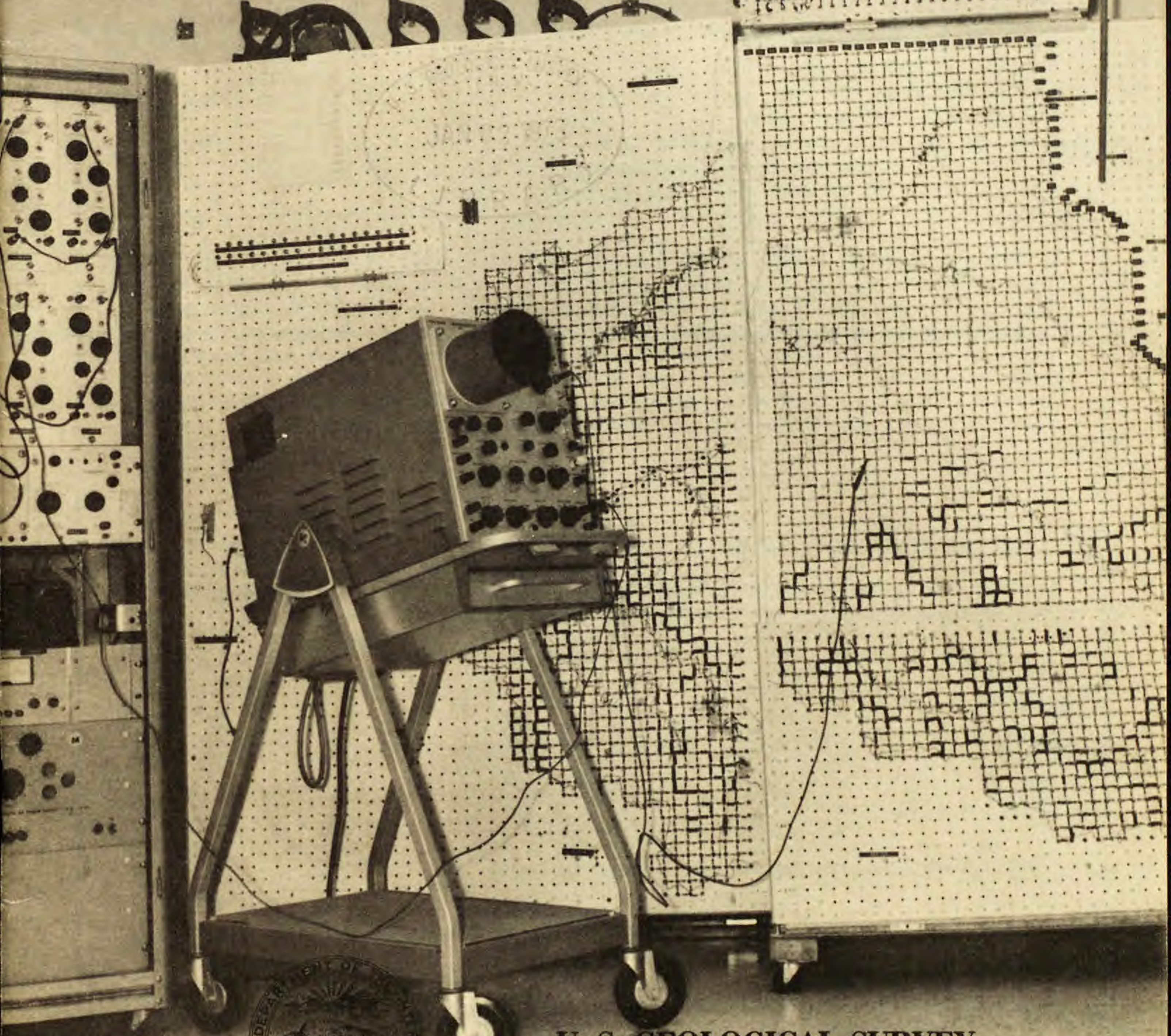
ELECTRIC-ANALOG SIMULATION NETWORK OF UNCONSOLIDATED AQUIFERS IN THE UPPER WABASH RIVER BASIN, INDIANA



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Electric-Analog Simulation Network of Unconsolidated Aquifers in the upper Wabash River basin, Indiana

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ELECTRIC-ANALOG SIMULATION NETWORK OF UNCONSOLIDATED AQUIFERS IN THE UPPER WABASH RIVER BASIN, INDIANA

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ABSTRACT

The ground-water budget of the unconsolidated deposits in the upper Wabash River basin was modeled. An electrical-simulation network was used to determine an integrated storage coefficient of 0.003 for the basin. Two practical problems were investigated: a municipal pumping problem and the change in flow regime due to the addition of surface-water reservoirs to the basin. Because these are demonstration exercises, the information presented is not intended to be used for construction justification. The network is available and can be used to determine the effect on basin hydrology due to local development of the ground-water resource.

INTRODUCTION

This paper describes an electric-analog model built to the specifications set forth in the report by Tate and others (1973), and suggests some applications in analyzing the water budget of the upper White River basin. Information taken from the report were the transmissivity and its areal variation and the base-flow discharge rates from the subbasins that make up the upper Wabash River basin. Of the 18 U.S. Geological Survey stream-gaging stations in the basin, 16 have records long enough to provide an average base-flow discharge. The analog network was divided into 16 subbasins, each with one of the stream-gaging stations at its discharge point.

The work described was done in cooperation with the Indiana Department of Natural Resources, Division of Water.

HYDROLOGY OF THE BASIN

The upper Wabash River basin consists of 3,779 sq mi (square miles in northeastern Indiana (fig. 1), containing three major rivers other than the Wabash itself: the Salamonie, the Mississinewa, and the Eel, having drainage areas of 560, 817, and 815 sq mi, respectively. The headwaters of the Wabash and the Mississinewa Rivers are in an area of Ohio where prospects are poor for the development of large ground-water supplies from unconsolidated deposits.

Precipitation averages 35.0 inches a year over the basin (Tate and others, 1973); precipitation is greatest in the southern part and decreases to the north. Annual precipitation at Wabash, which is centrally located in the basin, ranged from 26.5 inches to 44.2 inches during 1961-70.

The basin is underlain by the preglacial drainage features that formed the Teays system (Tate and others, 1973). This system complicates the ground-water flow, and, as a result, an integrated view of the basin must consider the aquifers that occupy these bedrock valleys.

Ground water usually is obtained from unconsolidated deposits in the basin. Where these deposits are missing or do not contain aquifers, it is necessary to get water from the bedrock. Places where the unconsolidated deposits are not adequate to store and supply water are the extreme eastern part of the basin, including all of the Ohio part, along the southwestern border in the Pipe Creek drainage basin, and in local areas along the major rivers, where the river has cut below the unconsolidated deposits into rock. Only the unconsolidated deposits were modeled for this study.

To construct the model it was necessary to estimate average transmissivity values of the aquifers. Drillers' well logs were used to determine the thickness of the unconsolidated deposits and the amount of sand and gravel in these deposits. Average values for the hydraulic conductivity of sand, sand and gravel, and other water-bearing deposits, were obtained from the literature, guided by the results of an aquifer test in the area (Ferris, 1945). These values were multiplied by the saturated thickness to obtain the transmissivity.

The vertical hydraulic conductivity of the material that forms the river bed was estimated from information obtained from two aquifer tests, neither in the study area. Norris and Fidler (1969, p. 45), report a value of 27 gpd per sq ft (gallons per day per square foot) for sediments in the Scioto River in Ohio. The Scioto receives most of its sediment from a basin similar to the study basin.

Walton and Ackroyd (1966) report an average infiltration rate from the White River near Anderson, Ind., of 42,000 gallons per day per acre. This is in geologic setting similar to and near the study area. The infiltration rate, Ia, is defined as $\frac{6.3 \times 10^7 \; Qr}{Ar}$, where Qr is the amount of induced infiltration, in gallons per minute, and Ar is the area of infiltration in the streambed, in square feet. Logs of wells used during this test indicate from 4 to 10 feet of "clay" at the surface, above the aquifer material. Using an average thickness of 7 feet for this material, 0.2 foot for the average head loss within the streambed area

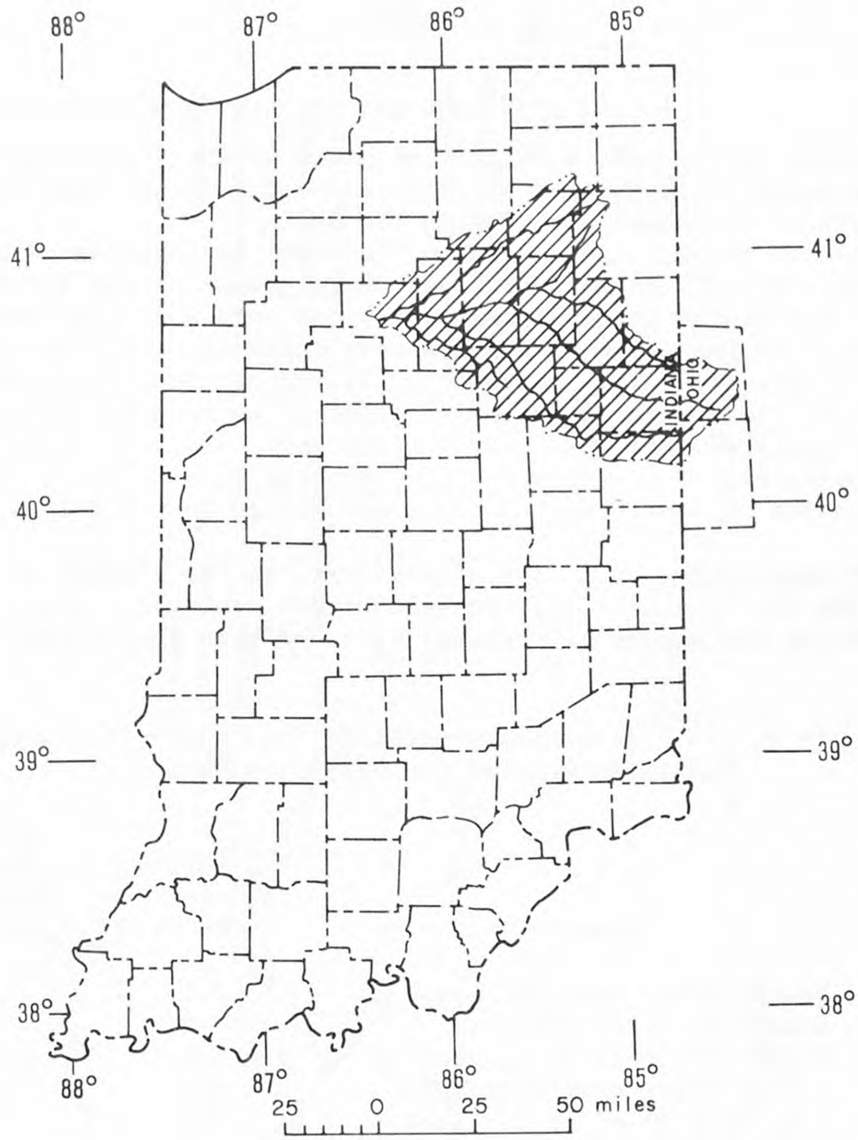

Figure 1.--Location of the study area.

of infiltration, as reported in Walton and Ackroyd's report, and converting to common units, the hydraulic conductivity of the streambed is:

$$P = \frac{Q}{I\,A} = \frac{Ia}{I} = \frac{42{,}000}{\frac{0.2}{7}}$$

$$= 1.47 \times 10^{6} \text{ gpd per acre or } 34 \text{ gpd per sq ft.}$$

This compares fairly well with that obtained in the Scioto basin in Ohio. An average value of 30 gpd per sq ft is assumed for the vertical hydraulic conductivity of streambed deposits in the basin.

A value for the average recharge to the unconsolidated deposits was obtained for each of the 16 subbasins. Hydrographs of the stream discharge at each of the gaging stations were separated into base-flow and flood-flow components. The base-flow components were accumulated and averaged for the period of record to give average yield figures (Tate and others, 1973). These average yield figures were interpreted as recharge to the aquifers. Base flows computed from the hydrographs were used as a foundation for proving the model. The value for the average base flow was computed from the hydrographs at the 16 gaging stations listed in table 1 and located on figure 2.

Evapotranspiration was not considered as an element of the budget because the aquifers were modeled as confined aquifers. Also, the method of determining the amount of recharge excluded this budget factor.

Table 1.--Stream-gaging stations from which data were analyzed to determine base-flow discharge

Number	Name	Subbasin drainage area (sq mi)	Average base-flow discharge (gallons per day per square mile)
03322500	Wabash River near New Corydon	262	111,000
03323000	Wabash River at Bluffton	270	54,000
03323500	Wabash River at Huntington	189	190,000
03324000	Little River near Huntington	263	167,000
03324200	Salamonie River at Portland	86	76,000
03324300	Salamonie River near Warren	339	105,000
03324500	Salamonie River at Dora	132	102,000
03325000	Wabash River at Wabash	227	170,000
03325500	Mississinewa River near Ridgeville	133	113,000
03326000	Mississinewa River near Eaton	177	113,000
03326500	Mississinewa River at Marion	372	161,000
03327000	Mississinewa River at Peoria	126	254,000
03327500	Wabash River at Peru	110	436,000
03328000	Eel River at North Manchester	417	252,000
03328500	Eel River near Logansport	372	242,000
03329000	Wabash River at Logansport	304	280,000
		basin average =	175,000

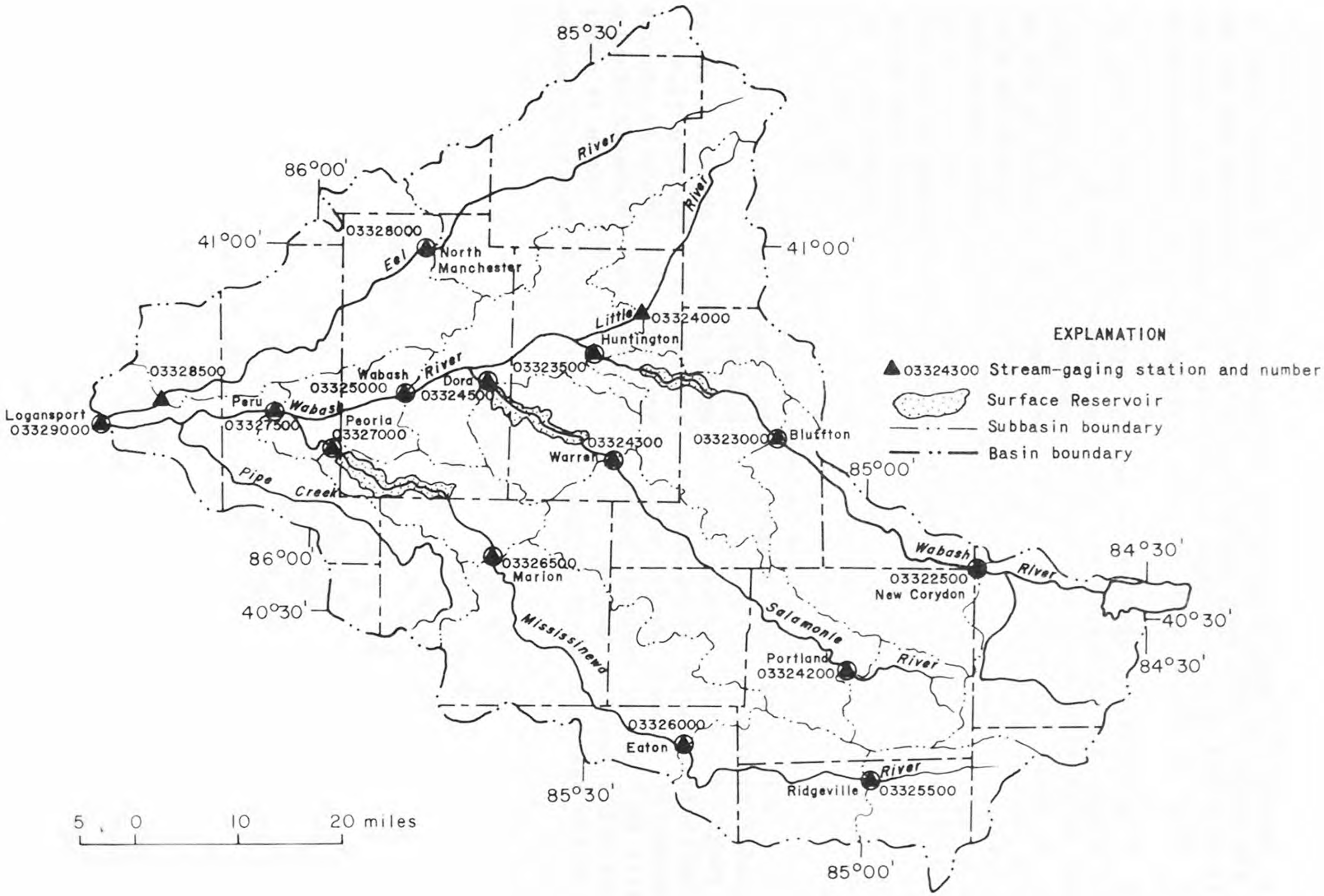

Figure 2--Location of stream-gaging stations and reservoirs.

WELLS IN THE BASIN

Water levels in many observation wells have been measured over the years in the basin. Some were monitored for specific problems and may have been wells that had been abandoned and replaced by another well. Many were used for only a short time and then dropped from the observation-well network.

Table 2 is a list of observation wells, both inactive and active, that are in the basin; the locations are shown on figures 7 and 9.

Several wells that have been used as observation wells in Miami County (Mi-3, 4, 6 thru 10) are not shown because they are all on the same property in the vicinity of Mi-2 and 5. Records of these wells are of short duration. They are rock wells and show similar water-level fluctuations as those around them.

As an indication of the relationship between water levels in the drift and in the rock, Blackford-1 (Bf-1), a drift well, fluctuated from about 1 foot to about 10 feet below the land surface, while the water level in a well in the rock in the same section was measured on April 21, 1962 at 60 feet below that of the water in the observation well. Both of these wells are near the topographic divide between the Mississinewa and Salamonie drainages. This condition suggests that water is moving from the unconsolidated aquifer into the rock in this area.

The range of water levels in these wells supports the value of the storage coefficient chosen in an exercise described in a later section.

The amount of ground water being pumped in the State is reported periodically in "Data on Indiana public water supplies", Bulletin SE-10 of the Indiana State Board of Health. The latest issue (1968) lists 34 public supplies in the basin, of which 31 are ground-water supplies.

Table 2.--Observation wells in the upper Wabash River basin

County	Designation	Period (years)	Depth to water (feet below land surface) high	Depth to water (feet below land surface) low	Formation*
Blackford	Bf-1	1945-66	0.7	9	U
Delaware	Dw-3	1946-49	6	13	R
Grant	Gt-1	1945-47	65	99	U
Grant	Gt-2	1945-50	58	86	U
Grant	Gt-5	1945-50	44	62	U
Grant	Gt-6	1945-48	6	17	U
Grant	Gt-7	1950-54	48	76 (dry)	U
Grant	Gt-9	1970-	26	53	U
Huntington	Hu-1	1945-62	0	41	U
Jay	Jy-1	one reading 1946	4		U
Jay	Jy-2	1950-52	31	35	R
Miami	Mi-1	1948-51	40	48	R
Miami	Mi-2	1957-71	19	54	R
Miami	Mi-5	1960-66	18	31	R
Miami	Mi-11	1966-67	70	115	R
Noble	No-2	1935-44	6	26	U
Wabash	Wb-1	1939-40	29	33	U
Wabash	Wb-2	1947-51	1	14	U
Wells	Wl-4	1967-71	19	23	R
Whitley	Wy-3	1966-71	50	52	U

*U = unconsolidated, R = rock

Theory and Design

The electric-analog model of the ground-water reservoir of the upper Wabash River basin consists of a network of resistors and capacitors representing the saturated, unconsolidated deposits as a unit at a scale of 1 inch to 1 mile. The model was designed and constructed so that simulated outflow from subbasins within the upper Wabash River basin could be measured and that recharge distribution could be varied within the subbasins.

The analog model is based on the analogy of the laws governing the flow of water through a saturated permeable zone and the laws governing the flow of electricity through a conductive medium. The theory and application of analog models for studying ground-water problems has been thoroughly discussed by Skibitzke and da Costa (1962), Walton (1964), and Winslow and Nuzman (1966). Because the ground-water reservoir is a continuous medium and the network consists of discrete elements, it only approximates the material medium. However, if the mesh size of the network is sufficiently small in comparison to the areal extent of the ground-water reservoir, the response of the network to excitation closely resembles the responses of the ground-water reservoir to any modeled stress, such as pumping, discharge and recharge.

The analogy between the electrical system and the aquifer system is indicated by the one-to-one correspondence between elements of the equation for electrical flow through a medium which conducts and stores charge and of equations for steady or nonsteady flow of fluid through the aquifer system. The analogous elements are: electrical current (amperes) and volume rate of flow (million gallons per day); potential (volts) in the model and potential (feet of head) in the hydrologic system; time (seconds) in the model and time (days) in the field; length (inches) in the model and length (miles) in the field.

By proper scaling of the elements, it is possible to construct an analog having complex boundaries that will respond to applied stress in a manner analogous to that of the ground-water reservoir. The model was constructed using the following initial scale factors:

Scale factor	Value	Units
K	10	ft/v (feet/volt)
K	5 x 10	gpd per amp (gallons per day per ampere)
K	3.2 x 10	days per sec (days of real time per second of computing time)

K was later modified to 8.1 x 10 days per sec, when it was determined from the model that the assumed storage coefficient was too low.

A grid was laid out on pegboard with resistors placed between adjacent holes to simulate the transmissivity of the aquifer. The scale chosen was 1 inch to 1 mile, which corresponds to 1 resistor to 1 mile. The resistors were selected for areas of the model corresponding to the transmissivity of each area, as shown on the transmissivity map in Tate and others (1973). Their values were determined by the following relationships:

$$R = \frac{K_I}{K_V T} \frac{\Delta X}{\Delta Y}, \qquad (1)$$

where R is the resistance, T is the transmissivity for that area, K_I and K_V are the transformation and constants, and ΔX and ΔY are the longitudinal and transverse dimensions of the area represented. Values of T ranged from less than 25,000 gpd per ft (gallons per day per foot) to more than 300,000 gpd per ft; consequently, values of R ranged from 27,000 ohms to 1,500 ohms.

Discharge to the streams is affected by the connection between the stream and the aquifer. Two measures were taken to place the discharge areas (rivers) into the working part of the model. The normal network spacing of 1 resistor to 1 mile was made finer to accommodate the position of the discharge point. Where a river crossed a grid element, that resistor was replaced by two resistors in series. This created an additional node, which was then used as the discharge point. The new resistors were chosen in the following manner:

If formula (1) is the formula for an element in the mesh and the river would cross the longitudinal element (ΔX) in the center so that ΔX were divided by the river into two equal parts, then the two resistors that would replace the normal element would be:

$$R' = \frac{K_I}{K_V T} \frac{\frac{1}{2}\Delta X}{\Delta Y},$$

$$\text{or } R' = \tfrac{1}{2} R.$$

To keep this process as simple as possible, the division of these elements was limited to individual resistances of one-fourth, one-half, and three-fourths of the full grid-element resistor.

The other measure used to model the river was to select and install a resistance to simulate the partial penetration and the limiting effect of the riverbed material.

Following a method described by Bedinger and others (1970), these resistors were chosen:

$$R_c = R_n + R_f$$

where R is the river-connecting resistor, R is the part of the resistor accounting for partial penetration, and R is the part of the resistor accounting for the fine-grained riverbed materials. For the upper Wabash mode, resistance R is determined by:

$$R_n = \frac{K_I}{K_V w P'} \frac{m}{2L},$$

where K_I, K_V have been previously defined, w is the width of the stream, P is average vertical permeability, m is the depth of the unpenetrated part of the aquifer, and L is the length of the reach simulated. For this purpose, it is assumed that the vertical permeability is uniform for the full depth of the unpenetrated part of the aquifer, m. The part of the river resistor that simulates the riverbed material, R_f, is determined by:

$$R_f = \frac{K_I \quad t}{K_V wP''L} ,$$

where R_f, K_I, K_V, w, and L are as previously defined, t is the thickness, and P'' is the vertical hydraulic conductivity of the bed material. P'' is taken as 30 gpd per ft, as previously discussed.

Information obtained from the separation of hydrographs indicate that there are three general areas in the basin where aquifer yields are significantly higher than the basin average: along the Wabash River from Wabash to Logansport, the Mississinewa River from Marion to the mouth, and along the Eel River.

Steady-state analysis

To determine if the values of transmissivity modeled adequately represented conditions in the basin, a steady-state analysis was made. As a basis for the steady-state analysis, it was necessary to make the following assumptions:

1. All the water considered in the budget moved only in the unconsolidated aquifers, which are interconnected and considered as one continuous formation.

2. All recharge is derived from vertical leakage through the confining till.

3. The ground-water divides coincide with the topographic divides.

4. The streams are partially penetrating and are, for the most part, separated from the aquifers by fine-grained fluvial material that is derived from the basin itself and are the only means of discharge.

5. Each square mile in a subbasin receives an equal amount of recharge.

6. Recharge is constant at all times.

7. Pumping, where present, is constant and continuous.

8. Storage does not affect the budget.

9. There is no change in water level with time.

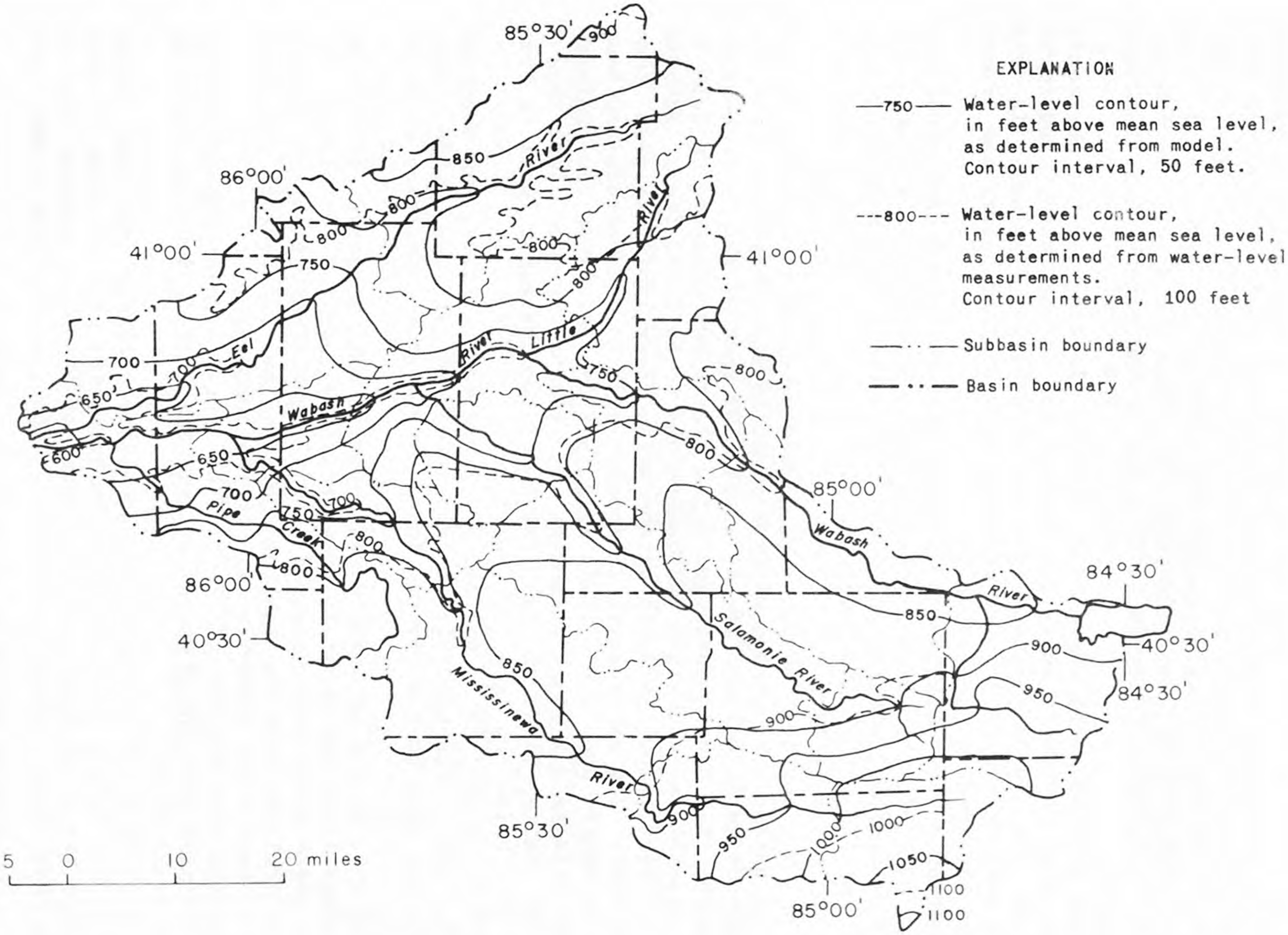

Figure 3.--Steady-State Analysis--simulated water levels obtained using average recharge values.

The first steady-state analysis of the network was an attempt to simulate ground-water levels based on work done by Tate and others (1973). An average ground-water discharge was determined by hydrograph separation for each of the subbasins. These values, listed in table 1, were fed into the modeled subbasins through each node (one per square mile), the nodes being isolated from the discharge sources by diodes. If assumption 3 were true and the transmissivity modeled correctly, then the water levels would be correctly simulated. Figure 3 is a map of the water levels obtained from the network.

There were several areas where the voltage levels did not correctly simulate the water levels or where the current fed to the network had to be adjusted from the value computed from the discharge figures. In the southeastern part of the basin more recharge was required before the proper discharge was obtained at Ridgeville.

The elevation of the Mississinewa River is about 1,000 feet above sea level in this part of the basin, whereas the elevation of the adjacent Salamonie River is about 900 feet. To the north of the Salamonie drainage, the elevation of the Wabash River is about 830 feet. Figure 3 indicates that the water levels reflect this topographic gradient in places.

Because of this apparent movement of ground water to the north, it was thought by the author that ground water was moving into this basin from the south. A bedrock well (W-1 on fig. 4) within 500 feet of the surface-water divide between the Mississinewa and White Rivers was measured on July 5, 1965. The water-level elevation was determined to be 1,075 feet. This is more than 10 feet below the level of the White River, which is less than half a mile away. The water level in another well in the bedrock had been measured on December 20, 1960. This well (W-2 on fig. 4) on the drainage divide is approximately 2 miles northeast of Winchester and less than a mile from the White River at its closest point. The water level in this well was 12 feet below the level of the river.

Another bedrock well (W-3 on fig. 4), close to the river, was measured February 16, 1968. The water-level elevation in it was 1,088 feet, approximately the same as the river, indicating a good connection with the river.

One well, W-4, in the unconsolidated deposits was measured in July of 1967. The water level was about 10 feet above the level of the river.

Although these wells were measured at different times, it seems likely that water is draining from the shallow aquifers into the bedrock in this area, which slopes to the north similar to the topography.

These observations support the conclusion that significant amounts of water are draining from the White River into the bedrock and then into the upper Wabash River basin.

Another problem noted during the steady-state analysis was the large discharge from the Wabash River at the Peru subbasin. The location of this subbasin within the total basin and the amount of discharge suggest that it is receiving flow from adjacent subbasins. Thus, assumptions 2 and 3 are invalid.

Discharge areas on the network were simulated along the larger rivers. In many cases the simulated ground-water divide did not approximate the topographic divide. Where this occurred, additional drains were put into the network to divert water back into the proper subbasin. The addition of these drains was predicated on the existence of drains (streams) in the area and on their discharge. The degree of connection between the smaller streams and the aquifers is uncertain, but in many places it can be inferred from water levels. In other places these smaller streams have no effect on the regional water levels.

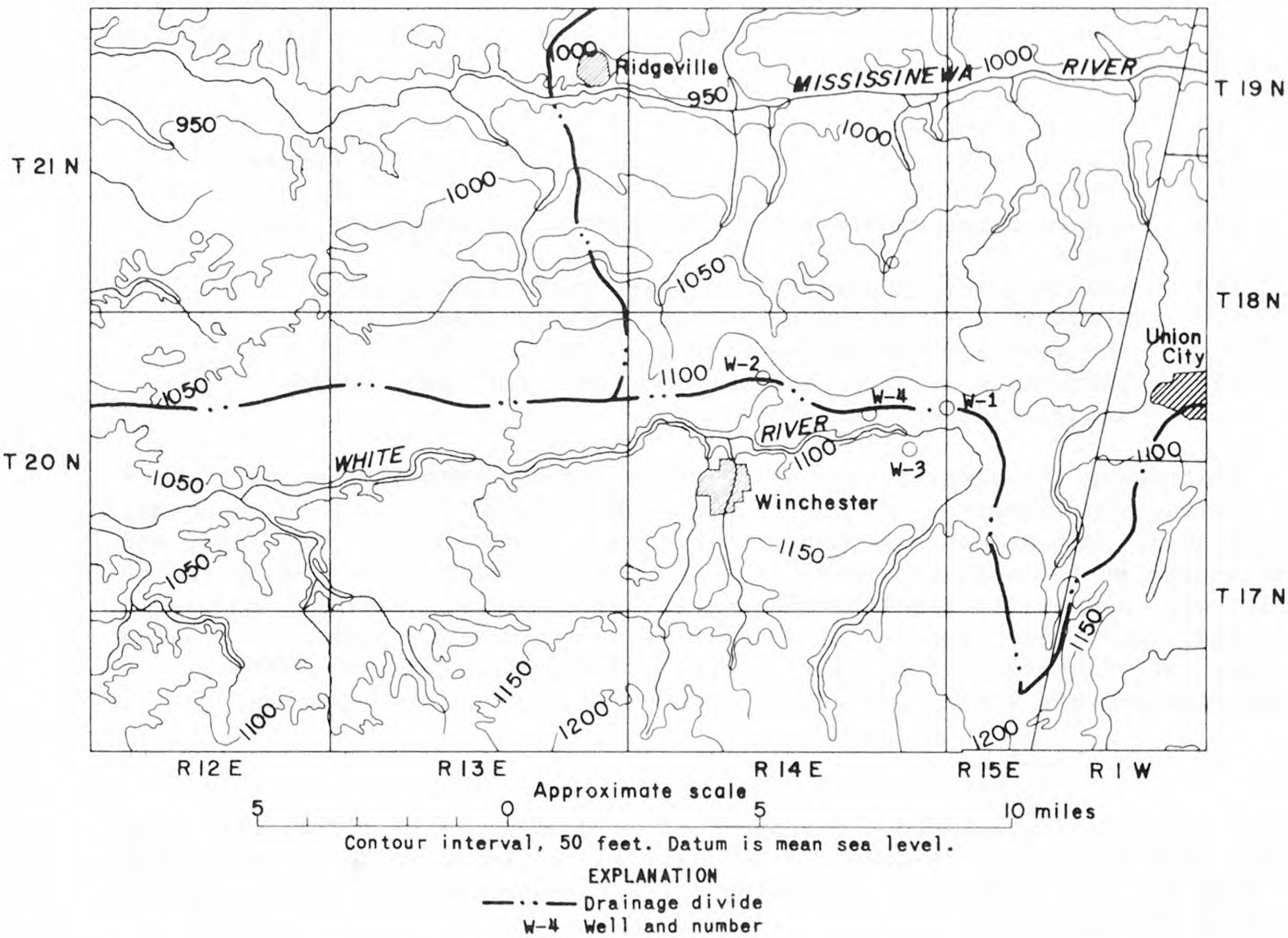

Figure 4.--Location of wells on divide between the Wabash and White River drainages near Winchester

Nonsteady-state analysis

Because recharge to the aquifer varies with time, the steady-state analysis does not completely describe the budget in the basin, nor does it allow for evaluation of the storage coefficient.

The extent of the water-level changes helps determine the amount of recharge that the aquifer can accept. When there is a short time between recharge events, more recharge will occur near the discharge ways or streams because the greatest water-level changes will have taken place there. As the recharge events are strung out in time, the water levels away from the streams will decline to a greater extent, and these areas will be able to accept more of the recharge.

Nonsteady-state analyses are difficult. Even with the aid of computers, certain assumptions must be made. In addition to assumptions 1 to 5, previously stated, the following assumptions were made for the nonsteady state:

(1) The aquifers are confined.
(2) One value is sufficient to define storage for the entire basin.
(3) The recharge events occur simultaneously throughout the basin.
(4) The recharge intensity is a fixed ratio from event to event in each of the subbasins, and recharge enters the aquifers instantaneously upon application.
(5) There is no surface storage involved, such as contributing lakes or channel storage.

Evaluating the distribution of recharge and determining the coefficient of storage are examples of the problems that can be analyzed by nonsteady techniques. Because of equipment limitations, availability of data, and the necessity to facilitate analysis, a period of 81 days in the spring of 1934 was chosen to simulate changes in ground-water levels, recharge, storage, and discharge. The base-flow hydrograph was used to provide values of recharge, following a method described by Butler (1957). A base-flow storage rating was established by evaluating the equation:

$$Q = \frac{K_1}{10^{t}K_2} ,$$

where Q is the instantaneous discharge, $K_1 = Q$ when $t + 0$, t is the time since peak Q, and $K_2 = t$ when $Q = 0.1\ K_1$. As can be seen, K_2 is the number of days for the base-flow recession curve to cover one log cycle. Butler derived the equation:

$$\frac{\Delta Q}{\Delta S} = \frac{2.3}{K_2} \text{, from which}$$

$$\Delta S = \frac{\Delta Q K_2}{2.3} \text{, where S is the change in storage.}$$

Table 3 lists the events, changes in discharge, and changes in storage for each event, as computed by this method. In the base-flow hydrograph (fig. 5), the value of K was chosen as 65 days.

To simulate these events, each change in storage volume is interpretated as the amount of recharge associated with a given event. This quantity of recharge is divided by a time interval representing the period of recharge, to obtain a recharge rate. The recharge periods were determined using National Weather Service records. The times used to simulate the recharge events are listed in table 3. The values of S on table 3 are considered as recharge volumes for the entire basin. The analogous electrical inputs were distributed to the subbasins on a per square mile basis.

Table 3.--Values for recharge determined by Butler method

Event	Q_1 (cfs)	Q_2 (cfs)	ΔQ (cfs)	ΔS (cfs-days)	T (days)
1	480	1120	640	17,700	5-7
2	800	1500	700	19,300	19-20
3	1200	2250	1050	29,000	29-31
4	1300	1350	50	1,380	43-44
5	580	620	40	1,110	70-72
			Total	68,490	

Q = discharge at beginning of recharge period
Q = discharge at end of recharge period
ΔQ = change in discharge
ΔQ = change in storage (interpreted as amount of recharge for the event)
T = days on which recharge occurred, measured from the beginning of the total period (total period = 81 days)

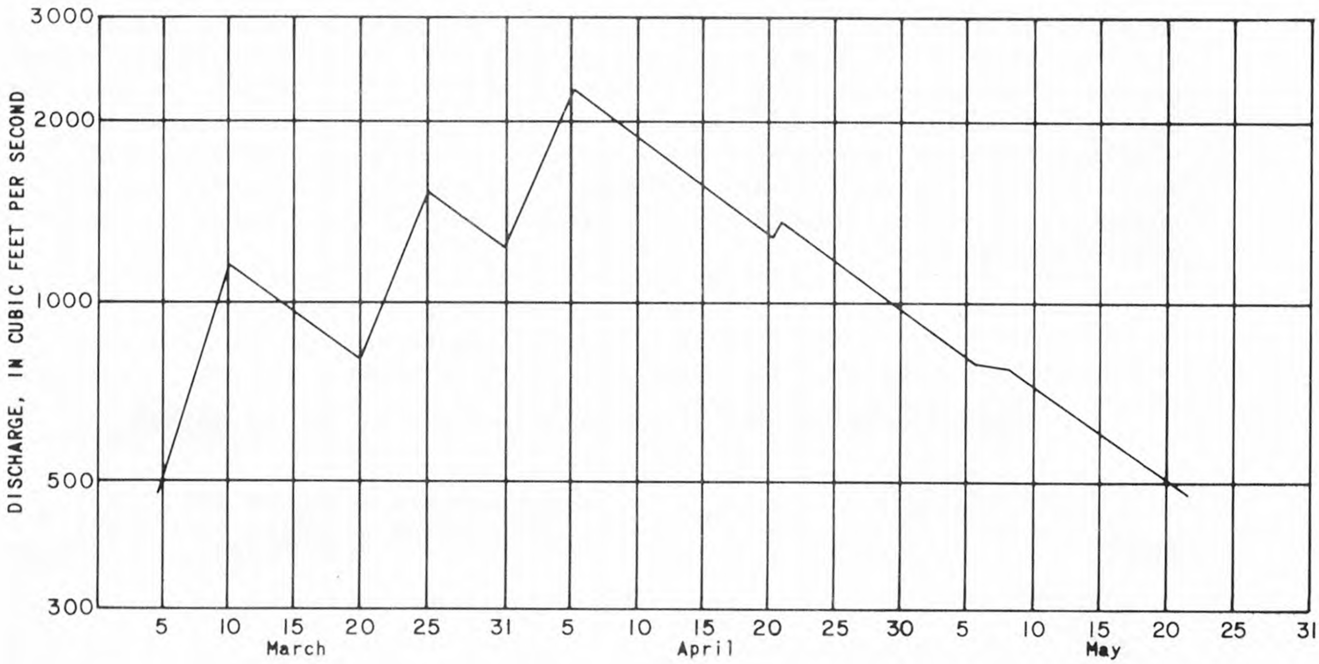

Figure 5.--Base-flow hydrograph, spring 1934, Wabash River at Logansport.

Under these conditions for recharge, the model was probed to determine the change in voltage level in the network. These measurements are important to determine if the storage is correctly simulated and, also, to determine where the recharge is being applied. Figure 6 is the hydrograph (redrawn to eliminate the logarithmic scale) and a simulated hydrograph, using a scale factor of $K_t = 3.2 \times 10^4$ days per sec. Figure 7 is a map of the basin, with simulated changes in water level with the applied recharge and scaling factor as above. The storage for this K is S = 0.001. Figures 8 and 9 are the same figures as the previous two but with a scaling factor of $K_t = 8.1 \times 10^4$ days per sec. The storage for this K_t is S = 0.003.

The water-level changes on figure 7 are greater than naturally occurring changes; they do not fit into the envelope of water-level changes observed in the wells in the unconsolidated formation listed in table 2. This fact suggests that the simulated value of storage coefficient, 0.001, was not representative of the ground-water system and was too low. To

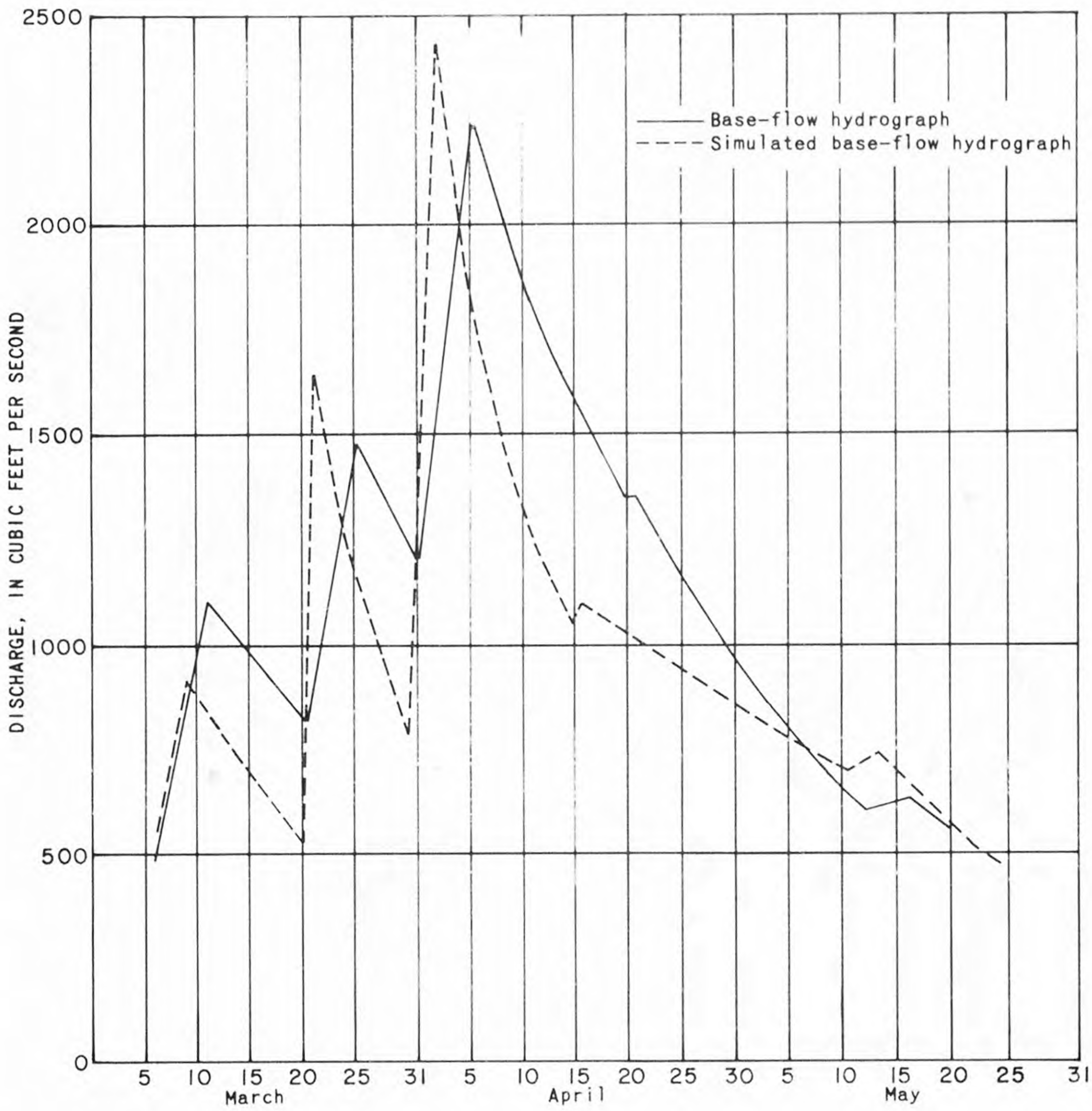

Figure 6.--Base-flow hydrograph and simulated base-flow hydrograph for spring 1934 (K_t = 3.2 x 10^4 days per sec., S = 0.001)

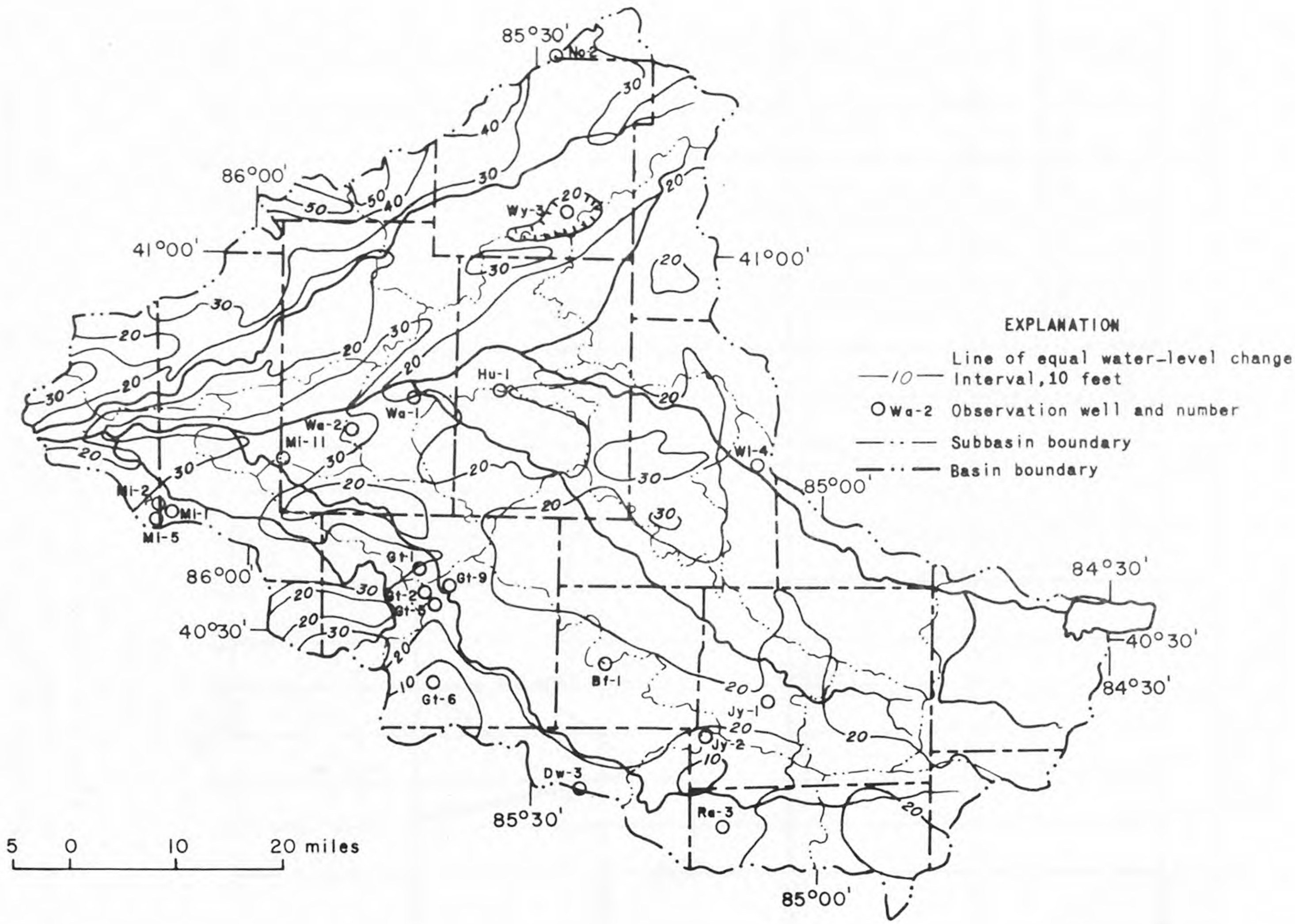

Figure 7.--Simulated changes in water level during recharge events of spring 1934 (K_t = 3.2 x 10^4 days per sec., S = 0.001)

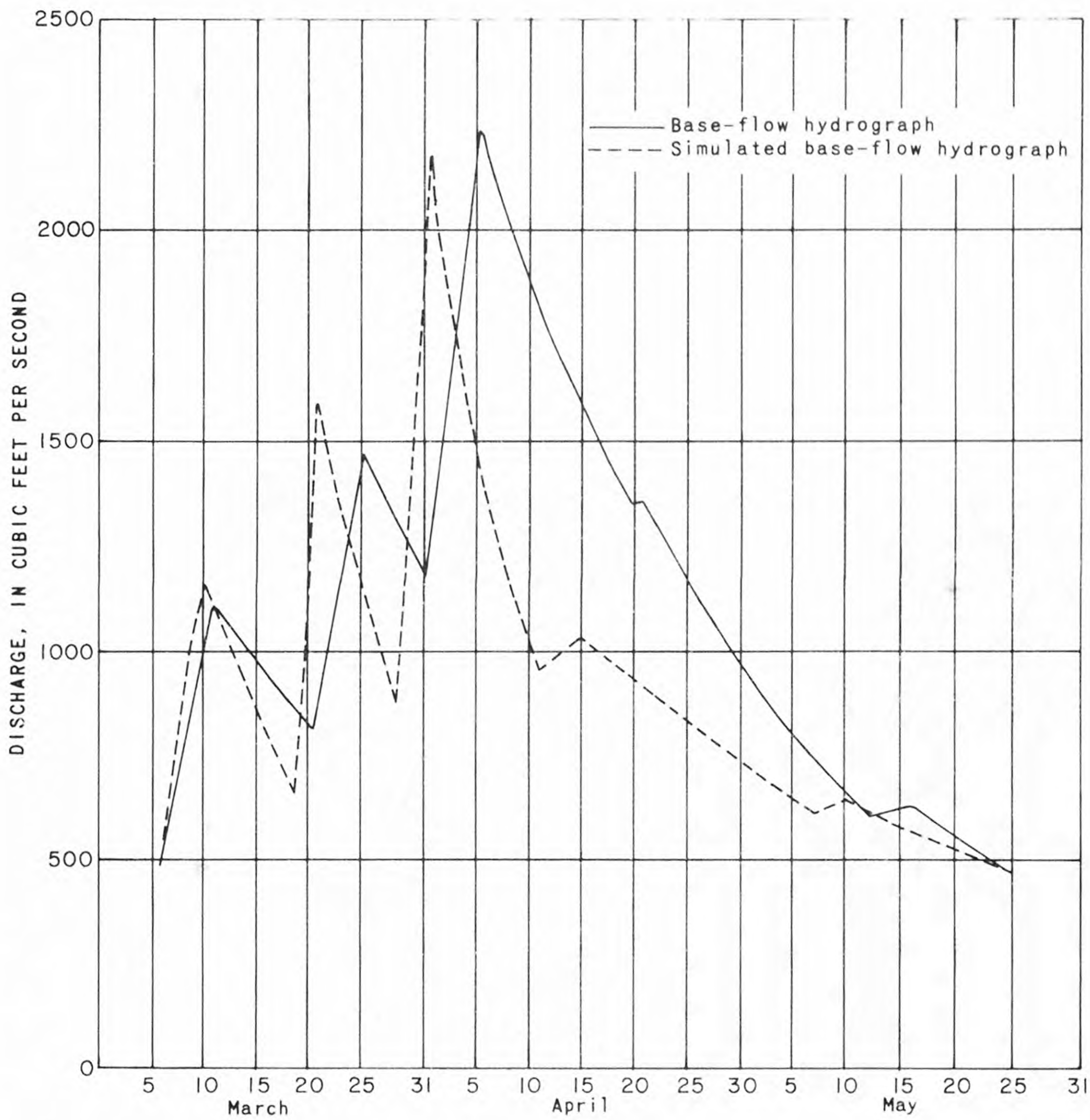

Figure 8.--Base-flow hydrograph and simulated base-flow hydrograph for spring 1934 (K_t = 8.1 x 10^4 days per sec., S = 0.003)

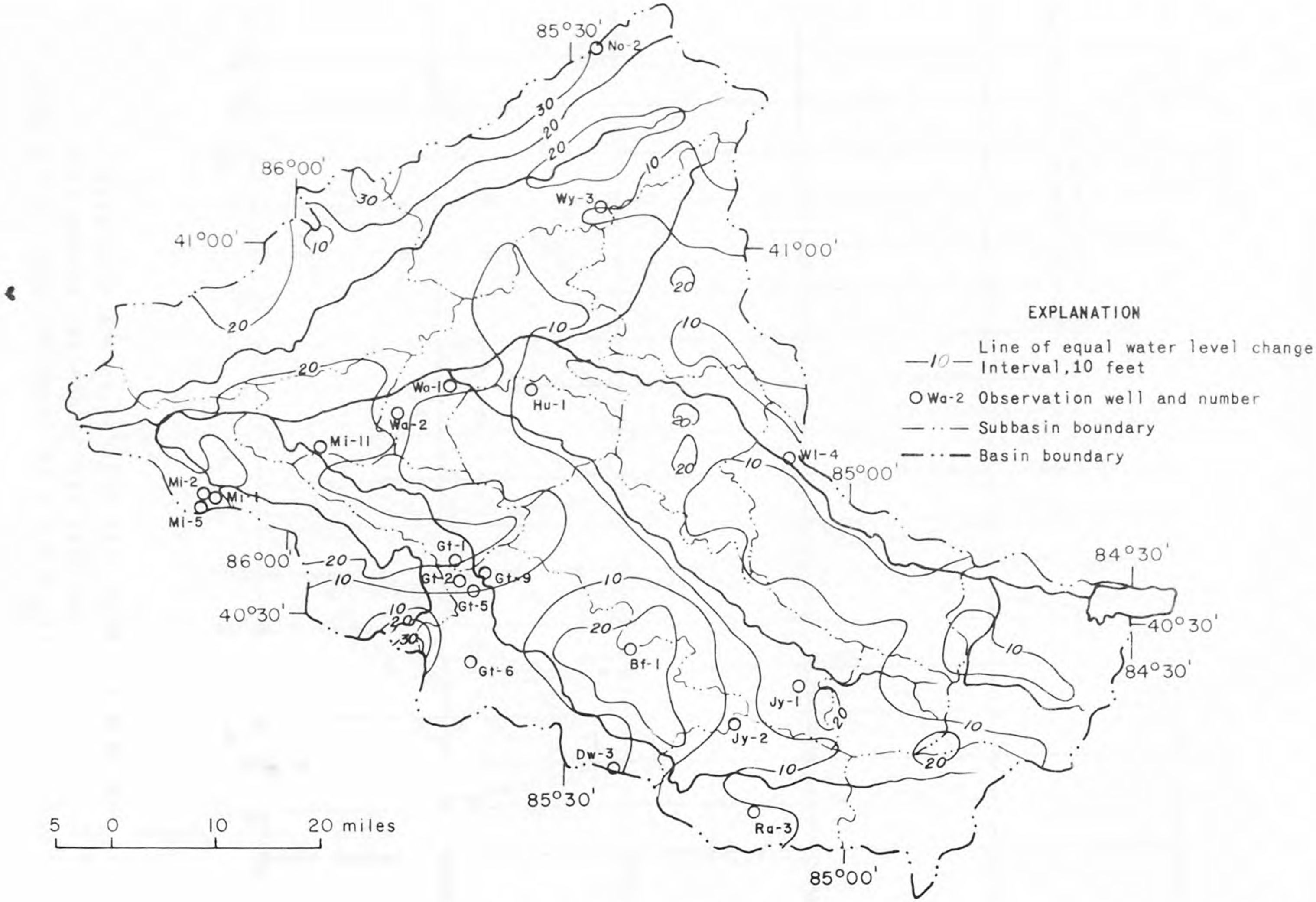

Figure 9.--Simulated changes in water level during recharge events of spring 1934
(K_t = 8.1 x 10^4 days per sec., S = 0.003)

reduce the water-level changes, it was necessary to change the simulated storage coefficient by changing the scale factor relating real time to simulation time. The simulated water-level changes seemed more reasonable when the scale factor $K_2 = 8.1 \times 10^4$ days per sec (fig. 9), corresponding to a storage coefficient of 0.003, was used.

The simulated and separated hydrographs do not coincide for several reasons. The time of application of the simulated recharge was determined by climatic records of the National Weather Service, whereas the rising part of the separated hydrograph was determined by streamflow records only. Recharge was not simultaneous throughout the basin, yet it was necessary to simulate recharge as if it were. No provision was made in the network for surface storage, such as stream-channel storage or lakes. The areal distribution of the recharge was significantly different for the different events; this effect was not modeled.

The recession part of the simulated base-flow hydrograph is steeper at the higher flows, but flattens out at lower discharges, indicating that near the discharge points (streams) storage is not adequate. This may be partly due to areas along the lower Wabash and Eel Rivers of restricted flood plains and water-table conditions. These restrictions were not considered in the network because of the scale of the grid. Also, the information used to provide the connection between the aquifer and the discharge point was based on tests outside of the basin and may be in error.

Demonstration

Two nonsteady-state problems were investigated to demonstrate the use of the network. It was not within the scope of this study to investigate the local aspects of individual problems but only to report on the availability of the model and the kind of problems that can be handled.

The first problem was to determine the effect of pumping 10 mgd (million gallons per day) on the ground-water levels at Marion and on the streamflow. Pumping was within a quarter of a mile of the Mississinewa River. The amount of water taken from the stream was determined by measuring the voltage change, V, across the resistors of known value, R, that simulated the river connection. The current was determined by Ohms law, $I = \frac{V}{R}$. The amount of water taken from the stream was half the total amount pumped, based on 1 year of pumping. The changes in water level are shown on figure 10. This problem is considered semiquantitatively. Pumping is simulated on an analog model by withdrawing current at a rate specified by the amount of pumping and the scale factor relating electrical current to discharge. The simulated pumping in this problem was approximately 480 feet east of the Mississinewa River. The transmissivity in the area is between 100,000 gpd per ft (gallons per day per foot) and 150,000 gpd per ft.

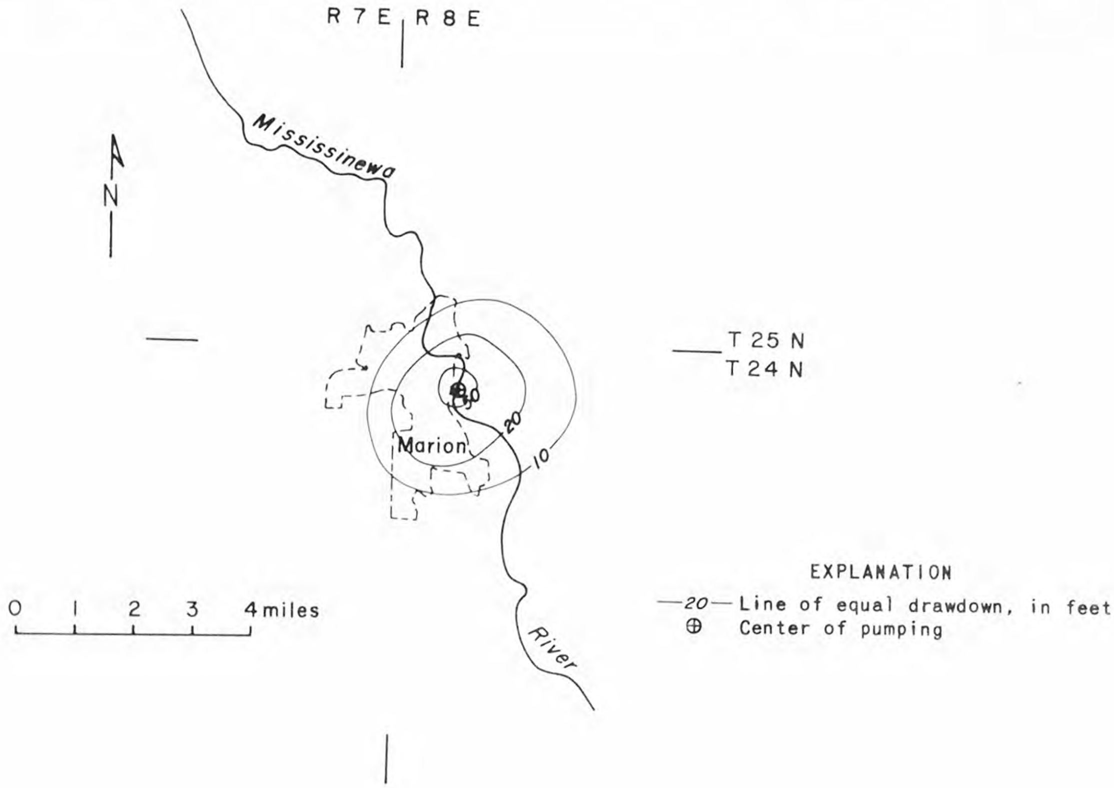

Figure 10.--Water-level changes with pumping 10 mgd at Marion, Ind. (Problem 1.)

The connection between the river and the aquifer is simulated at Ia = 213,000 gpd per acre. With an average water-level decline of 25 feet below the river in this area, the filtration rate per foot of drawdown is 8,500 gpd per acre per ft, which is comparatively low. If half the resistance used to connect the simulated stream to the network is considered as simulating partial penetration and half is considered to simulate the fine-grained materials, then:

P' = 89 gpd per sq ft (vertical hydraulic conductivity in aquifer)
and P" = 22 gpd per sq ft (vertical hydraulic conductivity in fine-grained riverbed material)

The second problem, one that is not considered quantitatively but is merely a demonstration, is the storage effect on high and low flows due to the three reservoirs in the basin. (See fig. 2 for the location of the reservoirs.) Figure 11 shows two hydrographs--one with none of the reservoirs active and one with all active. The storage utilized is approximately half the normal pool capacity, and water is released at a decreasing rate.

In the problem, the recharge is applied to each case identically and is a representation of a year's time. The base of the hydrograph with storage is approximately 175 mgd more than that without storage, an increase in low flow of 271 cubic feet per second. This problem cannot be used for planning because it is not based on a reasonable release program. A program can be applied, however, with minor circuit changes.

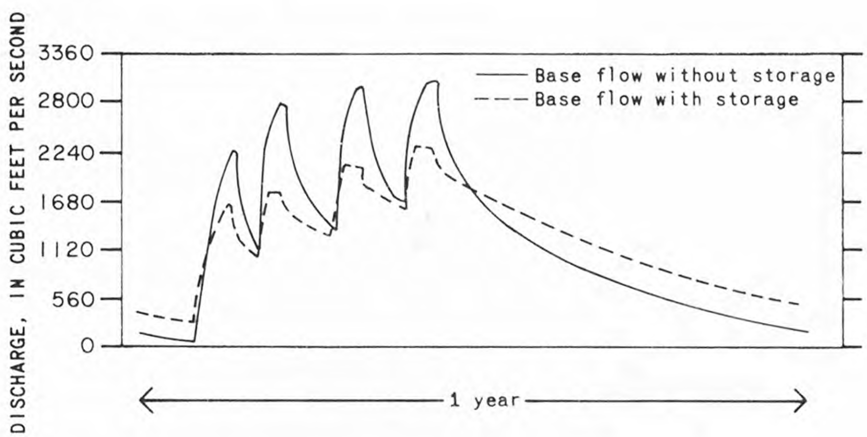

Figure 11.--Typical hydrographs with and without reservoir storage.

CONCLUSIONS

Model and hydrograph analyses of observed flows and water-level changes in observation wells lead to the conclusion that an overall storage coefficient for the basin approximates 0.003. This figure is based on a test of the basin as a whole and may vary among the subbasins. Subbasin tests, however, are possible.

The average base flows do not support the assumption that the ground-water and topographic divides are coincidental. Water levels support the contention that a significant amount of water is draining across the divide into the Mississinewa River basin from the White River basin as well as across some of the lesser divides. The general ground-water flow is northward in the basin.

The analog model at Marion indicates that withdrawing 10 mgd near the Mississinewa River will result in water-level declines greater than 10 feet below prepumping levels in an area of approximately 10 sq mi. Under these conditions, approximately half the pumped amount will be taken from the stream after a short period, during which the cone of influence develops.

When and if it is determined that storage facilities (reservoirs) are needed in the basin, the model can be used to determine the effects of the planned storage on hydrographs at any point, as they relate to base flows, and also on ground-water levels.

The model is available to solve problems similar to the above as long as the water is moving in the unconsolidated formations. Additional hardware would have to be developed to be able to include water moving in the bedrock aquifers.

SELECTED REFERENCES

Bedinger, M. S., Reed, J. E., Wells, C. J., and Swafford, B. F., 1970, Methods and applications of electrical simulation in ground-water studies in the lower Arkansas and Verdigris River valleys, Arkansas and Oklahoma: U.S. Geol. Survey Water-Supply Paper 1971, 71 p.

Butler, S. S., 1957, Engineering hydrology: Englewood Cliffs, N. J., Prentice-Hall, 356 p.

Ferris, J. G., 1945, Memorandum concerning a pumping test at Gas City, Indiana: Indiana Dept. Conserv., Div. Water Resources Bull. 1, 23 p.

Hoggatt, R. E., Hunn, J. D., and Steen, W. J., 1968, Water resources of Delaware County, Indiana: Indiana Dept. Natural Resources, Div. Water Bull. 37, 57 p.

Maclay, R. W., and Heisel, J. E., 1972, Electric analog model study of the upper White River basin, Indiana: U.S. Geol. Survey, Open-file rept., 27 p.

Norris, S. E., and Fidler, R. E., 1969, Hydrogeology of the Scioto River valley near Piketon, south-central Ohio: U.S. Geol. Survey Water-Supply Paper 1872, 70 p.

Norris, S. E., 1971, Availability of ground water from limetone and dolomite aquifers in northwest Ohio and its relation to geologic structure *in* Geol. Survey Research 1971: U.S. Geol. Survey Prof. Paper 750-B, p. B229-B235.

Skibitzke, H. E., and da Costa, J. A., 1962, The ground-water flow system in the Snake River Plain, Idaho--An idealized analysis: U.S. Geol. Survey Water-Supply Paper 1536-D, p. 47-67.

Tate, C. H., Davis, L. G., Johnson, L. E., and Pettijohn, R. A., 1973, Water resources of the upper Wabash River basin, northeastern Indiana: U.S. Geol. Survey Hydrol. Inv. Atlas HA-433, 3 sheets.

Walton, W. C., and Ackroyd, E. A., 1966, Effects of induced streambed in= filtration on water levels in wells during aquifer tests: Water Resources Research Center, Univ. of Minnesota Bull. 2, 43 p.

Watkins, F. A., Jr., and Ward, P. E., 1962, Ground-water resources of Adams County, Indiana: Indiana Dept. of Conserv., Div. Water Resources Bull. 9, 67 p.

Watkins, F. A., Jr., and Rosenshein, J. S., 1963, Ground-water geology and hydrology of Bunker Hill Air Force Base and vicinity, Peru, Indiana: U.S. Geol. Survey Water-Supply Paper 1619-B, 32 p.

Winslow, J. D., and Nuzman, C. E., 1966, Electronic simulation of ground-water hydrology in the Kansas River valley near Topeka, Kansas: Kansas Geological Survey Spec. Distrib. Pub. 29, 24 p.