GROUND-WATER RESOURCES OF THE WHITE RIVER BASIN, RANDOLPH COUNTY, INDIANA

By Wayne W. Lapham and Leslie D. Arihood

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FACTORS FOR CONVERTING THE INCH-POUND UNITS IN THIS REPORT TO THE INTERNATIONAL SYSTEM OF UNITS (SI)

Multiply inch-pound units	By	To obtain SI units
inch (in.)	25.40	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.590	square kilometer (km^2)
foot per day (ft/d)	0.3048	meter per day (m/d)
square foot per day (ft^2/d)	0.0929	square meter per day (m^2/d)
foot per year (ft/yr)	0.3048	meter per annum (m/a)
cubic foot per second (ft^3/s)	0.0283	cubic meter per second (m^3/s)
million gallons per day (Mgal/d)	0.0438	cubic meter per second (m^3/s)

DATUM

The datum used in this report is the National Geodetic Vertical Datum of 1929 (NGVD of 1929): A geodetic datum derived from a general adjustment of the first-order level nets of both the United States and Canada formerly called mean sea level. NGVD of 1929 is referred to as sea level in this report.

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ABSTRACT

The two major aquifer systems are (1) sand and gravel and (2) bedrock (limestone, dolomite, and shale of Silurian to Odovician age). Thickness of the areally discontinuous beds of sand and gravel averages 15 feet. The bedrock aquifer underlies the entire study area and is estimated to be 150 feet thick.

Six pumping plans simulated in the two systems by a five-layer, digital, ground-water-flow model provide data for an assessment of the water-yielding potential of the systems. On the basis of the pumping data, the authors estimate that as much as 2.5 million gallons per day can be pumped from the aquifers at some locations. This and similar rates of pumping cause drawdown greater than 5 feet in 10 to 50 percent of the study area. About half the stream reaches were reduced in flow by more than 10 percent by the simulated pumping. However, reaches where discharge exceeded more than 2 cubic feet per second were not affected to this degree.

INTRODUCTION

Previous Investigations

Although the water resources of Randolph County have been studied in general, the ground-water resources have not been quantitatively evaluated. A map report on water resources of Randolph County by Uhl (1969) emphasizes ground-water availability and provides general ground-water resource information useful in the current study.

Surface-water and ground-water resources and their quality, in the upper White River basin are described by Cable and others (1971). Data from the study were used by Maclay and Heisel (1972) to develop an analog model of the ground-water system of the area. Although Maclay and Heisel (1972) quantified the ground-water flow, the purpose of their study was to investigate the basin-wide effects of ground-water development. Their analog model did not have the precision needed to provide quantitative assessments of ground-water development locally within the basin.

In 1972, the U.S. Geological Survey, in cooperation with the Indiana Department of Natural Resources, began a 3-year study of the ground-water resources of the White River basin in Marion County, Ind. The objectives of the study were to (1) determine the quantity of ground water that could be pumped and (2) determine the effects of pumping on the ground-water system and on streamflow (Meyer and others, 1975, p. 2).

Purpose and Scope

After completing the Marion County study in 1975, the U.S. Geological Survey, in cooperation with the Indiana Department of Natural Resources began a similar study of the rest of the White River basin upstream from Marion County. The objective of the study, which began in July 1975, was to assess the ground-water resources in the White River basin upstream from Marion County. The assessment involved (1) mapping the aquifers and calculating the hydraulic properties of the aquifers and confining beds; (2) measuring the distribution of potentiometric head in the aquifers; (3) measuring ground-water discharge to streams; and (4) using a five-layer, digital, ground-water-flow model to (a) determine the water budget, (b) calculate the quantity of water that could be pumped without significant adverse effect on the ground-water system and streamflow, and (c) simulate the effect that a pumping rate of 1 Mgal/d would have on the ground-water system and streamflow.

Project Area

The project area, in central and east-central Indiana, consists of 1,500 mi^2 of the White River basin upstream from Marion County (fig. 1). The project area was divided into four study areas, each consisting of the principal county in that area. This division simplified detailed study. The four study areas, by county from west to east (fig. 1), are Hamilton, Madison, Delaware, and Randolph. The area of study in the current report is the part of Randolph County shown in figure 2 and the shaded parts of Henry, Delaware, and Wayne Counties shown in figure 1. The area, however, is referred to as the Randolph County study area in the remainder of this report for simplicity and because the area for which interpretations are made actually lie within Randolph County. Parts of other counties were included in this study to aid in analyses by the model, although no actual analyses are done in the other counties.

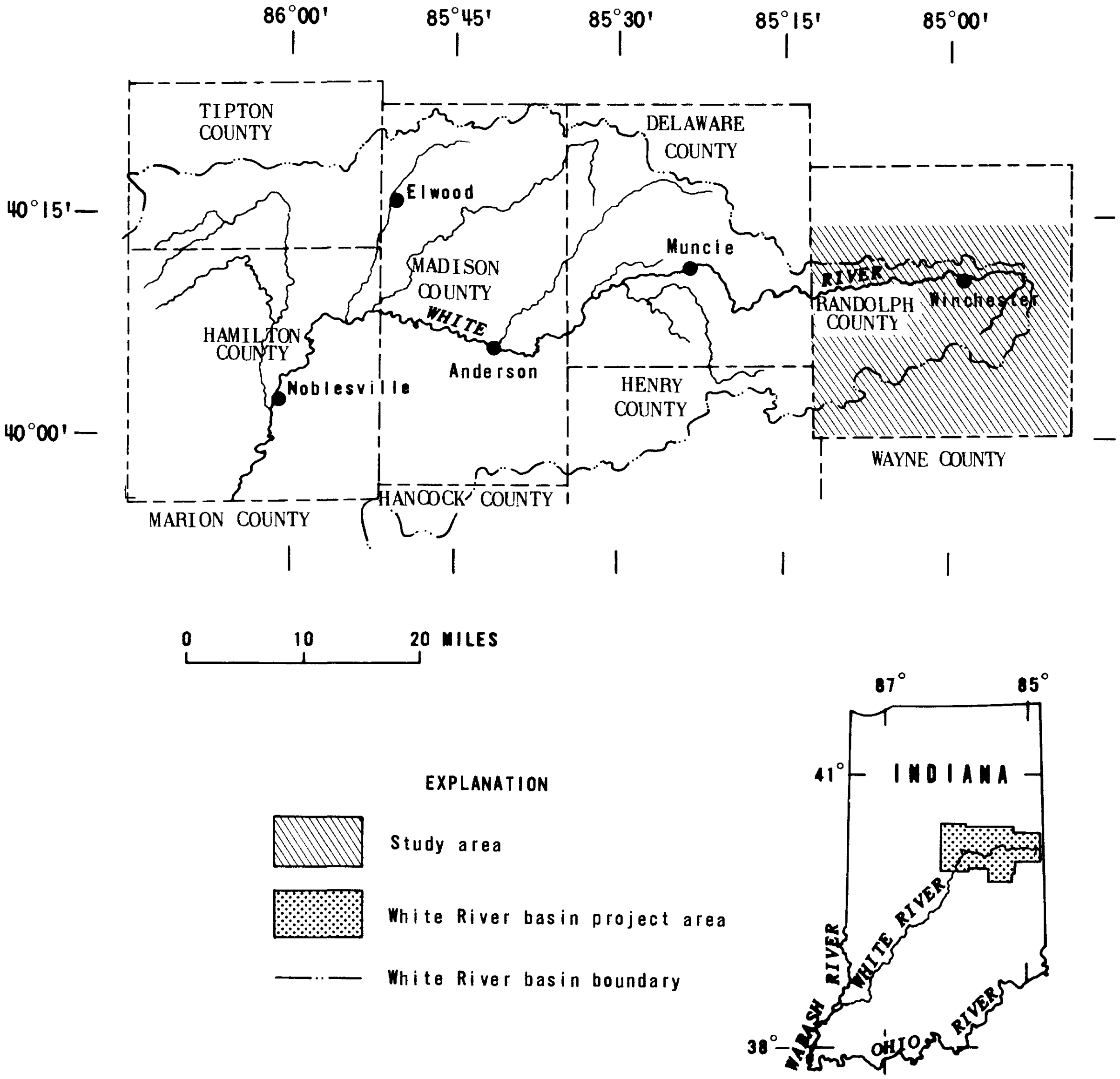


Figure 1.-- Location of Randolph County study area.

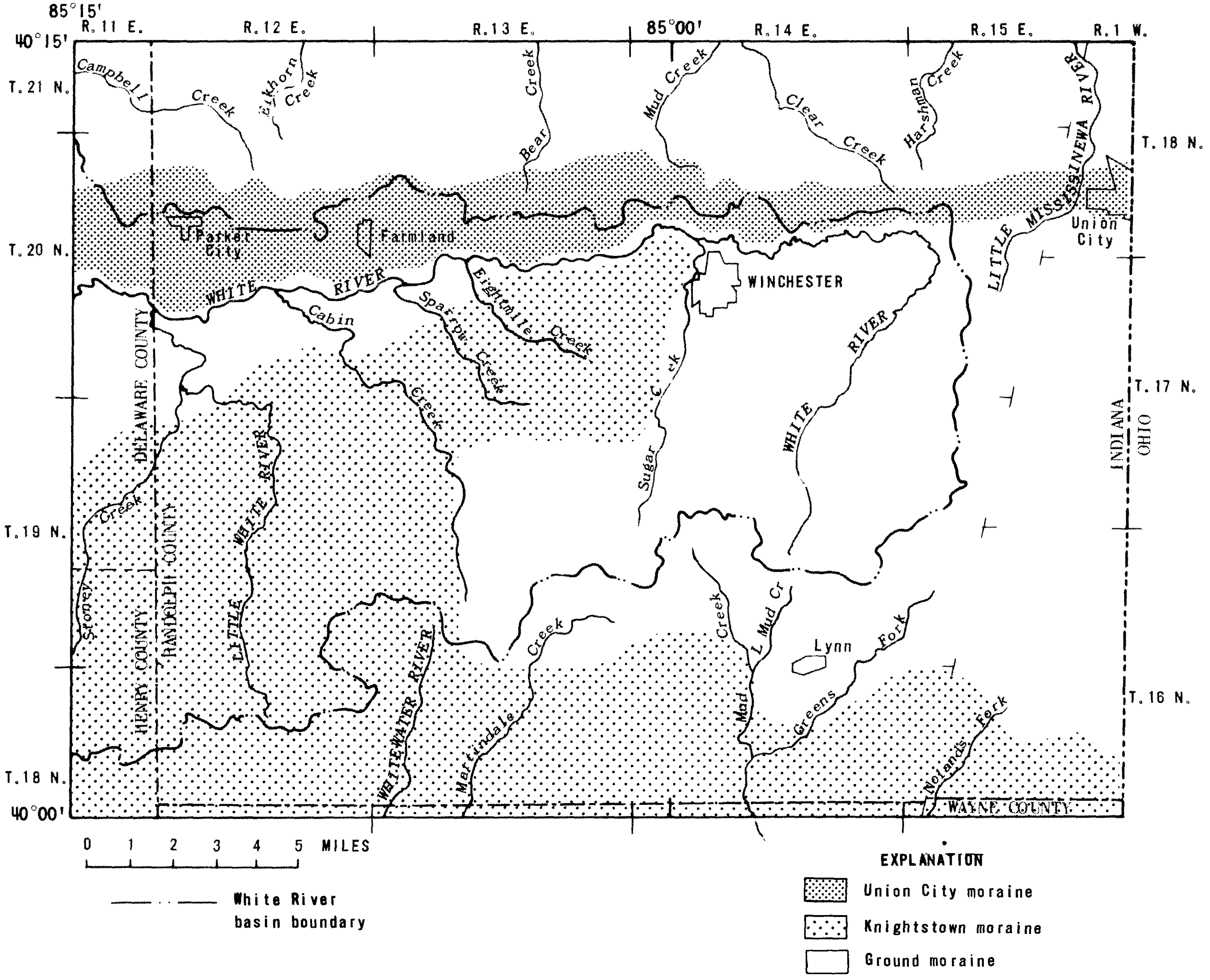


Figure 2.-- Study area and principal moraines in Randolph County.

Randolph County consists of 447 mi^2 (Uhl, 1969), but the study area (fig. 2) consists of about 400 mi^2. Major towns are Winchester, Union City, Farmland, and Lynn, whose populations are 5,659, 3,908, 1,560, and 1,250, (U.S. Department of Commerce, 1980, p. 26, 28, 30, and 31). Most of the land is farmed.

Randolph County is in the Tipton Till Plain of the Central Lowlands physiographic province (Wayne, 1956, p. 13). Ground surface is flat to gently rolling, and most of the local relief is due to stream incisement. The altitude of the land surface generally ranges from 950 to 1,200 ft.

The largest river is the White River; other streams include Stoney, Cabin, and Sugar Creeks and the Little White River. The White River flows from east to west, just south of the surface-water divide that separates the White River basin from the Mississinewa River basin (fig. 2). The tributaries of the White River generally flow north. The streams north of the White River basin divide generally flow north into the Mississinewa River; the streams south of the White River basin divide generally flow southwest into the Whitewater River.

The climate is temperate. During 1941-70, monthly average temperature at Winchester airport ranged from 3.1° C (Celsius) or 26.4° F (Fahrenheit) in January to 22.8° C (73.1° F) in July. The average annual temperature during this period was 10.2° C (50.4° F). For the same period, monthly average precipitation at the airport ranged from 2.14 in. in February to 4.61 in. in June. The 30-year average annual precipitation for 1941-70 at the airport was 39.0 in. (National Oceanic and Atmospheric Administration, 1973).

Methods of Investigation

Hydrologic data were collected to define the ground-water flow system. Specifically, the data were used to (1) map the areal extent, as well as the altitudes of the tops and thicknesses of the aquifers; (2) define the potentiometric surfaces of the aquifers; (3) calculate the hydraulic properties of the aquifers; and (4) estimate the discharge from the system.

Mapping the areal extent, thickness, and altitude of the top of the sand and gravel units was completed early in the project. Approximately 800 lithologic logs of domestic, industrial, and municipal wells on file with the Division of Water, Indiana Department of Natural Resources, and lithologic logs of several Geological Survey observation wells were used in the mapping. Differentiation of each unit into sand, sand and gravel, and gravel was not done because of inconsistent lithologic descriptions on well logs and the predominance of mixed sand and gravel. Consequently, although the units are composed of different combinations of sand and gravel, the author considers them to be mixed sand and gravel units.

Water levels in Geological Survey observation wells and in domestic wells were used to define the potentiometric surfaces of the aquifers. Analysis of specific-capacity data within the basin was used to calculate the hydraulic properties of the aquifers. Ground-water discharge to streams was determined by measuring changes in stream discharge throughout the area during low flow and adjusting for other inflows or outflows to or from the stream during the discharge measurements. Pumpage data were obtained from large-scale users (>0.1 Mgal/d) of ground water.

A five-layer digital model was constructed to simulate flow within the most permeable sediments and the bedrock. The model was used to simulate effects of six pumping plans on the ground-water system and streamflow and to determine, on a regional basis, the quantity of ground water that could be pumped without significantly affecting the ground-water system and streamflow.

Acknowledgments

The author is grateful to the Division of Water, Indiana Department of Natural Resources, for the use of water-well records obtained from their files and to the municipalities and industries that provided pumping information. Appreciation is also expressed to the private well owners in and around Randolph County, who permitted measurement of water levels in their wells.

GEOLOGY

The geology has been described in general by Wayne (1956 and 1963), and Uhl (1969). A generalized south-north geologic section of the area is shown in figure 3. The section illustrates the major geologic features--drift overlying the bedrock and composed predominantly of till; thin, generally horizontal, discontinuous sand and gravel units interbedded within the till; and the variable thickness of the drift. However, the thickness, location, width, and relief of lithologic units differ from those shown in the figure depending on location within the study area.

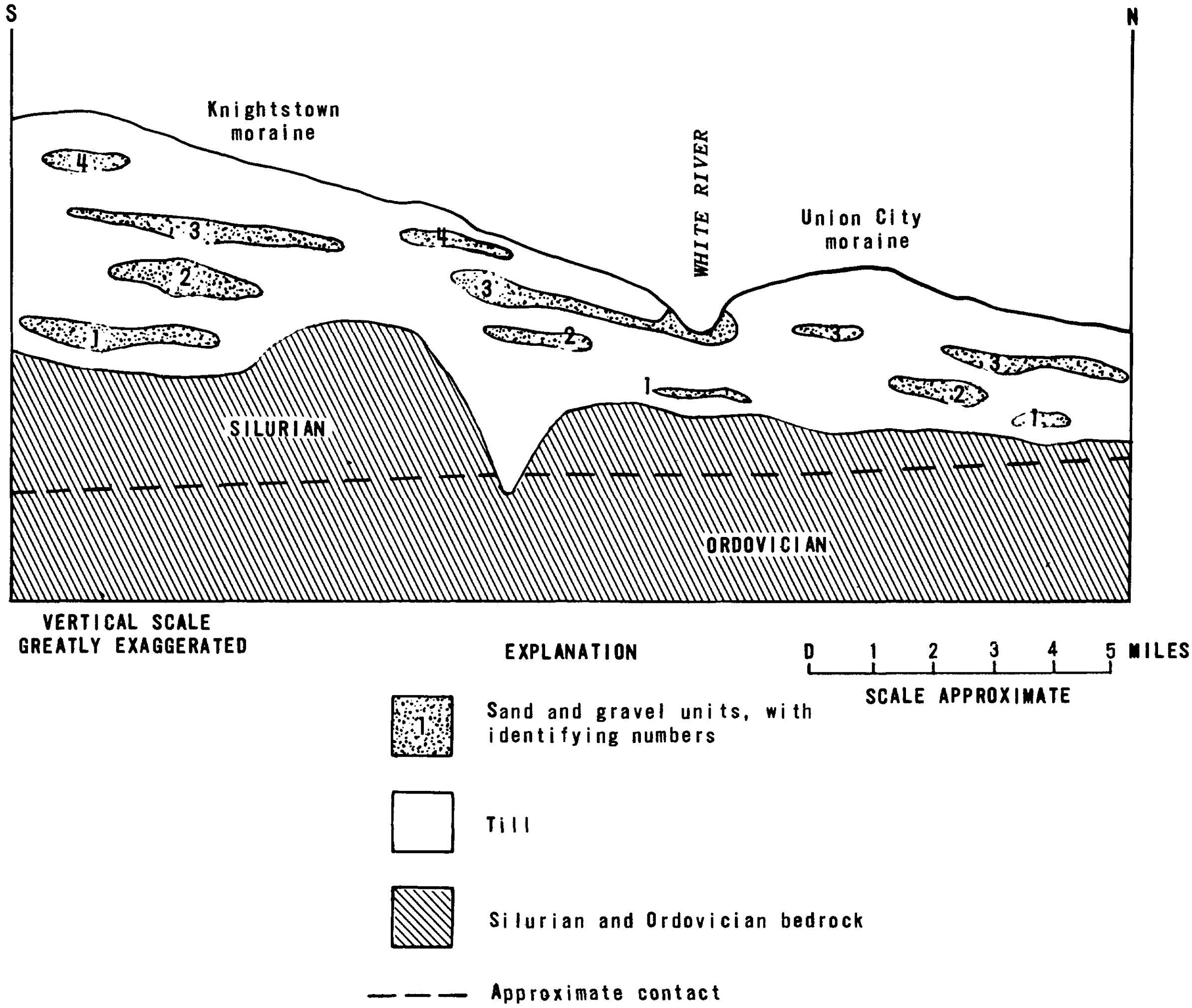


Figure 3.-- Generalized geologic section of study area.

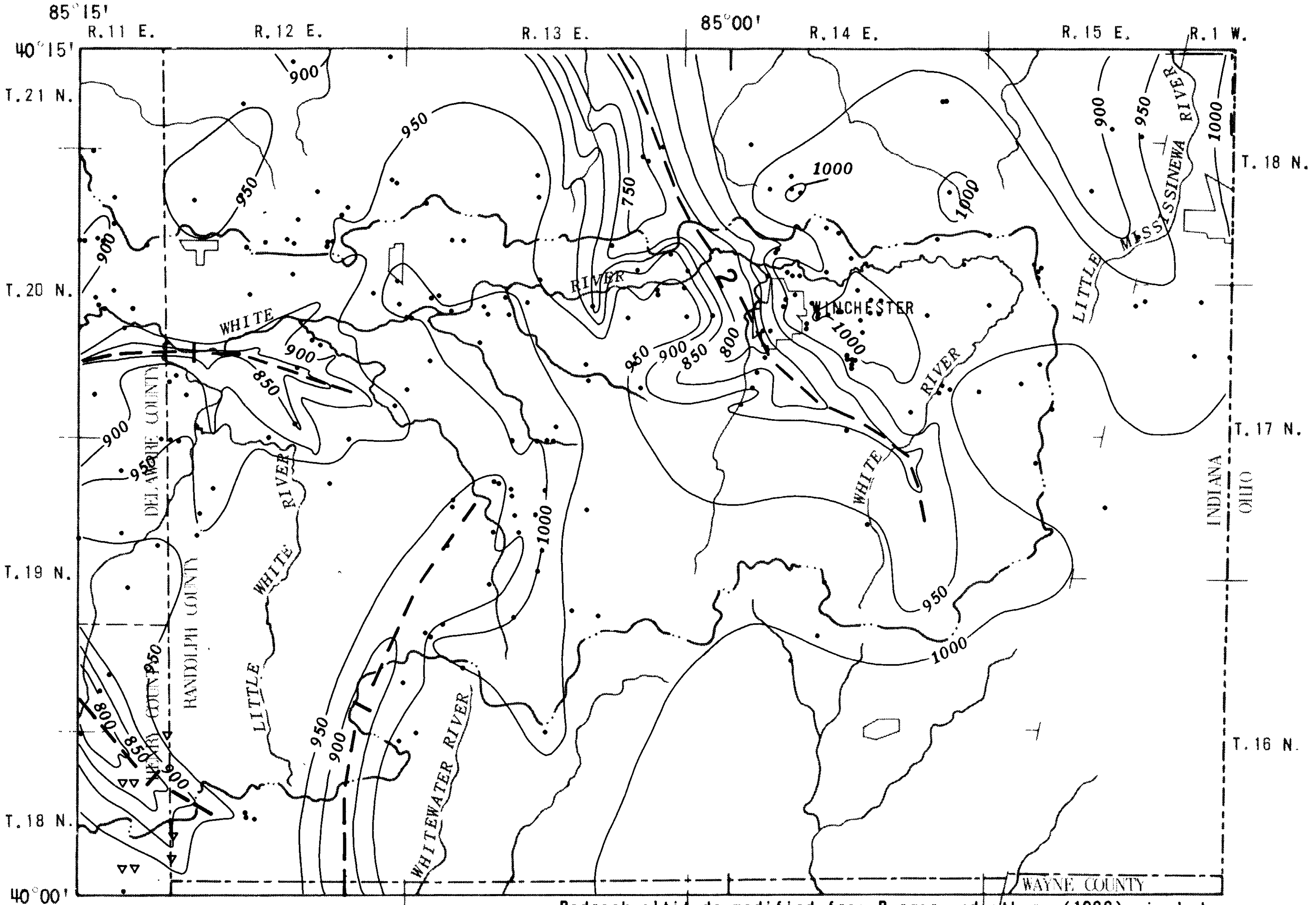
Bedrock

Rocks of Ordovician and Silurian, which dip to the southwest (Patton, 1956), underlie the whole area. Most of the bedrock is overlain by drift, although some is exposed southeast of Farmland near the White River (Uhl, 1969). In most places, limestone, dolomite, and shale of Silurian age underlie the drift (Burger and others, 1971, Muncie Sheet Part A). Ordovician shale, however, is exposed in the walls and at the base of buried preglacial valleys called the Anderson and Priam Valleys (Wayne, 1956, p 38-39). These valleys were formed as tributaries of the ancestral Teays River valley system. The Anderson Valley originates locally as three small tributaries; one trends west out of the west-central part, one trends northwest out of the southwest part, and one trends south just west of the Whitewater River (fig. 4). The Priam Valley trends northwest out of the north-central part, just west of Winchester (fig. 4). The valleys and their tributaries are generally narrow, and their walls are steep. Their maximum width is probbly no more than 2 miles. The altitude of the bedrock surface (fig. 4) generally ranges from 750 ft above sea level at the axis of the Priam Valley to 1,000 ft in the highlands of bedrock.

Glacial Drift

Drift covers most of the study area and generally ranges in thickness from 0 to 300 ft. The drift, composed mostly of till (poorly sorted clay, silt, and sand like the drift farther to the west), was probably deposited during at least three glaciations--the Kansan, Illinoian, and Wisconsin (Wayne, 1975, p. 7). Interbedded within the till are thin, sheetlike, areally discontinuous, stratified drift deposits of sand and gravel (fig. 3). Generally, these deposits are separated vertically by till. Locally, however, they coalesce vertically to form thick deposits of sand and gravel (not indicated in fig. 3).

An end moraine, the Union City moraine, extends east-west just north of the White River (fig. 3). Farther west, this moraine becomes a low, glacially formed ridge having about 30 ft of relief (Wayne, 1975, p. 3). The moraine forms the surface-water divide that separates the White River from the Mississinewa River to the north. The Knightstown moraine (Burger and others, 1971, Muncie sheet Part B), forms the surface-water divide that separates the White River from the Whitewater River to the south (fig. 3). This moraine generally occupies the southern one-third of the study area.



Bedrock altitude modified from Burger and others (1966); includes seismic data obtained by Indiana Geological Survey and test-hole data obtained by the U.S. Geological Survey

EXPLANATION

— · — White River basin boundary

— 850 — Line of equal altitude of bedrock surface. Interval 50 feet. Datum is sea level

— — 1 — — Axis of buried Anderson valley tributary

— — 2 — — Axis of buried Priam valley

● Data point based on well logs

▽ Data point based on seismic information

Datum is sea level

Figure 4.-- Altitude of top of bedrock.

GEOHYDROLOGY

The geohydrology was defined by mapping the aquifers, calculating their hydraulic properties and those of the semipermeable confining beds, describing the general characteristics of ground-water flow from analysis of potentiometric heads in the bedrock aquifer and sand and gravel aquifers in the drift, and determining the inflow to and outflow from the ground-water system.

Aquifer Geometry

On the basis of lithologic data, five hydrologic units were identified as potential aquifers. Four of these are sand and gravel units interbedded in the till, and the fifth is the upper 150 ft of predominately carbonate bedrock, which is more permeable than the deeper carbonate rocks owing to weathering along fractures.

The general stratigraphic relation between the sand and gravel aquifers can be seen in figure 3. The four sand and gravel aquifers are numbered so that aquifer 1 is the lowest stratigraphically and aquifer 4 the highest. Although some of the aquifers extend into other study areas within the White River basin and were mapped accordingly, numbers for the aquifers were assigned independently in each study area during the mapping. Correlation of each sand and gravel aquifer between study areas is given in table 1. For instance, aquifer 1 in Randolph County extends into Delaware County, where it is called aquifer 4, and into Madison and Hamilton Counties, where it is called aquifer 5.

Table 1.--Correlation of each sand and gravel aquifer between study areas in the White River basin upstream from Marion County, Indiana

Study area	Aquifer number used in individual study areas							
Hamilton	--	--	--	[a]5	[a]4	3	2	1
Madison	--	--	[a]6	[a]5	4	3	2	1
Delaware	[a]7	6	5	4	3	2	1	--
Randolph	4	3	2	1	--	--	--	--

[a]Not considered a significant aquifer in study area.

Configuration of the four sand and gravel aquifers is shown in figures 5 through 12. The aquifers are generally separated horizontally by low-permeability till but locally coalesce vertically to form one thick deposit. The aquifer maps are not meant to imply that the individual areas of sand and gravel in a figure were deposited by the same process. However, for ease of illustration and discussion, the individual units are grouped by altitude into four layers and shown as aquifers 1 through 4.

In some places, the mapping may simplify the complex distribution of sand and gravel units in a till system. However, the principal features of the confined sand and gravel units (thickness and altitude) are adequately defined in figures 5-12 for discussion. Thickness of the aquifers generally ranges from 5 to 50 ft (figs. 5, 7, 9, and 11) and averages approximately 15 ft. The tops of the aquifers (figs. 6, 8, 10, and 12) generally dip slightly to the northwest. Aquifer 4 (figs. 11 and 12) is found only in the south part of the study area. The areas delineated as "approximate area where horizontal projection of aquifer lies above land surface" (figs. 9-12) are general delineations only.

Locations of small, isolated sand and gravel outliers of the aquifers are shown in figures 5 through 12. These outliers are insignificant as aquifers because of their size and poor hydraulic connection to the major parts of the aquifer. However, they were included to provide information that may prove helpful if the mapping of the areal extent of the aquifers is refined in the future, as new data are collected.

Where sand and gravel aquifers are absent, the Silurian carbonate aquifer is an adequate source of water for many municipal, industrial, and domestic water users. Although Cable and others (1971) and Meyer and others (1975, p. 17) assumed an average thickness of 100 ft for the permeable part of the bedrock, an average thickness of 150 ft was assigned to the carbonate aquifer in the current study because many wells penetrate as deep as 150 ft into the aquifer. However, the thickness of the bedrock aquifer may exceed 150 ft. The significance of this assumption is discussed under "Bedrock Aquifer" in the section "Hydraulic Characteristics of the Ground-Water System."

Hydraulic Characteristics of the Ground-Water System

Sand and Gravel Aquifers

The hydraulic conductivity of the confined sand and gravel aquifers was calculated from specific-capacity data obtained at 13 sites, where 2-inch diameter observation wells were installed in the Madison County study area. An analysis of these data is given in the report of the ground-water resources of Madison County, Indiana (Lapham, 1981, p. 17). The results are briefly discussed in this section. Reference will be made to sand aquifers and sand

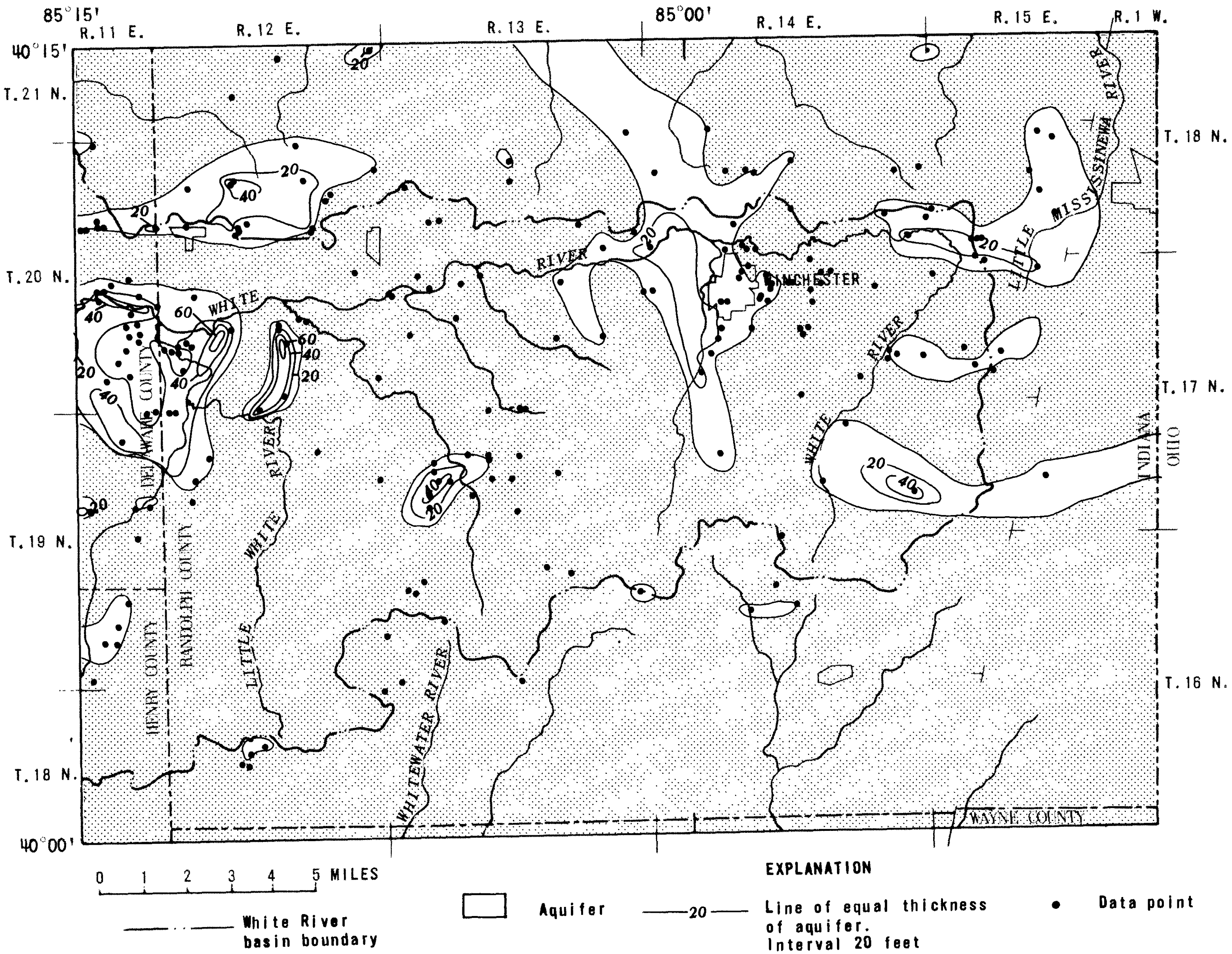


Figure 5.-- Thickness of aquifer 1.

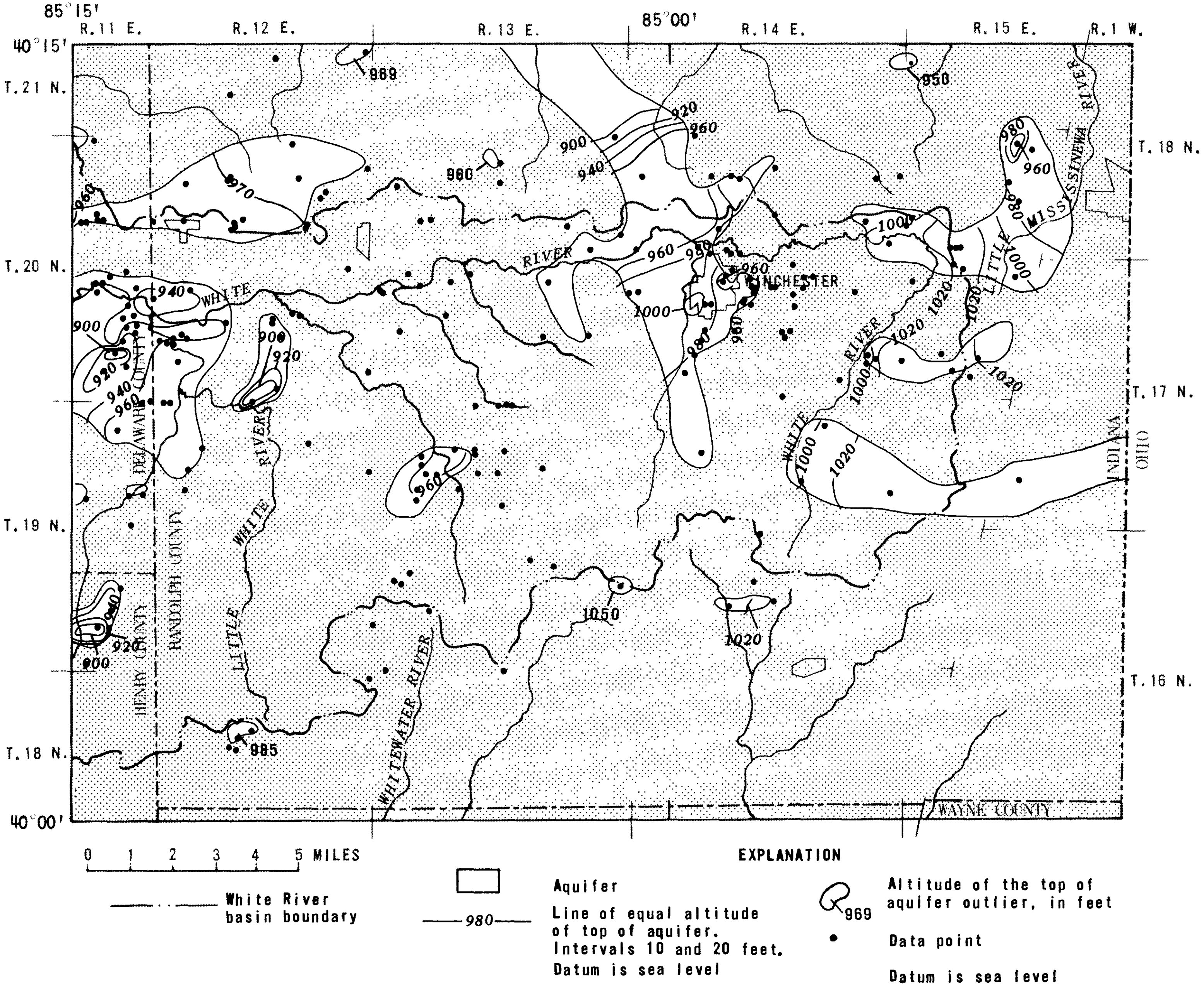


Figure 6.-- Altitude of top of aquifer 1.

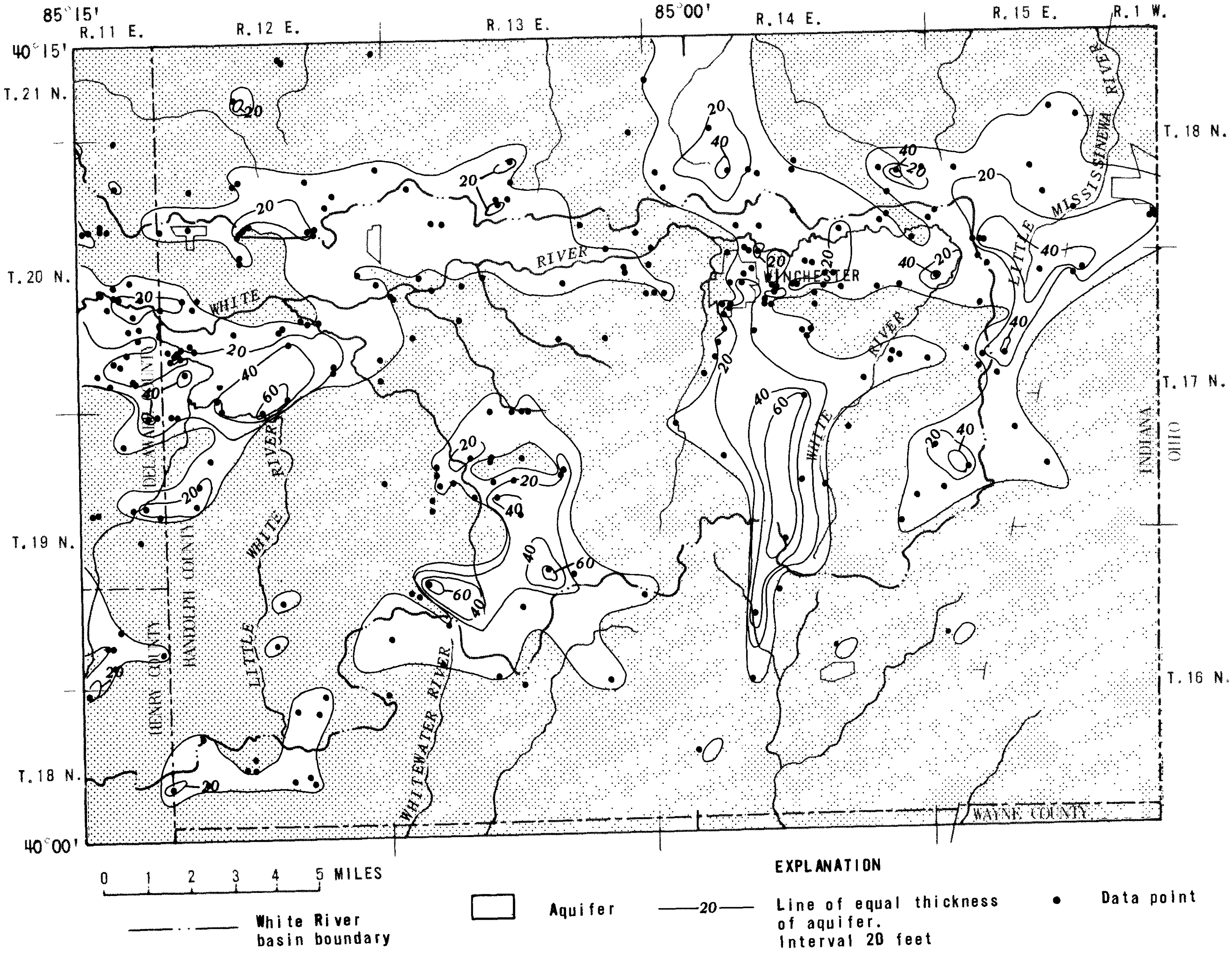
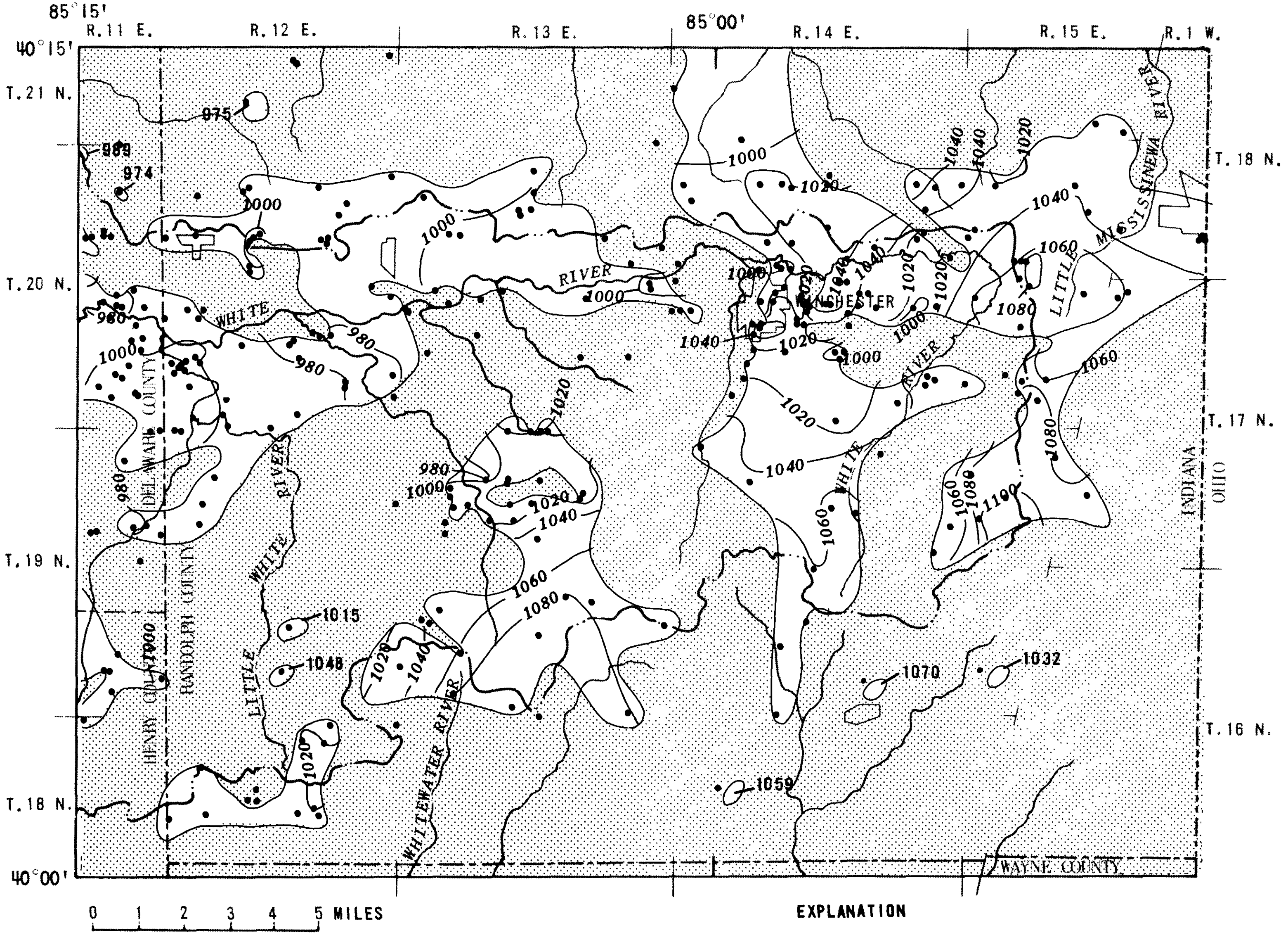


Figure 7.-- Thickness of aquifer 2.



EXPLANATION

White River basin boundary	
Aquifer	
1000	Line of equal altitude of the top of aquifer. Interval 20 feet. Datum is sea level
1070	Altitude of the top of aquifer outlier, in feet
•	Deta point
	Datum is sea level

Figure 8.-- Altitude of top of aquifer 2.

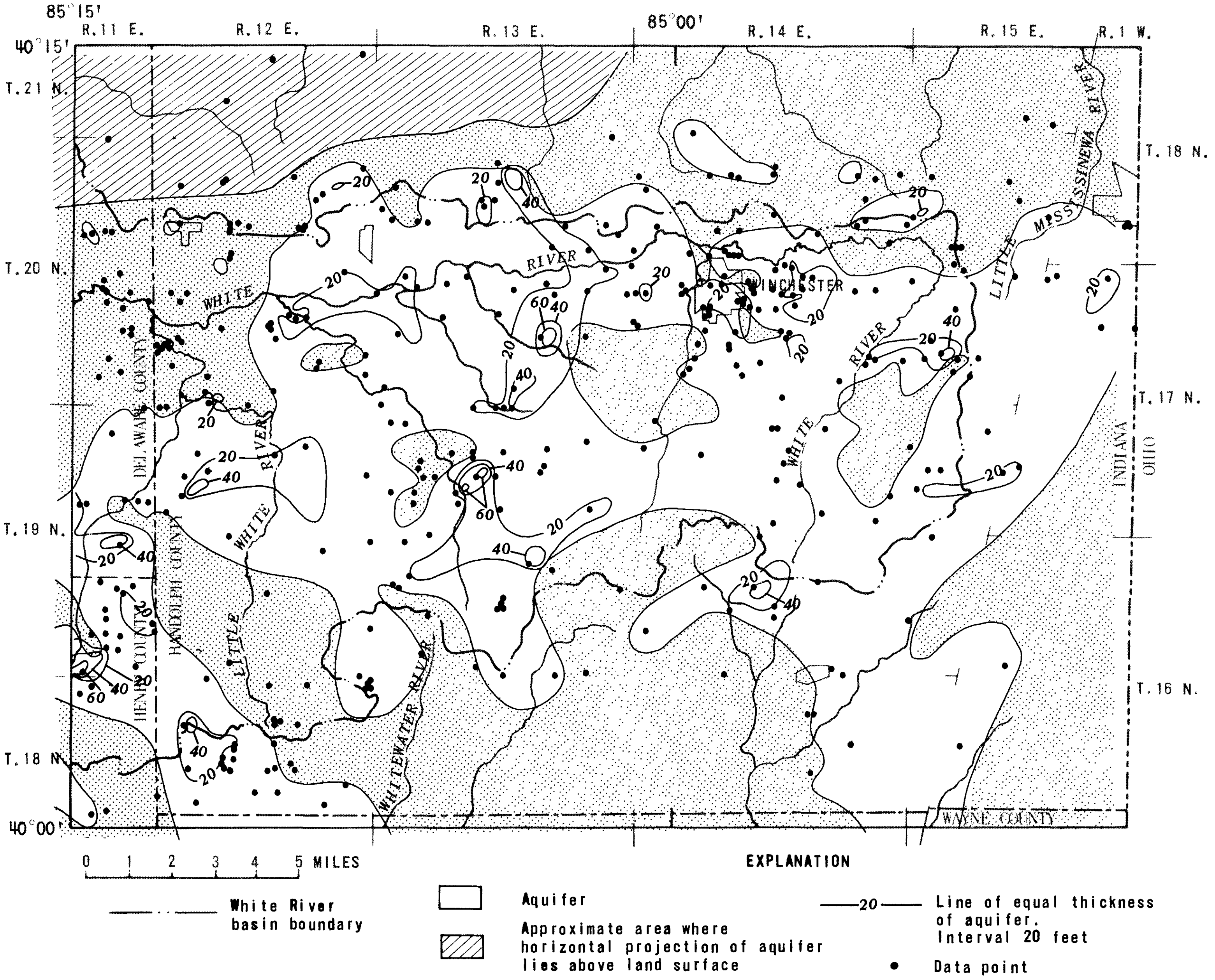


Figure 9.-- Thickness of aquifer 3.

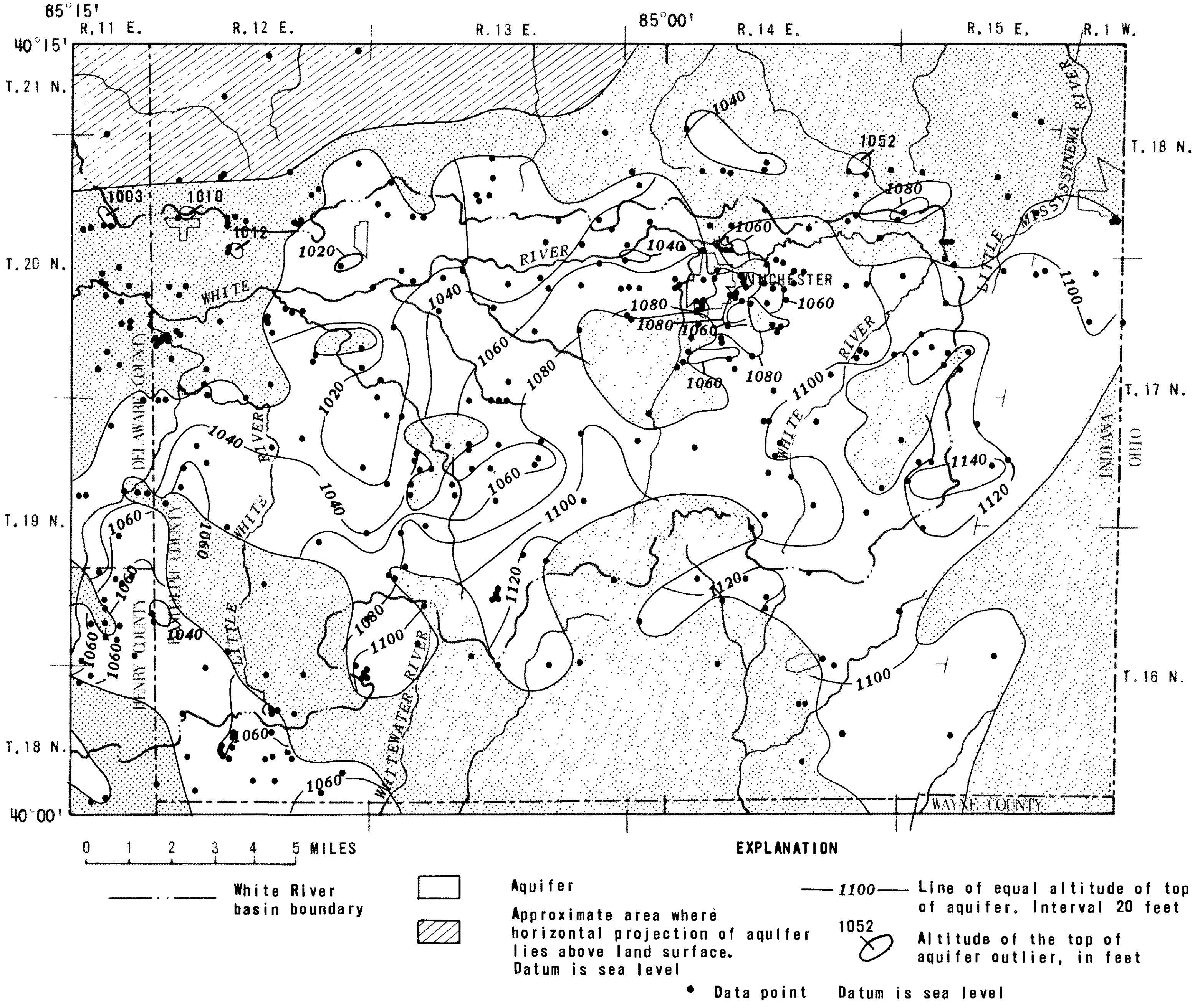


Figure 10.-- Altitude of top of aquifer 3.

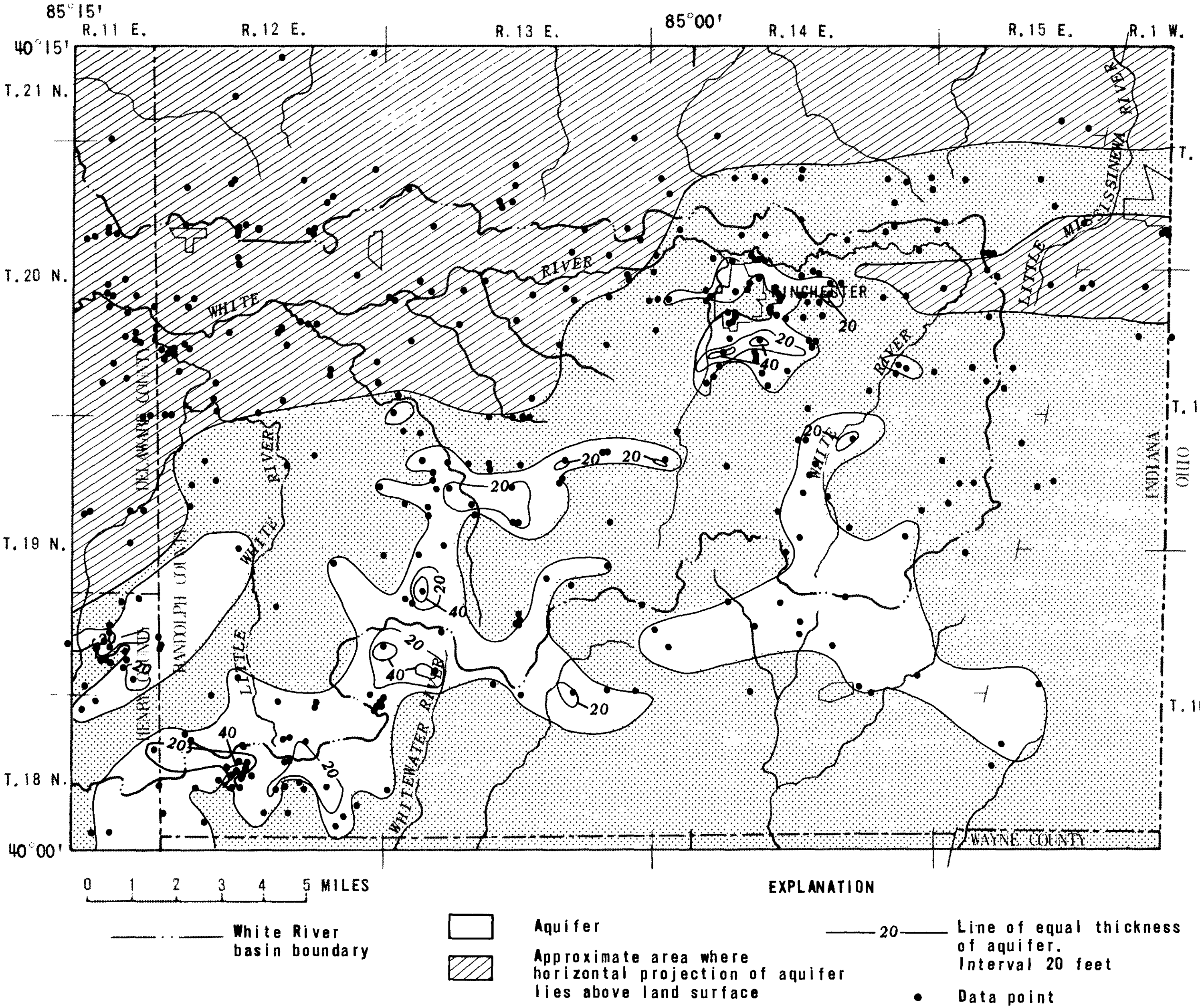


Figure 11.-- Thickness of aquifer 4.

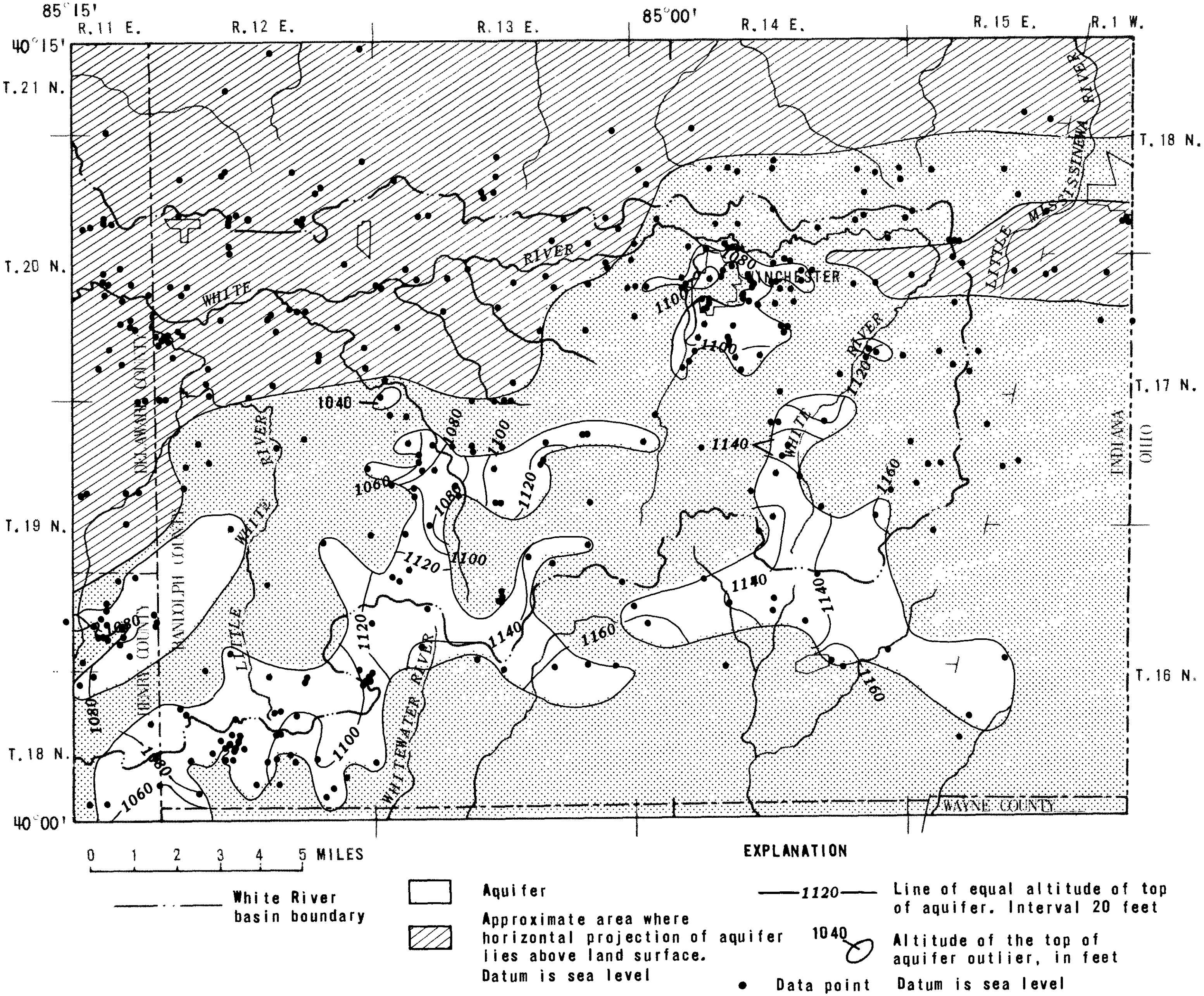


Figure 12.-- Altitude of top of aquifer 4.

and gravel aquifers in the hydraulic conductivity discussion in the following paragraphs. These names are given on the basis of the predominant material in the aquifer being tested. However, some sand and gravel are usually found in all the unconsolidated aquifer material. Therefore, the aquifers are referred to as "sand and gravel" in the remainder of the report.

The author calculated transmissivity of the predominantly sand and gravel aquifers for 10 sites and of the predominantly sand aquifers for 3 sites by applying specific-capacity data collected at each of the 13 well sites to the method of Brown (1963). Because only the bottom 3 ft of the aquifer was screened, the measured drawdowns were adjusted for partial penetration by the method described by Butler (1957). The ratio of horizontal to vertical hydraulic conductivity of the sand and gravel aquifers was assumed to be 10:1. The 10:1 ratio has been used in previous studies of the basin (Meyer and others, 1975, p. 19; Lapham, 1981). Hydraulic conductivity was calculated by dividing the calculated transmissivity by the total thickness of the aquifer.

Because most aquifer material determined from well logs was mixed sand and gravel, a representative hydraulic conductivity for the four aquifers (called sand and gravel aquifers, figs. 5-12) was calculated by averaging the hydraulic conductivity of the sand aquifers (156 ft/d) with that of the sand and gravel aquifers (710 ft/d). The average hydraulic conductivity thus calculated is 433 ft/d. The range for these 13 specific-capacity tests was 24 to 1,633 ft/d. The variability is large and suggests that the average, 433 ft/d, may differ considerably from the actual hydraulic conductivity at any point, but 433 ft/d is probably reasonable on a regional basis.

Using specific-capacity data, Cable and others (1971) calculated hydraulic conductivities of 200 ft/d for the confined sand and gravel aquifers and 334 ft/d for the unconfined sand and gravel aquifers. By applying specific-capacity data to the method of Brown (1963), Meyer and others (1975, p. 21) calculated an average hydraulic conductivity of 390 ft/d for the confined sand and gravel aquifers in Marion County.

Transmissivity distributions for each of the four sand and gravel aquifers (figs. 5-12) can be determined at a single point by multiplying the aquifer thickness shown in figures 5, 7, 9, and 11 by 433 ft/d, the average hydraulic conductivity of the confined sand and gravel aquifers. Because of the large range in hydraulic conductivity calculated from the specific-capacity tests, the transmissivity calculated by multiplying the thickness by the hydraulic conductivity (433 ft/d) results in an estimate of the transmissivity at that point.

Neither the storage coefficient nor the specific yield of the sand and gravel aquifers was calculated because only steady-state analyses were considered.

Bedrock

The transmissivity distribution of the bedrock (fig. 13) was calculated from specific-capacity data for approximately 100 wells. The specific-capacity data for wells that did not fully penetrate the assumed 150 ft of permeable bedrock were corrected for partial penetration (Butler, 1957). In the adjustment for partial penetration, the ratio of horizontal to vertical hydraulic conductivity of the bedrock was assumed to be 1:1, largely because much of the permeability probably results from fractures. Model analysis of the Madison County area indicated that the difference in potentiometric heads of the bedrock for anisotropies of 1:1 and 100:1 was insignificant (W. W. Lapham, oral commun., 1980). Also, changes in transmissivities corrected for partial penetration for anisotropies of 1:1 and 100:1 resulted in a maximum difference in the calculated transmissivity at a bedrock well of 50 percent and for many similar wells a difference of much less than 50 percent. Areally, however, the difference in the transmissivity of the 150-foot thickness of bedrock was as much as two orders of magnitude. Therefore, error in the anisotropy ratio does not significantly affect head distribution or transmissivity. The potential error in calculating transmissivity by assuming only a 150-foot thickness of aquifer is in the calculations that correct for partial penitration. The calculated transmissivity of bedrock increases directly with its assumed thickness. Therefore, the calculated transmissivities are an estimate of the upper part of the bedrock, where data were available and where the bedrock was probably most premeable. Even though transmissivity may double, depending on the the depth of bedrock chosen, transmissivity differs areally by as much as two orders of magnitude. Therefore, the possible error in assuming an incorrect thickness is small compared to the range in transmissivity.

Cable and others (1971) determined from bedrock well-log data that the thickness of the permeable bedrock in the White River basin is approximately 100 ft and estimated that the hydraulic conductivity of the bedrock is 13.4 ft/d. The transmissivity of the bedrock aquifer calculated from these data is 1,340 ft^2/d. These same values were used in the ground-water study of Marion County by Meyer and others (1975, p. 17). Cable and others (1971, p. C12) noted considerable areal variation in bedrock transmissivity. Although a large variation in the bedrock transmissivity is shown in figure 13, the average transmissivity is generally close to the average transmissivity of the bedrock (1,340 ft^2/d) calculated by Cable and others (1971).

Neither the storage coefficient nor the specific yield of the bedrock aquifer was estimated because only steady-state analyses were considered.

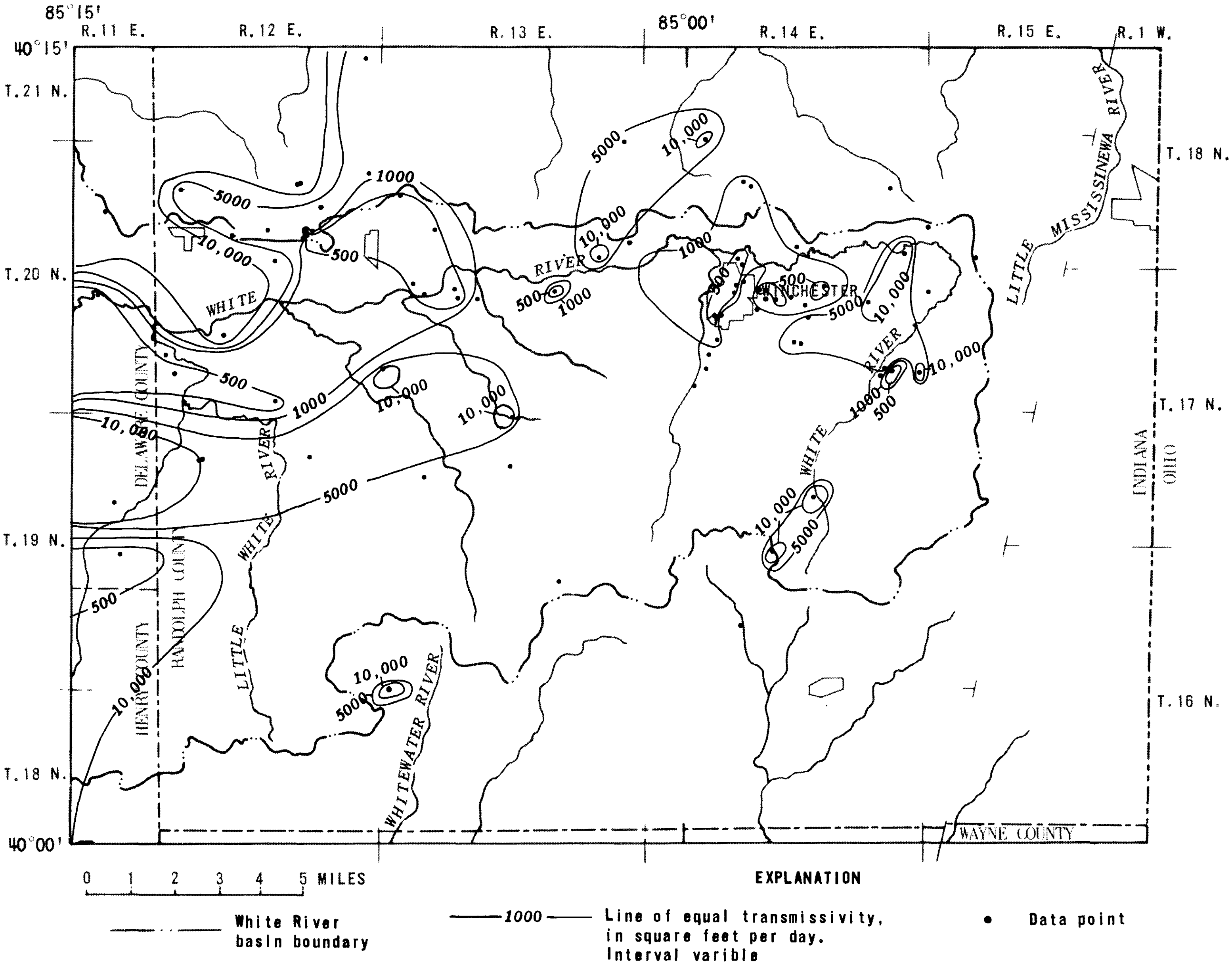


Figure 13.-- Transmissivity of bedrock calculated from specific-capacity data.

Semipermeable Confining Beds

The till between the aquifers, primarily a poorly sorted mixture of clay, silt, and sand, constitutes a semipermeable confining bed. Although the vertical hydraulic conductivity of the till is less than the horizontal, the area of vertical flow is about three orders of magnitude times that of horizontal flow. Thus, the till transmits little water horizontally, and vertical flow through the till between aquifers dominates.

No data for estimating the vertical hydraulic conductivity of the confining beds were collected. However, an analog model of Marion County by Meyer and others (1975, p. 26) indicated that the vertical hydraulic conductivity of similar material ranges from 10^{-4} to 1.3×10^{-3} ft/d. Therefore, an initial conductivity, equal to the average of these upper and lower ends of the range obtained from Meyer and others, 7×10^{-4} ft/d, was assumed. During calibration of the model, the initial conductivity was changed as needed to obtain a better match to field conditions. As in the Marion County study, the vertical hydraulic conductivity of confining beds in Randolph County has a wide range.

Ground-Water Flow

The flow patterns in the bedrock (fig. 14) and in sand and gravel aquifer 3 (fig. 15) indicate that the general direction of regional ground-water flow is from the southeast to the north and northwest. Ground water also flows south from the ground-water divide in the southeast corner of the study area (fig. 14). The flow patterns for aquifers 1, 2, and 4 are similar to those shown for the bedrock and aquifer 3.

As indicated in figures 14 and 15, the potentiometric heads in the two aquifers are about 1,150 ft above sea level at the ground-water divide and about 1,000 ft, northwest of the divide. This head difference results in a lateral hydraulic gradient of about 10 ft/mi.

Although downward flow is not necessarily evident from comparison of figures 14 and 15, it has been identified by comparing the potentiometric heads in adjacent observation wells, one screened in the bedrock aquifer and the other in aquifer 3. The differences between the potentiometric head in aquifer 3 and the lower head in the bedrock is as much as about 10 ft. This predominantly downward flow suggests a surface recharge area for the ground-water system. Small upward gradients in the shallow aquifers adjacent to streams indicate that some ground water discharges to the streams. The vertical and horizontal flows indicate the need to consider three-dimensional flow in an analysis of the ground-water system.

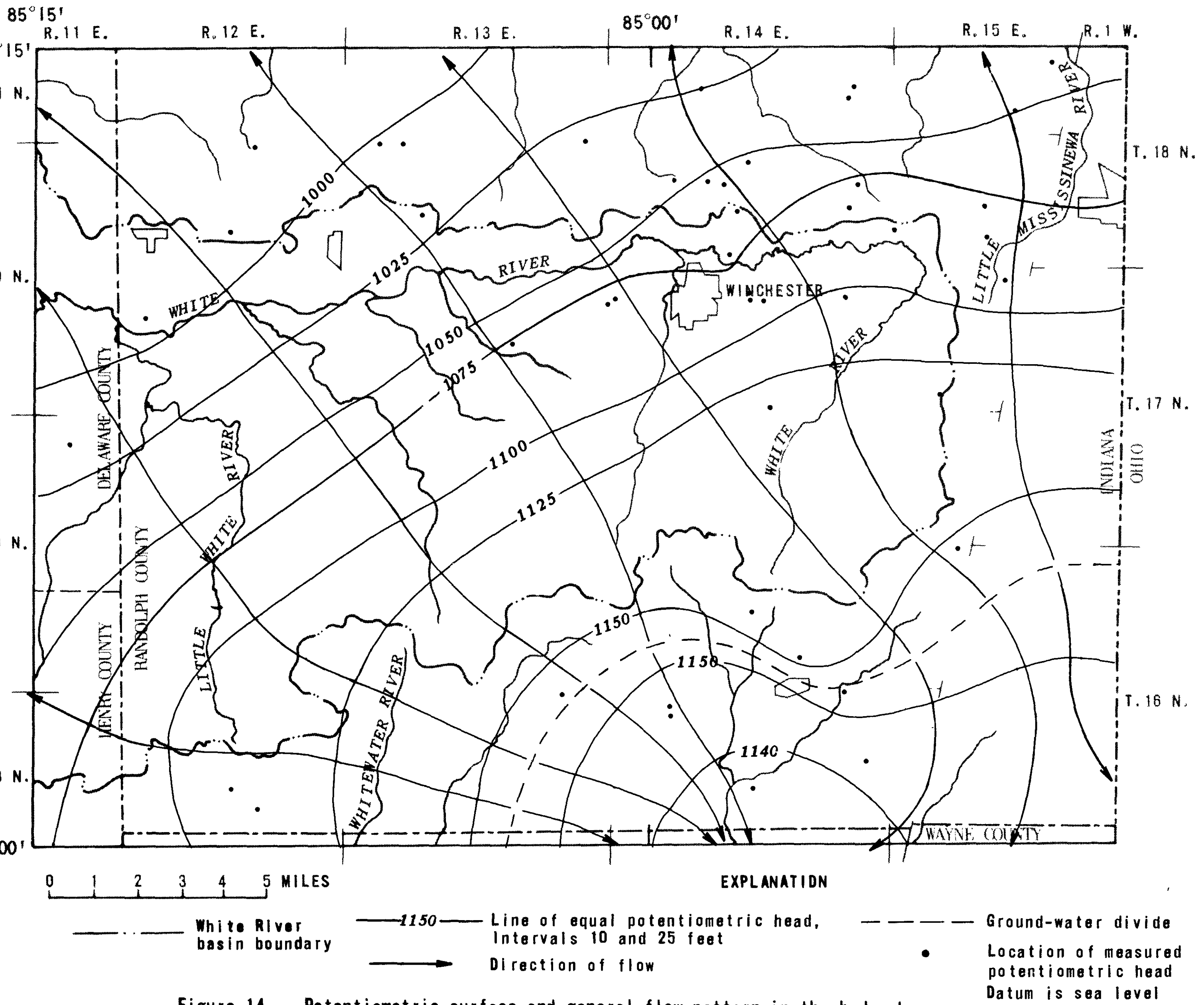


Figure 14.-- Potentiometric surface and general flow pattern in the bedrock.

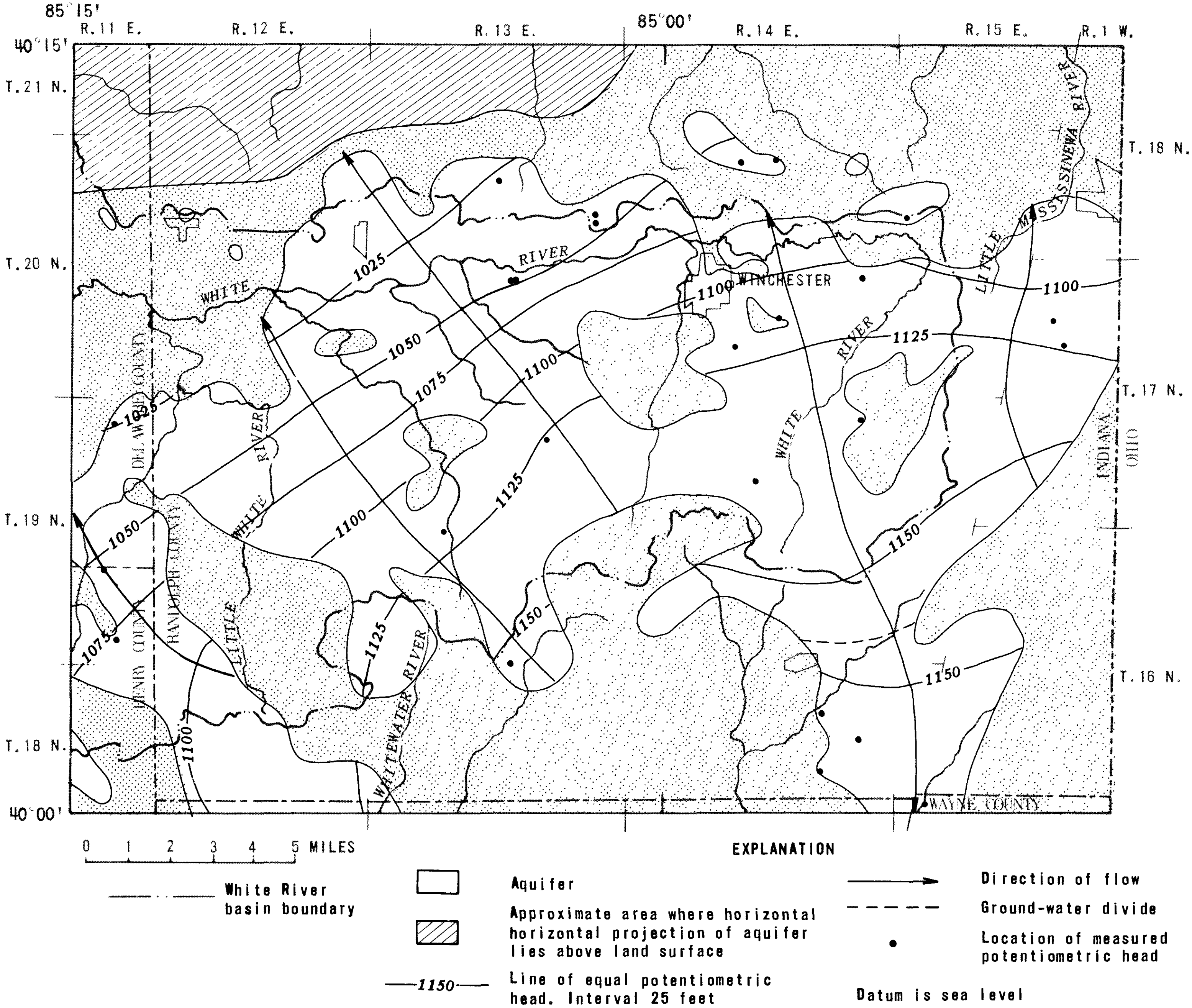


Figure 15.-- Potentiometric surface and general flow pattern in aquifer 3.

Water-Level Fluctuations

Water-level fluctuation in observation well Randolph 3 is shown in figure 16. The depth of this well is 54 ft, and it is open to the bedrock. Average annual water-level fluctuation in the well is about 5 ft. More important, however, is the stable, average annual water level shown during the period of record.

Water levels in Geological Survey and domestic wells screened in the bedrock and in the sand and gravel aquifers throughout the basin also indicate that water levels do not change significantly. Water levels in 28 wells were measured during at least two of the three autumns in 1976, 1977, and 1978 for comparison. The deviation in water levels from autumn to autumn ranged from 0.1 to 4.8 ft and averaged 2.2 ft. The average does not represent maximum annual water-level deviation because all measurements were made in the autumn. However, these data further support the conclusion that the long-term water levels remain nearly constant. Therefore, although ground-water levels fluctuate in response to seasonal variations in recharge, the ground-water system is in dynamic equilibrium and, thus, approximates a steady state.

Ground-Water Seepage to Streams

Streams are generally recipients of ground-water discharge. Some indication of the importance of the stream as a discharge area can be seen from a gain and loss study. Such a study was done October 3-4, 1978, when streamflow duration was about 70 percent. Any increase in streamflow within a section of stream at 70-percent flow duration is probably contributed by ground-water seepage, after correction for three man-induced surface inflows. Flow October 3-4 was 1.2 ft^3/s in the White River just upstream from Winchester and 14 ft^3/s upstream from the confluence of Stoney Creek; and 6.3 ft^3/s in Stoney Creek, 4.6 ft^3/s in Cabin Creek, and 0.4 ft^3/s in Sugar Creek at their confluences with the White River. The sections along which the study was made are shown in figure 17.

A ±5-percent error was assumed in all the discharge measurements of October 3-4, 1978. Minimum and maximum rates of seepage for each reach of stream are given in table 2. The ±5-percent error in individual measurements can create many possible combinations of discharges between sites and at tributaries. Therefore the range in rates of seepage in table 2 can exceed the ±5 percent error of individual measurements. Several of the rates are estimated (footnote c, table 2). The rates for these reaches were determined by decreasing the rate in proportion to the length of reach between measurement sites within the modeled area (fig. 17). On the basis of the ±5-percent error, ground-water seepage to streams at approximately 70-percent flow duration October 3-4, 1978, was estimated to range from 17.0 to 23.5 ft^3/s.

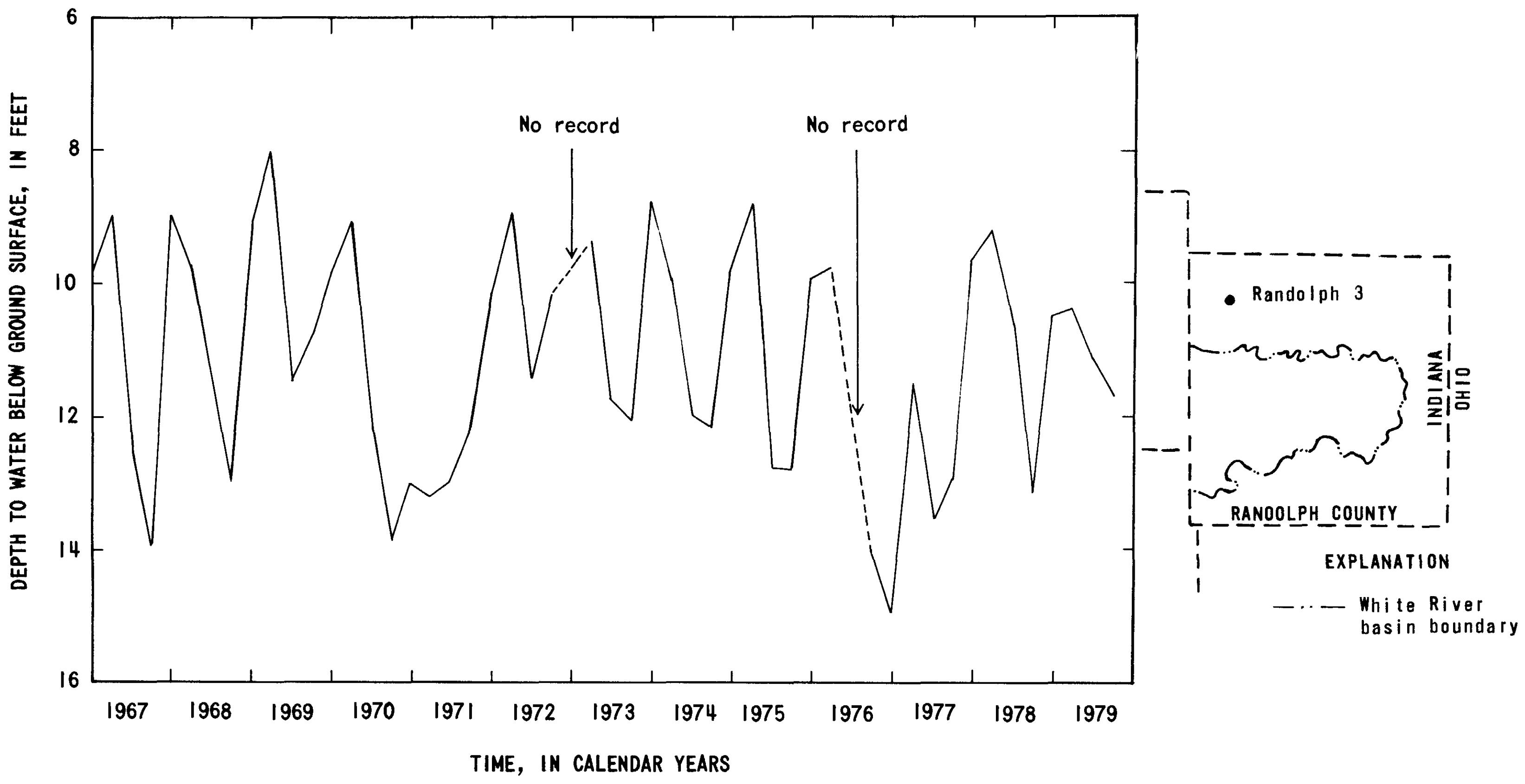


Figure 16.-- Water-level fluctuation in observation well Randolph 3.

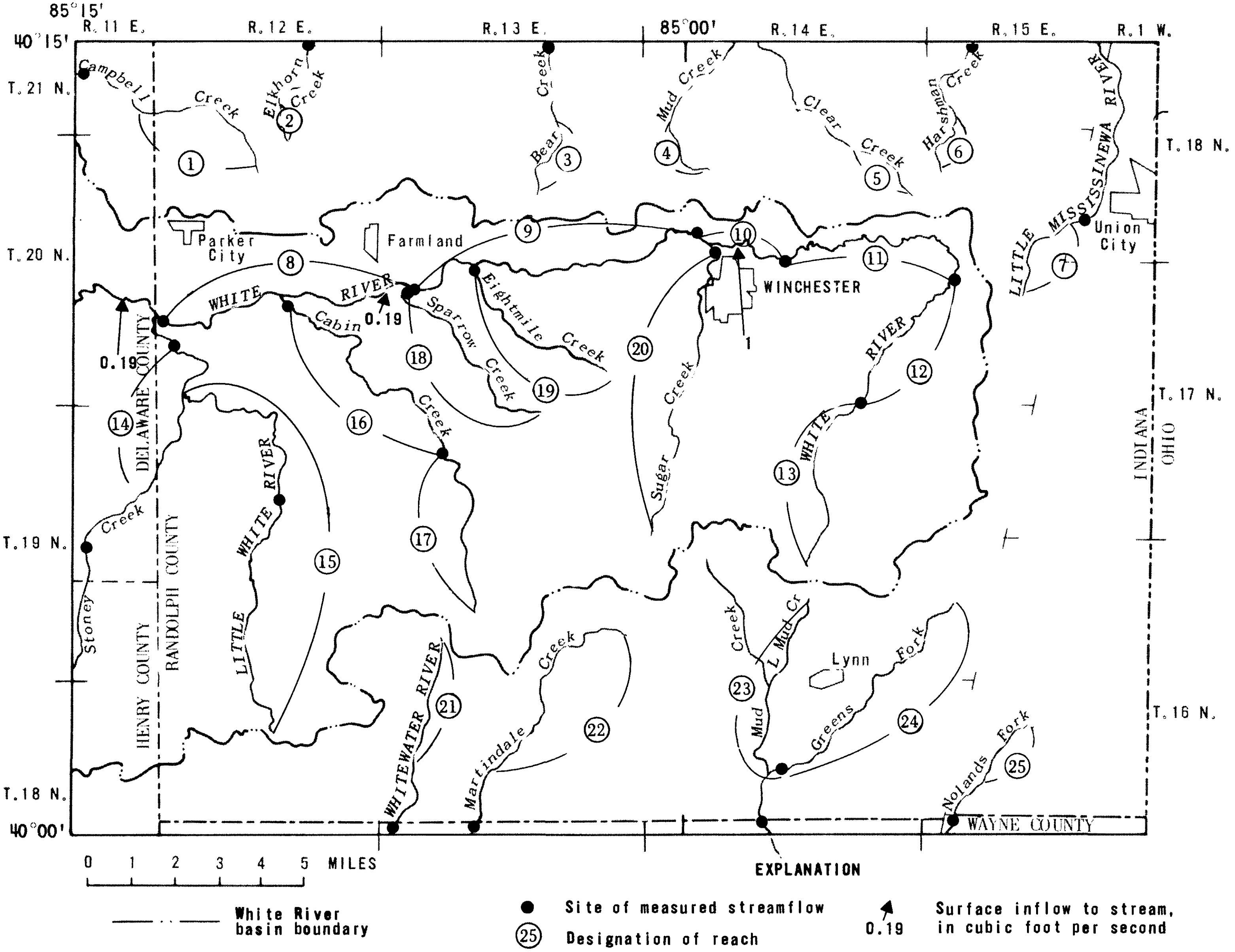


Figure 17.-- Reaches where ground-water-seepage rates were determined October 3-4, 1978.

Table 2.--Minimum and maximum rates of ground-water seepage to reaches at about 70-percent flow duration October 3-4, 1978

[For location of stream sections, see figure 17. Negative number indicates seepage into the ground-water system from the stream.]

Name of stream	Reach	Rates of seepage (ft^3/s)	
		Minimum	Maximum
Campbell Creek	1	[a]0.15	[a]0.15
Elkhorn Creek	2	[a].1	[a].1
Bear Creek	3	[a].05	[a].05
Mud Creek	4	[a].1	[a].1
Clear Creek	5	[a].05	[a].05
Harshman Creek	6	[a].05	[a].05
Little Mississinewa River	7	[a].05	[a].05
White River	8	.35	2.90
Do.	9	-.13	1.13
Do.	10	3.1	3.8
Do.	11	-.18	.05
Do.	12	-.05	.19
Do.	13	1.1	1.2
Stoney Creek	14	[a]2	[a]2
Little White River	15	[a]1.5	[a]1.5
Cabin Creek	16	1.9	2.7
Do.	17	2.2	2.5
Sparrow Creek	18	.50	.56
Eightmile Creek	19	.47	.50
Sugar Creek	20	.38	.42
Whitewater River	21	[a]1.0	[a]1.0
Martindale Creek	22	[a].6	[a].6
Mud Creek and Little Mud Creek	23	1.42	1.58
Greens Fork	24	.15	.17
Nolands Fork	25	[a].1	[a].1
Total		17.0	23.5

[a]Estimated.

Ground-Water Pumpage

Because pumpage can be a significant part of ground-water discharge in an area, historical and current pumpages were obtained from water companies. At the same time, historical and current static and pumping water levels in and near pumped wells also were obtained; however, few data were available. In determining pumpage, any pumpage less than 0.1 Mgal/d (0.16 ft^3/s) was not considered unless it was part of local pumpage totaling more than 0.1 Mgal/d (0.16 ft^3/s). The rates and the locations of significant pumpage for 1977 are shown in figure 18.

Pumpage during 1977 was 0.85 Mgal/d (1.3 ft^3/s). This rate is probably not significantly different from those for several years before 1977 or for 1978. Because the total pumpage was distributed throughout the study area and because it probably did not vary significantly during 1978 or several years earlier, the ground-water system was assumed to be in equilibrium with pumpage in 1978. This assumption is supported by observation of water level declines near a well field in drift in Madison County (Donald Davis, oral commun., 1978). At a pumping rate of 4 Mgal/d, water levels stabilized within about 2 years after the start of pumping. The effect of domestic pumping on the ground-water system is probably insignificant because the amount of water pumped is small and the wells are scattered.

The Governor's Water Resources Study Commission (1980, p. 269) reported that, as of 1975, the rate of ground-water withdrawal for public water supplies in Randolph County was 2.0 Mgal/d (3.1 ft^3/s). The pumping rate for the same public water supplies obtained in 1977 by the Geological Survey was 0.85 mgal/d (1.3 ft^3/s). The reason for the difference between the pumping rates is not known. Although the two rates differ, their effects on the ground-water flow system are not significant. Both rates of withdrawl by pumping are small compared to the rate of natural recharge to the ground-water system.

SIMULATION OF GROUND-WATER FLOW

Simplifying Assumptions Used To Simulate Ground-Water Flow in the Model

Simplifying assumptions on geometry, hydraulic properties, and other characteristics of the ground-water system were made for simulating ground-water flow with the model. These assumptions are necessary because the system cannot be simulated precisely. Nevertheless, the most important characteristics of the system can be modeled. A generalized geologic section of the study area that shows the design of the model is given in figure 19. The assumptions made in constructing the model are as follows:

1. Flow in the drift is quasi-three-dimensional, flow in the sand and gravel aquifers is horizontal, and flow (leakage) in the confining beds (till) between aquifers is vertical.

2. Flow in the bedrock is horizontal, except where the bedrock occupies more than one layer. (*See* the section "Selection, Design, and Construction of Model.")

3. The four confined sand and gravel aquifers are homogenous and horizontally isotropic, and their hydraulic conductivity is 433 ft/d.

4. The ratio of horizontal to vertical hydraulic conductivity of the bedrock aquifer is 1:1, the hydraulic conductivity is constant with depth, and the transmissivity is areally variable.

5. Only the upper 150 ft of the carbonate bedrock aquifer is permeable.

6. Streambeds are 1 ft thick and are composed of a material of lower vertical hydraulic conductivity than that of the sand and gravel aquifers.

7. Some minor streams are insignificant discharge points for the ground-water system and can be ignored. (This simplification eliminates some shallow ground-water circulation.)

8. The ground-water system is in steady state.

Selection, Design, and Construction of Model

The quasi-three-dimensional, finite-difference model of Trescott (1975) was used to simulate ground-water flow. The program of the model was altered slightly from the documented version so that a more realistic approximation of the ground-water flow system could be simulated. This alteration included the flexibility to simulate areal recharge directly to any layer in the model and to simulate streams in any layer.

The finite-difference grid used in modeling the area is shown in figure 20. The grid consists of 1,845 (41 x 45) grid blocks representing 390 mi^2. The active area of the grid, where ground-water flow is simulated, is 336 mi^2. Spacing in the grid ranges from 800 to 4,000 ft, and area of nodes ranges from 0.06 to 0.57 mi^2.

The area was divided into five layers (fig. 19) for modeling the ground-water flow. Layer 1 (the bottom layer) generally represents the bedrock aquifer. Layer 2 was used to simulate aquifer 1. Where aquifer 1 is absent, layer 2 simulates either bedrock or till. Layers 3 through 5 generally represent sand and gravel aquifers 2 through 4.

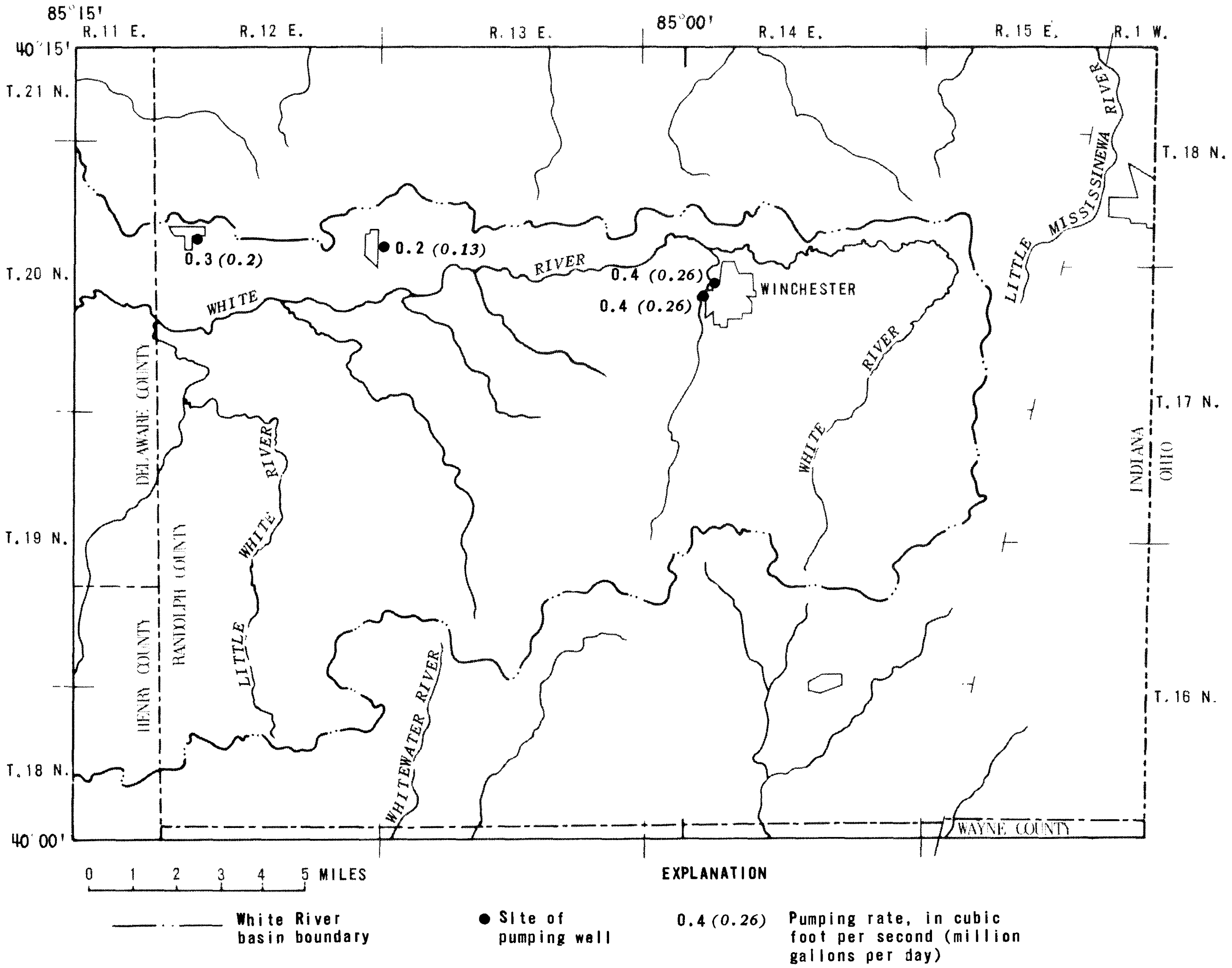


Figure 18.-- Sites of significant ground-water pumpage, 1977.

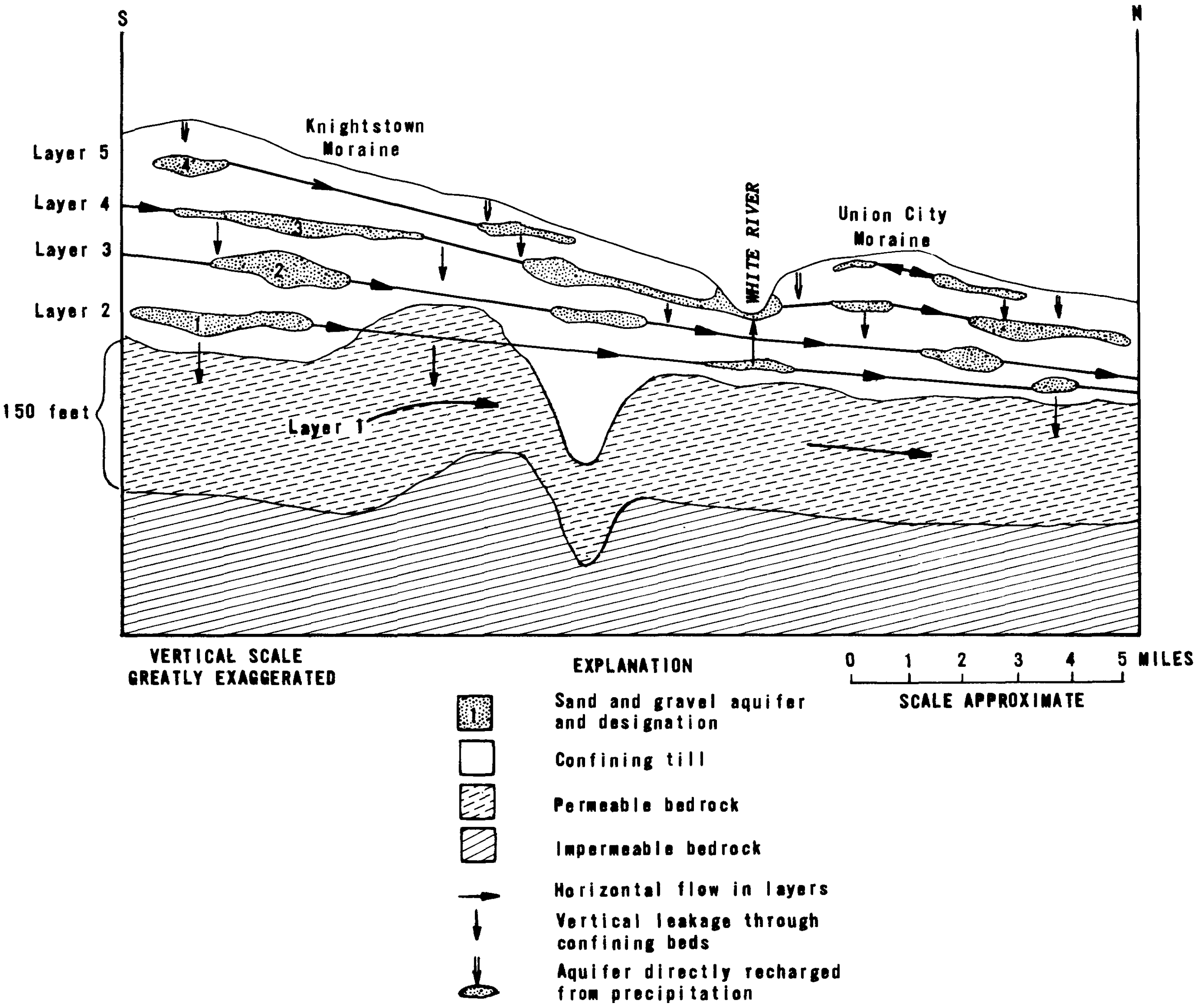


Figure 19.-- Generalized geologic section showing relations between geology and model design through Randolph County.

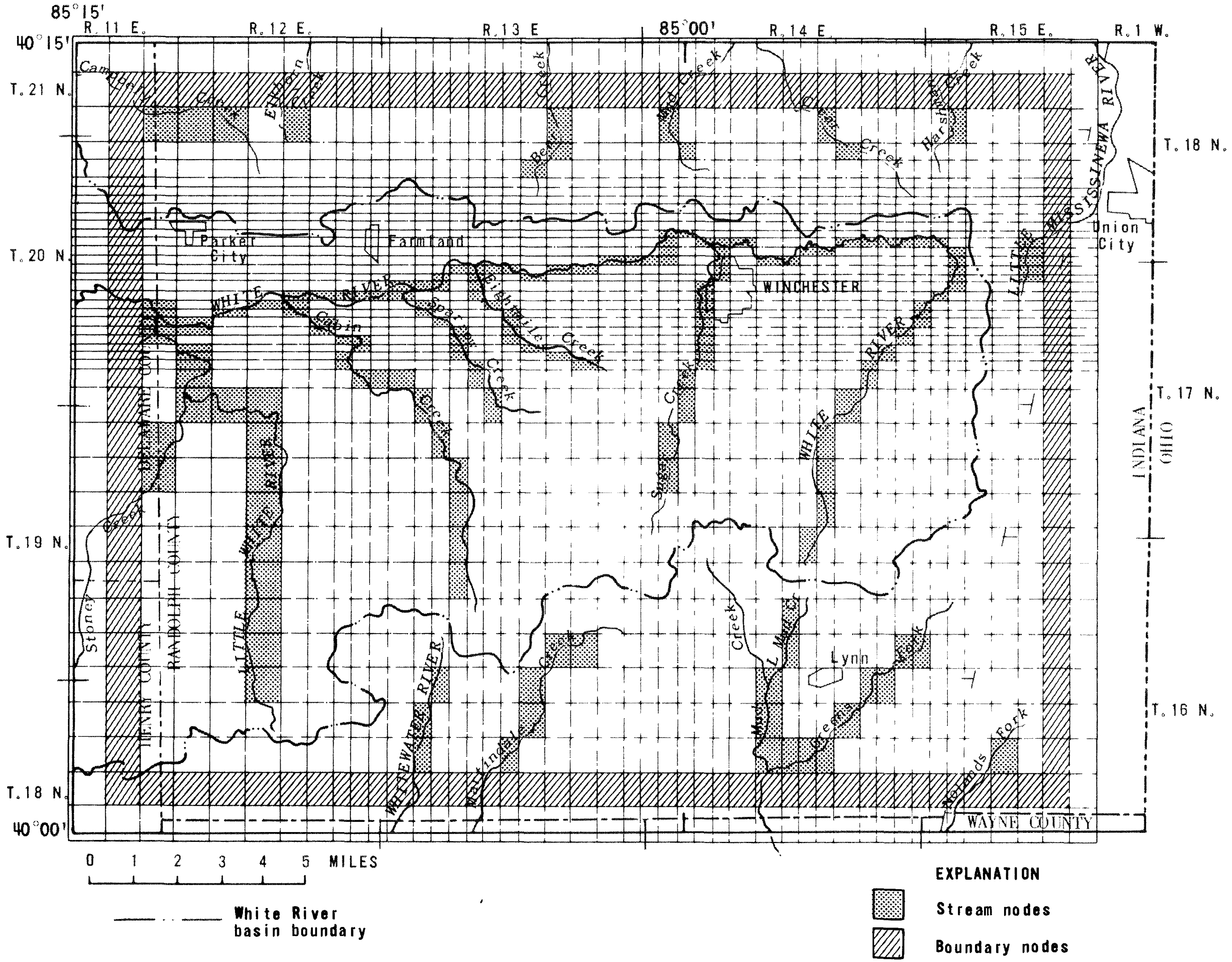


Figure 20.-- Finite-difference grid used to simulate ground-water flow.

The transmissivity of the bedrock for each node (center point of grid block) in the model in layer 1 was assigned by overlaying the grid on the bedrock transmissivity map (fig. 13) and estimating the average transmissivity within each grid block. Some of the upper parts of the 150 ft of permeable bedrock were laterally within one or more of the layers above layer 1 (fig. 19). For this condition, the total bedrock transmissivity (normally assigned completely to layer 1) was divided among each of the layers containing bedrock in proportion to the thickness of bedrock in each layer.

The technique of a grid overlay was also used to assign transmissivity for aquifers 1 to 4. However, the transmissivities could not be estimated directly. Figures 5, 7, 9, and 11 were used first to estimate the thickness of aquifers 1 through 4 for each node in layers 2 through 5. These thicknesses were then multiplied by 433 ft/d, the average hydraulic conductivity for the confined sand and gravel aquifers, to calculate the appropriate transmissivity.

Because the four sand and gravel aquifers are areally discontinuous, some areas in layers 2 to 5 consist of till or bedrock, rather than sand and gravel (fig. 19). In these areas, transmissivities were assigned as follows: For till, a transmissivity of 2.8 ft^2/d was assigned. This transmissivity was based on a hydraulic conductivity of till equal to 0.14 ft/d [about (1 gal/day)/ft^2]. The value within the range of hydraulic conductivity for till given by Freeze and Cherry (1979, p. 29, table 2.2) and Todd (1959, p. 53, table 3.4), is assumed to represent the average hydraulic conductivity of the till in the study area. The 0.14 ft/d was multiplied by 20 ft, the average thickness of till separating the laterally discontinuous sand and gravel aquifers. This procedure provided a small but finite transmissive connection laterally between the discontinuous parts of each aquifer. Transmissivities of bedrock were assigned as discussed in the second preceding paragaph. Zero transmissivities were imposed in each layer wherever the layer intercepted land surface.

Vertical flow in the ground-water system was simulated by allowing leakage between model layers. A leakage coefficient for each model node was calculated by first determining the thickness of the least permeable material (till, bedrock, or sand and gravel) between model layers. The thickness was then divided into the vertical hydraulic conductivity estimated for that material. The vertical hydraulic conductivity of the bedrock was calculated individually for each bedrock-leakage coefficient. This calculation was done by first assuming that the ratio of vertical to horizontal hydraulic conductivity of the bedrock was 1:1. The vertical hydraulic conductivity was then calculated by estimating the average bedrock transmissivity at each node (fig. 13) and dividing that transmissivity by the assumed thickness of permeable bedrock, 150 ft. An estimate of the vertical hydraulic conductivity of sand and gravel was based on the assumption that the ratio of horizontal to vertical hydraulic conductivity of the sand and gravel was 10:1. Because the average horizontal hydraulic conductivity of the sand and gravel in the study area was estimated to be 433 ft/d, the resulting estimate of the vertical hydraulic conductivity of the sand and gravel was 43 ft/d. The estimate of the vertical hydraulic conductivity of the till, 7×10^{-4} ft/d, was the average of the range of vertical hydraulic conductivity for the confining beds reported by Meyer and others (1975, p. 26).

Streams were modeled (fig. 20) to simulate the areal distribution of ground-water discharge. All streambeds were modeled as leakage boundaries. The head in the stream, held as a constant, was equal to the stream-surface elevation measured October 3-4, 1978.

Leakage coefficients for the streambeds were determined by dividing an assumed thickness of streambed, 1 ft, into the assumed vertical hydraulic conductivity of the streambeds, 4×10^{-2} ft/d. The vertical hydraulic conductivity was the average of a range reported by Meyer and others (1975, p. 19) for a clay lense in the outwash aquifer. Because the clay had a vertical conductivity two orders of magnitude times that of the till, the clay layers possibly originated as a less compacted fluvial sediment. Therefore, as a first estimate, the hydraulic properties of the streambed material, also a fluvial sediment, were assumed to be similar to that of the clay layer.

The rate of ground-water pumpage simulated as part of the steady-state ground-water system was 0.85 Mgal/d (1.3 ft^3/s), distributed as shown in figure 18.

Potentiometric heads at the boundaries of all five layers of the model were assumed to be constant. Potentiometric head at each boundary node in each layer was assigned on the basis of heads measured in each aquifer. Using these specified heads, the model computed ground-water flow across its boundaries, either into or out of the modeled area. Because this boundary flux at nodes representing till was very small, the parts of the boundaries at these nodes could probably have been modeled as no-flow boundaries. Almost all the boundary flux crosses at aquifers.

Calibration of Model

The steady-state-flow model was calibrated to a set of measured potentiometric heads and seepage rates to streams. The model was considered to be calibrated when model-simulated potentiometric heads and seepage rates matched corresponding, measured potentiometric heads and estimated seepage rates to an acceptable degree.

Model-simulated potentiometric heads were matched to heads measured in the study area during the summer and the autumn of 1978. Approximately 15 of the heads in the west one-fourth of the area, however, were measured during the summer of 1977 and were included with the heads measured in the summer of 1978. This inclusion was justified because, in that area, the average difference in head between measurements during the summers of 1977 and 1978 was only 1.7 ft for each of 12 other observation wells. At most the difference was less than 4 ft. Also, on the basis of the Marion County study by Meyer and others (1975, p. 48), the author expected an acceptable difference of as much as 15 ft between model-simulated and measured heads.

Model-simulated, ground-water-seepage rates to streams were matched to the seepage rates for various reaches of stream (table 2) estimated from discharge measurements October 3-4, 1978, when flow duration of all streams in the study area was about 70 percent. The author assumed that the potentiometric heads measured during the summer of 1978 were the potentiometric heads during the calibration period, October 3-4, 1978.

The calibration procedure consisted of changing the values of hydrologic variables in the model until model-simulated heads and seepage rates matched the measured values. The transmissivity distributions of the aquifers and the boundary heads were based on data collected in the field and, therefore, were well defined. However, vertical hydraulic conductivity of the till, effective areal recharge to the ground-water system, and the streambed-leakage coefficient were not based on field data and were least well defined. Therefore, changes during model calibration centered largely on changes in the last three variables.

Final calibrated vertical hydraulic conductivity of the till generally ranged from 10^{-4} to 10^{-2} ft/d. These data suggest that the variability in the vertical hydraulic conductivity of the till is probably large.

Effective recharge to the ground-water system in the calibrated model ranged from 0.04 to 0.4 ft/yr and averaged about 0.2 ft/yr. The range can probably be attributed to areal differences in the slope of the land surface and the vertical hydraulic conductivity of the till above the top aquifer. Both variables cause variation in infiltration rates. The effective recharge from precipitation represents the recharge to the regional flow system and does not include recharge that circulates in the shallow ground-water system and discharges locally to small streams that were not modeled. However, the amount of recharge that discharges locally as shallow ground-water circulation is probably insignificant relative to the amount of recharge that enters the regional flow.

A streambed thickness of 1 ft and a streambed vertical hydraulic conductivity of 4×10^{-2} ft/d were assumed in the first estimate of the vertical-leakage coefficient of the streambeds modeled. The latter assumption implies that all streambeds were composed of material having a low vertical hydraulic conductivity. Adjustments during calibration resulted in calibrated streambed hydraulic conductivities generally ranging from 7×10^{-4} to 0.9 ft/d for a streambed of 1-foot thickness.

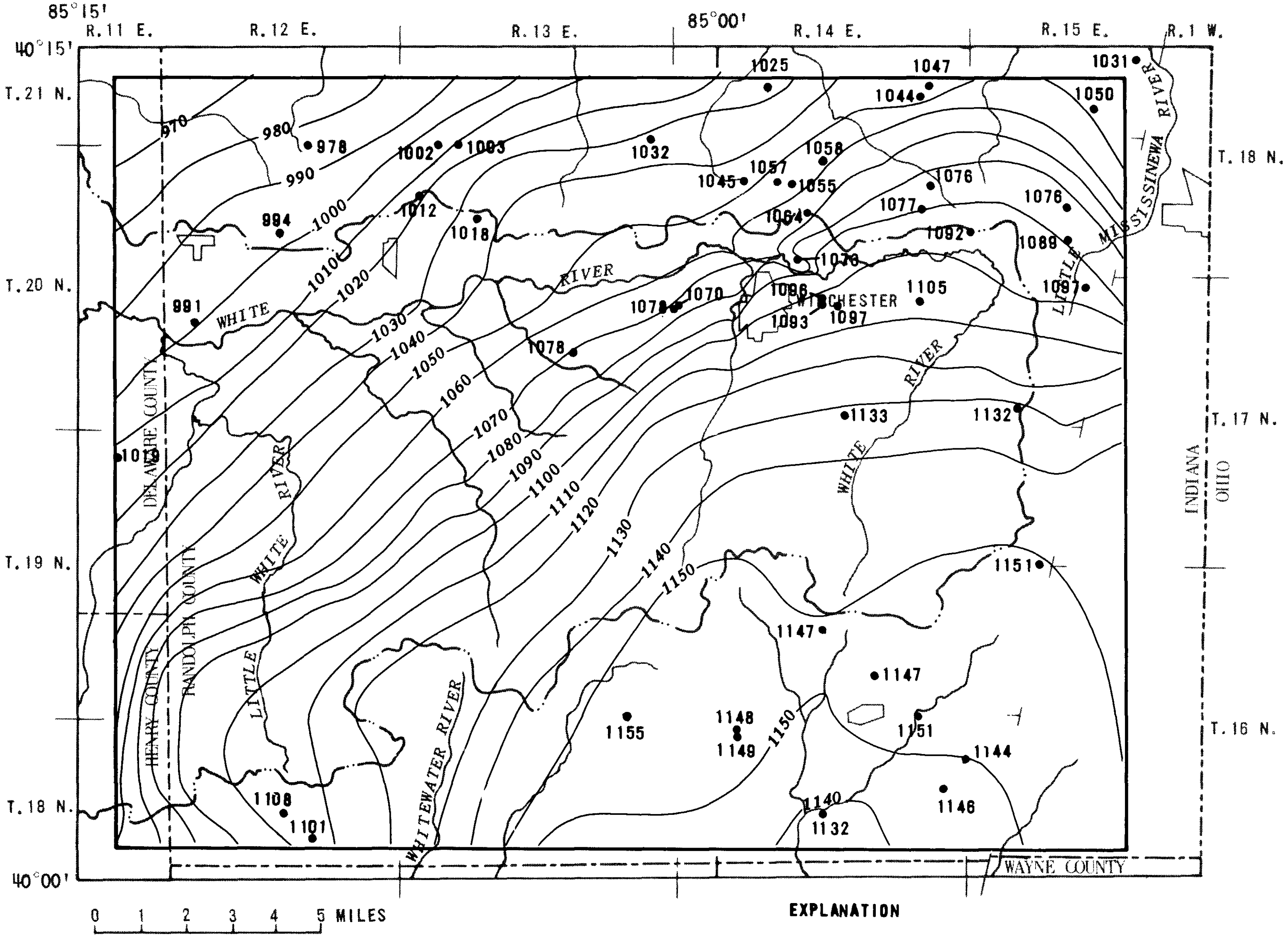
Final distributions of transmissivity for the bedrock and the sand and gravel aquifers after calibration differed only slightly from the original estimates. Because calculation of the transmissivities for the sand and gravel aquifers were based on an estimate of hydraulic conductivity, the modeled distributions are also estimates of the transmissivities of the aquifers at any point.

Model-simulated, steady-state potentiometric surfaces of the bedrock and the four sand and gravel aquifers are shown in figures 21 to 25. Generally, the difference between model-simulated heads and the corresponding measured heads for autumn 1978 was less than 10 ft. Because calibration of the model involved measuring and matching ground-water heads in the aquifers only, the reliability of simulated heads in areas that are not aquifer is not known.

The model-simulated ground-water-flow pattern in the aquifers suggests that most of the ground water discharges out of the study area rather than to streams in the study area. Comparison of figures 14 and 21 and of 15 and 24 indicates that model simulation of the ground-water flow regionally duplicates the observed pattern of ground-water flow.

Model-simulated and estimated seepage rates for streams in the modeled area are shown in table 3. With few exceptions, model-simulated rates were within ±5 percent of the estimated rates. The differences in rates in some sections are probably due to either some lack of knowledge of the ground-water system or the inability of the model to simulate the flow system in enough detail in those sections. However, these differences should not seriously limit the usefulness of the model in simulating the effect of future stresses on ground-water levels and streamflow.

The water budget for the calibrated model is shown in table 4. This tabulation represents the distribution of ground-water inflow and outflow for the period of calibration . Ninety percent of the total inflow to the ground-water system is effective areal recharge from precipitation and 10 percent is flow across boundaries. Of the total outflow, 2 percent is pumpage, 29 percent is seepage to streams, and 69 percent is flow across boundaries. Areal recharge to the till for an average base period was estimated from model results in Meyer and others (1975, p. 48) to be 2 in./yr in Marion County. This recharge is equivalent to 49.5 ft^3/s in the 336-mi^2 area modeled. Model-simulated recharge at 70-percent flow duration is 56.7 ft^3/s in Randolph County. This recharge is generally consistent with the recharge to the till for similar conditions in other areas of the basin.



EXPLANATION

White River basin boundary

970 — Line of equal potentiometric head. Interval 10 feet

Boundary of modeled area

960 — Well and measured potentiometric head, in in feet

Datum is sea level

Figure 21.-- Model-simulated, steady-state potentiometric surface of the bedrock.

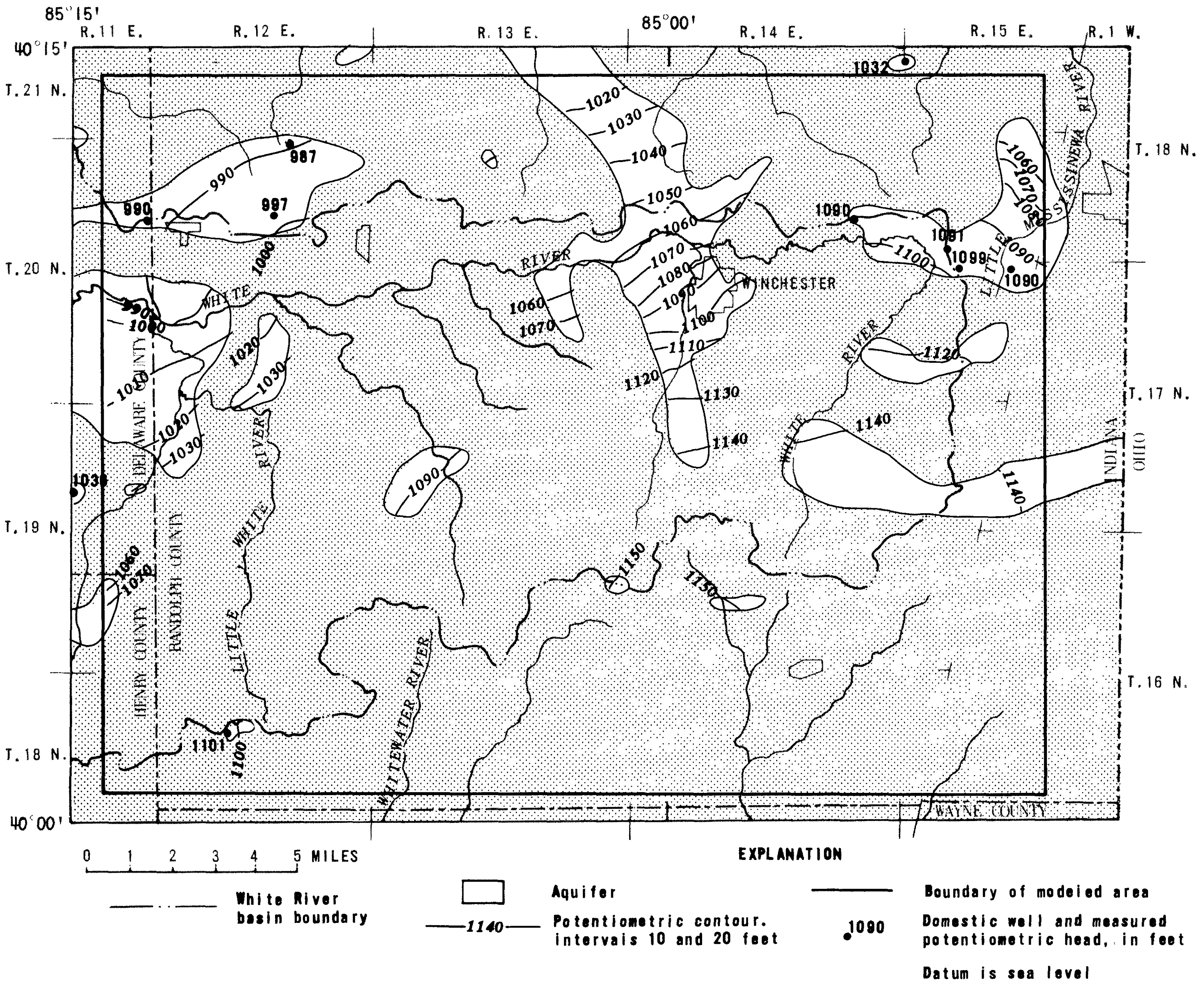


Figure 22.-- Model-simulated, steady-state potentiometric surface of aquifer 1.

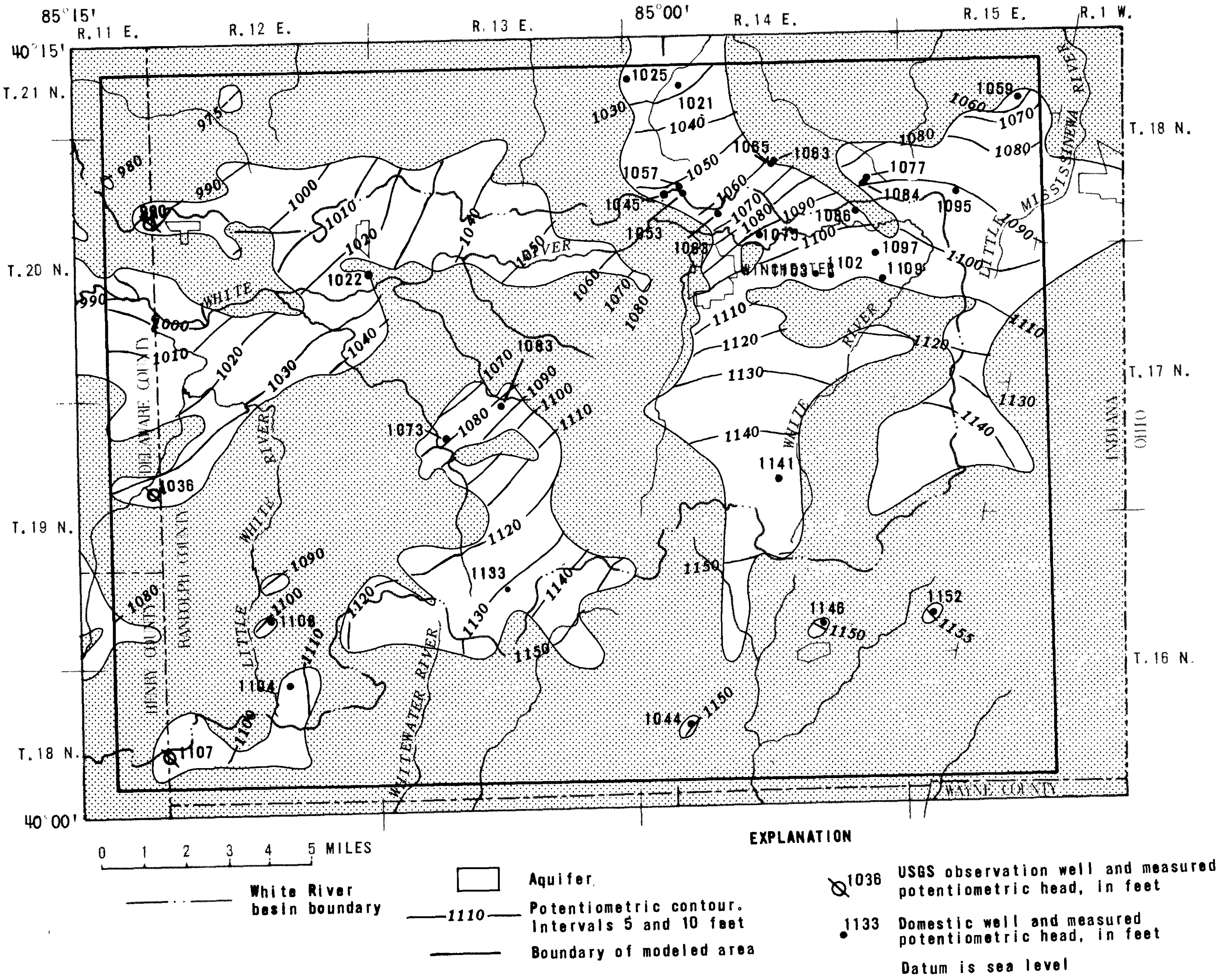


Figure 23.-- Model-simulated, steady-state potentiometric surface of aquifer 2.

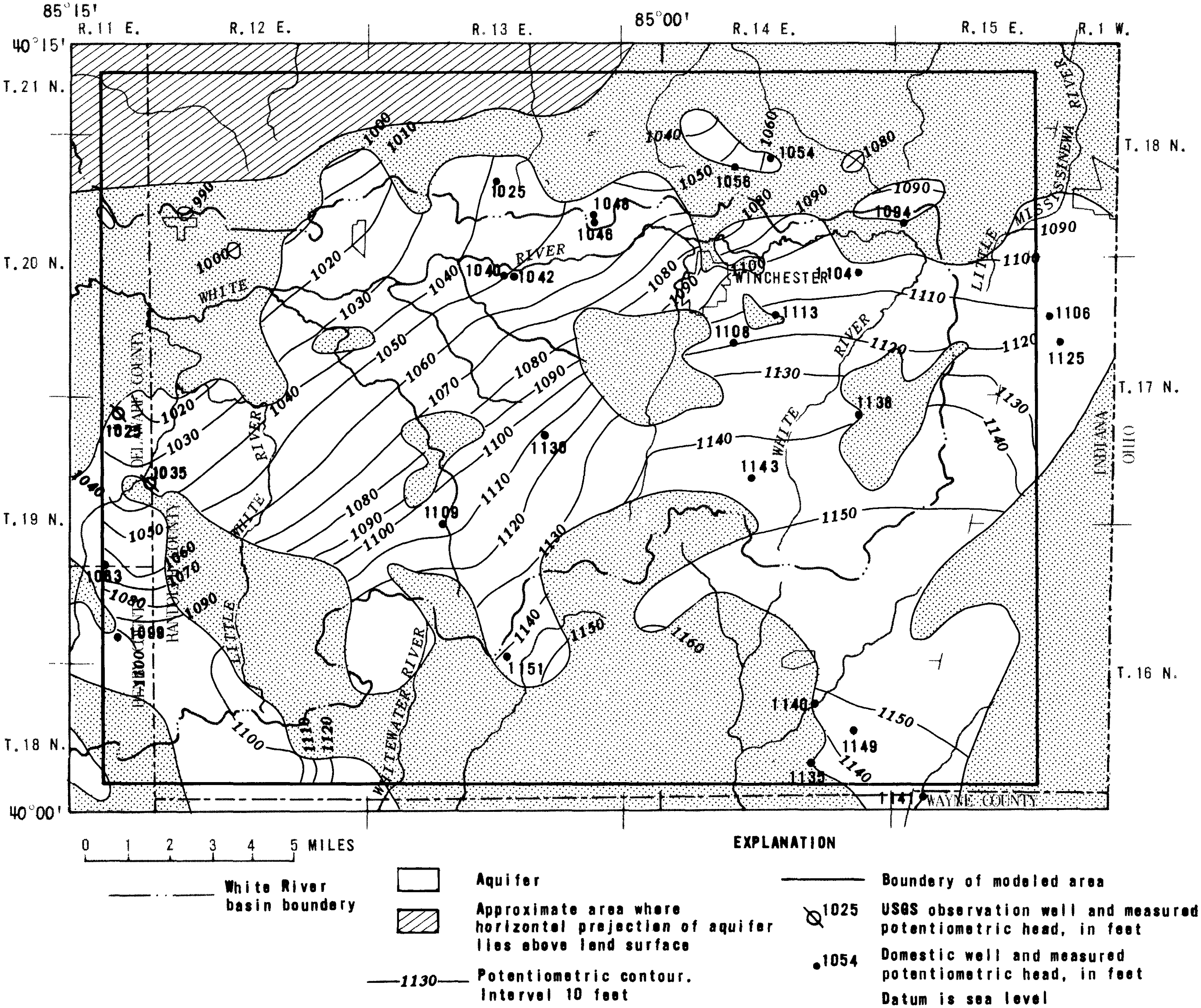


Figure 24.-- Model-simulated, steady-state potentiometric surface of aquifer 3.

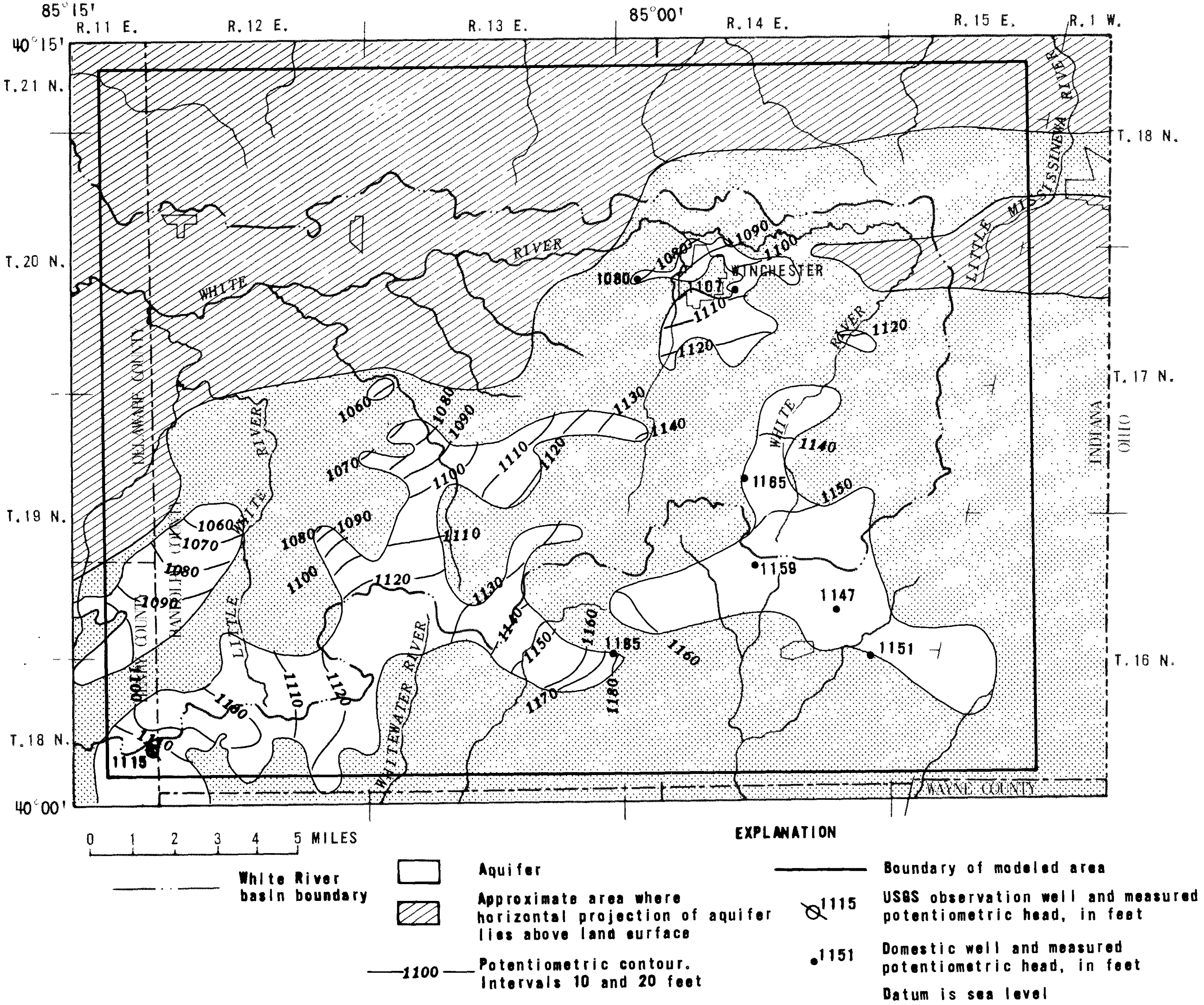


Figure 25.-- Model-simulated, steady-state potentiometric surface of aquifer 4.

Table 3.--Estimates and model-simulated rates of ground-water seepage for streams in Randolph County

[For location of reaches, see figure 17. Negative minimum indicates seepage into the ground-water system from the stream. A total of the model-simulated seepage in this table would not agree with ground-water seepage to streams in table 4 because of round-off error in the numbers of this table]

Stream	Reach	Estimates of seepage (ft^3/s)		Model-simulated seepage (ft^3/s)
		Minimum	Maximum	
Campbell Creek	1	0.15	0.15	0.13
Elkhorn Creek	2	.1	.1	.2
Bear Creek	3	.05	.05	.15
Mud Creek	4	.1	.1	.1
Clear Creek	5	.05	.05	.3
Harshman Creek	6	.05	.05	.15
Little Mississinewa River	7	.05	.05	.02
White River	8	.35	2.90	1.51
Do.	9	-.13	1.13	.91
Do.	10	3.1	3.8	.7
Do.	11	-.18	.05	.20
Do.	12	-.05	.19	.12
Do.	13	1.1	1.2	1.0
Stoney Creek	14	2	2	1.8
Little White River	15	1.5	1.5	1.4
Cabin Creek	16	1.9	2.7	2.3
Do.	17	2.2	2.5	2.2
Sparrow Creek	18	.50	.56	.52
Eightmile Creek	19	.47	.50	.40
Sugar Creek	20	.38	.42	.45
Whitewater River	21	1.0	1.0	1.3
Martindale Creek	22	.6	.6	.5
Mud Creek and Little Mud Creek	23	1.42	1.58	1.36
Greens Fork	24	.15	.17	.22
Nolands Fork	25	.1	.1	.05

Table 4.--Steady-state water budget from the calibrated model at 70-percent flow duration

	Inflow (ft^3/s)		Outflow (ft^3/s)
Effective recharge from precipitation	56.7	Ground-water pumpage	1.3
		Ground-water seepage to streams	18.5
Ground-water flow across model boundaries into study area	6.1	Ground-water flow across model boundaries out of study area	44.8
Total inflow	62.8	Total outflow	64.6

ASSESSMENT OF GROUND-WATER AVAILABILITY

The model was used to investigate the potential for additional ground-water development locally by assessing the effect of six pumping plans on water levels and streamflow (fig. 26). Results of these plans should indicate the feasability of pumping at specific rates from the two major aquifer systems--the sand and gravel aquifers (within the drift) and the bedrock aquifer. Pumping plans consist of 0.57-square mile pumping areas, each of which is shown by capital letters in figure 26. Although other locations have the potential for ground-water development that are probably equal to that of the locations investigated in plans A through F, the location chosen for each pumping plan should represent one of the areas of greatest potential for the particular aquifer system investigated.

Pumping plans A, B, and D through F simulated pumping from the sand and gravel aquifers: plans B and E from aquifer 2, plans A and D from aquifer 3, and plan E from aquifer 4.

Plan C was designed to investigate the potential for ground-water development in the bedrock. The results of this simulated pumping in an area of high transmissivity should provide a reasonable estimate of the maximum yield of a well field in the bedrock.

Two variations of pumping plans A through F were used to investigate the potential for ground-water development in the two major aquifer systems. First, pumping was simulated in each plan so that in all plans the same drawdown was maintained over an equal area. The rates of pumping and distributions of drawdown derived were then compared. Second, equal pumping was simulated in all the plans. The resulting distributions of drawdown for the plans were then compared. Both variations included model-simulated depletion of streamflow and percent depletion in streamflow at about 70-percent flow duration for each pumping simulation.

The model was designed to simulate stresses and to determine system response on a regional scale. The pumping plans are suited to the scale of the model because of the area of imposed drawdown and the amount of pumpage simulated by the plans. However, the model does not provide the needed detail of drawdown where a pumping plan has, for example, only one well that causes a drawdown cone that includes only one or two model nodes.

Model simulations in Randolph County may not be as accurate as those in the other three counties in the White River basin project area because fewer wells were available in Randolph County than in the other counties. Construction of the model in the eastern three-fourths of the county was based on lithologic information from commercial drillers' logs only. Geological Survey and contract driller logs supplemented the commercial driller logs in constructing the model in the western one-fourth of the county.

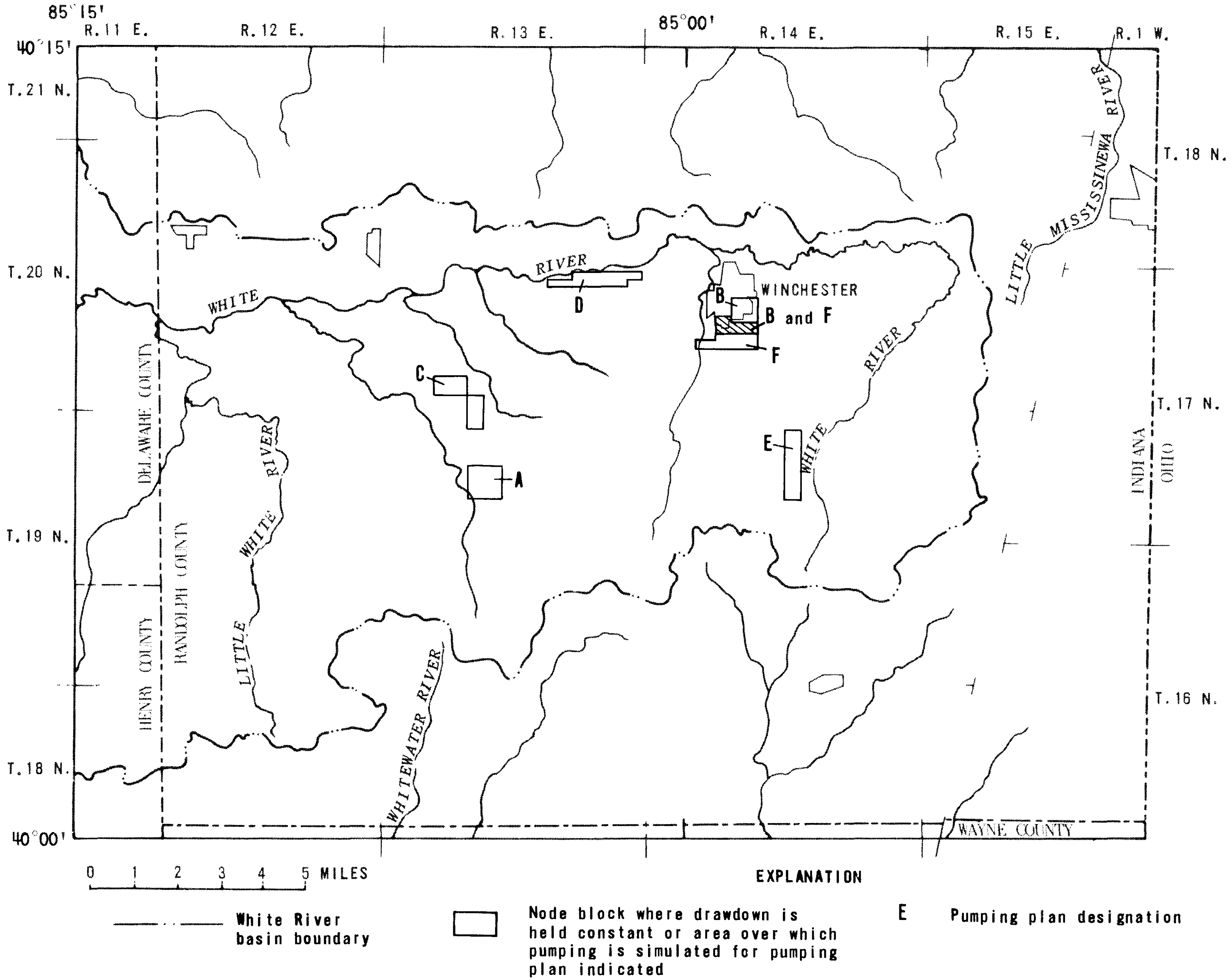


Figure 26.-- Locations of pumping plans A through F.

Constant-flux boundaries were simulated on the perimeter of all layers in the model, and all simulations were run to steady state. A constant-flux boundary tends to maximize water-level change and streamflow depletion attributable to pumpage by maintaining constant flow in or out of the model boundaries. Maintaining constant-flux boundaries results in conservative model predictions of water availability.

To ensure that the constant-flux boundary was not significantly affecting the results of the pumping simulations, the author simulated a constant-head boundary where necessary. If the water-level decline in a simulation using a constant-flux boundary exceeded 1 ft at the boundary, a simulation using constant-head boundaries was also run. A constant-head boundary tends to minimize water-level changes attributable to pumpage by maintaining zero drawdown at the model boundary. Therefore, a constant-head boundary may contribute an artificial source of water at that boundary. If the result of a simulation with a constant-flux boundary is virtually identical with a simulation with a constant-head boundary, the results of the simulations are not being affected by the model boundaries. In several simulations, either the pumping or the drawdown distribution between simulations for which the two boundries were used differed significantly. For these simulations, the differences are discussed and illustrated. Otherwise, only simulations using the constant-flux boundary condition are discussed and illustrated.

Running a pumping simulation to steady state produces the equilibrium response and thus the maximum water-level change and streamflow depletion attributable to the pumping because no water is derived from storage. Although no transient simulations were made, steady state is probably reached within 5 yr or less for the pumping rates simulated. This conclusion is based on the observed time for stabilization of water levels in and near a well field in Madison County. (See the section "Ground-water pumpage.") Water levels stabilized within approximately 2 yr after the beginning of 4-Mgal/d pumping.

Although the Madison County well-field drawdown should be fairly representative of how quickly the ground-water flow system in Randolph County stabilizes to the pumping simulations, specific factors control response at any one site. Water-level stabilization is a function of (1) distance to the boundary from where recharge is derived, (2) the hydraulic properties of the medium between the pumping sites and the source of recharge, and (3) the storage coefficient of the medium.

Model-simulated drawdown and streamflow depletions discussed in this report are caused only by the simulated pumping. The simulations do not account for any other stresses on the ground-water system within or near the modeled area except those simulated during model calibration. For instance, increased pumping of the ground-water system outside the modeled area may change the boundary fluxes simulated in the calibrated model, which, in turn, may alter the distribution of flow from that established during calibration.

For a given pumping plan, drawdowns in aquifers other than the aquifer or aquifers in which pumping was simulated are less than the drawdowns shown for that plan. Because model calibration involved measuring and matching ground-water heads in the aquifers only, how well the model simulates drawdown in areas not shown as aquifer is not known. The results of the model simulations should not be accepted as precise predictions of what will happen in the field, but rather a best estimate of what will happen.

The streams were modeled as leakage boundaries, and the heads in the streams were held constant. The streams were modeled so that the seepage could flow from the streams to the ground-water system. Therefore, during pumping simulations, the rate of inducement from a stream section to the ground-water system should not exceed flow in that section.

Simulations of Pumping Plans A through F with Uniform Drawdown

Pumping in plans A through F was limited to the rates that would cause an average drawdown of 20 ft in the 0.57 mi^2 where pumping was simulated. This limit was established to minimize the effect of pumping on nearby wells. The resulting distribution of drawdown in the aquifer pumped in the six plans (A through F) are shown in figures 27-37.

A constant-flux boundary resulted in drawdowns exceeding 1 ft at one or more of the model boundaries for all the pumping plans except plan A (fig. 27). For comparison, simulations with a constant-head boundary were also made and illustrated for pumping plans B-F. Results obtained for plan B are shown in figures 28 and 29 and for plans C, D, E, and F, in figures 30-31, 32-33, 34-35, and 36-37. The model-simulated pumping rates for the 6 plans are listed in table 5.

With a constant-flux boundary, the entire pumping stress is satisfied by reducing ground-water seepage to streams. With a constant-head boundary, probably too much of the pumping stress is satisfied by reduced outflow or increased inflow at the boundaries. Therefore, the actual distribution of drawdown, reduction in stream seepage, and simulated pumping for plans B-F are between the results obtained with the constant-flux and constant-head boundaries. The distribution of drawdown for pumping plan B is between the distributions shown in figures 28 and 29, and the pumping derived is between 1.9 and 2.8 Mgal/d (table 5).

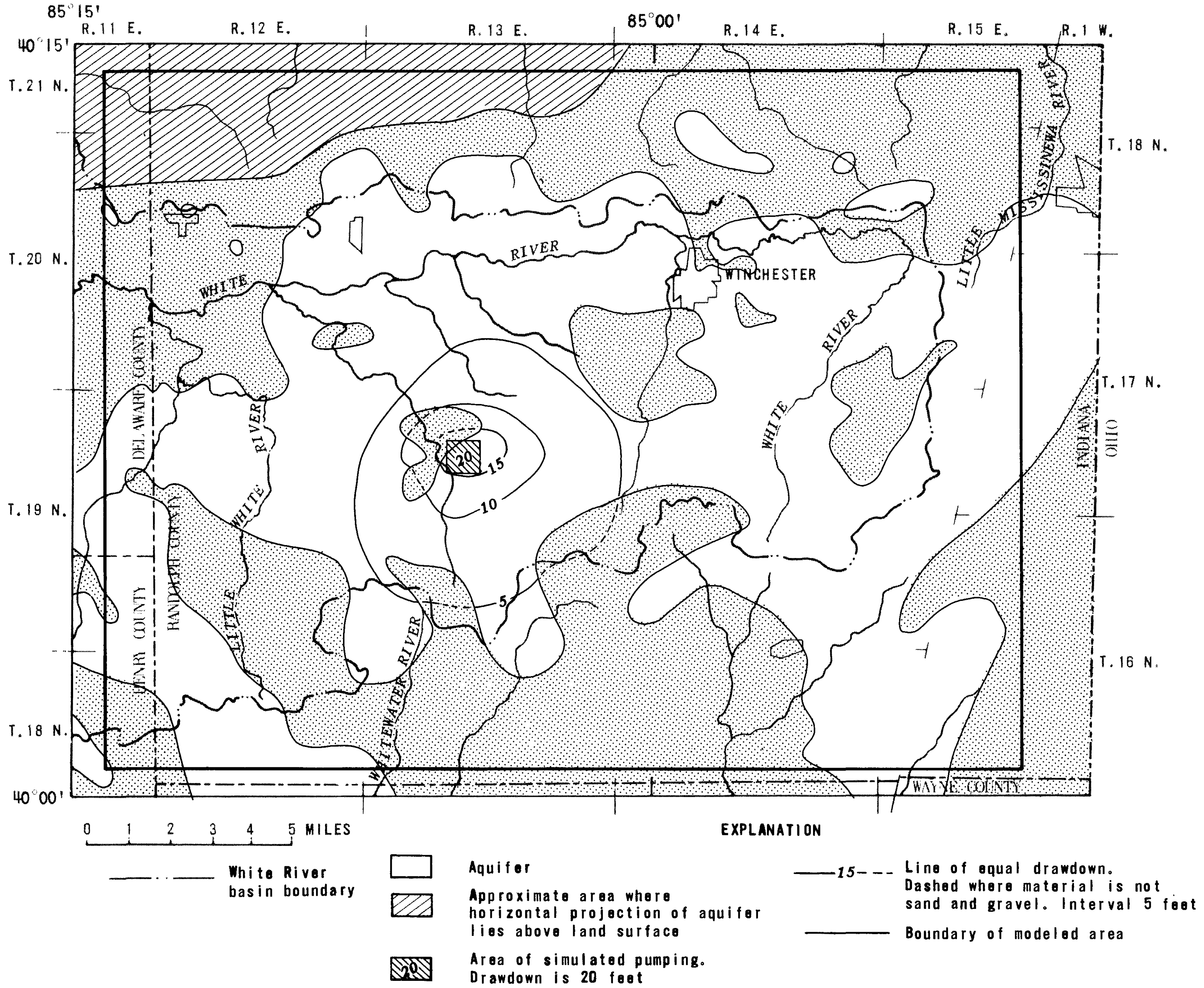


Figure 27.-- Model-simulated drawdown in aquifer 3 for pumping plan A and constant-flux boundaries. Simulated pumping is 2.1 million gallons per day.

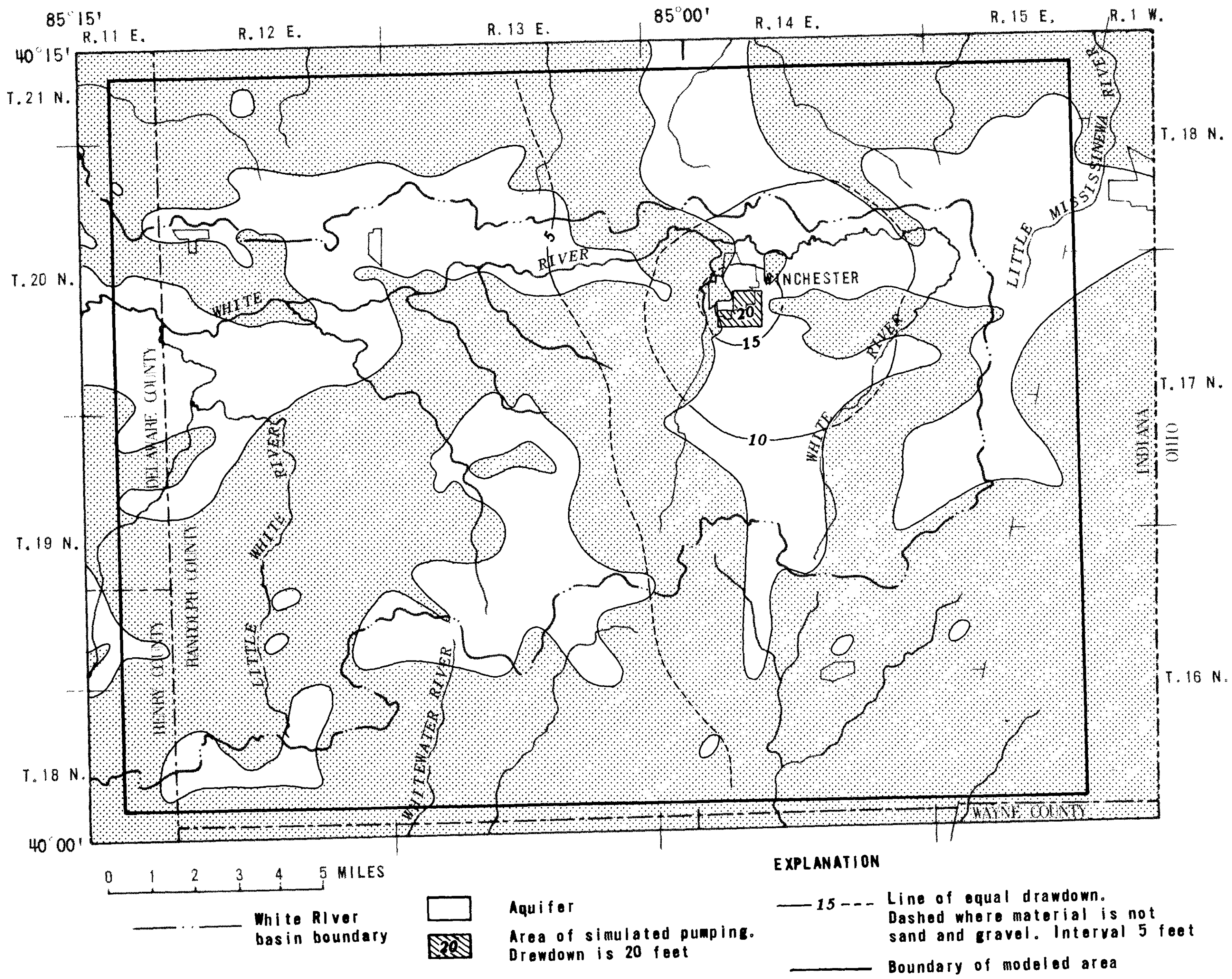


Figure 28.-- Model-simulated drawdown in aquifer 2 for pumping plan B and constant-flux boundaries. Simulated pumping is 1.9 million gallons per day.

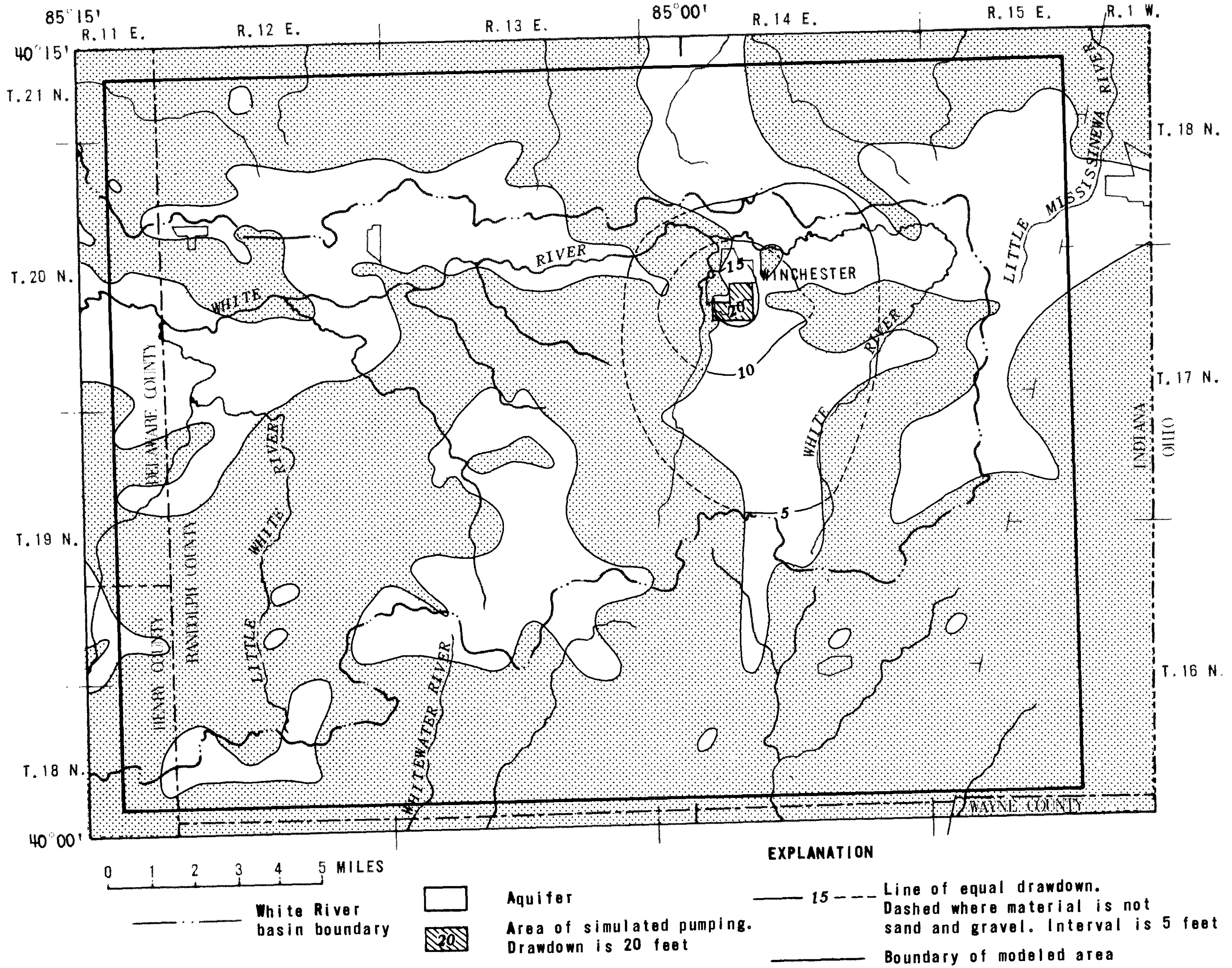


Figure 29.-- Model-simulated drawdown in aquifer 2 for pumping plan B and constant-head boundaries. Simulated pumping is 2.8 million gallons per day.

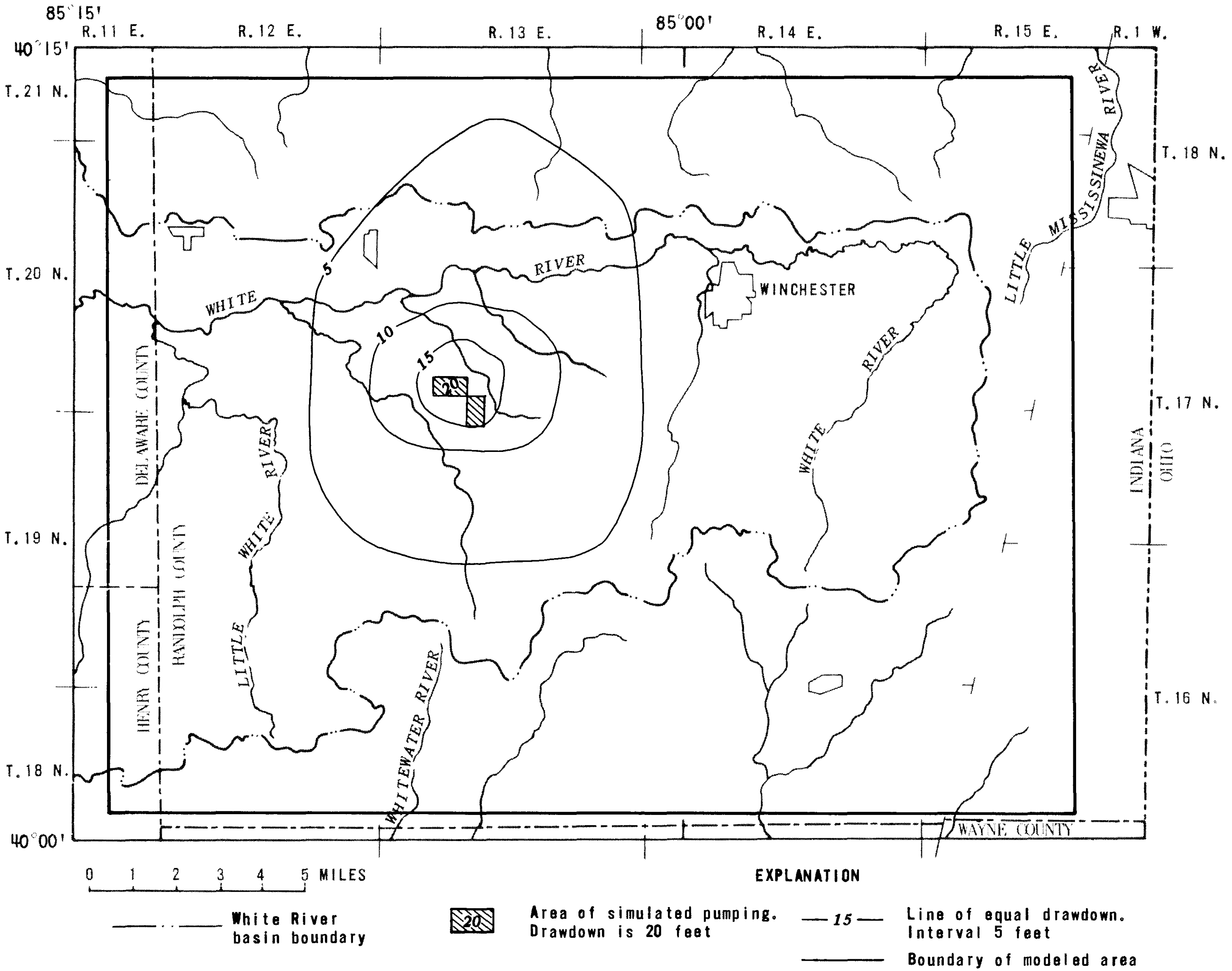


Figure 30.-- Model-simulated drawdown in the bedrock for pumping plan C and constant-flux boundaries. Simulated pumping is 3 million gallons per day.

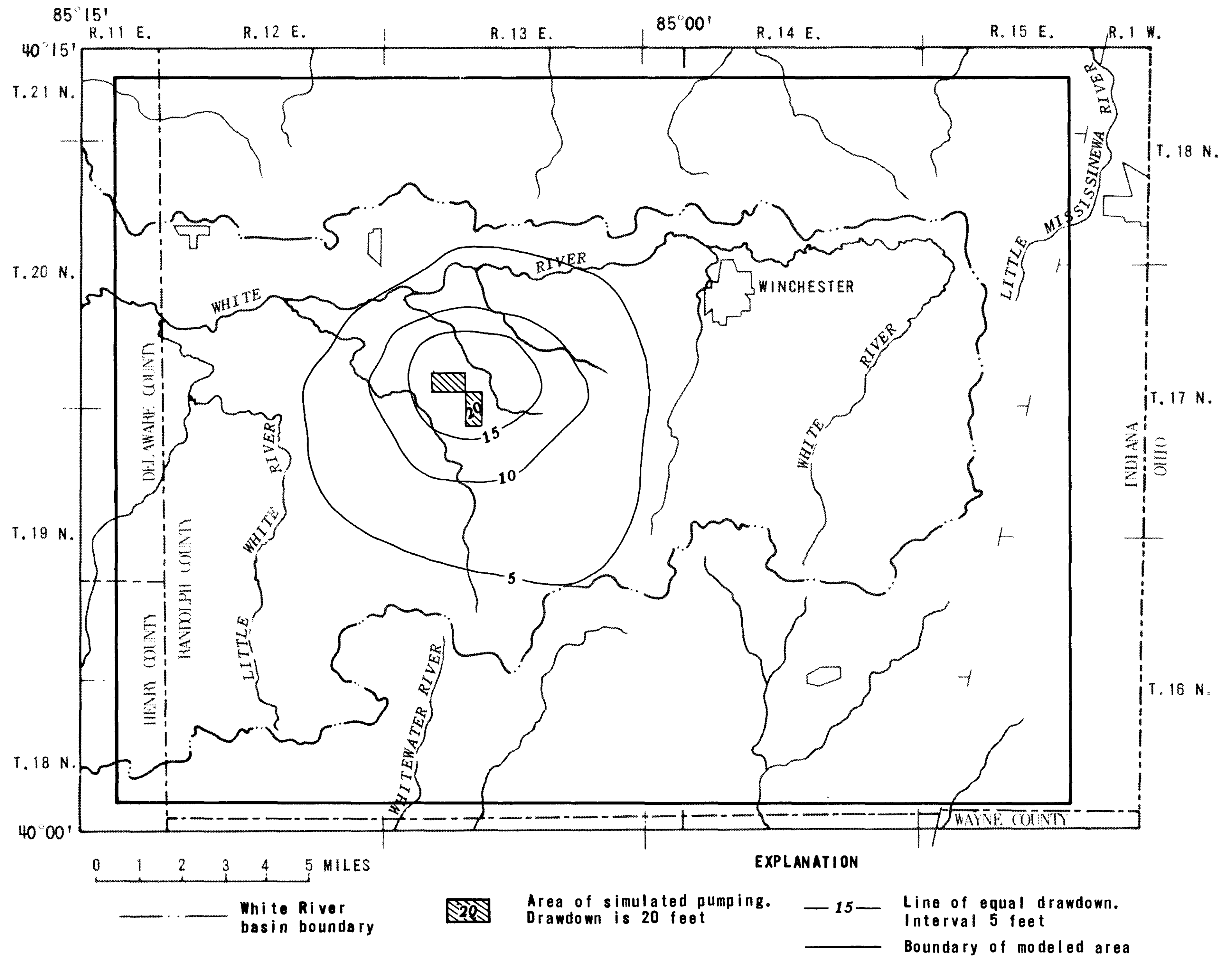


Figure 31.-- Model-simulated drawdown in the bedrock for pumping plan C and constant-head boundaries. Simulated pumping is 4.4 million gallons per day.

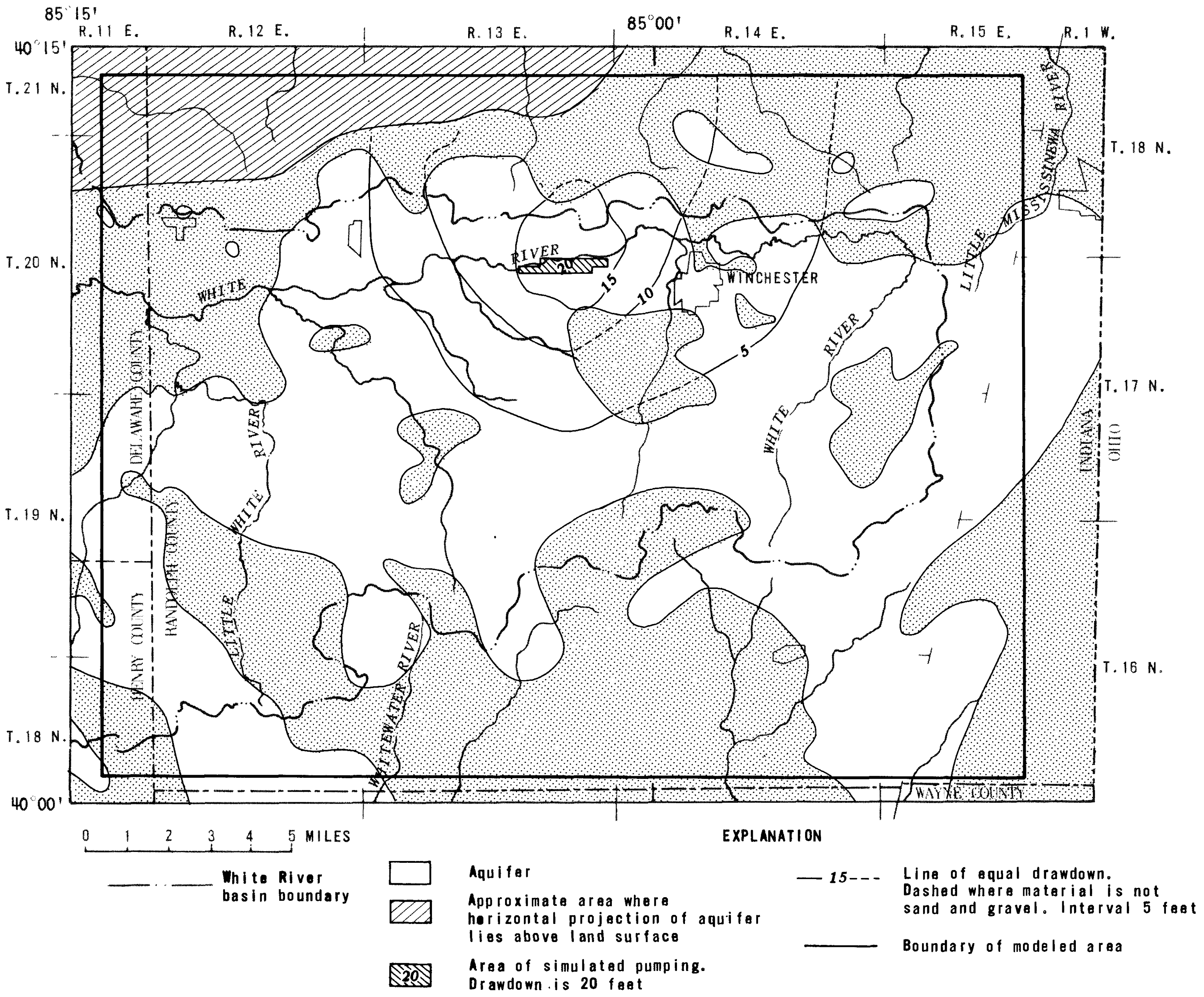


Figure 32.-- Model-simulated drawdown in aquifer 3 for pumping plan D and constant-flux boundaries. Simulated pumping is 2.4 million gallons per day.

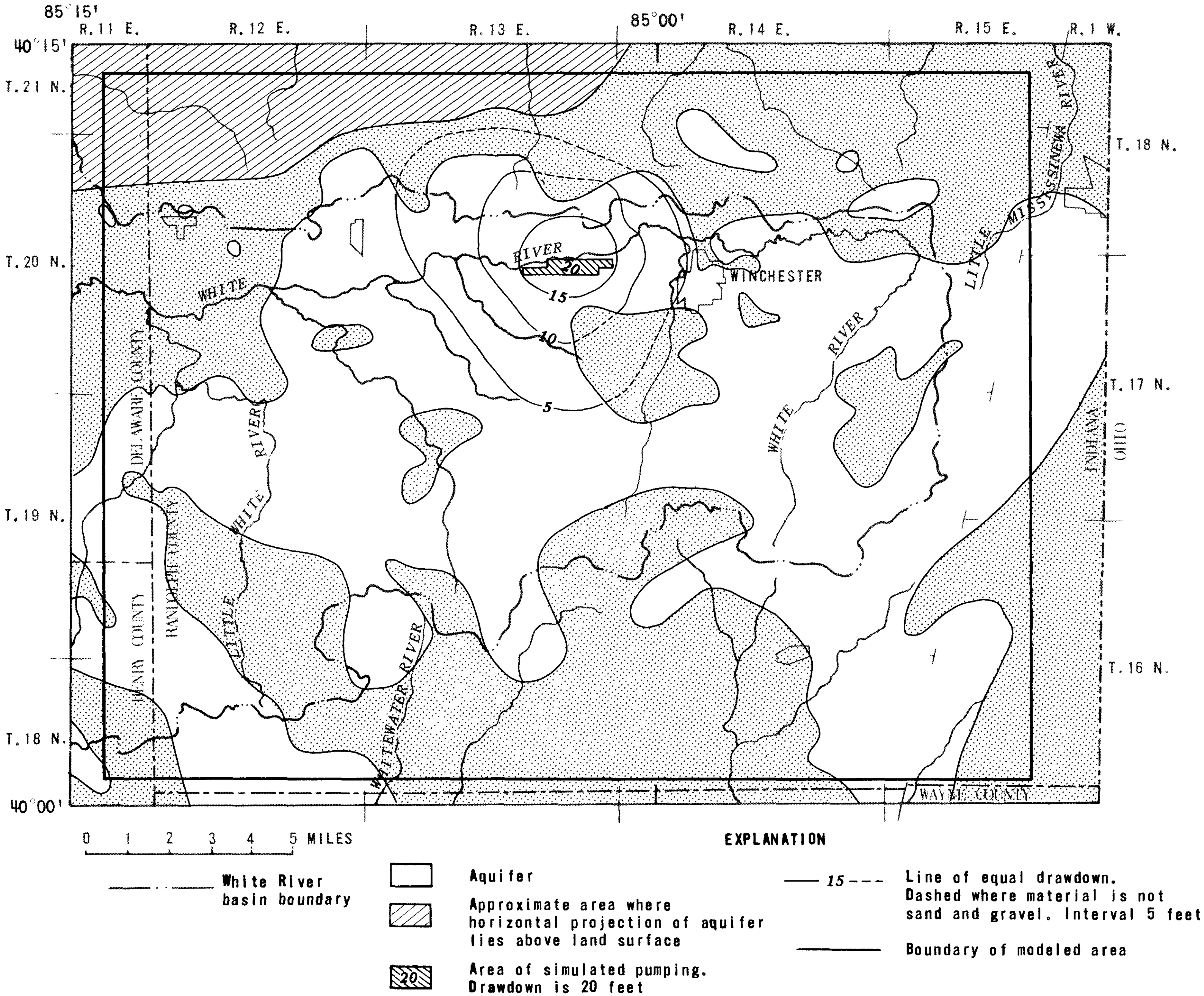


Figure 33.-- Model-simulated drawdown in aquifer 3 for pumping plan D and constant-head boundaries. Simulated pumping is 3.3 million gallons per day.

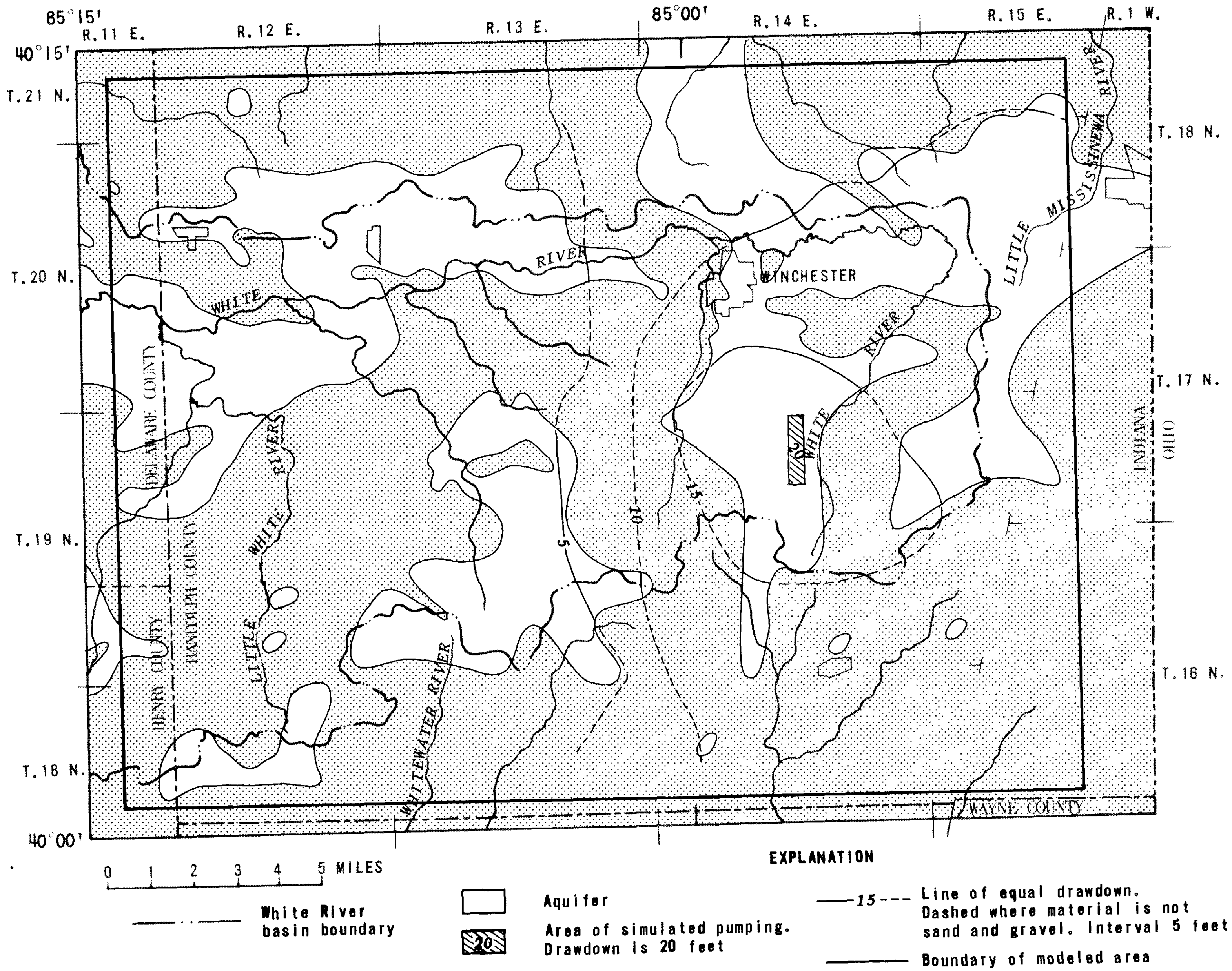


Figure 34.-- Model-simulated drawdown in aquifer 2 for pumping plan E and constant-flux boundaries. Simulated pumping is 2.1 million gallons per day.

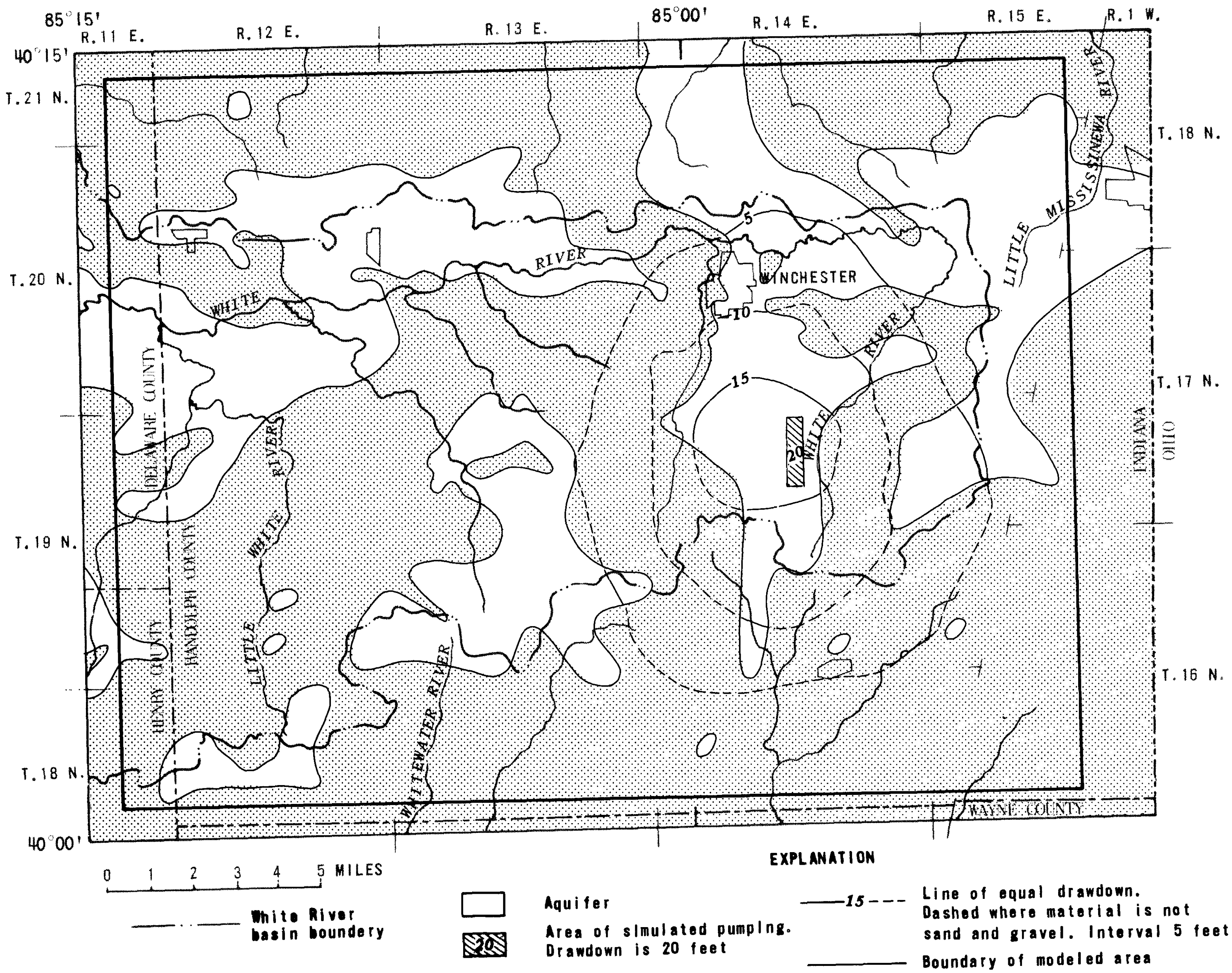


Figure 35.-- Model-simulated drawdown in aquifer 2 for pumping plan E and constant-head boundaries. Simulated pumping is 3.3 million gallons per day.

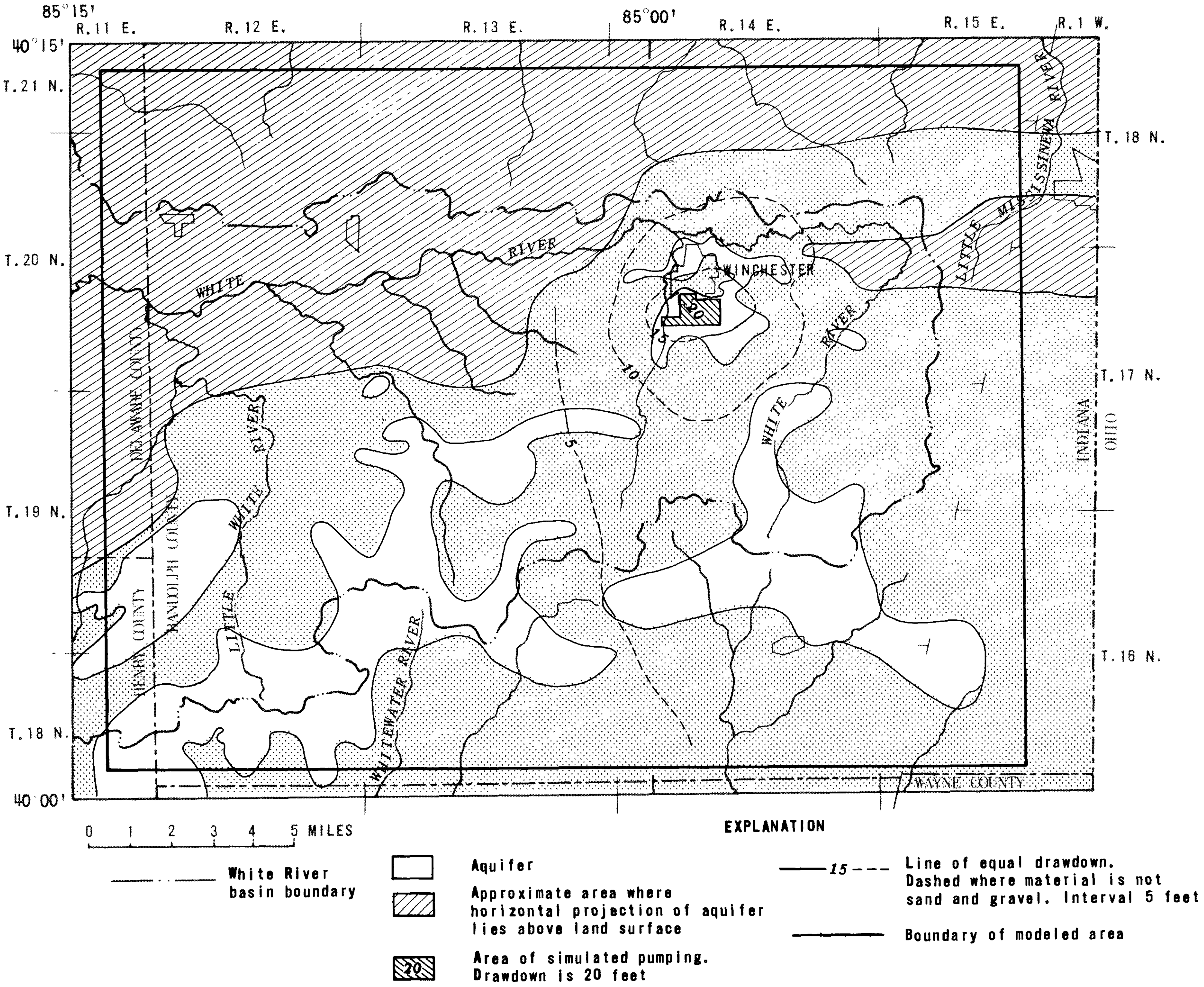


Figure 36.-- Model-simulated drawdown in aquifer 4 for pumping plan F and constant-flux boundaries. Simulated pumping is 1.8 million gallons per day.

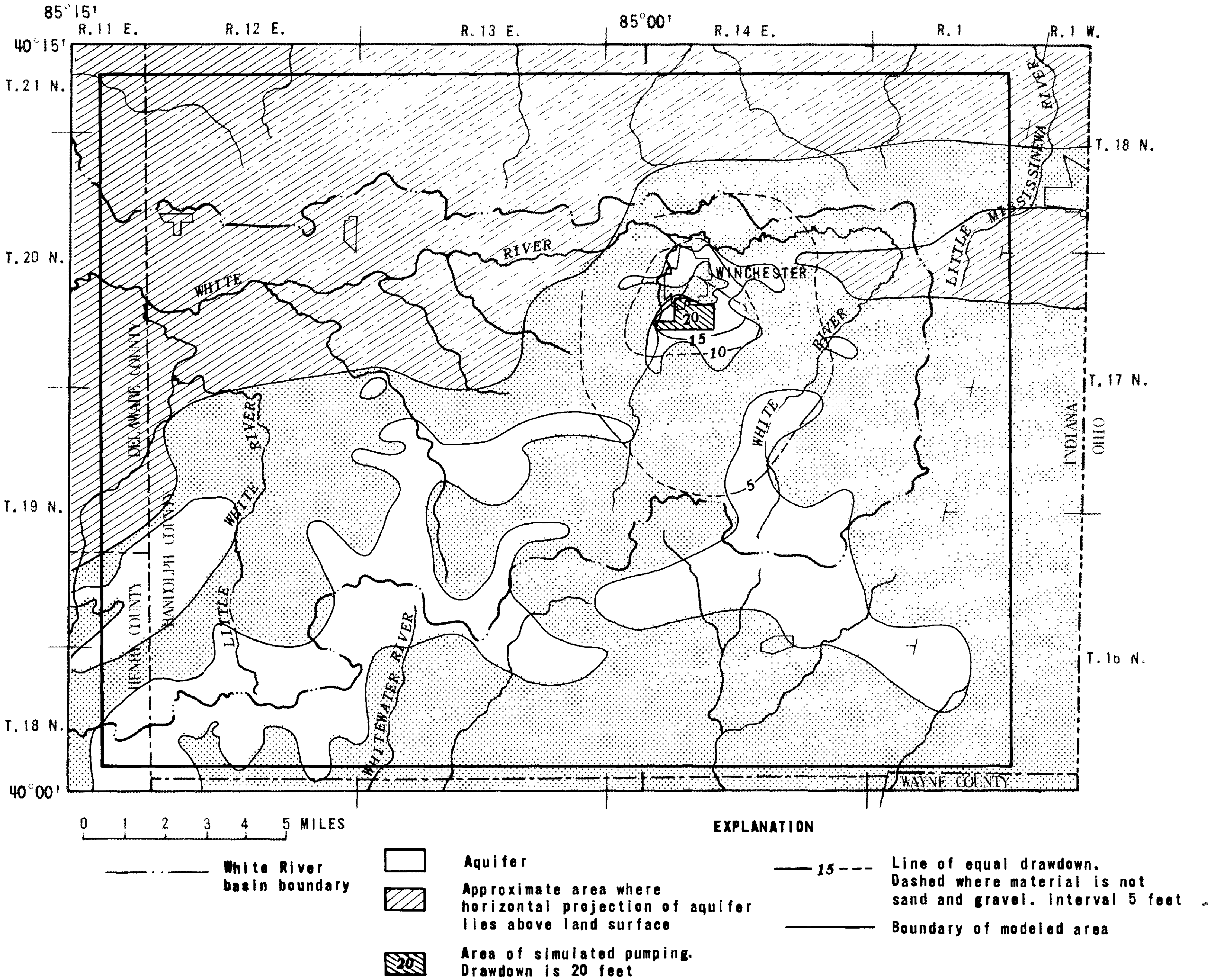


Figure 37.-- Model-simulated drawdown in aquifer 4 for pumping plan F and constant-head boundaries. Simulated pumping is 2.4 million gallons per day.

Table 5.--Pumping rates of plans A through F needed to simulate a 20-foot drawdown in 0.57 square mile for constant-flux and constant-head boundaries, Randolph County

[CF, constant-flux boundary; CH, constant-head boundary; ft^3/s, cubic foot per second Mgal/d, million gallons per day; Cr, creek; R, river. Model-simulated ground-water seepage to reach: From calibrated steady state model. (See table 3.) Locations of reaches are shown in figure 17. Streamflow at 70 percent flow duration based on discharge measurements October 3-4, 1978. Streamflow does not agree with the cummulative ground-water seepage of all reaches upstream from point of measurement because some small tributary inflows have not been included.]

Pumping plan	Model-simulated pumping (ft^3/s) / (Mgal/d)		Reach most affected by pumping	Stream	Model-simulated ground-water seepage to reach (ft^3/s)	Approximate flow at downstream end of reach at about 70-percent flow duration (ft^3/s)	Decrease model-simulated flow in reach (ft^3/s)		Approximate decrease in flow in reach (percent)	
	CF	CH					CF	CH	CF	CH
A[a]	3.2 / 2.1	--- / ---	1	Campbell Cr	0.13	0.15	0.01	---	7	---
			2	Elkhorn Cr	.2	.1	.01	---	10	---
			3	Bear Cr	.15	.05	.02	---	40	---
			4	Mud Cr	.1	.1	.04	---	40	---
			5	Clear Cr	.30	.05	.02	---	40	---
			6	Harshman Cr	.15	.05	.01	---	20	---
			7	Little Mississinewa R	.02	.05	.03	---	60	---
			8	White River	1.51	14.0	.15	---	1	---
			9	do.	.91	7.0	.11	---	2	---
			11	do.	.2	1.2	.01	---	1	---
			13	do.	1.0	1.1	.18	---	16	---
			14	Stoney Cr	1.8	6.3	.13	---	2	---
			15	Little White R	1.4	1	.53	---	53	---
			16	Cabin Cr	2.3	4.6	.36	---	8	---
			17	do.	2.2	2.3	.24	---	10	---
			18	Sparrow Cr	.52	.5	.10	---	20	---
			19	Eightmile Cr	.4	.5	.12	---	24	---
			20	Sugar Cr	.45	.4	.06	---	15	---
			21	Whitewater R	1.3	1.0	.16	---	16	---
			22	Martindale Cr	.5	.6	.40	---	67	---
			23	Mud Cr	1.36	1.5	.07	---	5	---
			24	Greens Fork	.22	.16	.02	---	13	---
			25	Nolands Fork	.05	.1	.01	---	10	---

Table 5.--Pumping rates of plans A through F needed to simulate a 20-foot drawdown in 0.57 square mile for constant-flux and constant-head boundaries, Randolph County--Continued

Pumping plan	Model-simulated pumping (ft^3/s) / (Mgal/d)		Reach most affected by pumping	Stream	Model-simulated ground-water seepage to reach (ft^3/s)	Approximate flow at downstream end of reach at about 70-percent flow duration (ft^3/s)	Decrease model-simulated flow in reach (ft^3/s)		Approximate decrease in flow in reach (percent)	
	CF	CH					CF	CH	CF	CH
B	3.2 / 2.1	--- / ---	1	Campbell Cr	0.13	0.15	0.01	<0.01	7	<7
			2	Elkhorn Cr	.2	.1	.01	<.01	10	<10
			3	Bear Cr	.15	.05	.02	.01	40	20
			4	Mud Cr	.1	.1	.08	.03	80	30
			5	Clear Cr	.30	.05	.05	.04	100	80
			6	Harshman Cr	.15	.05	.05	.01	100	20
			7	Little Mississinewa R	.02	.05	.05	.02	100	40
			8	White River	1.51	14.0	.11	.05	1	<1
			9	do.	.91	7.0	.11	.09	2	1
			10	do.	.7	6.0	.08	.26	1	4
			11	do.	.2	1.2	.03	.05	3	4
			12	do.	.12	1.2	.02	.05	2	4
			13	do.	1.0	1.1	.33	.28	30	25
			15	Little White R	1.4	1	.12	.03	12	3
			16	Cabin Cr	2.3	4.6	.17	.08	4	2
			17	do.	2.2	2.3	.15	.09	7	4
			18	Sparrow Cr	.52	.5	.08	.14	16	28
			19	Eightmile Cr	.4	.5	.12	.06	24	12
			20	Sugar Cr	.45	.4	.08	.36	20	90
			21	Whitewater R	1.3	1.0	.04	.02	4	2
			22	Martindale Cr	.5	.6	.22	.04	37	7
			23	Mud Cr	1.36	1.5	.14	.12	9	8
			24	Greens Fork	.22	.16	.04	.01	25	6
			25	Nolands Fork	.05	.1	.01	<.01	10	<10

Table 5.--Pumping rates of plans A through F needed to simulate a 20-foot drawdown in 0.57 square mile for constant-flux and constant-head boundaries, Randolph County--Continued

Pumping plan	Model-simulated pumping (ft^3/s) / (Mgal/d)		Reach most affected by pumping	Stream	Model-simulated ground-water seepage to reach (ft^3/s)	Approximate flow at downstream end of reach at about 70-percent flow duration (ft^3/s)	Decrease in model-simulated flow in reach (ft^3/s)		Approximate decrease in flow in reach (percent)	
	CF	CH					CF	CH	CF	CH
C	4.6 / 3	6.8 / 4.4	1	Campbell Cr	0.13	0.15	0.04	<0.01	27	<7
			2	Elkhorn Cr	.2	.1	.03	.01	30	<10
			3	Bear Cr	.15	.05	.02	.02	40	40
			4	Mud Cr	.1	.1	.08	.02	80	<20
			5	Clear Cr	.30	.05	.05	.01	100	20
			6	Harshman Cr	.15	.05	.02	<.01	40	<20
			7	Little Mississinewa R	.02	.05	.05	<.01	100	<20
			8	White River	1.51	14.0	.18	.44	1	3
			9	do.	.91	7.0	.11	.28	2	4
			10	do.	.7	6.0	.06	.04	1	1
			11	do.	.2	1.2	.02	.01	2	1
			12	do.	.12	1.2	.02	.01	2	1
			13	do.	1.0	1.1	.24	.20	22	18
			14	Stoney Cr	1.8	6.3	.31	.12	5	2
			15	Little White R	1.4	1	.76	.27	76	27
			16	Cabin Cr	2.3	4.6	.40	1.2	9	26
			17	do.	2.2	2.3	.24	.77	10	33
			18	Sparrow Cr	.52	.5	.10	.45	20	90
			19	Eightmile Cr	.4	.5	.12	.27	24	54
			20	Sugar Cr	.45	.4	.08	.07	20	18
			21	Whitewater R	1.3	1.0	.12	.09	12	9
			22	Martindale Cr	.5	.6	.28	.09	47	32
			23	Mud Cr	1.36	1.5	.07	.07	5	5
			24	Greens Fork	.22	.16	.02	.01	13	6
			25	Nolands Fork	.05	.1	.01	<.01	10	<10

Table 5.--Pumping rates of plans A through F needed to simulate a 20-foot drawdown in 0.57 square mile for constant-flux and constant-head boundaries, Randolph County--Continued

Pumping plan	Model-simulated pumping (ft^3/s) / (Mgal/d)		Reach most affected by pumping	Stream	Model-simulated ground-water seepage to reach (ft^3/s)	Approximate flow at downstream end of reach at about 70-percent flow duration (ft^3/s)	Decrease in model-simulated flow in reach (ft^3/s)		Approximate decrease in flow in reach (percent)	
	CF	CH					CF	CH	CF	CH
D	3.7 / 2.4	5.1 / 3.3	1	Campbell Cr	0.13	0.15	0.03	<0.01	20	<7
			2	Elkhorn Cr	.2	.1	.03	<.01	30	<10
			3	Bear Cr	.15	.05	.02	.05	40	100
			4	Mud Cr	.1	.1	.08	.06	80	60
			5	Clear Cr	.30	.05	.10	.02	100	40
			6	Harshman Cr	.15	.05	.07	<.01	100	<20
			7	Little Mississinewa R	.02	.05	.08	.01	100	20
			8	White River	1.51	14.0	.17	.24	1	2
			9	do.	.91	7.0	.11	.47	2	7
			10	do.	.7	6.0	.08	.09	1	2
			11	do.	.2	1.2	.03	.02	3	2
			12	do.	.12	1.2	.02	.02	2	2
			13	do.	1.0	1.1	.33	.00	30	<1
			14	Stoney Cr	1.8	6.3	.11	.03	2	<1
			15	Little White R	1.4	1	.28	.08	28	8
			16	Cabin Cr	2.3	4.6	.39	.33	8	7
			17	do.	2.2	2.3	.21	.17	9	7
			18	Sparrow Cr	.52	.5	.10	.17	20	34
			19	Eightmile Cr	.4	.5	.12	.30	24	60
			20	Sugar Cr	.45	.4	.08	.15	20	38
			21	Whitewater R	1.3	1.0	.04	.02	4	2
			22	Martindale Cr	.5	.6	.14	.04	23	7
			23	Mud Cr	1.36	1.5	.11	.06	7	4
			24	Greens Fork	.22	.16	.04	.01	25	6
			25	Nolands Fork	.05	.1	.01	<.01	10	<10

Table 5.--Pumping rates of plans A through F needed to simulate a 20-foot drawdown in 0.57 square mile for constant-flux and constant-head boundaries, Randolph County--Continued

Pumping plan	Model-simulated pumping (ft^3/s) / (Mgal/d)		Reach most affected by pumping	Stream	Model-simulated ground-water seepage to reach (ft^3/s)	Approximate flow at downstream end of reach at about 70-percent flow duration (ft^3/s)	Decrease in model-simulated flow in reach (ft^3/s)		Approximate decrease in flow in reach (percent)	
	CF	CH					CF	CH	CF	CH
E	3.3	5.2	1	Campbell Cr	0.13	0.15	0.01	0.00	7	<1
	2.1	3.3	2	Elkhorn Cr	.2	.1	.01	.00	10	<1
			3	Bear Cr	.15	.05	.02	.01	40	20
			4	Mud Cr	.1	.1	.08	.02	80	20
			5	Clear Cr	.30	.05	.05	.03	100	60
			6	Harshman Cr	.15	.05	.05	.01	100	20
			7	Little Mississinewa R	.02	.05	.05	.02	100	40
			8	White River	1.51	14.0	.09	.04	1	<1
			9	do.	.91	7.0	.10	.00	1	<1
			10	do.	.7	6.0	.08	.07	1	1
			11	do.	.2	1.2	.03	.03	3	3
			12	do.	.12	1.2	.02	.02	2	2
			13	do.	1.0	1.1	.33	.33	30	30
			15	Little White R	1.4	1	.14	.06	14	6
			16	Cabin Cr	2.3	4.6	.16	.09	3	2
			17	do.	2.2	2.3	.21	.15	9	7
			18	Sparrow Cr	.52	.5	.07	.04	14	8
			19	Eightmile Cr	.4	.5	.12	.08	24	16
			20	Sugar Cr	.45	.4	.08	.08	20	20
			21	Whitewater R	1.3	1.0	.06	.03	6	3
			22	Martindale Cr	.5	.6	.39	.21	65	35
			23	Mud Cr	1.36	1.5	.14	.1	9	9
			24	Greens Fork	.22	.16	.04	.04	25	25
			25	Nolands Fork	.05	.1	.01	<.01	10	<10

Table 5.--Pumping rates of plans A through F needed to simulate a 20-foot drawdown in 0.57 square mile for constant-flux and constant-head boundaries, Randolph County--Continued

Pumping plan	Model-simulated pumping (ft^3/s) / (Mgal/d)		Reach most affected by pumping	Stream	Model-simulated ground-water seepage to reach (ft^3/s)	Approximate flow at downstream end of reach at about 70-percent flow duration (ft^3/s)	Decrease in model-simulated flow in reach (ft^3/s)		Approximate decrease in flow in reach (percent)	
	CF	CH					CF	CH	CF	CH
F	2.8 / 1.8	3.7 / 2.4	1	Campbell Cr	0.13	0.15	0.01	0.00	7	<1
			2	Elkhorn Cr	.2	.1	.01	.00	10	<1
			3	Bear Cr	.15	.05	.02	.01	40	20
			4	Mud Cr	.1	.1	.08	.03	80	30
			5	Clear Cr	.30	.05	.05	.04	100	80
			6	Harshman Cr	.15	.05	.05	.01	100	20
			7	Little Mississinewa R	.02	.05	.05	.02	100	40
			8	White River	1.51	14.0	.09	.04	1	<1
			9	do.	.91	7.0	.10	.07	1	1
			10	do.	.7	6.0	.08	.07	1	1
			11	do.	.2	1.2	.03	.03	3	3
			12	do.	.12	1.2	.02	.02	2	2
			13	do.	1.0	1.1	.33	.33	30	30
			15	Little White R	1.4	1	.11	.04	11	4
			16	Cabin Cr	2.3	4.6	.15	.07	3	2
			17	do.	2.2	2.3	.14	.08	6	3
			18	Sparrow Cr	.52	.5	.07	.03	14	6
			19	Eightmile Cr	.4	.5	.12	.08	24	16
			20	Sugar Cr	.45	.4	.08	.08	20	20
			21	Whitewater R	1.3	1.0	.04	.01	4	1
			22	Martindale Cr	.5	.6	.22	.08	37	13
			23	Mud Cr	1.36	1.5	.14	.10	9	7
			24	Greens Fork	.22	.16	.04	.02	25	13
			25	Nolands Fork	.05	.1	.01	<.01	10	<1

[a]A constant-head boundary was not used with Pumping plan A.

The effect of the model boundary on the results of pumping simulations was generally insignificant in Hamilton, Madison, and Delaware Counties (fig. 1). Cones of depression continued to spread until a quantity of water equal to the pumpage was derived from ground water previously discharged from the ground-water system. Because most of the ground-water discharge is across model boundaries, much of the water derived from the ground-water system during pumping is probably from boundary flux that normally flows out of the study area.

Determining the hydrologic advantages of one plan over another is difficult because of the influence of boundaries on simulations, even if the only criterion is the extent of the spread of the cone of depression. For instance, comparison of figures 27 and 28 indicates that pumping plan A is more favorable than B. However, if a constant-head boundary simulates the response of the boundaries to pumping plan B (fig. 29) better than a constant-flux boundary does, then, in general, both plans are equally advantageous. In addition, because the pumpage of each plan may vary considerably, depending on the actual response at the boundaries, as well as among plans, comparison of the advantages of the plans is difficult. However, for an average drawdown of 20 ft, simulations can be used to assess the approximate regional effect of developing a well field at each location (A-F).

The preceding simulations are useful in estimating yields for the two major aquifer systems. Comparisons of the model-simulated pumping rates (table 5) indicates that, for the 20-foot drawdown in 0.57 mi^2, the minimum pumping rate was the rate derived for plan F (1.8 Mgal/d) and a constant-flux boundary, and the maximum pumping rate was the rate derived for plan C (4.4 Mgal/d) and a constant-head boundary. Although the maximum simulated pumping rate is for the bedrock, its yield is not sufficiently greater than that of the sand and gravel pumping plans to infer a higher yielding system.

Model-simulated depletion of flow and the percentage of depletion of flow at 70-percent flow duration caused by the six pumping plans are also listed in table 5. Reaches having depletion in flow of less than 1 percent for constant-flux simulations are not included. Depletion of flow along a reach causes an identical loss in flow in all other reaches downstream. However, if more than one reach is upstream from a measuring point, then actual discharge at that point is generally large compared with the simulated depletion in flow attributable to pumping. Therefore, ignoring the decrease in streamflow attributed to depletion in streamflow upstream, in general, does not significantly affect the results shown in table 5. However, for larger rates of pumping than those in table 5, the "approximate flow at the downstream end of the reach at about 70-percent flow duration" (table 5) may be reduced by upstream decrease in flow attributed to that pumping. This observation applies to reaches 8, 9, 10, 11, 12, and 16 (fig. 17). Consideration of the observation in an analysis of any future simulations in the study area would be useful. Depletion in flow of most of the streams for each pumping plan is 1 percent or greater, probably because flow in most of the streams is the same magnitude as the pumping. Simulations by the model indicate that, at 70-percent flow duration, the effect of pumping on flow in the reaches most distant upstream in Randolph County may be considerable.

Some of the largest differences in decreases in flow are the differences between constant-flux and constant-head simulations of one plan. The actual decrease in flow for each reach should be between the results derived for the two boundary conditions.

Decreases in flow for a simulation having a constant-head boundary are generally smaller than for a simulation having a constant-flux boundary. However, decreases can be larger in reaches near the pumping center, where a constant-head boundary is used, because the pumping rate is higher. (See table 5, plan B, reaches 10 and 20.)

About half the stream reaches are significantly affected (more than 10-percent reduction in flow for constant-flux boundaries) by the simulated pumpage. The effect is understandable because the streams in Randolph County are generally smaller than those of the downstream study areas. However, streams in Randolph County are similar to those in the rest of the upper White River basin in that pumpage does not reduce streamflow by more than 10 percent for streams discharging more than 2 ft^3/s.

Simulations of Pumping Plans A through F with Uniform Pumping Rate

Because of the effect of the boundaries on pumping and because the pumping rates for the six simulations in table 5 differ, an assessment of the relative potential for ground-water development in the two major aquifer systems based on the simulations is difficult. Furthermore, comparison of the distribution of drawdown in well fields having unequal pumping rates does not allow direct comparison of the effect of well-field development in various locations.

To assess the relative potential for ground-water development in the two major aquifer systems and provide results by which direct comparison of the effect of pumping at each well-field location is possible, the author selected a pumping rate of 1 Mgal/d for simulating in pumping plans A through F. One Mgal/d was used because, although it may represent a lower yield than is typical from a well field, the rate is generally small enough so that the boundaries do not significantly affect the results. Thus, although constant-flux boundaries were used in the six pumping plans, nearly identical results would have been obtained if constant-head boundaries had been used.

The resulting distribution of drawdowns from the pumping rate of 1 Mgal/d used in plans A through F are shown in figures 38-43. Constant-flux boundaries were used in all these plans. Comparison of the figures indicates that the area where drawdown was 3 ft or more is least for plan C (fig. 40) and greatest for plan E (fig. 42). Simulated drawdown in the well field did not exceed 10 ft in any of the plans. Because drawdown at part of the boundary of the model for plan E, in particular, was greater than 3 ft, comparison of the results shown in figure 42 with results for plans A through D (figs. 38-41) and F (fig. 43) should be done carefully.

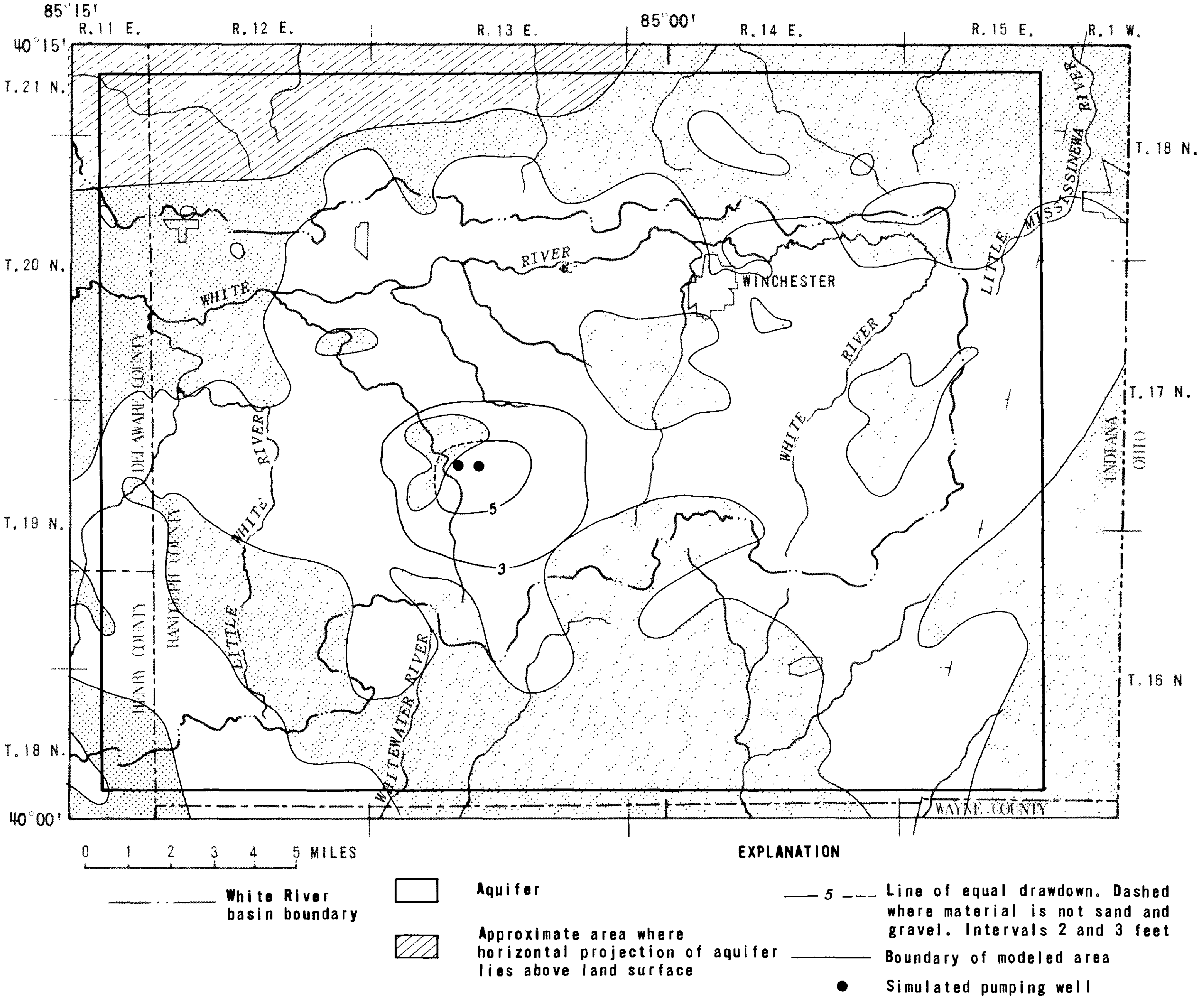


Figure 38.-- Model-simulated drawdown in aquifer 3 for pumping plan A and constant-flux boundaries. Simulated pumping is 1 million gallons per day.

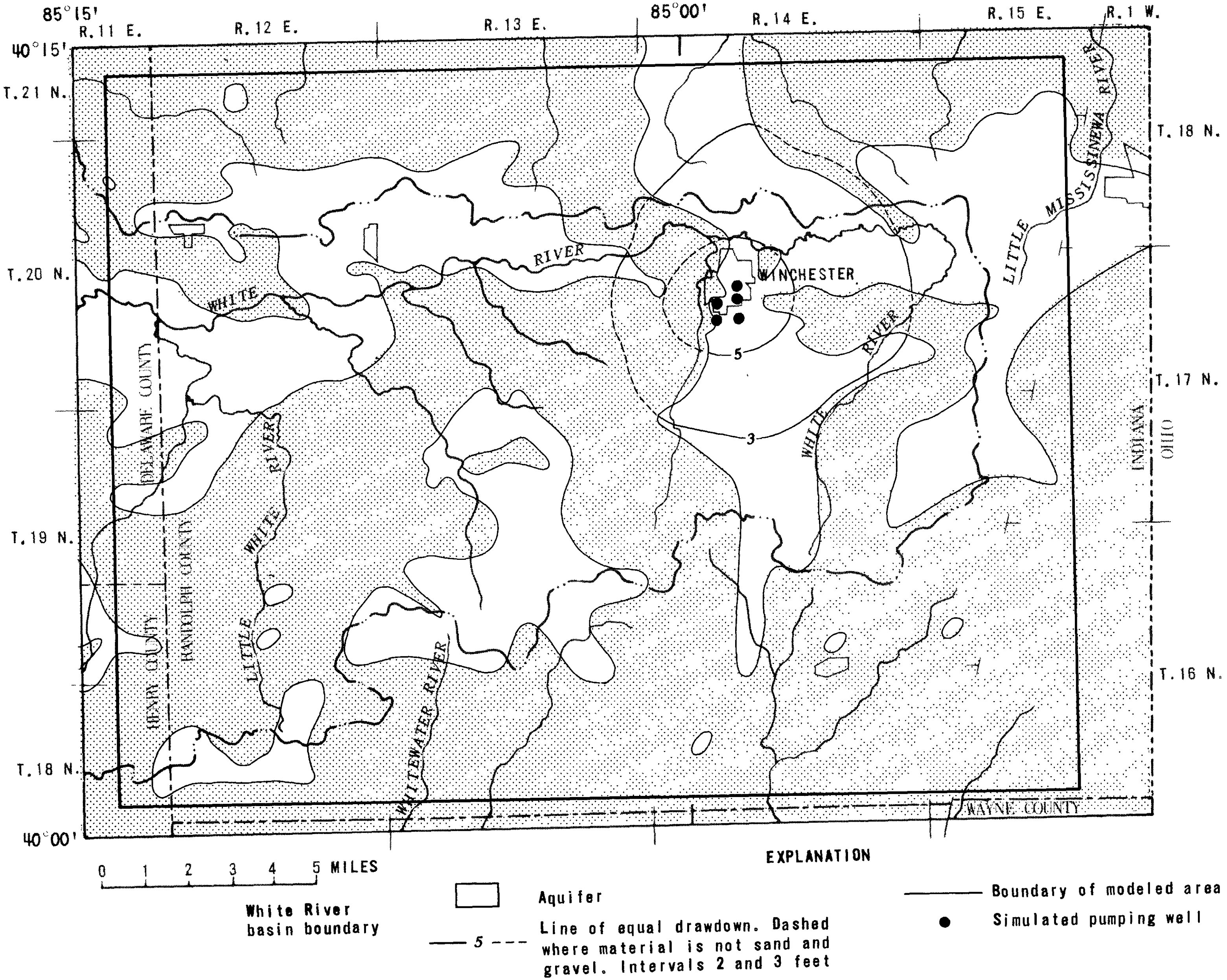


Figure 39.-- Model-simulated drawdown in aquifer 2 for pumping plan B and constant-flux boundaries. Simulated pumping is 1 million gallons per day.

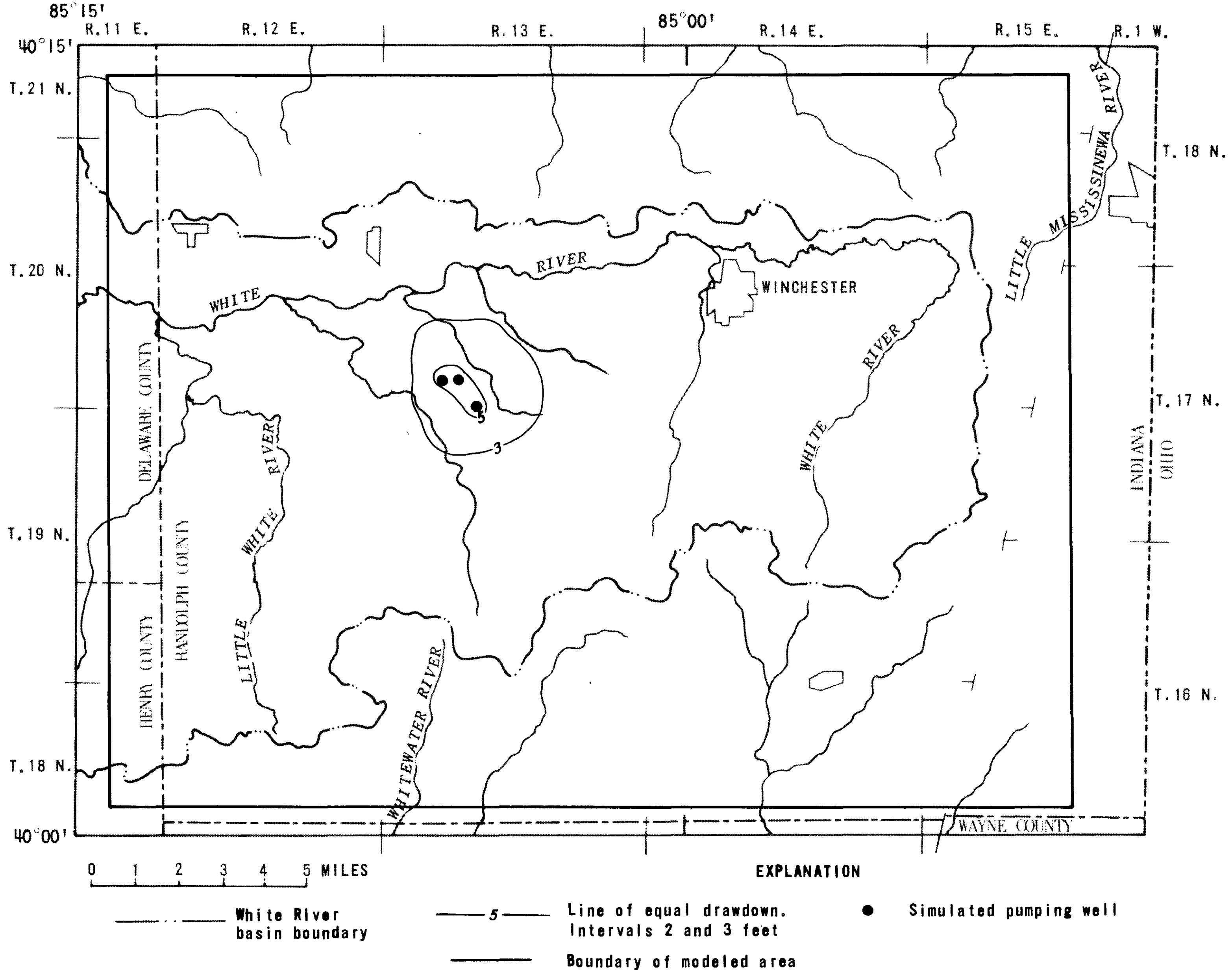


Figure 40.-- Model-simulated drawdown in the bedrock for pumping plan C and constant-flux boundaries. Simulated pumping is 1 million gallons per day.

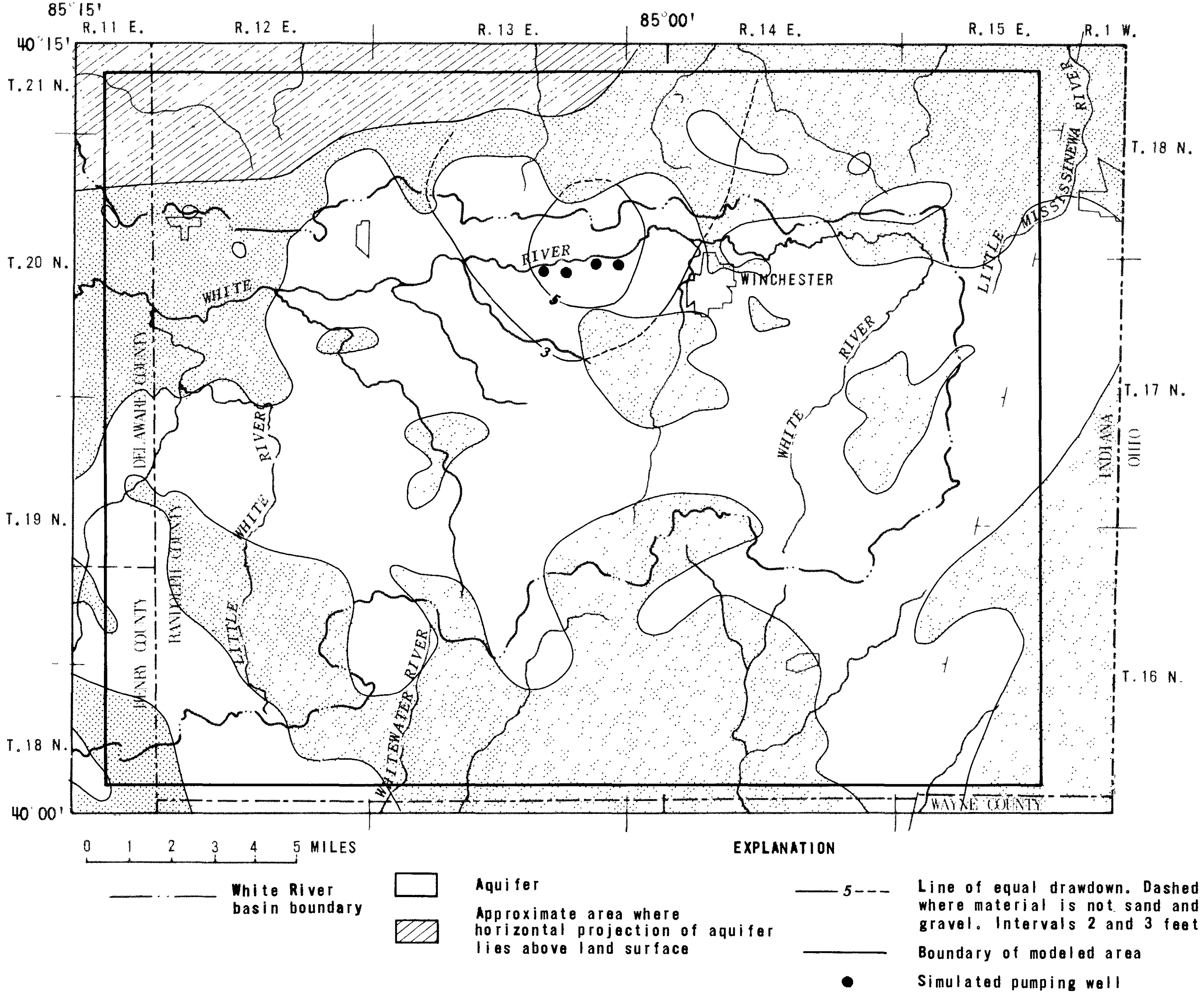


Figure 41.-- Model-simulated drawdown in aquifer 3 for pumping plan D and constant-flux boundaries. Simulated pumping is 1 million gallons per day.

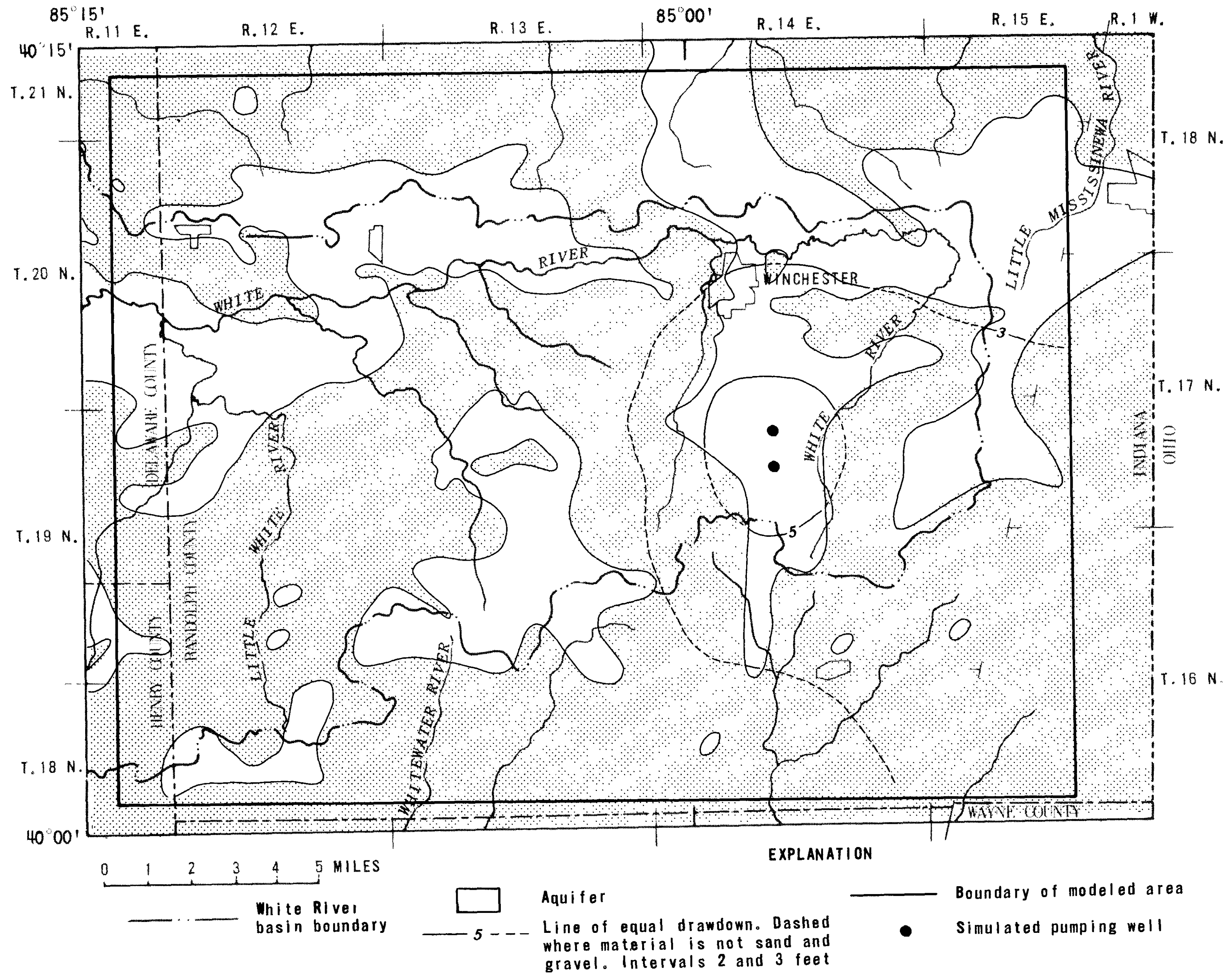


Figure 42.-- Model-simulated drawdown in aquifer 2 for pumping plan E and constant-flux boundaries. Simulated pumping is 1 million gallons per day.

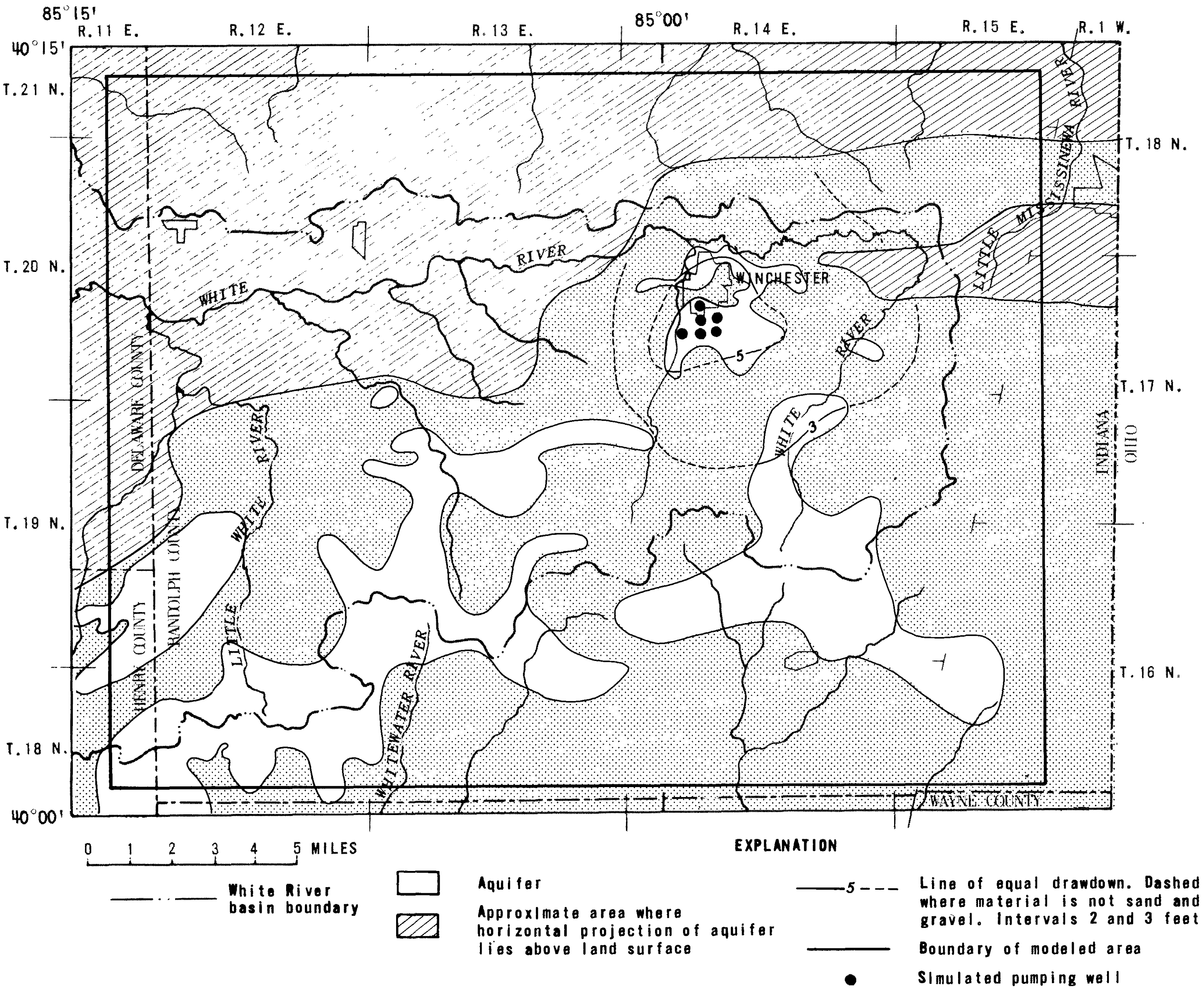


Figure 43.-- Model-simulated drawdown in aquifer 4 for pumping plan F and constant-flux boundaries. Simulated pumping is 1 million gallons per day.

Because only one pumping plan was simulated in the bedrock, the variability of drawdown distribution in that aquifer cannot be predicted. Plan C was simulated in an area where the transmissivity of bedrock was high. Pumping elsewhere in the bedrock would probably result in a more extensive drawdown distribution than is shown in figure 40.

Comparison of the pumping simulated in the sand and gravel aquifers (plans A, B, and D-F, figs. 38, 39, and 41-43) indicates that the variablitiy in drawdown is great in the area where drawdown is 3 ft or more . Thus, the effect of developing well fields in the sand and gravel aquifers in the till would probably be variable.

Model-simulated decrease in flow and the percentage decrease that this represents are given in table 6. With the constant-flux boundary used for plans A through F, the sum of the model-simulated decreases in streamflow in the reaches for each plan should equal the 1.55-ft^3/s model-simulated pumping because the pumping can only be satisfied by loss of seepage to the streams if a constant-flux boundary is used. However, the sum does not equal 1.55 ft^3/s because (1) not all reaches are listed in column 7 for each plan (Only reaches whose decreases in flow are greater than or equal to 1 percent are included for each plan.); (2) round-off error in column 7 makes equality improbable; and (3) the total model-simulated inflow did not equal the total outflow at the end of each pumping simulation. In the author's opinion, the discrepancy between total inflow and outflow should not significantly affect the results given in table 6.

As was also indicated for the six pumping plans, decrease in flow in most of the streams was 1 percent or greater. (See section "Simulations of Pumping Plans A through F with Uniform Drawdown" and Table 5.) Therefore, the results presented in tables 5 and 6 suggest that development of the ground-water system under the conditions simulated for these pumping plans will cause some decrease in flow in most streams in the area and a large decrease in flow in small streams.

In comparing and contrasting the results of the pumping plans, the reader should be aware that variations in transmissivity of the aquifer, vertical hydraulic conductivity of the confining beds, and the stream-aquifer connection near the simulated pumping centers all contribute to the differences between simulations. Although the pumping plans were designed to minimize the effect of these variations, some of the variations cannot be eliminated. As much as 2.5 Mgal/d can probably be developed at some locations with the criteria used in the simulations. This and similar rates of pumping may cause drawdowns exceeding 5 ft. in 10 to 50 percent of the study area. Drawdowns probably spread into nearby river basins.

A study of the effect of more intensive, larger scale development of the ground-water system than is presented here could include, for example, the concurrent simulation of pumping plans B and E. The effect of the concurrent pumping of two or more plans can be estimated by adding together the individual effects of each plan. For instance, the combined drawdown caused by the simulation of plans B and E can be estimated by adding the drawdown in aquifer 2, caused by plan B, to the drawdown in aquifer 2, caused by plan E at any point. Decreases in streamflow in each reach can also be estimated similarly if depletion does not exceed streamflow. Simulated drawdown obtained with a

Table 6.--Pumping plans A through F, which simultate 1 Mgal/d and a constant-flux boundary

[ft^3/s, cubic foot per second; Mgal/d, million gallons per day; Cr, creek; R, river. Model-simulated ground-water seepage to reach: from calibrated model. (See table 3.) Locations of stream in figure 17]

Pumping plan	Model-simulated pumping (ft^3/s) / (Mgal/d)	Reach most affected by pumping	Stream	Model-simulated ground-water seepage to reach (ft^3/s)	Approximate flow at downstream end of reach at about 70-percent flow duration (ft^3/s)	Decrease in model-simulated flow in reach (ft^3/s)	Approximate decrease in flow in reach (percent)
A	1.55 / 1.0	1	Campbell Cr	0.13	0.15	<0.01	<7
		2	Elkhorn Cr	.2	.1	<.01	<10
		3	Bear Cr	.15	.05	.01	20
		4	Mud Cr	.1	.1	.01	10
		5	Clear Cr	.30	.05	.01	20
		6	Harshman Cr	.15	.05	<.01	<20
		7	Little Mississinewa R	.02	.05	.01	20
		13	White R	1.0	1.1	.07	6
		15	Little White R	1.4	1	.21	21
		16	Cabin Cr	2.3	4.6	.20	4
		17	do.	2.2	2.3	.24	10
		18	Sparrow Cr	.52	.5	.07	14
		19	Eightmile Cr	.4	.5	.07	14
		20	Sugar Cr	.45	.4	.03	8
		21	Whitewater R	1.3	1.0	.06	6
		22	Martindale Cr	.5	.6	.16	27
		23	Mud Cr	1.36	1.5	.03	2
		24	Greens Fork	.22	.16	.01	6
		25	Nolands Fork	.05	.1	<.01	<10

Table 6.--Pumping plans A through F, which simultate 1 Mgal/d and a constant-flux boundary--Continued

Pumping plan	Model-simulated pumping (ft^3/s) / (Mgal/d)	Reach most affected by pumping	Stream	Model-simulated ground-water seepage to reach (ft^3/s)	Approximate flow at downstream end of reach at about 70-percent flow duration (ft^3/s)	Decrease in model-simulated flow in reach (ft^3/s)	Approximate decrease in flow in reach (percent)
B	1.55 / 1.0	1	Campbell Cr	0.13	0.15	<0.01	<7
		2	Elkhorn Cr	.2	.1	<.01	<10
		3	Bear Cr	.15	.05	.02	40
		4	Mud Cr	.1	.1	.08	80
		5	Clear Cr	.30	.05	.05	100
		6	Harshman Cr	.15	.05	.04	80
		7	Little Mississinewa R	.02	.05	.05	100
		10	White R	.7	6.0	.07	1
		11	do.	.2	1.2	.03	3
		12	do.	.12	1.2	.02	2
		13	do.	1.0	1.1	.33	30
		15	Little White R	1.4	1	.04	4
		16	Cabin Cr	2.3	4.6	.05	1
		17	do.	2.2	2.3	.05	2
		18	Sparrow Cr	.52	.5	.03	6
		19	Eightmile Cr	.4	.5	.06	12
		20	Sugar Cr	.45	.4	.08	20
		21	Whitewater R	1.3	1.0	.01	1
		22	Martindale Cr	.5	.6	.06	10
		23	Mud Cr	1.36	1.5	.08	5
		24	Greens Fork	.22	.16	.03	19
		25	Nolands Fork	.05	.1	.01	10

Table 6.--Pumping plans A through F, which simultate 1 Mgal/d and a constant-flux boundary--Continued

Pumping plan	Model-simulated pumping $\frac{(ft^3/s)}{(Mgal/d)}$	Reach most affected by pumping	Stream	Model-simulated ground-water seepage to reach (ft^3/s)	Approximate flow at downstream end of reach at about 70-percent flow duration (ft^3/s)	Decrease in model-simulated flow in reach (ft^3/s)	Approximate decrease in flow in reach (percent)
C	$\frac{1.55}{1.0}$	1	Campbell Cr	0.13	0.15	0.01	7
		2	Elkhorn Cr	.2	.1	.01	10
		3	Bear Cr	.15	.05	.01	20
		4	Mud Cr	.1	.1	.02	20
		5	Clear Cr	.30	.05	.01	20
		6	Harshman Cr	.15	.05	<.01	<20
		7	Little Mississinewa R	.02	.05	.01	20
		8	White R	1.51	14.0	.11	1
		9	do.	.91	7.0	.07	1
		13	do.	1.0	1.1	.05	5
		15	Little White R	1.4	1	.17	17
		16	Cabin Cr	2.3	4.6	.30	7
		17	do.	2.2	2.3	.15	7
		18	Sparrow Cr	.52	.5	.10	20
		19	Eightmile Cr	.4	.5	.11	22
		20	Sugar Cr	.45	.4	.02	5
		21	Whitewater R	1.3	1.0	.02	2
		22	Martindale Cr	.5	.6	.06	10
		23	Mud Cr	1.36	1.5	.02	1
		24	Greens Fork	.22	.16	<.01	<6
		25	Nolands Fork	.05	.1	<.01	<10

Table 6.--Pumping plans A through F, which simultate 1 Mgal/d and a constant-flux boundary--Continued

Pumping plan	Model-simulated pumping $\frac{(ft^3/s)}{(Mgal/d)}$	Reach most affected by pumping	Stream	Model-simulated ground-water seepage to reach (ft^3/s)	Approximate flow at downstream end of reach at about 70-percent flow duration (ft^3/s)	Decrease in model-simulated flow in reach (ft^3/s)	Approximate decrease in flow in reach (percent)
D	$\frac{1.55}{1.0}$	1	Campbell Cr	0.13	0.15	0.01	.7
		2	Elkhorn Cr	.2	.1	.01	10
		3	Bear Cr	.15	.05	.02	40
		4	Mud Cr	.1	.1	.08	80
		5	Clear Cr	.30	.05	.05	100
		6	Harshman Cr	.15	.05	.02	40
		7	Little Mississinewa R	.02	.05	.03	60
		8	White R	1.51	14.0	.10	1
		9	do.	.91	7.0	.10	1
		10	do.	.7	6.0	.06	1
		11	do.	.2	1.2	.02	2
		12	do.	.12	1.2	.01	1
		13	do.	1.0	1.1	.13	12
		15	Little White R	1.4	1	.07	7
		16	Cabin Cr	2.3	4.6	.13	3
		17	do.	2.2	2.3	.06	3
		18	Sparrow Cr	.52	.5	.07	14
		19	Eightmile Cr	.4	.5	.12	24
		20	Sugar Cr	.45	.4	.07	18
		21	Whitewater R	1.3	1.0	.01	1
		22	Martindale Cr	.5	.6	.04	7
		23	Mud Cr	1.36	1.5	.03	2
		24	Greens Fork	.22	.16	.01	6
		25	Nolands Fork	.05	.1	<.01	<10

Table 6.--Pumping plans A through F, which simultate 1 Mgal/d and a constant-flux boundary--Continued

Pumping plan	Model-simulated pumping $\frac{(ft^3/s)}{(Mgal/d)}$	Reach most affected by pumping	Stream	Model-simulated ground-water seepage to reach (ft^3/s)	Approximate flow at downstream end of reach at about 70-percent flow duration (ft^3/s)	Decrease in model-simulated flow in reach (ft^3/s)	Approximate decrease in flow in reach (percent)
E	$\frac{1.55}{1.0}$	1	Campbell Cr	0.13	0.15	<0.01	<7
		2	Elkhorn Cr	.2	.1	<.01	<10
		3	Bear Cr	.15	.05	.01	20
		4	Mud Cr	.1	.1	.04	40
		5	Clear Cr	.30	.05	.04	80
		6	Harshman Cr	.15	.05	.03	60
		7	Little Mississinewa R	.02	.05	.05	100
		10	White R	.7	6.0	.06	1
		11	do.	.2	1.2	.03	3
		12	do.	.12	1.2	.02	2
		13	do.	1.0	1.1	.33	30
		15	Little White R	1.4	1	.04	4
		17	Cabin Cr	2.2	2.3	.06	3
		18	Sparrow Cr	.52	.5	.02	4
		19	Eightmile Cr	.4	.5	.04	8
		20	Sugar Cr	.45	.4	.08	20
		21	Whitewater R	1.3	1.0	.02	2
		22	Martindale Cr	.5	.6	.11	18
		23	Mud Cr	1.36	1.5	.14	9
		24	Greens Fork	.22	.16	.04	25
		25	Nolands Fork	.05	.1	.01	10

Table 6.--Pumping plans A through F, which simultate 1 Mgal/d and a constant-flux boundary--Continued

Pumping plan	Model-simulated pumping $\frac{(ft^3/s)}{(Mgal/d)}$	Reach most affected by pumping	Stream	Model-simulated ground-water seepage to reach (ft^3/s)	Approximate flow at downstream end of reach at about 70-percent flow duration (ft^3/s)	Decrease in model-simulated flow in reach (ft^3/s)	Approximate decrease in flow in reach (percent)
F	$\frac{1.55}{1.0}$	1	Campbell Cr	0.13	0.15	<0.01	<7
		2	Elkhorn Cr	.2	.1	<.01	<10
		3	Bear Cr	.15	.05	.02	40
		4	Mud Cr	.1	.1	.06	60
		5	Clear Cr	.30	.05	.05	100
		6	Harshman Cr	.15	.05	.04	80
		7	Little Mississinewa R	.02	.05	.05	100
		10	White R	.7	6.0	.07	1
		11	do.	.2	1.2	.03	3
		12	do.	.12	1.2	.02	2
		13	do.	1.0	1.1	.33	30
		15	Little White R	1.4	1	.04	4
		16	Cabin Cr	2.3	4.6	.05	1
		17	do.	2.2	2.3	.05	2
		18	Sparrow Cr	.52	.5	.02	4
		19	Eightmile Cr	.4	.5	.05	10
		20	Sugar Cr	.45	.4	.08	20
		21	Whitewater R	1.3	1.0	.01	1
		22	Martindale Cr	.5	.6	.06	10
		23	Mud Cr	1.36	1.5	.09	6
		24	Greens Fork	.22	.16	.03	19
		25	Nolands Fork	.05	.1	.01	10

constant-flux boundary in plans A through F differed significantly from drawdowns obtained with a constant-head boundary in the same pumping plans. (See "Assessment of Ground-Water Availability in the Study Area.") With concurrent simulation of two or more plans, the influence of the boundary of the model may also be significant. The influence can only be investigated by simulating pumping with each of the boundaries (constant head and constant flux) separately and then comparing the results.

SUMMARY AND CONCLUSIONS

The ground-water resources of the White River basin in and near Randolph County, Ind., were investigated by mapping the aquifers, calculating their hydraulic properties, measuring the distribution of potentiometric head in the aquifers, measuring the ground-water discharge to streams, and determining some of the components of the ground-water budget. This information was used to construct and calibrate a five-layer, digital, ground-water-flow model. The flow model, constructed and calibrated to ground-water-level and seepage data collected during the study, simulated conditions during October 1978. The model was used to assess ground-water potential in terms of yield, drawdown, and decrease in streamflow.

Drift generally ranges in thickness from 0 to 300 ft and covers most of the study area. The drift is underlain by limestone, dolomite, and shale of Ordovician to Devonian age. The buried Priam and Anderson Valleys, tributaries of the Teays Valley system, trend north and southwest out of the area.

The two major aquifer systems within Randolph County are (1) four areally discontinuous, confined, sand and gravel aquifers within the drift and (2) bedrock. The average thickness of the four sand and gravel aquifers is 15 ft. Locally, they coalesce vertically to form one thick deposit. On the basis of specific-capacity data from Madison County, the average hydraulic conductivity of the sand and gravel aquifers was calculated to be 433 ft/d. The average permeable thickness of the bedrock aquifer underlying the entire study area is estimated to be 150 ft; the average transmissivity was estimated to be 1,340 ft^2/d in previous studies of the White River basin.

Water-level fluctuations in observation wells indicate that the ground-water system is in dynamic equilibrium. Seepage to streams at 70-percent flow duration October 3-4, 1978, was estimated to be between 17.0 and 23.5 ft^3/s. Pumpage for 1977 and 1978 was 0.85 Mgal/d (1.3 ft^3/s).

In the water budget simulated in the model, the rate of inflow to the ground-water system in the area represented by the model is 62.8 ft^3/s. Of this, 90 percent is effective areal recharge of precipitation, and 10 percent is flow across the boundaries. Two percent of the ground-water outflow is pumpage, 29 percent is seepage to streams, and the remaining 69 percent is ground-water flow across the boundaries.

The digital flow model was used to assess the potential for development of the two major aquifer systems and to investigate an alternative for future development. Results of six pumping plans indicate that pumping rates of as much as 2.5 Mgal/d can be developed at some locations. Much of the water derived from the ground-water system during pumping is probably from boundary flux that normally flows out of Randolph County. As a result, drawdowns probably spread into nearby river basins. Drawdowns caused by the pumping plans were greater than 5 ft. in 10 to 50 percent of the study area. About half the stream reaches are significantly affected (more than 10 percent reduction in flow with constant-flux boundaries) by the simulated pumpage. The effect is understandable because the streams in Randolph County are generally smaller than those in counties downstream. However, streams in Randolph County are similar to those in the rest of the upper White River basin in that pumpage does not reduce streamflow by more than 10 percent for streams discharging more than 2 ft^3/s. Although well hydraulics and pumping constraints were not considered in the pumping simulations, their consideration in the evaluation of the results of these and any other simulations may be advantageous if the hydrology of the area is studied in the future.

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