 in an approximate finite difference form for uniform $T$ and a square network of nodes ($\Delta x = \Delta y$) as

\begin{equation}
\frac{(h_1 + h_2 + h_3 + h_4 - 4h_0) + W_0(\Delta x)^2 - Q_0(\Delta x)^2}{T} = 0
\end{equation}

(6)

where $h_1$ through $h_4$ are the heads at the four nearest neighbors of $h_0$ or alternately

\begin{equation}
h_0 = \frac{(h_1 + h_2 + h_3 + h_4) + W_0(\Delta x)^2 - Q_0(\Delta x)^2}{4T} = 0
\end{equation}

(7)

Without recharge or discharge at a node, head is just the average of the head at the four nearest neighbors; recharge tends to raise the head and discharge to lower it. The above equations can be written in a more general form and the interested reader may refer to the thesis by Geisser (1975) or standard texts such as the one by Rushton and Redshaw (1979). In this report we will use the simpler forms of the equations for clarity.

The solution technique will be explained with a one-dimensional aquifer model. Figure 1 shows a typical cross-section for the Chipuxet aquifer and figure 2 an idealized model representing it. Assume that a series of wells is proposed to intercept ground water and induce river infiltration. As a first approximation flow is considered to be one dimensional towards the river and the line of wells to behave effectively as a slot. This means that head varies only with distance away from the river ($x$) and not along it ($y$). The governing flow equations can be written in finite difference
Figure 1.-- Generalized Cross-section of the Chipuxet Stratified-drift Aquifer.
Figure 2.-- Cross-section of Idealized Aquifer.
form or represented by a resistor network.

To solve for the nodal heads the necessary boundary conditions must first be introduced. Boundary conditions in this case are the head at the river and the flow from the till area; values must also be assigned to input and output terms.

As an example, consider the following situation for the idealized aquifer, assume aquifer transmissivity is uniform at 8,000 ft.²/day, nodes are spaced 1,000 ft. apart, average recharge is at the rate of 20 in/yr and leakage from the till area 4,000 feet from the river is negligible and can be ignored. A typical nodal equation is written as

\[ h_{i-1} + h_{i+1} - 2h(i) + W(i) - Q(i) = 0 \] (8)

Assume also that the head at the river is 95 ft. and that the proposal is to extract 73% of the average ground water recharge (20 in/yr.) at the first node, 1,000 ft. from the river. The equations are written in matrix form as

\[
\begin{bmatrix}
\begin{bmatrix}
hr \\
o \\
o \\
0
\end{bmatrix}
&
\begin{bmatrix}
W_1 \\
W_2 \\
W_3 \\
W_4
\end{bmatrix}
&
\begin{bmatrix}
Q_1 \\
Q_2 \\
Q_3 \\
Q_4
\end{bmatrix}
\end{bmatrix}
= [0]
\] (9)

The column matrices can be combined into a single matrix [b] so that

\[
[A] [h] - [b] = 0
\] (10)
which for this problem in expanded form is
\[
\begin{bmatrix}
2 & -1 & 0 & 0 \\
-1 & 2 & -1 & 0 \\
0 & -1 & 2 & -1 \\
0 & 0 & -2 & 2
\end{bmatrix}
\begin{bmatrix}
h \\
h \\
h \\
h
\end{bmatrix}
= 
\begin{bmatrix}
93.90 \\
.57 \\
.57 \\
.28
\end{bmatrix}
\]

The next step is to find the inverse of \([A]\) and compute \([h]\) from \([h] = [A]^{-1} [b]\). If it is only necessary to solve this problem once we can compute \([b]\) find \([A]^{-1}\), which in this case can be done with a programmable calculator, and multiply to obtain the heads \([h]\).

Some results from this problem are tabulated below in Table 1.

TABLE 1 - Heads in idealized aquifer for development of 73% of average recharge rate

<table>
<thead>
<tr>
<th>Recharge rate in inches per year</th>
<th>20</th>
<th>10</th>
<th>5</th>
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<tr>
<td>Node</td>
<td>95</td>
<td>95</td>
<td>95</td>
</tr>
<tr>
<td>Node Head, water table altitude, in feet above mean sea level</td>
<td>95</td>
<td>95</td>
<td>95</td>
</tr>
<tr>
<td>River</td>
<td>95</td>
<td>95</td>
<td>95</td>
</tr>
<tr>
<td>2</td>
<td>95.18</td>
<td>94.24</td>
<td>93.79</td>
</tr>
<tr>
<td>3</td>
<td>96.46</td>
<td>94.88</td>
<td>94.11</td>
</tr>
<tr>
<td>4</td>
<td>97.17</td>
<td>95.23</td>
<td>94.29</td>
</tr>
<tr>
<td>5</td>
<td>97.31</td>
<td>95.30</td>
<td>94.32</td>
</tr>
</tbody>
</table>

| River infiltration as percent of development | 0 | 28 | 34 |
Also shown in table 1 are heads for recharge rates of 10 and 5 in/yr, which could represent conditions more severe than the average. Withdrawals are still at the rate of 73% of the average recharge rate and during periods where recharge is less (5 and 10 in/yr for example) results show that heads are lowered and more water must be obtained from the stream. The number in parentheses for the 20 in/yr rate are the heads if there were no pumping at that recharge rate. By comparison it can be seen that with pumping, heads are lowered nearly two feet. This is a crude model but it illustrates how a modeler may easily study a variety of conditions. Additional constraints could be imposed to account for partial penetration of the stream or leakage from the till area. Solutions with an analog model proceed similarly; after a model is set up voltages (heads) are measured.

If it is necessary to solve this type of problem many times on a digital computer, a program could be written which would take as input, transmissivities, heads, recharges, etc.; assemble the equations; and solve for heads and flows. When a program is developed and applied to a particular problem (data are introduced), a model of the ground water system is the result.

The reader may ask how good a crude model of the actual conditions shown in Figure 1 would be. The cross-section in Figure 1 is shown with vertical exaggeration because the aquifer's thickness is very small relative to its lateral dimensions. If the section were drawn without vertical exaggeration, the outwash would be visible only as a thin layer. Also shown in Figure 1 are some possible ground water flow lines; these are intended only to show that flow in the stratified drift aquifer is nearly horizontal.
except where ground water enters the Chipuxet River. Near the river where flow lines curve upward, there is a much greater head loss per horizontal foot of aquifer than in areas further from the river, even when conditions are otherwise the same. If flow is primarily toward the river, then flow can reasonably be approximated as 1-dimensional and modeled as shown in Figure 2. For conditions prior to the development of the Chipuxet ground water reservoir the assumption of one-dimensional or horizontal flow is reasonable. Some additional simplifications may also be explained using the one-dimensional model. In the idealized model (Figure 2), seepage from the till-bedrock area was neglected. This would not be reasonable for the Chipuxet aquifer, since inflow from the till-bedrock areas is an important input. On the loss side, wetlands adjacent to the river are areas of relatively high evapotranspiration losses which have not been directly modeled. Since in this case wetlands are adjacent to a river node, this node actually includes outflow by ground-water flow and by evapotranspiration from the adjacent wetlands. There would be no effect on the predicted heads in the aquifer but the modeled outflow would be larger than the observed outflow by the amount of evapotranspiration. Head losses near the stream due to curvature of flow lines and lower vertical hydraulic conductivities can be accounted for by inserting nodes immediately adjacent to the stream to impede flow.

Some of the simplifications necessary for representing real aquifers with idealized computer models have been discussed. Both the till-bedrock input and the evapotranspiration losses could be included in a computer model but were beyond the scope of this study.
In calibration, which will be discussed for the Chipuxet ground-water reservoir in the next section, both transient and steady-state simulations were used. Equation 11
\[
\frac{\partial h}{\partial x^2} + W - Q = \frac{Sah}{\partial t}
\]  
(11)
is the partial differential equation governing transient one-dimensional aquifer flow. For transient conditions, \( h \) is a function of time and the boundary conditions, recharge inputs and discharges which may all vary with time. If a modeler wants to calibrate a transient model against actual aquifer behavior, aquifer quantities must be known as a function of time. Often only a few values of head and some flows are known as a function of time and the need for or appropriateness of a transient model may be questionable. For example, in the Chipuxet aquifer which has been monitored in some detail, good records of streamflow (stream stage at one point) and water levels at one observation well are available. Withdrawal records at the URI well field have been until recently poor and other records are of relatively short duration. (Allen et al 1966).

To understand how steady-state simulations can be used in calibration it is necessary first to look at the physical meaning of the term \( \frac{Sah}{\partial t} \). If the water level at an observation well drops [0.5 feet] in a month and the specific yield at that point is estimated to be \( .10 \) then
\[
S_y \frac{\Delta h}{\Delta t} = -.10 \times \frac{0.5 \times 12}{1/12} = -7.2 \text{ in/yr}
\]
This means that the water table at the end of a month may be considered to result in part from an accretion of this magnitude. Equation 11 may then be written

\[
\frac{T \alpha^2 h + (\bar{W} - S \frac{\Delta h}{\Delta t})}{\alpha x^2} - Q = 0
\]

or

\[
T \frac{\alpha^2 h}{\alpha x^2} + \bar{W} - Q = 0
\]

(12)

where

\[
\bar{W} = W - S \frac{\Delta h}{\Delta t}
\]

(13)

and \(\bar{W}\) is the recharge which will simulate aquifer response at the desired time. This expedient is often used in calibrating aquifer models where measurements of aquifer conditions are known for only a few times and this is the approach we have used.

Another reason for assuming steady-state conditions would be to represent average yearly conditions. If water levels are not declining on a yearly basis, then \(\Delta h = 0\) and equation 11 becomes

\[
T \frac{\alpha^2 h}{\alpha x^2} + W - Q = 0
\]

(14)

where \(W, Q,\) and \(h\) now would represent average yearly values.

Still retaining a one-dimensional model for simplicity model calibration will be explained. Before making predictions, a model
must be calibrated to accurately represent field conditions. Assume the modeler has a water-table map constructed from water level measurements made in late summer, some stream-flow measurements made during the same period representing base flow conditions and is using the simplified model shown in Figure 2. Calibration would involve adjusting transmissivities, and recharge and underflow from the till-bedrock area until the predicted water table map matches the observed water table to any desired degree. Also outflow from the model should, in this case, match the observed baseflow with a recharge rate consistent with known specific yields and rates of water level decline, and an input from the till-bedrock area consistent with independent estimates. The degree of agreement between observed and predicted behavior is an indication of the accuracy with which a model represents the conditions under which it is calibrated.

After the model has been calibrated in steady-state, transmissivities, leakages, and underflows which are not time-dependent are fixed and the model is calibrated in a transient mode by introducing values for storage coefficients and time varying inputs and outputs and checking the model against a period of historical record. By adjusting storage coefficients and possibly underflows a match is obtained after which the model is considered to be calibrated for conditions similar to the period for which the model was calibrated. Verification would involve checking the calibrated model against another period of record. For transient calibration two approaches are normally used. In the first the model is calibrated by testing it against a large-scale pumping test. The pumping must be great enough and of sufficient
duration so that the modeled portion of the aquifer is stressed. Measurements of drawdowns and other relevant parameters must be available over the aquifer since these values are compared with modeled drawdowns. When a large-scale pumping test is not feasible the necessary data can be obtained by careful monitoring of existing large-scale ground water withdrawals.

In cases where a large-scale pumping test is not feasible and where the aquifer is undeveloped the alternative is to compare predicted natural fluctuations with observed natural fluctuations during a period of historical record. When a model is calibrated against natural conditions only, predictions of the effects of future development must be clearly qualified since development conditions will in general not be comparable with the conditions under which the model was calibrated.

When complete calibration and verification are not possible or when a modeler wants to learn more about aquifer behavior, a sensitivity analysis may be performed. A sensitivity analysis could consider the sensitivity of water levels to changes in transmissivity. Results from sensitivity analyses besides giving modelers a better feel for aquifer behavior may also be useful as a guide in decisions such as the need for additional field data and the types of data of most value.

Figure 3 shows an idealized two-dimensional model and Figure 4 some results from sensitivity analyses by Beckman. The model was operated in a transient mode with recharge applied over half the items; the response at a single observation node and the baseflow are shown. By the end of the second year of simulation, the effects of the initial conditions are negligible and the aquifer is
TRANSMISSIVITY = 3200 ft²/day
STREAMBED PERMEABILITY = 1 ft/day
STORAGE COEFFICIENT = 0.20

Figure 3.-- Idealized Two-dimensional Model.
Figure 4.-- Results of Sensitivity Analysis.
essentially in equilibrium with discharge balancing recharge. This condition could be considered representative of average aquifer conditions.

Figure 4 shown results of varying transmissivity and storage coefficient in the idealized two-dimensional model. Heads (water table elevation) are higher at any point in the aquifer as transmissivity is lowered, and the rate of aquifer discharge is greater at higher transmissivities. For storage coefficient, a higher storage coefficient mean slower rates of aquifer depletion.

DESIGN AND CONSTRUCTION OF THE CHIPUXET GROUND-WATER RESERVOIR MODELS

The first step in designing a model is to develop a conceptual model of the ground-water reservoir from available hydrologic information. Next decisions as to the actual area to be modeled are made. This will depend on several factors including the amount and quality of data available and the degree of detail desired. Figure 5 shows the area of the Chipuxet ground-water reservoir modeled, the mesh spacing, and the types of boundary conditions used. A constant mesh spacing of 400 feet was used mainly to take advantage of aerial photographs available at that scale. This scale is adequate for modeling, although, we have used variable mesh spacing in more recent digital models. With variable mesh spacing, areas of primary interest can be modeled with a finer mesh and areas of less interest with a coarser mesh. This may reduce computer storage requirements and execution time since less total nodes are generally involved. For the same number of nodes a more detailed water level picture
would be obtained in areas with a finer mesh. Unfortunately, with finite difference methods, mesh spacing may not be varied arbitrarily without introducing computational problems.

Initial values of transmissivity were estimated from saturated thicknesses and hydraulic conductivities given by Allen, et al. (1966). Transmissivities used in the digital model range from 135 ft.$^2$/day for till areas to 20,100 ft.$^2$/day for stratified-drift areas beneath and south of Thirty Acre Pond. In the analog model a transmissivity of 670 ft.$^2$/day was used in till areas, but proved to be too high. In the analog model the quantity affecting model performance is the rate of seepage from till areas which is proportional to the product of transmissivity and hydraulic gradient. In the analog model the final gradients determined in calibration are much smaller than those observed, compensating for the fact that the transmissivity used is higher than the probable value.

Streambed parameters were initially selected based on published data (Rosenshein et al. 1968; Gonthier et al., 1974). However, these estimates did not prove reasonable and it was necessary to resort to selection by trial and error. Leakage into or out of a stream is computed as

$$Q_s = K_s \left( \frac{h}{s} - \frac{a}{M_s} \right) A_s = K(h_s - h_a)$$  \hspace{1cm} (15)

where $Q_s$ is leakage $h_s$ and $h_a$ are heads in the stream and aquifer respectively, and $A_s$ is the area of the streambed through which infiltration occurs. In reality it is impossible to separate the effects of hydraulic conductivity ($K_s$), area ($A_s$) and
thickness ($M_s$) of the streambed which are effectively lumped together ($K$). Also as indicated earlier, these parameters represent, in part, the increased head loss due to flow line convergence and anisotropy of the aquifer in the vicinity of the stream. Even heads in the aquifer are not well known beneath the streams. The absolute value of predicted water levels away from the stream are dependent on these streambed parameters which are not well defined physically. For example, depending on how flow-path dependent these parameters are, the values could change as a stream changes from gaining to losing in response to pumping.

For transient predictions a value of storage coefficient (specific yield) must be introduced (see equation 3). We used two values in the analog model .10 in till areas and .3 in stratified drift areas. In the digital model we used a value of .15 for both till and stratified drift. These values are based on values given by Allen et al. The storage coefficients affect transient simulations in several ways. First, when a well is pumped the volume of aquifer dewatered will be larger and the cone of pumping influence will expand faster for a lower storage coefficient. Second water levels and streamflows will decrease faster for a lower storage coefficient if all other factors remain constant.

Underflows are flows entering or leaving the modeled area as ground-water flow. Initial estimates of these quantities were made using the transmissivities used in the model and the hydrogeologic map given by Allen, et al (1960). Most underflow enters the model along its western boundary; this underflow was initially estimated to be about 460,000 gal/day which was reduced to 350,000 gal/day during calibration of the analog model.
MODEL CALIBRATION

Models were calibrated in the steady-state mode by matching the observed water table map from late summer 1959 (Allen, et al., 1966) and miscellaneous streamflow data.

The analog model was not equipped with an accretion (recharge) network; instead boundary conditions consisted of heads interpolated from water-level data along boundaries in the till areas and along streams, and estimated underflows. Initial simulations reproduced the general shape of the water table but absolute values of heads were too low. Heads were increased primarily by increasing streambed resistances and lowering boundary heads in the till area.

The different boundary conditions used in the analog and digital models required slightly different transmissivity distributions to calibrate them. However both these transmissivity distributions and the different boundary conditions led to essentially the same trial and error values for streambed hydraulic conductivities. Therefore, a discussion of the influence of the different boundary conditions on calibration will not be included. Geisser (1975) obtained a reasonably good water table match by trial and error adjustment of streambed resistors alone.

After a satisfactory water table match was obtained for the analog model, streambed hydraulic conductivities in the digital model were set equal to those obtained in the analog calibration.

In the digital model, steady-state calibration was slightly different since a uniform recharge rate was applied over the entire model. Except for till areas, streambed properties and transmissivities in the two models are essentially the same. Boundary conditions in the digital model are either constant flow
(underflows) or no flow and constant head along the stream. Again the late summer 1959 water-table map and miscellaneous streamflow data were used for calibration. The final underflows in the digital model were determined to be 1,450,000 gpd versus 300,000 gpd in the analog model. This difference is significant and is the major difference between the two models. Geisser did not discuss it suggesting that the differences in underflows were small. Possible reasons for this difference will be discussed below.

Figure 6 shows the predicted and observed water table. Agreement is fair and could be improved by varying transmissivities and steady-state accretions. However, because of the limited water-table data available, further adjustment of transmissivities and accretions was not warranted.

Several points need clarification since they probably contribute, in part, to some of the discrepancies between measured and modeled water levels. White Horn Brook which appears to be a gaining stream when ground-water levels are high and a losing stream when water levels are low was not included in either model. Chickasheen Brook which appears from the water-table map (Allen, et al., 1966) to be a losing stream, at least as it crosses part of the aquifer in West Kingston, was also not included. With reliable measurements of stream flow loss along the Chickasheen Brook, an estimate of the recharge from this stream to the aquifer could be made and entered in the model.

The underflow along the western model boundary may include part of this recharge. Differences between measured and modeled water tables in the West Kingston area could then be due to the fact that recharge has been input to the modeled area as underflow but
Figure 6.-- Comparison of Observed and Predicted Water Table Map
actually enters the aquifer as stream infiltration. Additional flow data and modeling would be necessary to resolve these questions. Sensitivity testing could be done to determine how sensitive model predictions are to these differences.

An additional check of model performance was to look at the variation of baseflow with accretion rate. To match the steady-state water table, underflows and accretions can be varied recognizing that the accretion rate is related to baseflow or ground-water runoff for a declining water table. Figure 7 shows the variation of predicted water level in the vicinity of SNW6 with accretion rate (solid line) and some average monthly baseflow rates with the corresponding average monthly water table altitudes. Here the steady-state simulation represents an average declining water table condition. Ground water baseflows are those reported by Allen, et al. for summer and fall 1959 and the corresponding water levels are those reported by Allen, et al. Additional runoffs were determined by Zeneski (1975) for the 1974 water year.

The models could be modified to improve the match between predicted and observed values. However, the parameters used in the models were based on the best data available and if the values of interest (heads) are relatively insensitive to minor changes in these parameters, little improvement in model performance would result from further parameter adjustment.

Figure 8 shows the variation of predicted water table altitude at SNW 6 for different ratios of the transmissivities used in the digital model. Variations of 20% have only a minor effect on water levels (less than .2 ft.). Figure 9 shows the variation of water level at SNW6 with model streambed hydraulic conductivity. Water
Figure 7.-- Modeled Ground Water Runoff versus Water Level SNW 6.
Figure 8.-- Sensitivity of Chipuxet Model to Variations in Transmissivity.
Figure 9.-- Sensitivity of Chipuxet Model to Variations in Streambed Hydraulic Conductivity.
levels are about as sensitive to streambed hydraulic conductivities as to aquifer transmissivities.

Finally, we may look at the streambed properties. These were determined by trial and error and because they are derived quantities and cannot be measured directly they need further discussion. In the models, streams are modeled as aquitards of limited areal extent. Kuenenberger (1975) analyzed data from a pumping test at Thirty Acre Pond as a leaky confined aquifer and determined a transmissivity of 26,700 ft.²/day and a leakance (Ks/Ms) of .11 gpd/ft³. In the vicinity of Thirty Acre Pond these values may be compared with the leakance used in the models by Geisser since these are full 400 by 400 foot nodes. Values of model leakance are .62 gpd/ft³ with a corresponding transmissivity of 20,100 ft.²/day. These values are for practical purposes equivalent; for example, at a pumping rate of 650 gpm, steady-state drawdown 150 ft. from the pumping well would be about 1.6 ft. for the higher T, and 1.5 ft. for the lower T. This difference is insignificant when it is noted that Geisser calibrated steady-state behavior against a water-table map contoured in five foot intervals.

Very little transient calibration has been done since stresses imposed on the aquifer have so far been slight. One opportunity for which limited data is available is the pumping which occurred during dewatering for the sewage pumping station along White Horn Brook just south of Rte. 138 (Figure 6). Accurate records of pumpage are not available and as already noted White Horn Brook was not included in the model.

Dewatering started in April, 1976 and was crudely estimated to average 3 mgd. Dewatering continued at this rate for four months.
Predictions were made with the digital model using a uniform specific yield of .15. These predictions indicated only a light drop in water level at well SNW6 and very little drawdown was observed. Perhaps of more interest would be depleted streamflows which would lag dewatering by several months. Additional interpretation would be necessary to see if this effect could be separated from available streamflow and water level records.

Comparisons were made between the analog and digital models by comparing drawdowns due to pumping for several hypothetical schemes. Agreement was excellent.

MODEL SIMULATION OF PUMPING SCHEMES

Five pumping schemes have been proposed by the Rhode Island Water Resources Board for the Chipuxet ground-water reservoir. Proposed schemes I through V are summarized in table 2; the locations of existing and proposed pumping centers are shown in figure 10. To test these proposed pumping schemes two types of simulations were done: steady-state simulation and transient simulation. All simulation results reported, herein, were obtained with the digital model.

Steady-state simulations of low-flow conditions are the basis for comparing pumping schemes at this time. Under nonpumping conditions, the water table is falling and stream flow decreasing during late summer and fall. Therefore, on a monthly basis the water table may be assumed to be in steady-state equilibrium. With pumping the water table is assumed to reach a new equilibrium with average pumping rates. Under pumping conditions some water that
would have become streamflow is diverted to wells, some streamflow is infiltrated, and the equilibrium water table is generally lower than under nonpumping conditions.

Figures 11 and 12 show the predicted steady-state water table for proposed pumping schemes I and V. Figure 13 shows how the predicted stream pickup would vary under steady state conditions for these two proposed pumping schemes.

Results from pumping schemes II, III and IV fall between Scheme V, which concentrates the pumping at Thirty Acre Pond, and Scheme I, which spreads the pumping throughout the basin. Scheme V would result in the greatest absolute lowering of the water table; it would also significantly modify flow patterns and induce large amounts of stream and pond infiltration in the Thirty Acre Pond Area. Figure 13 indicates that there would be little or no flow in the stream between 30 Acre Pond and the gage at Route 138 for Scheme V. Scheme I on the other hand, would result in less ground water lowering over a larger area, would not significantly modify existing ground water flow patterns and would involve more ground water interception and less induced infiltration.

Transient 180-day simulations were run under conditions similar to those used by Allen, et al. for an image well model. Simulation conditions were as follows:

1) the water table was assumed flat and equal to the stream elevation; 2) specific yield was assumed constant and equal to .15; and 3) there was no underflow - boundaries where underflow does occur were assumed to be no-flow boundaries.

Under these conditions there is no flow before pumping starts. Wells were pumped at the average rates shown in table 2. The
Figure 12.--Predicted Steady-state Water Table with Proposed Scheme V.
Figure 13.-- Stream Pickup for Natural Conditions and with Proposed Schemes I and V.
maximum predicted drawdowns at each pumping center after 180 days for each scheme are tabulated in table 3.

The sixth pumping scheme in table 2 is not proposed by the RIWRB; instead it is designed to test the higher withdrawal rates estimated by Allen et al. Results tabulated in table 3 show that even for pumping at these rates drawdowns are not excessive.

This type of transient simulation yields results comparable to image well models but allows a more accurate representation of the aquifer including the actual location of wells, varying transmissivities, storage coefficients and boundary conditions (reeds, changes in stream infiltration) during a simulation period. These simulations accurately predict short term pumping effects.

During a growing system the amount of induced infiltration and aquifer drawdowns increase and these changes can be predicted with this type of transient simulation. The absolute values of streamflow and water levels would depend on the initial conditions which are the conditions at the start of the growing season. A complicating factor is that streamflows and water levels decline naturally during the growing system and these changes as well as changes due to pumping would have to be accounted for in predicting actual late season conditions. Although this can be done, it involves a great deal of judgement.

More realistic transient modeling would involve simulating monthly or weekly water levels with and without pumping. With this type of modeling, conditions at the beginning of the growing season, the effects of pumping and seasonally declining water tables are accounted for in model predictions. Beckman (1978) modified Trescott's (1972) computer program so that monthly water levels
could be simulated. Beckman's modifications were primarily to the input and output portions of the program. Beckman also did transient simulations on proposed Schemes I and V on a monthly basis. The resulting simulated water-table altitudes at observation well SNW6 are shown in Figure 14. Also shown in Figure 14 for comparison are the observed end-of-month water table altitudes at SNW6 for the 1959 water year (Allen et al., 1963). The model simulates only part of the ground water component of the hydrologic cycle so that on an annual basis, stream pick-up in the modeled area must equal aquifer recharge. Beckman used a recharge rate of approximately 14 inches/year which is representative of conditions occurring about once every ten years. Actually, these results are more representative of two successive years with a recharge rate of 14 in/year. It should be noted that Beckman found it necessary to increase some streambed leakances by a factor of two over those determined by Geisser.

Figure 15 compares results of stream pickup between transient simulations and steady-state simulation for pumping scheme I. This comparison suggests that the steady-state analysis is conservative. However, it should be noted that what is simulated with the transient model are average monthly baseflows and daily low flows will be considerably below monthly averages.

Results from continuous transient simulations confirm that for continuous withdrawals at constant rates, an equilibrium condition will be achieved in the aquifer where the reduction in streamflow will be approximately equal to the withdrawal rate. Figure 16 shows this comparison of nonpumping conditions with pumping conditions under the proposed pumping schemes I and V.
Figure 14.—Simulated and Observed Water Levels at Well SNW 6.
Table 2 - Average rates of continuous pumping used in transient 180-day simulation for proposed schemes

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<th>III</th>
<th>IV</th>
<th>V</th>
<th>VI</th>
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<td></td>
<td>E-F</td>
<td>(15,18)</td>
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<td>(15, 8)</td>
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<td>H</td>
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1) See Figure 10 for well location.
2) See Figure 5 for node location.
Table 3 - Drawdowns for proposed pumping schemes after 180-day simulation with continuous pumping

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<th>Node $^2$</th>
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<td>11.36</td>
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<td>(15, 8)</td>
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1) See Figure 10 for well location. Well radius assumed equal to 1/2 foot.
2) See Figure 5 for node location.
3) Drawdown not connected for dewatering, partial penetration, or well loss.
Figure 16.-- Comparison of Natural Conditions and Conditions with Proposed Pumping Schemes.
Conclusions

Results from analysis of the Chipuxet Ground-Water Reservoir with digital and analog computer models indicate that flow in the Chipuxet river will be reduced by an amount approximately equal to the average withdrawal rate and that groundwater levels will be lowered from less than 1 foot near the river to as much as 3-4 feet at points most distance from the stream.

All of the modeled pumping schemes have essentially the same effect on streamflow - a reduction in streamflow of approximately 3 mgd. Under field conditions there will probably be some reduction in evapotranspiration so that the actual reduction in streamflows are expected to be less than 3 mgd. A small difference in the effects on total flow at the gaging station between Schemes I and V will occur because planned withdrawals of .75 mgd at the Liberty Lane site would have little or no effect at the West Kingston gaging station. Scheme V would reduce flow at the gaging station by that additional amount.

Schemes that spread pumping out are able to derive more of their yield from ground-water runoff rather than induced infiltration. This means that ground-water moves directly to wells rather than moving into the stream and then to wells as induced infiltration. The percentage of induced infiltration will increase as the rate of ground-water runoff decreases. For scheme V during periods of low ground-water runoff, approximately 30 percent of the water would be infiltrated from the Chipuxet River in the Thirty Acre Pond area.

Because of the generally high transmissivity of the aquifer and the location of pumping wells near streams, drawdowns are relatively small. Even for scheme V drawdowns are not excessive. In a water emergency, the storage capacity of the aquifer could be utilized and the
full yield of the aquifer, estimated by Allen et al (1966) to be more than 8.6 mgd, could be utilized. Withdrawals of this amount for short periods of time, half a year or less, would have little carry over effect on streamflow.

Further refinements in the model should be made as the sustained yield of the reservoir is more fully developed. Although the best available data and modeling techniques were used in this study, additional data has become available and modeling techniques are continually improving.

References


Geisser, D., 1975, An electric analog and digital computer model of the Chipuxet ground water aquifer, Kingston, Rhode Island: Thesis presented to the University of Rhode Island, in partial fulfillment of the requirements for the Degree of Master of Science.

Kukenberger, R., 1975, Induced infiltration in the vicinity of the University of Rhode Island well field: Thesis presented to the University of Rhode Island, in partial fulfillment of the requirements for the Degree of Master of Science.


ACKNOWLEDGMENTS

The Rhode Island Water Resources Center has worked cooperatively with the Rhode Island Water Resources Board since 1975 in their ground-water modeling program. The Board has provided financial support to the Center for this work. A number of graduate students have been involved in this work and have contributed directly or indirectly to this report. In addition to the student's whose theses are referenced, H. Niles, P. Reiter, and N. Ozbilgin have contributed. Herbert E. Johnston and David Dickerson, U.S. Geological Survey, Providence, Rhode Island, reviewed earlier versions of this report and made many valuable suggestions. The illustrations were prepared by personnel in the U.S. Geological Survey's Providence, Rhode Island office.
APPENDIX A.

Data Input for Steady-State Model
TRANSIENT ANALYSIS OF THE CHIMNEY WLSHER

PROBLEM OPTIONS: ARTESIAN LEAKAGE

INTERMEDIATE MESH:
BASIC LENGTH UNIT IS MULTIPLIED BY 1.00 TO GET FEET.
NUMBER OF UNITS PER INCH = 100.00
CONTOR INTERVAL = 1.00

NUMBER OF PUMPING PERIODS = 1
NUMBER OF NODES IN COLUMN = 13
NUMBER OF NODES IN ROW = 51
PRINTOUT EVERY 160 STEPS
ERROR CRITERIA FOR CLOSURE = 0.0000000E-03
CONSTANT RECHARGE RATE(=T) = 0.1586000E-02
SPECIFIC STORAGE OF CONFINING BED = 0.0
MAXIMUM PERMITTED NUMBER OF ITERATIONS = 100
TERMINATION OF COMPUTATION WHEN MAX(PII(T)-PII(T-1)) = 0.10000000E-02

EVAPOTRANSPIRATION RATE = 0.0
EFFECTIVE DEPTH OF ET = 0.0

PARAMETER MATRICES WERE ASSIGNED THE FOLLOWING INITIAL VALUES:

\[
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\theta &= 0.1 \\
\phi &= 0.002631 \\
\beta &= 0.000000 \times 10^{-10} \\
\gamma &= -1.00 \\
\alpha &= 0.0 \\
\tau &= 0.1544444444 \times 10^{-3} \\
\rho &= 0.0 \\
\kappa &= 0.0 \\
\lambda &= 100.0 \\
\delta &= 400.0 \\
\end{align*}
\]

* THESE VALUES WERE REPLACED BY VARIABLE DATA IN ANY MATRICES PRINTED OUT IN FULL BELOW.

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### Transmissivity Matrix

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### 33 Wells

#### Pumping Rates

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#### Steady State at Time Step

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### Simulation Details

- **Time Step Number:** 3
- **Size of Time Step in Seconds:** 7366734.00
- **Total Simulation Time in Seconds:** 15551994.00
- **Minutes:** 259299.50
- **Hours:** 4322.00
- **Years:** 0.49
- **Duration of Current Pumping Period in Days:** 180.00
- **Years:** 0.50

### Balances

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<td>Evapotranspiration</td>
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<tr>
<td>Constant Head</td>
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<td>Leakage</td>
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### Rates for This Time Step

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<tr>
<td>Pumping</td>
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### Steady State

- **Storage:** 0.00
- **Change in Head for This Time Step:** 0.00
- **Min of the Absolute Value of Head Changes:** 0.00