HYDROLOGIC EFFECTS OF GROUND- AND SURFACE-WATER WITHDRAWALS IN THE MILFORD AREA, ELKHART AND KOSCIUSKO COUNTIES, INDIANA

By Heler A. Lindgren, James G. Peters, David A. Cohen, and E. James Crompton

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Dallas L. Peck, Director

For additional information write to:

District Chief
U.S. Geological Survey
6023 Guion Road, Suite 201
Indianapolis, Indiana 46254

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TABLES--Continued

FACTORS FOR CONVERTING INCH-POUND UNITS TO METRIC (INTERNATIONAL SYSTEM) UNITS

Multiply Inch-Pound Unit	By	To Obtain Metric Unit
inch (in.)	25.40	millimeter (mm)
foot (ft)	0.3048	meter (m)
square foot (ft^2)	0.0929	square meter (m^2)
foot per day (ft/d)	0.3048	meter per day (m/d)
foot squared per day (ft^2/d)	0.0929	meter squared per day (m^2/d)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.590	square kilometer (km^2)
inch per year (in/yr)	2.540	centimeter per year (cm/a)
cubic foot per second (ft^3/s)	0.0283	cubic meter per second (m^3/s)
gallon per day (gal/d)	3.785	liter per day (L/d)
gallon per minute (gal/min)	3.785	liter per minute (L/min)
million gallons per day (Mgal/d)	0.04381	cubic meter per second (m^3/s)
million gallons per year (Mgal/yr)	3,785.0	cubic meter per year (m^3/a)
cubic foot per day per foot [$(ft^3/d)/ft$]	0.0929	cubic meter per day per meter [$(m^3/d)/m$]
gallon per day per square mile [$(gal/d)/mi^2$]	0.0982	cubic meter per day per square kilometer [$(m^3/d)/km^2$]
cubic foot per second per square mile [$(ft^3/s)/mi^2$]	0.01093	cubic meter per second per square kilometer [$(m^3/s)/km^2$]

To convert degree Fahrenheit (°F) to degree Celsius (°C)

$$5/9\ (°F - 32°) = °C$$

HYDROLOGIC EFFECTS OF GROUND- AND SURFACE-WATER WITHDRAWALS IN THE MILFORD AREA, ELKHART AND KOSCIUSKO COUNTIES, INDIANA

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ABSTRACT

Agricultural irrigation in northern Indiana has increased rapidly since 1975 and might double by the year 2000. A 16.5 square-mile area in north-central Indiana was studied to determine possible effects of increased irrigation on local water supply. In 1982, an average of 2 inches of water was used to irrigate 975 acres of sandy soil overlying highly transmissive outwash deposits. Irrigational pumpage was 75 percent of the summer water use but was less than potential irrigational pumpage because (1) only one-third of the suitable land was irrigated, and (2) precipitation was near normal for the year.

A three-dimensional digital flow model, calibrated with data collected in 1982, was used to simulate four hypothetical pumping plans representing various irrigational schemes and possible rainfall conditions: (1) 1982 acreage irrigated and 1982 (above normal) precipitation; (2) 1982 acreage irrigated and below-normal precipitation; (3) maximum acreage irrigated and normal precipitation; and (4) maximum acreage irrigated and below-normal precipitation. A fifth pumping plan was used to simulate maximum year-round water use. Plan 5 was not designed to simulate irrigational development but rather a maximum rate of withdrawal sustainable year-round until steady-state is reached.

Of the four pumping plans that simulated irrigational pumpage, plan 4 had the greatest effect on ground- and surface-water supply. Compared with 1982 pumpage, this plan represented a thirteenfold increase in the volume of water pumped for irrigation from wells and from Turkey Creek, a stream bordering the area of study. The model predicted a potentiometric decline of as much as 20.7 feet over an 8-acre area of the aquifer. This decline was one-fourth of the available drawdown and would not dewater the source aquifer. Streamflow in Turkey Creek would be reduced 39 percent by simulated ground-water and surface-water pumpage but remaining flow would still be twice the 7-day, 10-year low flow. However, the model predicted that flow in two smaller streams would be reduced to zero.

The rate of pumping used in plan 5 was nearly 4 times the pumping rate in 1982. Potentiometric decline for plan 5 was as much as 40 percent of available drawdown, and predicted streamflow reduction would cause flow in Turkey Creek to decrease below the 7-day, 10-year low flow.

Results of plans 1, 2, 3, and 4 indicate that the outwash system provides adequate water for current (1982) needs and substantial growth for irrigation. However, maximum irrigational development might cause temporary, local competition for water in several parts of the area. Plan 5 indicates that water use could increase substantially before effects of pumping would prevail year-round.

INTRODUCTION

Background

The Indiana Department of Natural Resources (IDNR) anticipates that it will have a larger role in state water-management within the next few years. In preparation, IDNR recently developed an assessment of the State's water-resource needs. This work was done as a contribution to the Governor's Water Resources Study Commission (GWRSC) which delineated several areas of the State where conflicts in water use may occur.

During the past ten years, increases in irrigation in northern Indiana have caused local competition for water among irrigators and other users. In two northwestern counties, water withdrawals for irrigation from a confined bedrock aquifer have caused temporary water-level declines of more than 20 feet throughout a 175-square mile area (Indiana Department of Natural Resources, 1982). The conflict in this area has caused concern about irrigation in other parts of northern Indiana.

Of the other areas where potential conflicts may occur, IDNR selected the 1,800-square mile part of the St. Joseph River basin in Indiana (fig. 1) as a top-priority area for study. Results of studies by Purdue University for the GWRSC indicate that agricultural irrigation is extensive in this part of the State and might double by the year 2000 (Governor's Water Resource Study Commission, 1980, p. 179). Many natural lakes, streams, and marshes are used for recreation and wildlife habitat. Many summer homes are built in areas adjacent to lakes and marshes. The lakes and marshes are sensitive to changes in streamflow and ground-water levels. The State is concerned about possible effects of withdrawals for irrigation on surface- and ground-water supplies in the basin.

As a first step in preparing for increased responsibilities in water-resource management, the GWRSC and IDNR compiled much of the water-resource information available for the basin, including ground-water availability, irrigational potential of soils, and ground- and surface-water withdrawals. In addition, IDNR updated water-use information, identified and mapped natural lakes and wetlands, and provided estimates of future irrigation. Beyond this preliminary work, the State was interested in developing management tools that effectively use this information to evaluate the effect of ground- and surface-water withdrawals on water supply.

The St. Joseph River basin project was begun in 1981 to assist IDNR in developing management tools in two ways. The first entails application of selected methods of hydrologic analysis to areas in the basin with potential problems. Two areas were selected for intensive study (fig. 1)--one in Lagrange County (Howe study area) and the other in Elkhart and Kosciusko Counties (Milford study area). They were selected because they were intensively irrigated and representative of other potential problem areas in the basin. The second entails evaluation of the effectiveness of the present hydrologic data-collection network in the basin. This network includes gages on streams and lakes as well as records on observation wells. Suggestions about improvements to the network were also developed.

Purpose and Scope

This report discusses the effects of surface- and ground-water withdrawals on the hydrologic system of the Milford area of the St. Joseph River basin in Elkhart and Kosciusko counties (fig. 2). The report describes the water resources of the area and includes discussions of streamflow, ground-water flow, geology, aquifer properties, and interaction between surface water and ground water. Estimating techniques are used to evaluate the effects of surface-water withdrawals on streamflow. A digital model is used to predict the effects of ground-water withdrawals on drawdown and streamflow in five hypothetical pumping plans.

Previous Studies

Several hydrologic studies (Pettijohn, 1968; Hunn and Rosenshein, 1969; and Reussow and Rhone, 1975) have been done in the St. Joseph River basin in Indiana. These studies were helpful in providing preliminary hydrogeologic information.

Pettijohn (1968) discussed the different types of aquifers in the Indiana part of the basin. He summarized the geometry; transmissivity; and rate of recharge of sand and gravel aquifers in the outwash plains, preglacial valleys, kames and eskers, and till and lake sediments. Most useful to this study were his estimates of recharge to outwash aquifers through different surficial materials including soil, dune sand, and till or lake clay and through kames and eskers.

Hunn and Rosenshein (1969) described the geologic and the hydrologic characteristics of aquifers in St. Joseph County (fig. 1). Their evaluation included division of the glacial drift into four units. They describe the geometry and lithology and in each unit and estimated transmissivity, storage coefficient, and recharge.

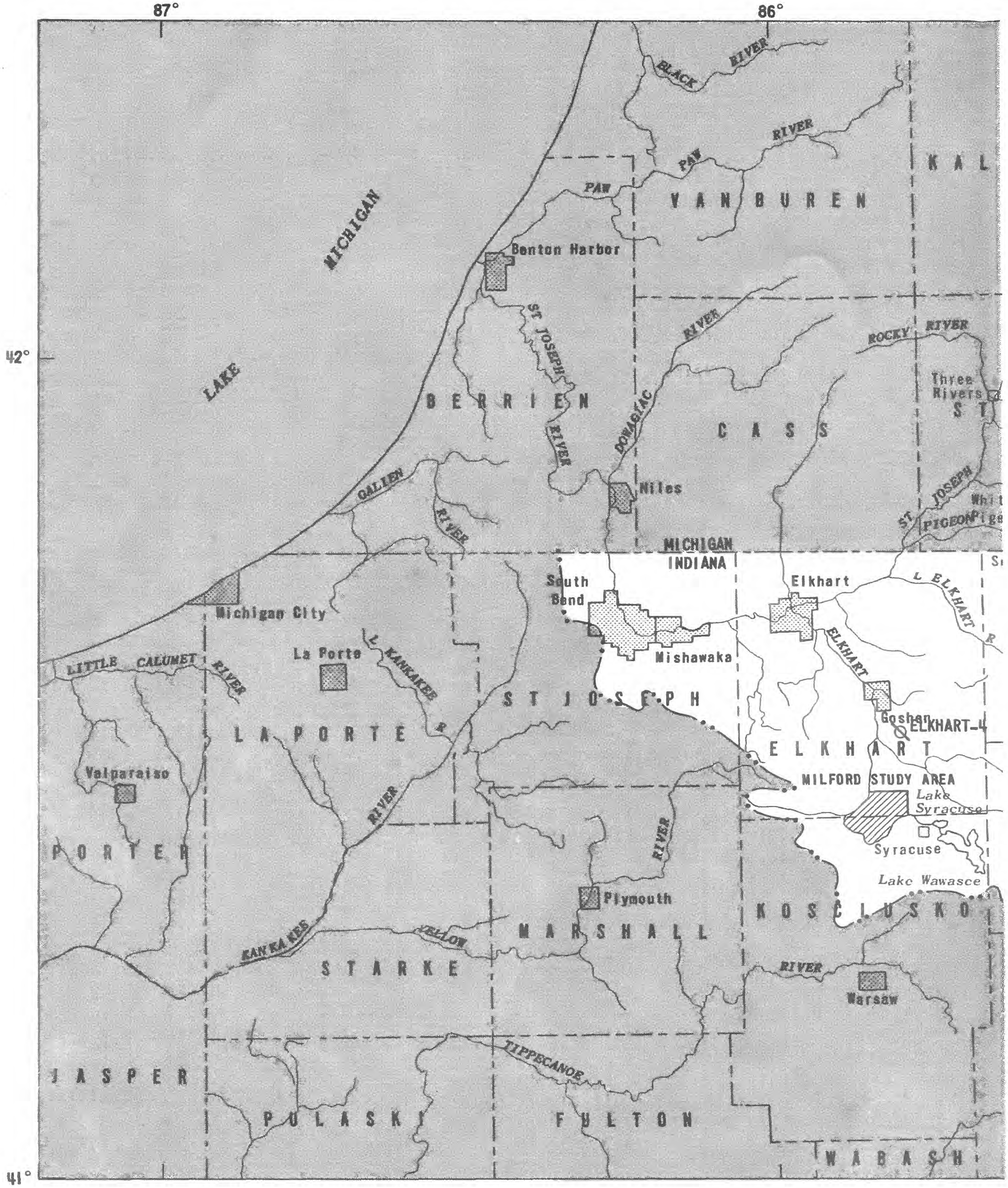
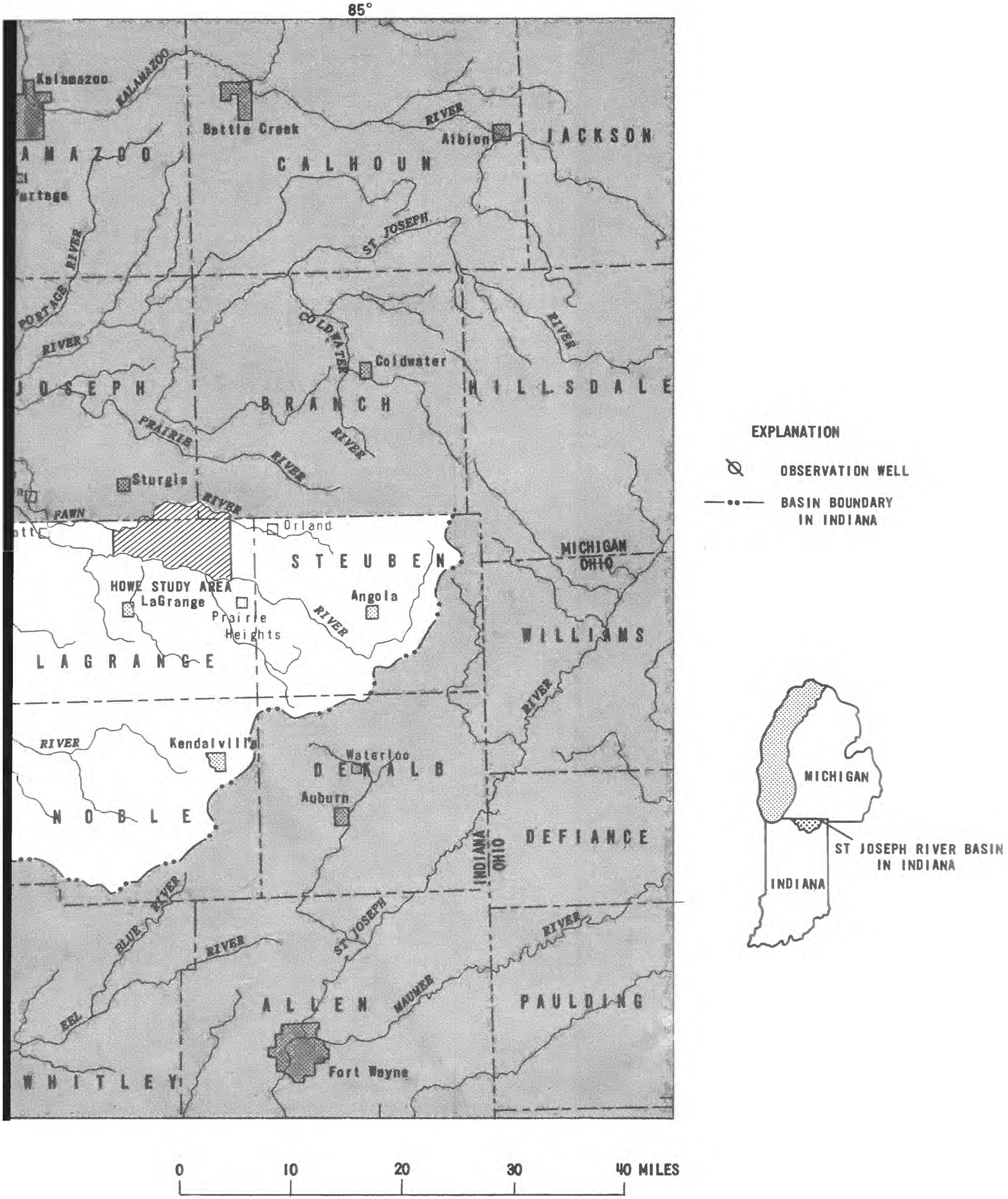


Figure 1.-- Milford and Howe areas of study and the boundary of the St. Joseph River basin in Indiana.


85°
Kalamazoo
KALAMAZOO
Battle Creek
RIVER
Albion
JACKSON
CALHOUN
Portage
ST JOSEPH
RIVER
PORTAGE
RIVER
COLDWATER
Coldwater
RIVER
JOSEPH
BRANCH
HILLSDALE
PRAIRIE
RIVER
RIVER
Sturgis
RIVER
FAWN
Orland
STEUBEN
MICHIGAN
OHIO
HOWE STUDY AREA
LaGrange
Prairie Heights
Angola
RIVER
WILLIAMS
LAGRANGE
RIVER
RIVER
Kendallville
Waterloo
DE KALB
Auburn
NOBLE
INDIANA
OHIO
DEFIANCE
BLUE RIVER
ST JOSEPH
RIVER
RIVER
EEL
ALLEN
MAUMEE
PAULDING
Fort Wayne
WHITLEY
EXPLANATION
OBSERVATION WELL
BASIN BOUNDARY IN INDIANA
MICHIGAN
INDIANA
ST JOSEPH RIVER BASIN IN INDIANA
0
10
20
30
40 MILES

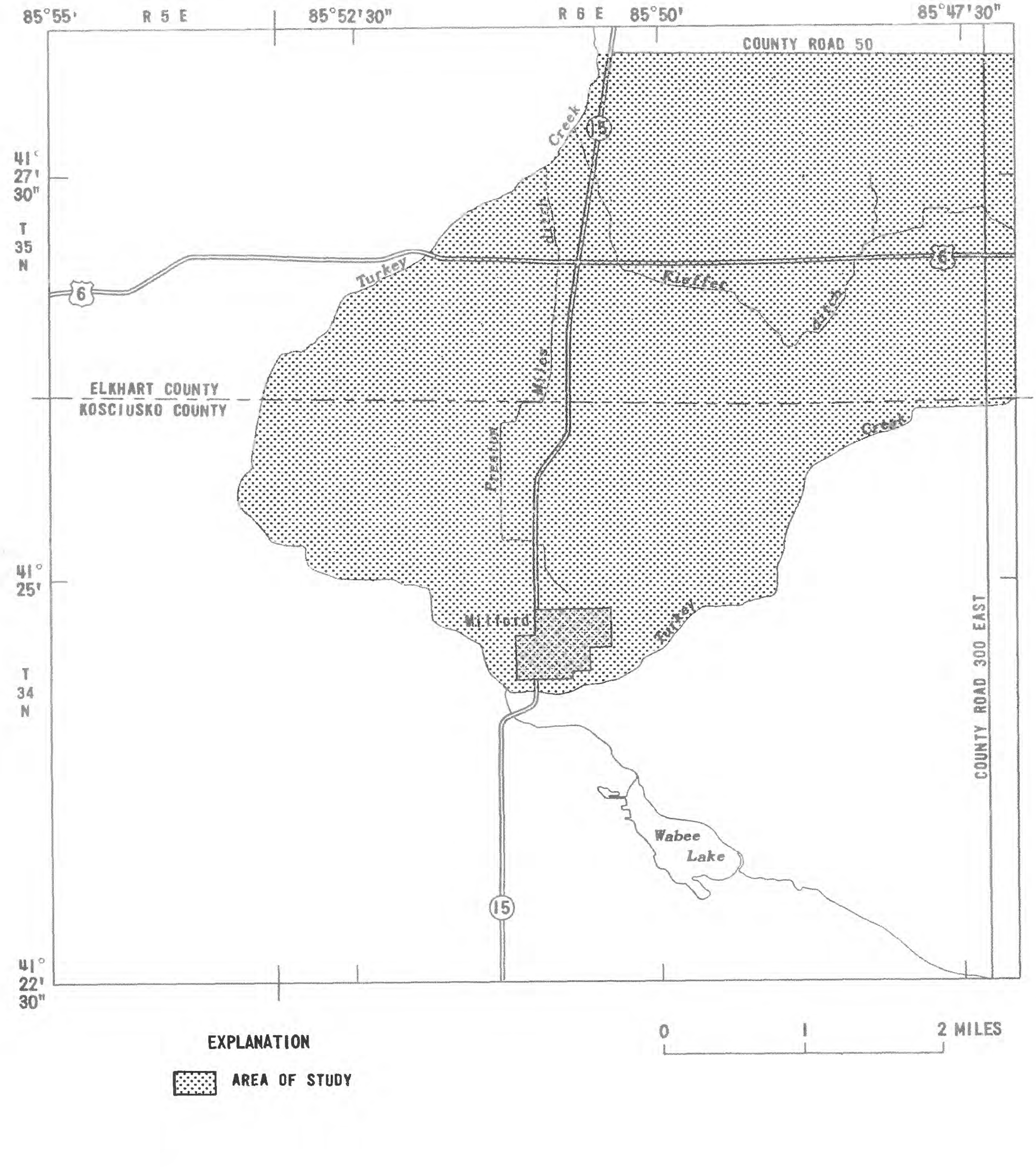


Figure 2.-- Milford area of study.

Reussow and Rhone (1975) present generalized maps of the Indiana part of the basin. Information on geology, water budget, water use, water levels in aquifers, specific capacity, and low-flow frequency of streams was presented in maps. Water-budget maps included precipitation, runoff, and evapotranspiration.

Acknowledgments

IDNR provided much of the information needed for the study. The IDNR Division of Water updated an inventory of irrigation wells, provided driller's logs for high production wells, compiled land-use information, and delineated areas with irrigable soils in the two areas of study. The Division also provided valuable suggestions and advice to the project staff at each stage of the project.

Dr. Daniel Wiersma, retired chairman of the Indiana Water Resources Research Center at Purdue University, and Dr. Rolland Wheaton, Agricultural Engineer at Purdue University, provided technical guidance in various aspects of agricultural irrigation. Dr. Wheaton wrote the calculator program for estimating requirements of water for irrigation.

Mike Jewett and Vic Virgil, the agricultural extension agents for Elkhart and Kosciusko Counties, coordinated activities and meetings between the project staff and irrigators.

Charles Phillips of Phillips and Sons Irrigation Company[1], Bristol, and James Fizel of J. and J. Irrigation Company, Lagrange, provided technical information about the irrigational systems involved in the study.

All irrigators in the study area permitted the authors to monitor their irrigational systems and to drill observation wells on their property. Several irrigators collected field data during the 1982 irrigational season. Without this support, much of the data collection would not have been possible.

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[1]Use of trade names and firm names in this report is for identification purposes only and does not constitute and endorsement by the U.S. Geological Survey.

DESCRIPTION OF THE STUDY AREA

Location

The 16.5-square-mile area is in parts of Elkhart and Kosciusko Counties (fig. 2). The area is bounded on the south and west by Turkey Creek, on the north by county road 50, and on the east by a line one-fourth mile east of county road 300 East.

Climate

Thirty years of climatic data (1941-1970) from the weather stations at Goshen and Warsaw (fig. 1) were used to determine a climatic "norm" (or average) for the study area (U.S. Department of Commerce, 1973). Average annual temperature is 50° F. The average temperature in July, the warmest month, is 73° F. Average annual precipitation is 36 in. and average precipitation for the summer (June through August) is 11 in. Annual and summer precipitation for 1982 was 32 and 9 in. (fig. 3). Thus, both annual and summer precipitation in 1982 were near the 30-year averages. Average potential evapotranspiration for north-central Indiana is 27 in/yr (Newman, 1981). Potential evapotranspiration for 1982 was estimated using average weekly air temperature (Thorthwaite, 1948) to be 26 in.

Geology

Geomorphology

The geomorphology of the Milford area is the result of deposition and erosion of glaciofluvial material during northward-retreating Wisconsin glaciation. The altitude of the land surface ranges from 810 to 1040 ft. above sea level. Outwash-plain and valley-train deposits composed of sand and gravel lie on either side of Turkey Creek and extend beyond the Milford area south to Wabee Lake and east and north along Turkey Creek (fig. 4). These deposits are referred to as outwash in the remainder of the report. Muck, peat, and marl have been deposited on the outwash in a few areas.

Deposits of till, composed mostly of clay, border the outwash. The altitude of the till is generally higher than that of the outwash. The hills near the east edge of the area are kame deposits.

Most of the soils derived from the outwash deposits are the Oshtemo-Fox association. These course-textured soils are well drained to excessively drained and respond favorably to agricultural irrigation. The major soil association derived from the deposits of till and kame is the Riddles-Crosby-Miami association. These are well drained to poorly drained soils and do not respond favorably to irrigation (Chelf, 1983).

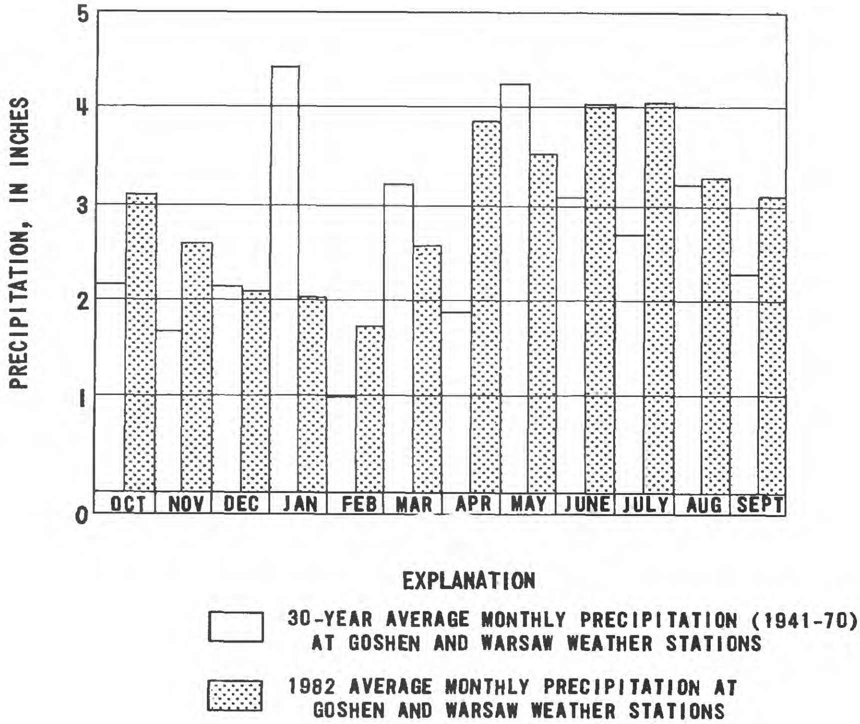

Figure 3.-- Thirty-year average monthly precipitation (1941-70) and 1982 average monthly precipitacion measured at Goshen and Warsaw weather stations.

Glacial deposits

Eighteen observation wells were drilled at fourteen locations to improve knowledge of the lithology and provide water levels (fig. 5). Gamma radiation logging was done on all wells. Gamma-radiation logs and driller's logs were used to construct a composite log for each well. These data and the glacial history of the area were used by the Indiana Geological Survey and the authors to develop an interpretation of the lithology.

The thickness of glacial drift generally ranges from 100 to 400 feet. Two distinct layers of sand and gravel, situated one above the other in the uppermost 100 ft are seperated by a layer of clay, silt, and sand (fig. 6).

REVISED FIGURE 4

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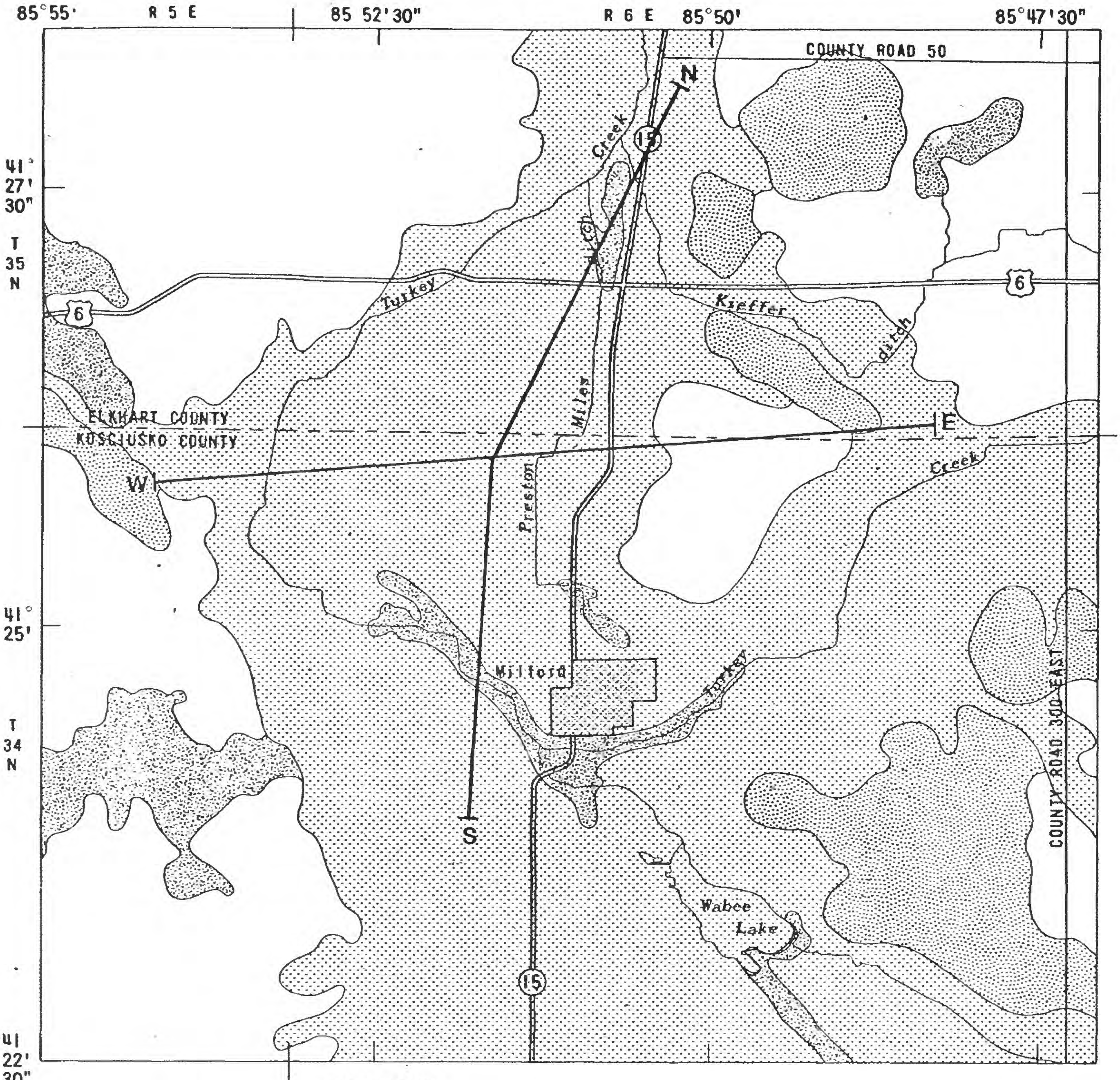


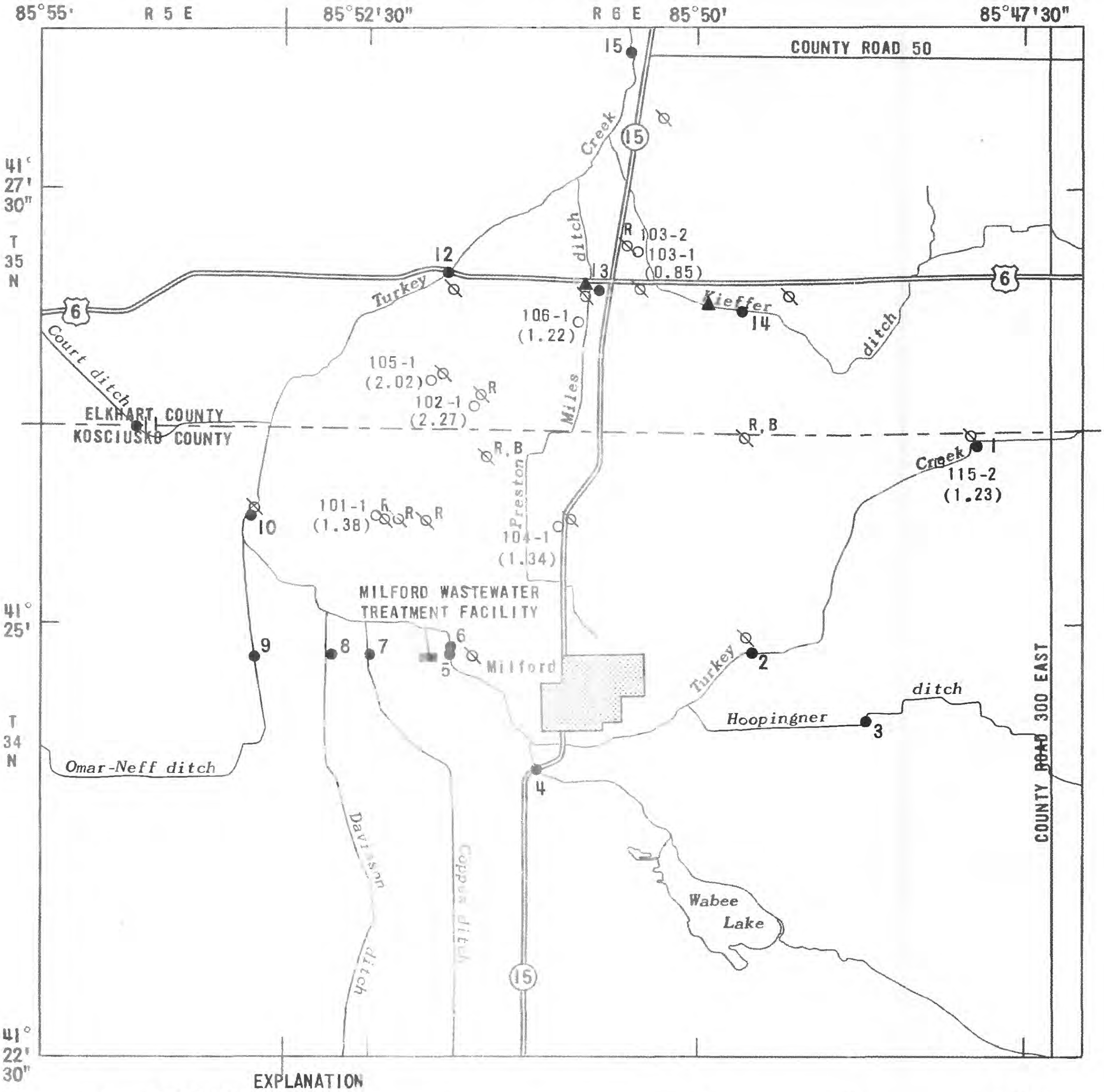
EXPLANATION

- MUCK, PEAT, AND MARL--Recharge equals 10.5 inches per year
- SAND AND GRAVEL--Outwash. Recharge equals 10.5 inches per year
- TILL--Recharge equals 4.2 inches per year
- GRAVEL, SAND, AND SILT--Kames. Recharge equals 6.3 inches per year

0 1 2 MILES

W—E TRACE OF GEOLOGIC SECTION

Figure 4.-- Surficial geology, corresponding recharge rates, and locations of geologic sections.



EXPLANATION

OBSERVATION WELL

CONTINOUS-RECORD OBSERVATION WELL

WELL PUMPED FOR IRRIGATION

STREAM PUMPED FOR IRRIGATION

101-1 WELL DESIGNATION

(1.38) RATE OF PUMPING--In cubic feet per second

GAGING STATION

4 STREAMFLOW MEASUREMENT SITE AND DESIGNATION

OBSERVATION WELL DRILLED TO BEDROCK SURFACE

Figure 5.-- Data-collection sites.

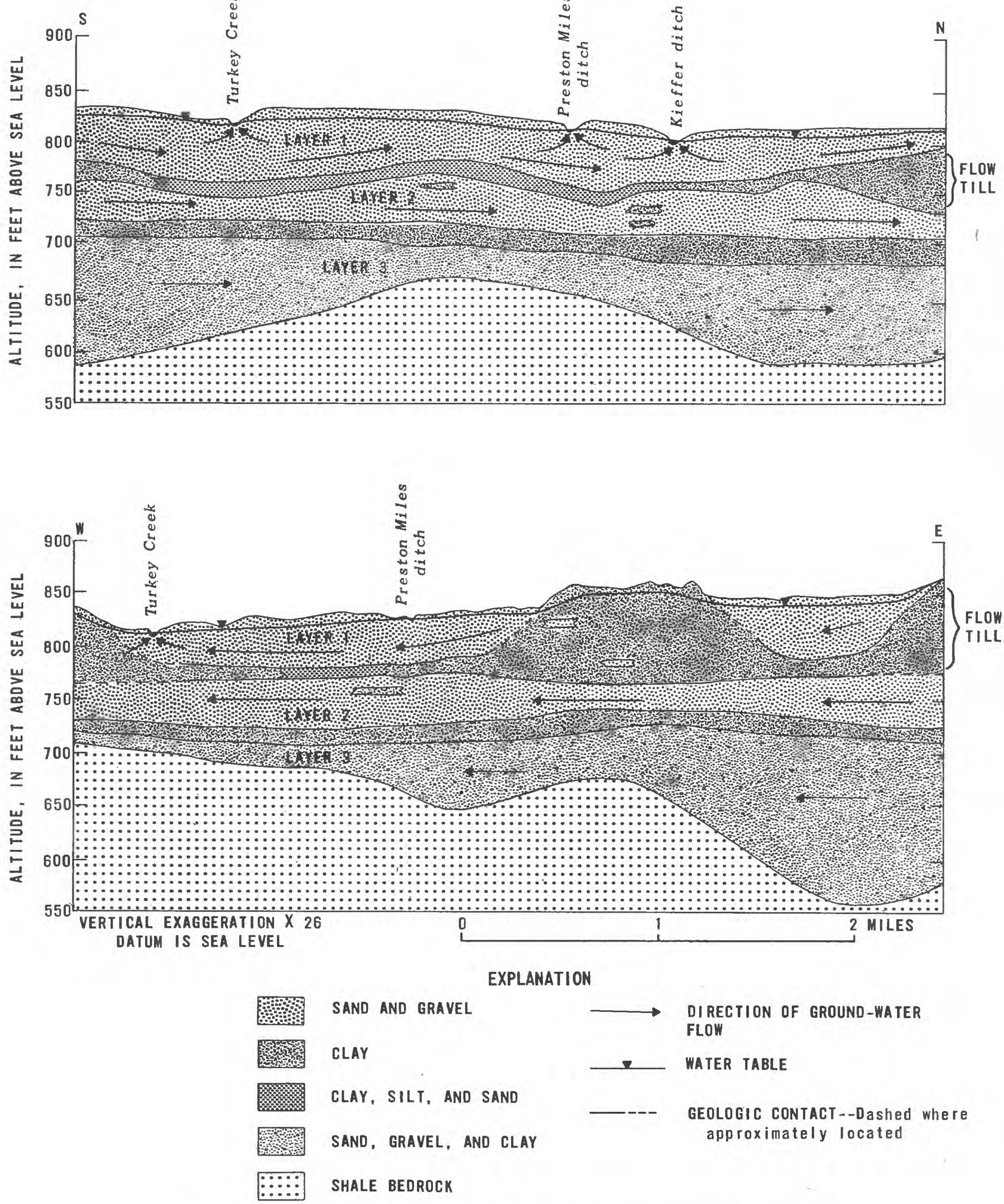


Figure 6.-- Generalized north-south and east-west geologic sections.

Thickness of the upper layer of sand and gravel ranges from 5 to 68 ft and averages 40 ft (fig. 7). The areal distribution of this layer coincides with the surficial outwash deposits shown in figure 5 but also includes muck, peat, and marl, and the kame deposits in the northeast near Kieffer ditch. The altitude of the bottom of layer 1 varies by 50 ft and is highest near till boundaries (fig. 8).

The clay layer beneath the surficial sand and gravel (layer 1) has been identified as a "flow till" (Ned Bleuer, Indiana Geological Survey, oral commun., 1983). The layer formed downslope from a retreating glacier as a semiplastic flowage. Large-grained material settles to the bottom of flow tills (Sugden and John, 1976, p. 224). The authors assumed that this layer was composed of clay, silt and sand and was continuous over the area. The thickness of the flow till ranges from 3 to 45 ft and averages 17 ft (fig. 9).

The deeper sand and gravel ranges from 17 ft to 62 ft in thickness and averages 37 ft in thickness (fig. 10). It generally coincides in areal distribution with layer 1 but is also present beneath the isolated surficial till deposit south of Kieffer ditch (fig. 4).

Little is known about the sediments below a depth of 100 ft. A clay till at least 8 ft thick below layer 2 (fig. 6) is mentioned on four driller's logs on file with the IDNR. Logs from the two U.S. Geological Survey wells drilled to bedrock (fig. 5) indicate that sand with gravel and clay (layer 3 in fig. 6) is present below the clay till. The thickness of the deposits between layer two and the bedrock surface generally ranges from 25 ft in the west to 150 ft in the east (fig. 11). No production wells pump water from layer 3 probably because the clay content of these deeper deposits is greater than that of the shallow deposits and because layer 2 is highly productive.

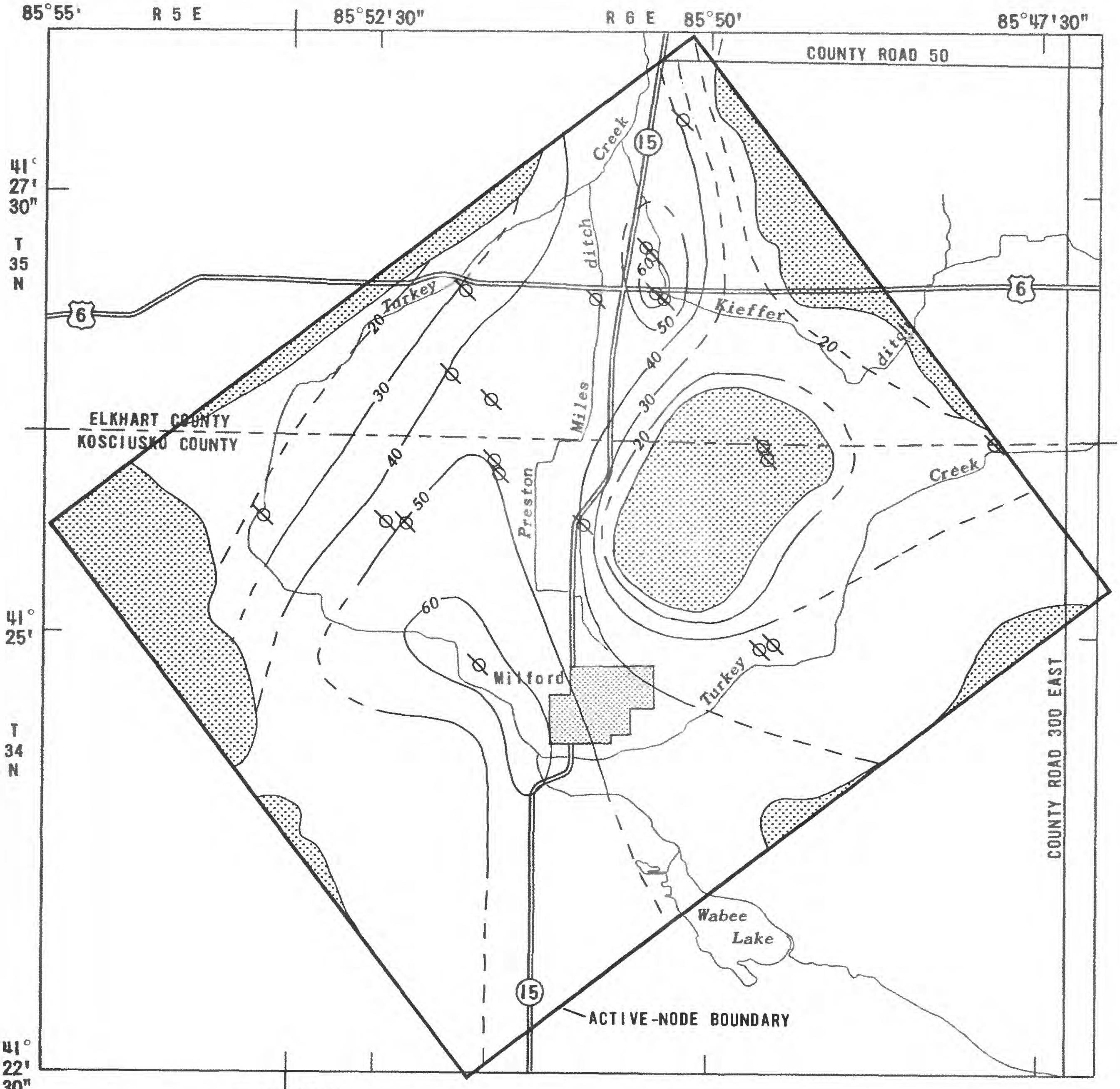
Bedrock

Seismic refraction data were collected by the Indiana Geological Survey. These data together with logs of oil and gas wells were used to map the surface of the bedrock (fig. 12).

The bedrock underlying the glacial deposits in order of increasing age is composed of the Sunbury and Ellsworth Shales of Mississippian and Devonian age and the Antrim Shale of Devonian age (Johnson and Keller, 1972). The altitude of the surface of the bedrock ranges from 525 ft to 700 ft above sea level (fig. 12).

Land use

Land use for the study area was mapped for the following categories: forested, urban, and agricultural (Chelf, 1983). The amount of land in the forested and urban categories was measured from maps by using a computer-



EXPLANATION

0 1 2 MILES

OUTWASH

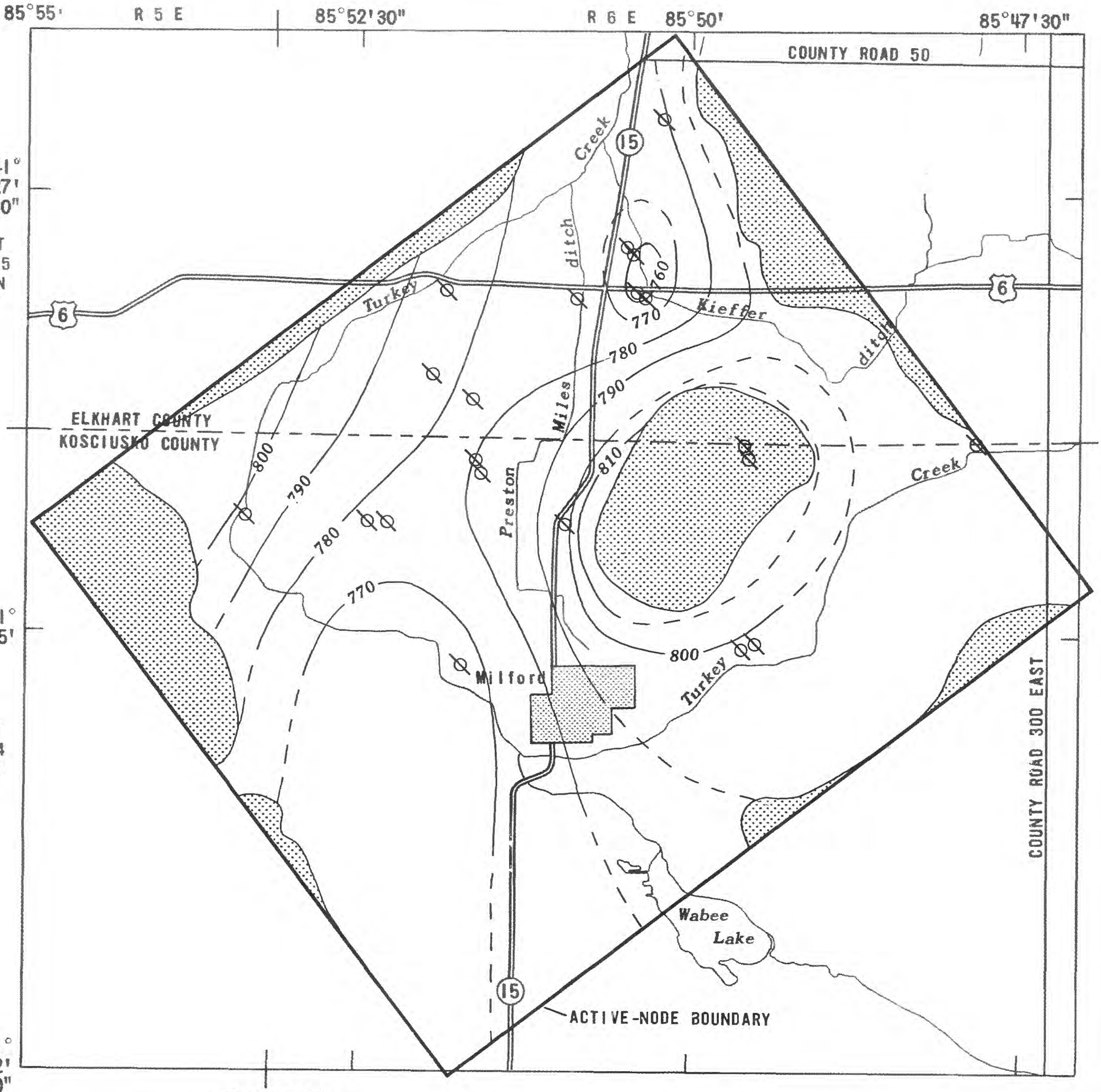
TILL

—50— — LINE OF EQUAL THICKNESS OF AQUIFER, LAYER 1--Dashed where approximately located. Interval 10 feet

OBSERVATION WELL--Lithologic information site

(The thickness of layer 1 was estimated on the basis of lithologic information from observation wells and interpretation of the nature of the glacial sediments)

Figure 7.-- Thickness of layer 1.

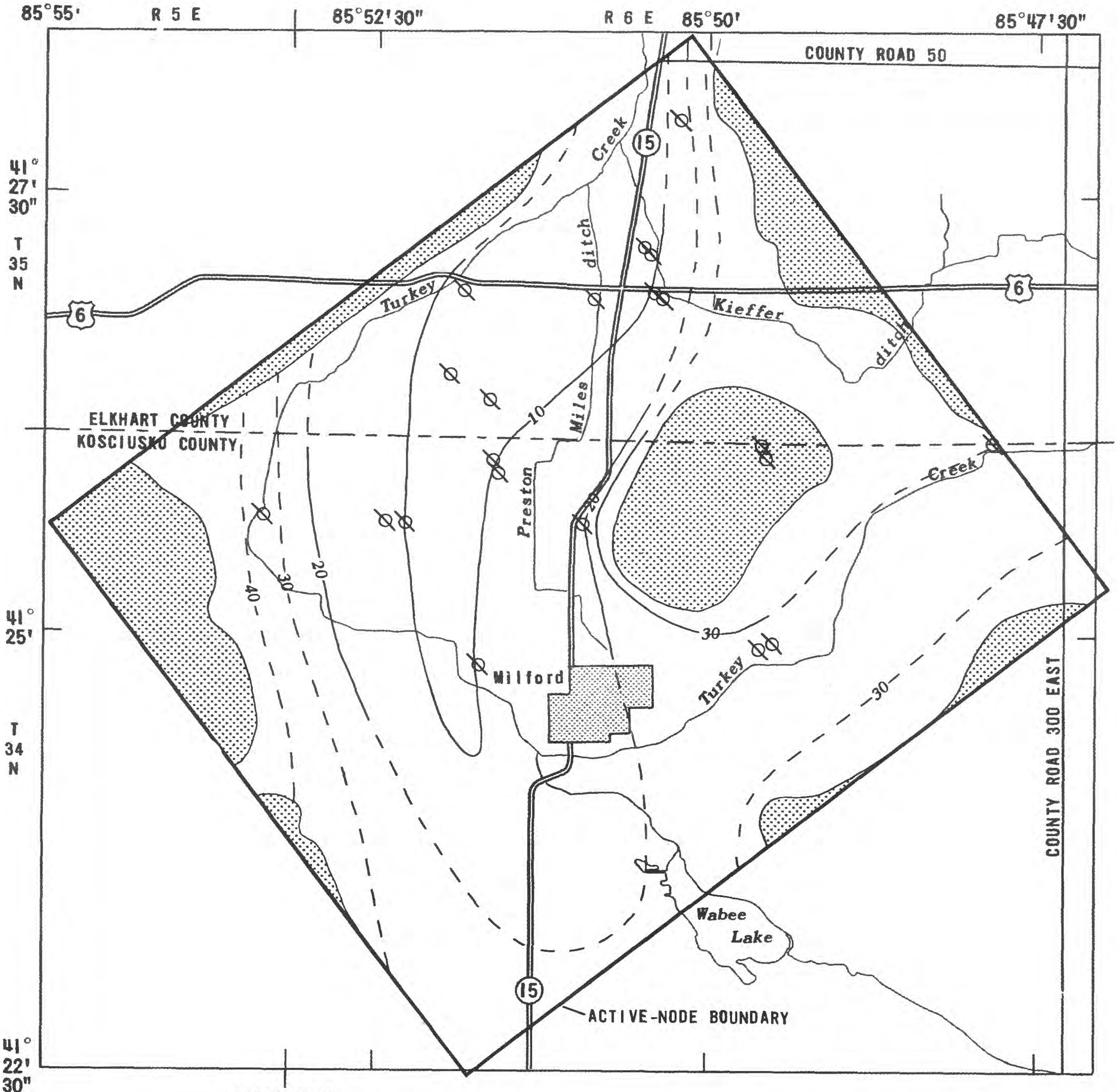


EXPLANATION

	OUTWASH
	TILL
—770— —	STRUCTURE CONTOUR--Shows altitude of base of layer 1. Dashed where approximately located. Contour interval 10 feet. Datum is sea level
	OBSERVATION WELL--Lithologic information site

(The altitude of the base of layer 1 was estimated on the basis of lithologic information from observation wells and interpretation of the nature of the glacial sediments)

Figure 8.-- Altitude of base of layer 1.



EXPLANATION

OUTWASH

TILL

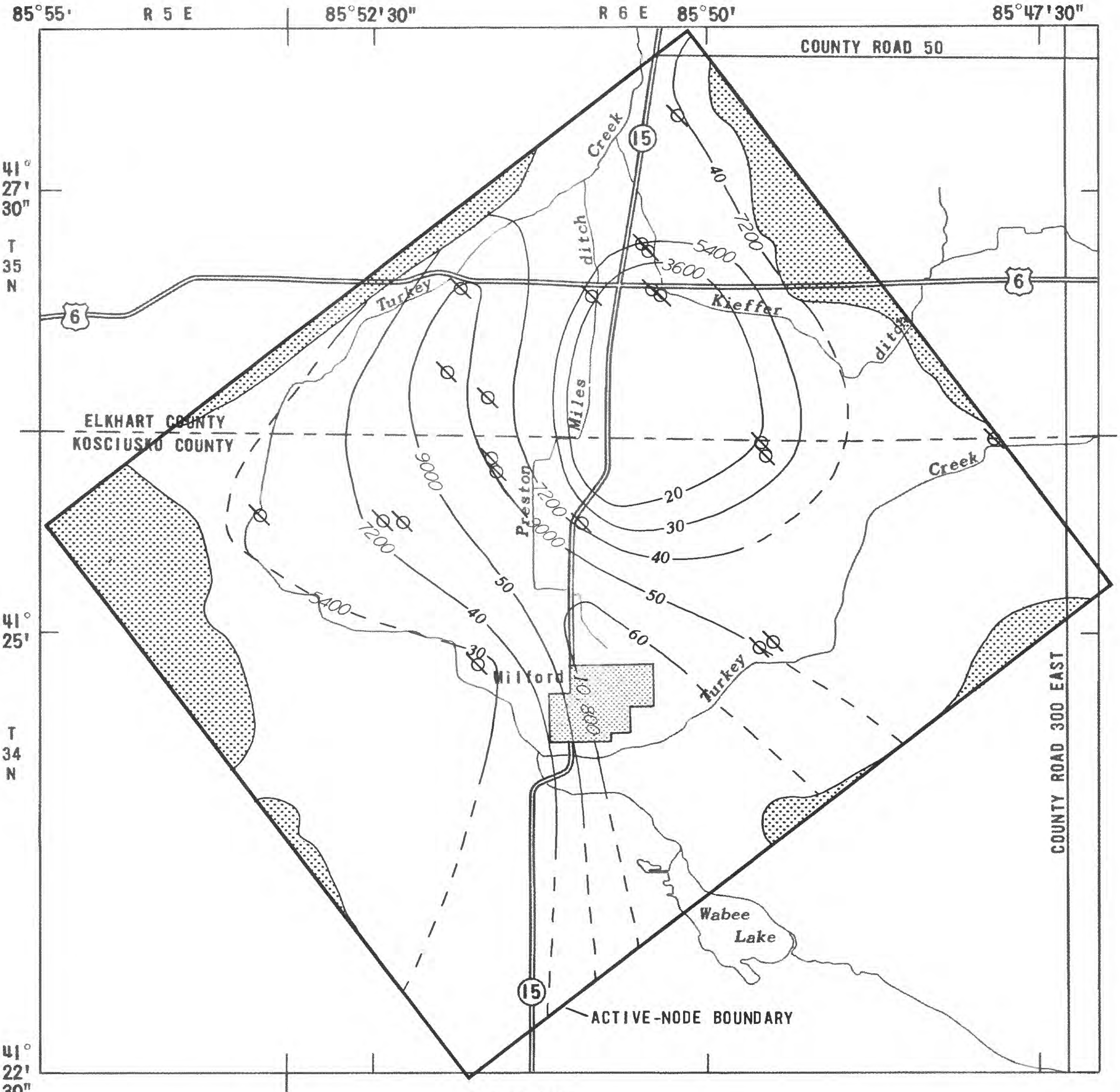
—20— — LINE OF EQUAL THICKNESS OF FLOW TILL BENEATH LAYER 1--Dashed where approximately located. Interval 10 feet

OBSERVATION WELL--Lithologic information site

0 1 2 MILES

The thickness of the flow till beneath layer 1 was estimated on the basis of lithologic information from observation wells and interpretation of the nature of the glacial sediments)

Figure 9.-- Thickness of the flow till beneath layer 1.



EXPLANATION

0 1 2 MILES

OUTWASH

TILL

—30— — LINE OF EQUAL THICKNESS OF AQUIFER, LAYER 2--Dashed where approximately located. Interval 10 feet

OBSERVATION WELL--Lithologic information site

—5400— — LINE OF EQUAL TRANSMISSIVITY, LAYER 2--Dashed where approximately located. Interval 1800 square feet per day

(The thickness of layer 2 was estimated on the basis of lithologic information from observation wells and interpretation of the nature of the glacial sediments)

Figure 10.-- Thickness and transmissivity of layer 2.

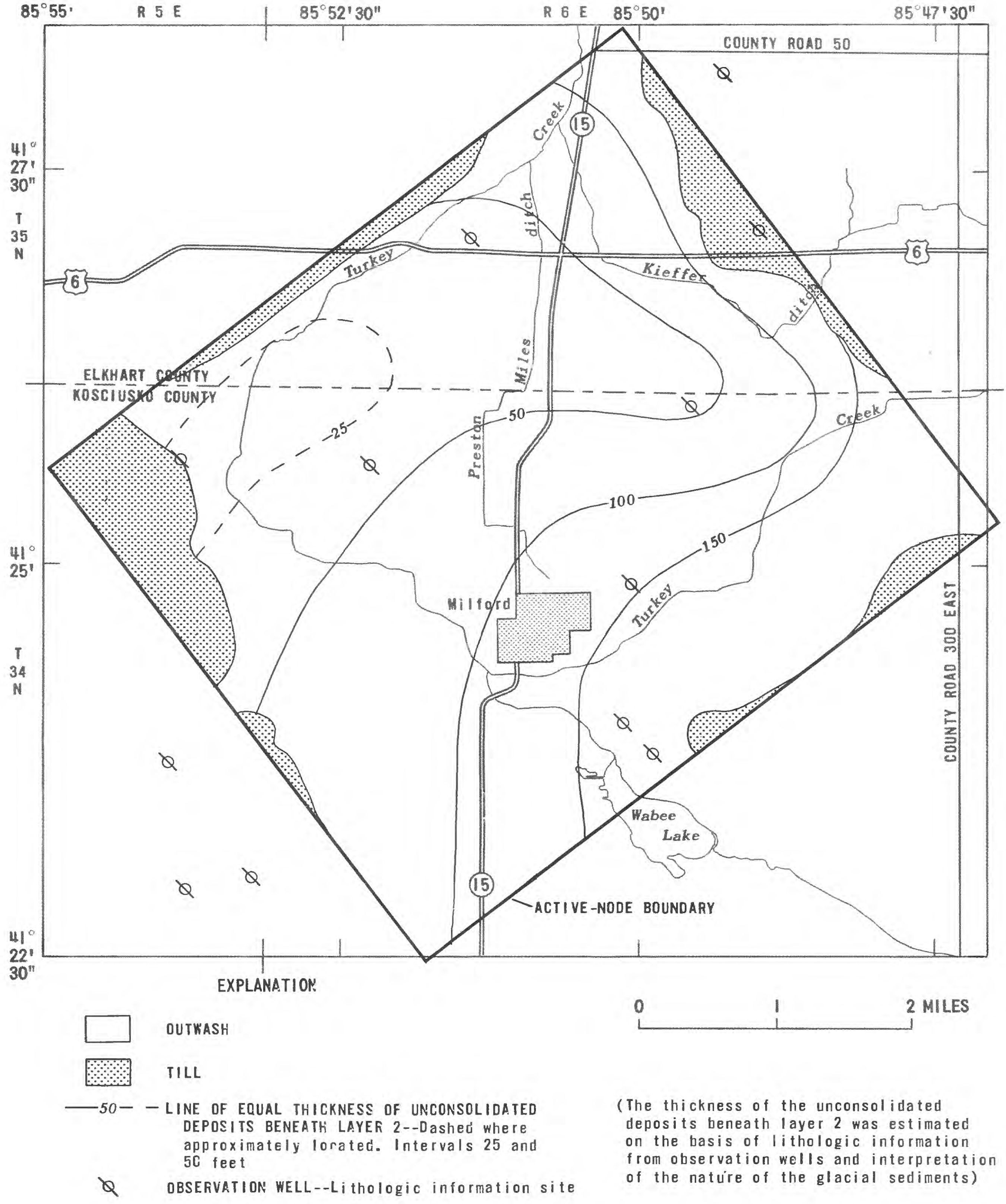
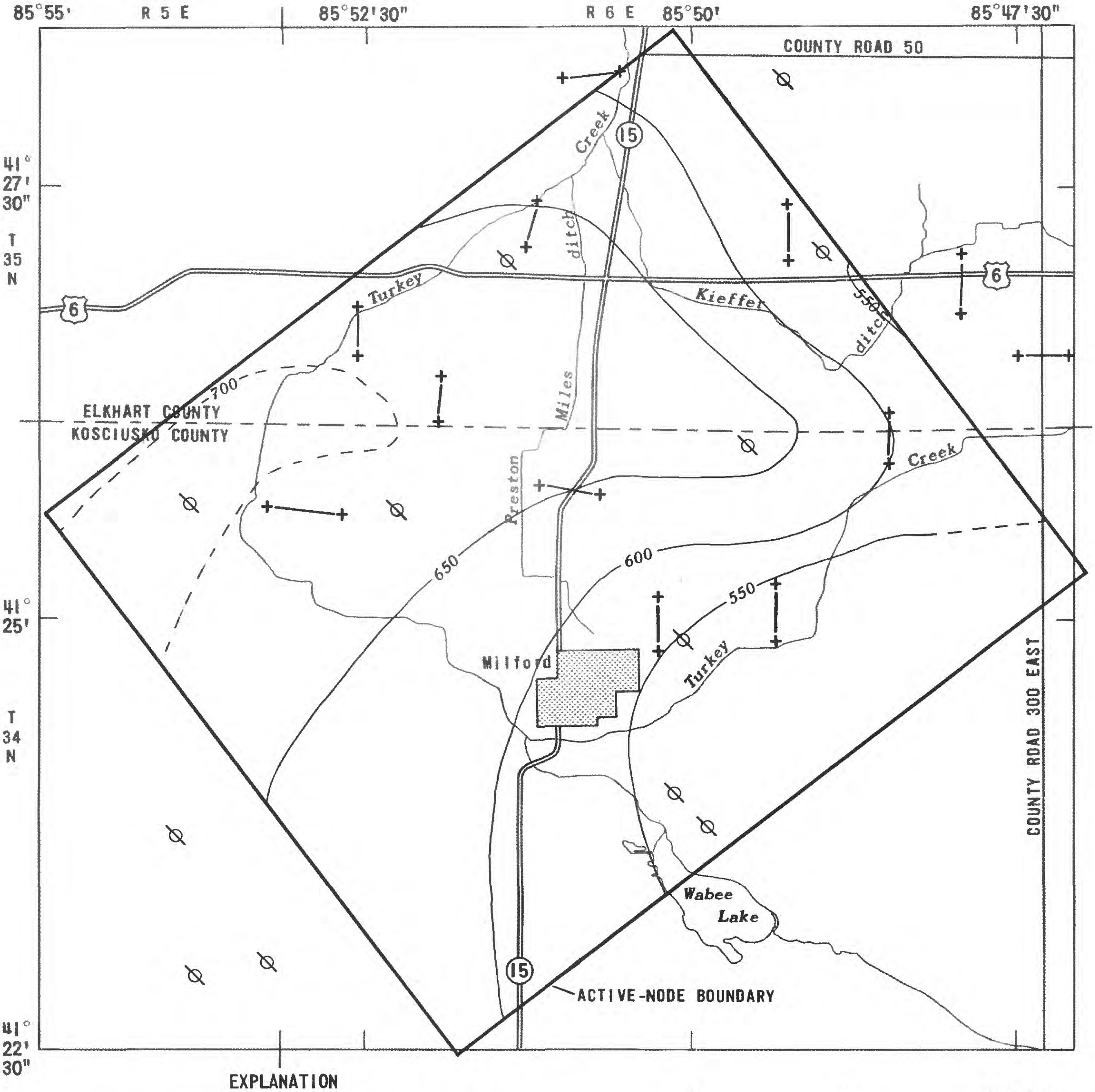


Figure 11.-- Thickness of unconsolidated deposits beneath layer 2.



EXPLANATION

—600— — BEDROCK CONTOUR--Shows altitude of bedrock surface. Dashed where approximately located. Contour interval 50 feet. Datum is sea level

OBSERVATION WELL--Lithologic information site

SEISMIC REFRACTION TRANSECT

0 1 2 MILES

(The altitude of the surface of the bedrock was estimated on the basis of (1) observation wells that reach the bedrock, (2) seismic refraction transects, and (3) geologic interpretation of the nature of the bedrock and overlying galcial sediments)

Figure 12.-- Altitude of the surface of the bedrock.

assisted digitizing program. Agricultural land was estimated as a residual (fig. 13). Most of the 16.5 mi^2-area (90 percent) is used for agricultural purposes (table 1). The remaining 10 percent comprise forested and urbanized areas. Several natural lakes, including Wabee Lake, which lie immediately south and east of the Milford area, are used for recreation. No lakes are within the study area. The few marshy areas were too small to be mapped.

Table 1.--Area of land use and percentage of total area

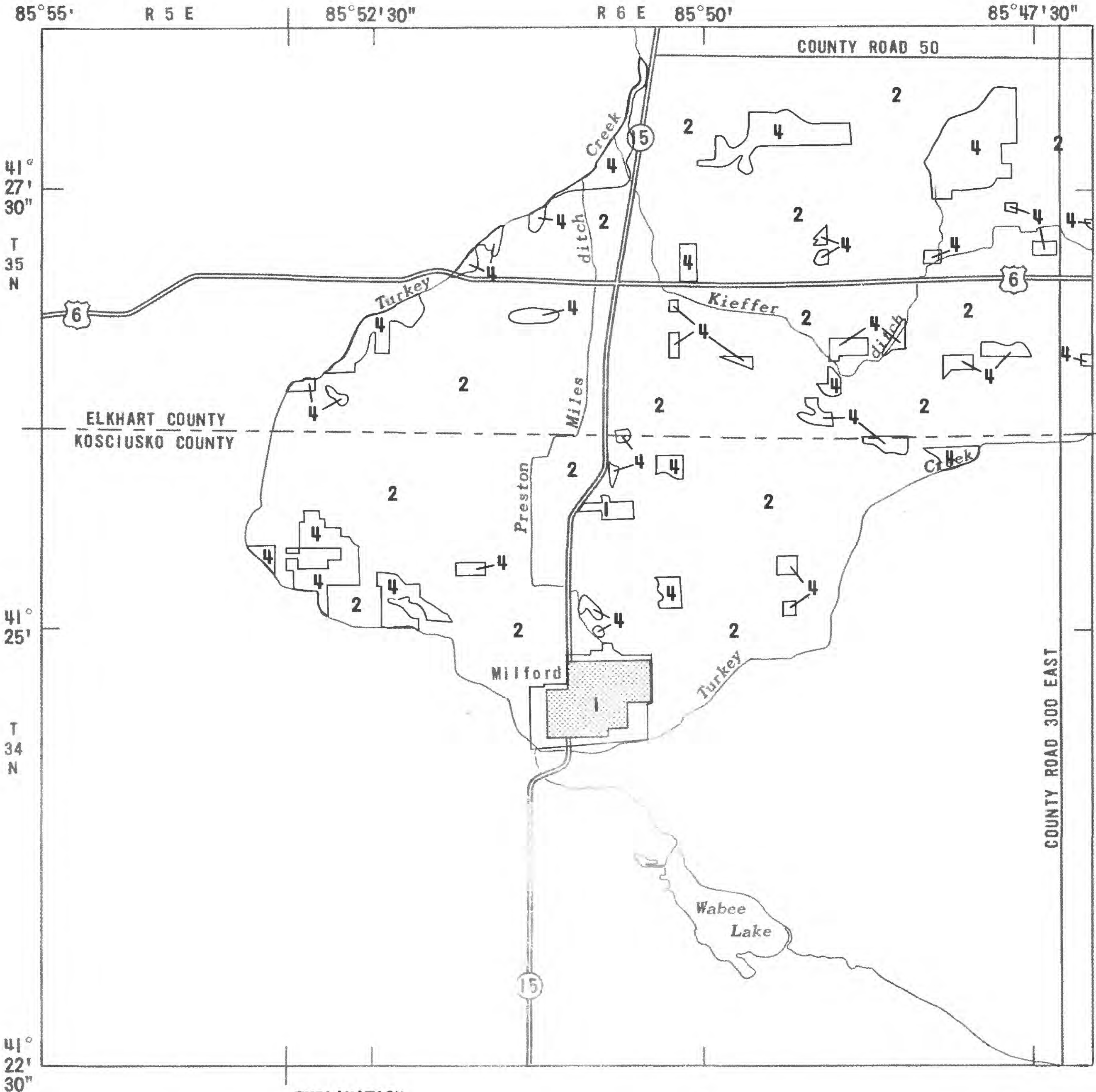
Category	Area, (mi^2)	Percent of total
Agricultural	14.8	90
Forested	1.3	8
Urban	0.4	2
Total	16.5	100

Water Use

Water use was estimated for water year 1982 in several ways. Where feasible, a Clampitron portable flow meter (Luckey and others, 1980) was used to measure flow in pipes, and a time totalizer was used to measure total hours of pump operation. Where it was not practical to use this equipment, an estimate of use was obtained from the user and (or) from the installer. Estimates of irrigated acreage were obtained from irrigators and (or) equipment installers.

Water use from private wells was estimated by assuming a rural population density of 45 people per square mile for Kosciusko and Elkhart Counties (Patricia Watkins, Van Buren Township, Kosciusko County, oral commun., 1983) and a rate of water use of 76.3 gal/d per person (Governor's Water Resources Study Commission, 1980, p. 479).

Water use can be divided into two categories: Consumptive use--use that results in water not available for reuse locally, and nonconsumptive use--use that results in water available for reuse locally. Agricultural irrigation was the largest consumptive use of water in 1982 (table 2). In June, July, and August, six irrigational wells and one surface-water withdrawal system pumped a total of 55.8 Mgal of water to irrigate row crops (mostly corn). This pumpage was 46 percent of the annual pumpage and about 73 percent of the pumpage during the irrigational season of June through August.



EXPLANATION

LAND USE CLASSIFICATIONS

1 Towns

2 Agriculture

4 Forest

Figure 13.-- Land use.

Table 2.--Water use for summer (June through August) 1982 and for the 1982 water year

Use	Annual pumpage (Mgal)	Percentage of annual pumpage	Summer pumpage (Mgal)	Percentage of summer pumpage
Irrigational:[1]				
Well	31.7	42.4	51.7	67.9
Surface withdrawals	4.1	3.4	4.1	5.4
Total	55.8	45.8	55.8	73.3
Domestic:[2]				
Municipal wells	32.8	31.3	11.5	15.1
Private wells	20.0	16.4	6.7	8.8
Total	58.2	47.7	18.2	23.9
Industrial wells[2]	7.9	6.5	2.1	2.8
Total	121.9	100.0	76.1	100.0

[1] Primarily consumptive use.
[2] Primarily nonconsumptive use.

Domestic use in water year 1982, including water pumped by the Milford municipal wells and private wells, was 58.2 Mgal or 48 percent of the total annual pumpage. Though this represents more water than that used for irrigation, it is primarily a nonconsumptive use of water--that is, most of the water is returned to Turkey Creek as treated sewage or to ground water as septic-tank drainage. During the irrigational season (June through August), domestic use was approximately 18.2 Mgal or 24 percent of the pumpage during the 3-month irrigational season. Municipal pumpage for water year 1982 by the town of Milford was estimated to be 32.8 Mgal for the year and 11.5 Mgal for June, July, and August (Ronald Connelly and William Knowles, Milford Municipal Water Company, oral commun., 1983). Private use was estimated to be 20 Mgal for water year 1982. One-third of the water for private use was assumed to have been used during June, July, and August because of the higher than average demand during these 3 months.

Industrial water use was approximately 7.9 Mgal or 6 percent of the total for the year, but only 2.1 Mgal or 3 percent of the total for the 3-month irrigational season.

The Milford wastewater treatment plant, located 1 mile downstream of Milford, discharged 11 Mgal of treated sewage into Turkey Creek in water year 1982 (Donald R. Daley, Sanitarian, Indiana State Board of Health, oral commun., 1983).

WATER RESOURCES

Surface-water System

All surface drainage is to Turkey Creek either directly to the main channel or by way of Preston Miles or Kieffer ditches (fig. 2). All three streams have been dredged along much of their length in the study area. The flow in Turkey Creek is regulated by a variable-head dam at the outlet to Lake Syracuse 2 miles upstream from the east edge of the study area. The total drainage area of Turkey Creek at the north (downstream) edge of the area is 150 mi^2. The average flow at this location is 105 ft^3/s. The drainage areas for Preston Miles and Kieffer ditches at their mouths are 3.8 and 5.3 mi^2 and average flows are 1.2 and 0.7 ft^3/s. Average flows for Turkey Creek, Preston Miles ditch and Kieffer ditch were based on flow duration curves (see section "Connection Between Ground Water and Surface Water"). Turkey Creek and Preston Miles ditch flow over primarily sand and gravel (outwash). Kieffer ditch flows over sand and gravel (outwash) downstream from the stream gaging station and over mostly clay (till) in its headwaters (figs. 4 and 5).

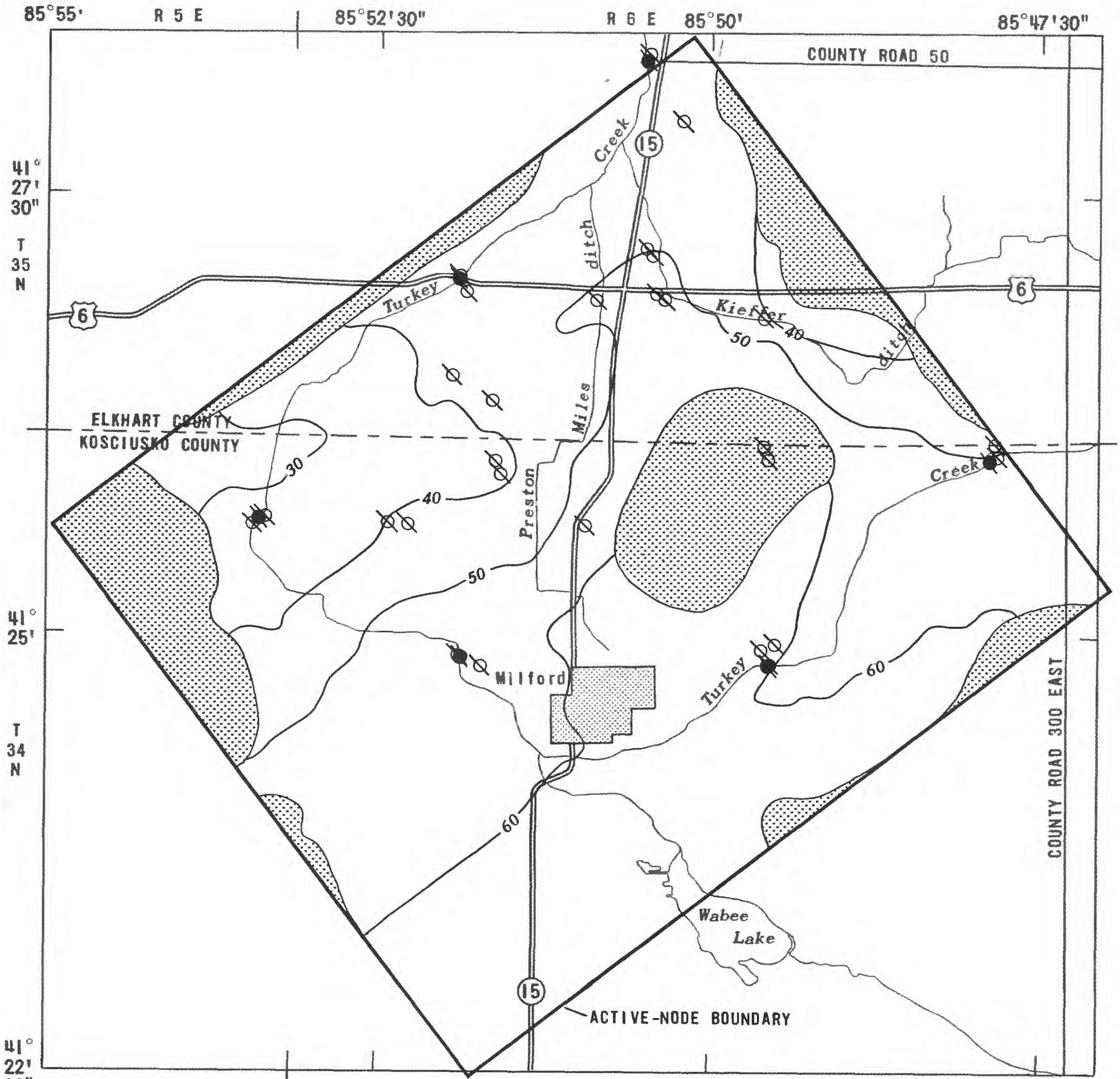
Ground-water System

Hydraulic Characteristics of the Glacial Deposits and the Bedrock

Glacial deposits

The glacial drift was assumed to comprise three aquifers (layers 1, 2, and 3 in fig. 6) separated by two clay layers. Aquifer 1 is unconfined and aquifers 2 and 3 are confined. Aquifers 1 and 2 are composed primarily of highly permeable sand and gravel. Aquifer 3 was assumed to be less permeable because of its higher clay content. The composition of the two confining layers ranged from sandy to silty clay.

Hydraulic conductivity was determined from specific-capacity data for four wells screened in aquifer 1 and for three wells screened in aquifer 2 by using techniques described in Theis (1963) and Brown (1963). Transmissivity was calculated as the product of hydraulic conductivity and aquifer thickness. Values of conductivity for aquifer 1 ranged from 220 to 390 ft/d and averaged 280 ft/d. The unit of hydraulic conductivity, ft/d, used in this report is the reduced form of cubic foot per day per square foot of aquifer $[(ft^3/d)/ft^2]$. The values are near the middle of the range from 13 to 668 ft/d reported by Pettijohn (1968, p. 10) for outwash in the St. Joseph River basin. The saturated thickness of aquifer 1 ranges from almost 20 ft near the edge of the till to 60 ft in the southern part of the area (fig. 14). For an average



EXPLANATION

OUTWASH

TILL

—60— LINE OF EQUAL THICKNESS OF SATURATED MATERIAL, AQUIFER 1--Interval 10 feet

OBSERVATION WELL--Lithologic information site

STREAMFLOW MEASUREMENT SITE

0 1 2 MILES

The saturated thickness of aquifer 1 was estimated on the basis of water levels from observation wells and streamflow measurement sites, and on interpretation of the nature of the glacial material

Figure 14.-- Saturated thickness of aquifer 1.

hydraulic conductivity of 280 ft/d, the transmissivity of the aquifer would range from about 5,600 to 16,800 ft^2/d. The unit of transmissivity, ft^2/d, used in this report is the reduced form of cubic foot per day per foot of aquifer $[(ft^3/d)/ft]$.

Several values of vertical hydraulic conductivity for the flow till were calculated by using aquifer-test data collected near well 100-1 (fig. 3). These values range from 0.044 to 0.21 ft/d (J. G. Peters, U.S. Geological Survey, oral commun., 1983) and are near the high end of the range of values for glacial till (1×10^{-6} to 1 ft/d) reported by Freeze and Cherry (1979, p. 29). The conductivity of the flow till was assumed to range from 0.01 ft/d in the eastern part of the area, where the clay content of the till is high, to 0.1 ft/d in the west-central part of the area where till has a higher sand content.

Hydraulic conductivity of aquifer 2 ranges from 60 to 355 ft/d and averages 180 ft/d. Aquifer thickness generally ranges from 20 to 60 ft (fig. 10). Based on average hydraulic conductivity, transmissivity ranges from 3,600, beneath the isolated surficial till, to more than 10,800 ft^2/d in the southeast.

No specific capacity data were available for aquifer 3; however information from driller's logs indicates more clay in aquifer 3 than in either aquifers 1 or 2. The hydraulic conductivity of aquifer 3 is probably less than that of aquifers 1 or 2.

Specific yield for aquifer 1 and storage coefficient for aquifer 2 were calculated from aquifer-test data to be 0.15 and 0.0003 (J. G. Peters, U.S. Geological Survey, oral commun., 1983).

Bedrock

No hydraulic data were available for the shale bedrock which generally has low hydraulic conductivity (Freeze and Cherry, 1979, p. 29). The bedrock was assumed to be impermeable compared with the highly permeable outwash.

Connection Between Ground Water and Surface Water

The hydraulic connection between ground water and surface water determines how rapidly water moves into or out of a stream in response to differences in heads in the stream and in the aquifer. This response is influenced by the hydraulic conductivity and thickness of the streambed. Neither of these variables was measured; however, inferences about the connection between ground and surface water were made from unit discharge, flow duration, and streamflow hydrographs.

Unit discharge, which is discharge divided by drainage area, can be helpful in assessing streambed leakage. In general, the higher the unit discharge during periods of low flow, the greater the streambed leakage.

In late summer 1982, after a 3-week period of no rain, discharge was measured concurrently at six sites on Turkey Creek and nine surface-water inflows to Turkey Creek (fig. 5). Unit discharge was calculated for 14 of these 15 sites (table 3). Unit discharge for site 5, the Milford waste-water treatment plant, could not be defined.

Table 3.--Flow in Turkey Creek and tributaries, August 31 through September 2, 1982

Site[1]	Site discription	Flow (ft^3/s)	Unit discharge (ft^3/s)mi^2
1	Turkey Creek at county road 250 East	5.88	0.12
2	Turkey Creek at county roads 1250 North and 100 East	8.05	.15
3	Hoopingarner ditch at county road 175 East	.11	.03
4	Outlet to Wabee Lake at State Road 15	4.75	.30
5	Outfall for Milford waste-water treatment plant	.03	--
6	Turkey Creek at county road 1250 North	16.30	.21
7	Coppes ditch at county road 1250 North	10.10	.47
8	Davisson ditch at county road 1250 North	2.36	.51
9	Omar-Neff ditch at county road 1250 North	1.79	.17
10	Turkey Creek at county road 1350 North	37.20	.27
11	Court ditch at county road 300 West	1.78	.10
12	Turkey Creek at U.S. Route 6	37.70	.27
13	Preston Miles ditch at U.S. Route 6	1.02	.28
14	Kieffer ditch at unnamed gravel road	.19	.04
15	Turkey Creek at county road 50	42.10	.28

[1]Locations of sites are given in figure 5.

Unit discharges at sites 3 and 14 on Hoopingarner and Kieffer ditches were less than 0.05 (ft^3/s)/mi^2. These ditches drain areas where soils are primarily clay, especially in their headwaters (figs. 4 and 5), whereas other sites with greater unit discharges drain areas where soils have greater amounts of sand and gravel.

The unit discharge for Preston Miles ditch (site 13) that drains sand and gravel was several times the unit discharge at Kieffer ditch (site 14). Flow in Preston Miles ditch is presumably maintained during periods of low flow by ground water, owing to greater aquifer recharge, more transmissive aquifer, and (or) a more permeable streambed, than these for Kieffer ditch.

Data from table 3 were also used to estimate the amount of ground water seeping into Turkey Creek. Accretion was 42.1 ft^3/s - 5.9 ft^3/s = 36.2 ft^3/s. Surface-water flow to Turkey Creek from sources outside the study area was 20.9 ft^3/s. Thus, total ground-water seepage from the study area to Turkey Creek, Preston Miles ditch and Kieffer ditch was 36.2 ft^3/s - 20.9 ft^3/s = 15.3 ft^3/s. Ground-water gain to Turkey Creek was indicated between each site except between sites 10 and 12, where a loss of 1.3 ft^3/s was measured.

Flow-duration curves of unregulated streams provide a second way to examine qualitatively the hydraulic connection between surface water and ground water. A flow-duration curve is a plot of unit discharge (vertical axis) and the percentage of time the discharge is equaled or exceeded (horizontal axis). Generally, a curve with a low slope is an indication of a stream with a large component of flow coming from ground water. A curve with a high slope is indicative of a stream with a small ground-water component of streamflow.

On the basis of two measurements of flow in Turkey Creek at county road 50 (site 15), flow duration at this location more closely resembles flow duration at Elkhart River at Goshen than flow duration at Turkey Creek at Syracuse (fig. 15). The gage for Turkey Creek at Syracuse is 0.2 mi downstream from the controlled outlet to Lake Syracuse. The outlet is used to maintain higher-than-normal water levels in the lake during periods of low streamflow; this results in an increased slope of the flow duration curve at its lower end. The drainage areas for Turkey Creek and Elkhart River comprise similar geologic deposits. Thus, the flow-duration curve for Turkey Creek at county road 50 was extrapolated visually from the Elkhart River flow-duration curve. Data for plotting flow-duration curves for Preston Miles and Kieffer ditches were insufficient. Therefore, the curves for the two ditches were extrapolated by correlating 11 measurements of flow in the ditches to corresponding flow at Elkhart River at Goshen (fig. 16) with techniques described by Riggs (1972).

The slope of the flow-duration curve for Preston Miles ditch is less than that of the curve for Kieffer ditch. This difference indicates that the ground-water component of flow in Preston Miles ditch is greater than that in Kieffer ditch. Measurements of flow on Kieffer ditch made at the gaging station are an indication of the characteristics of the ditch upstream from the gage. Downstream from the gage, Kieffer ditch drains an area of outwash and probably receives more seepage from the ground-water system than it does upstream.

When the flows in table 3 were measured, the flow for Elkhart River at Goshen corresponded to a 70-percent flow duration. Unit discharges at 70-percent flow duration for Preston Miles ditch, Kieffer ditch and Turkey Creek at the measurement sites described in the two preceding paragraphs are 0.24, 0.05, and 0.33 $(ft^3/s)/mi^2$.

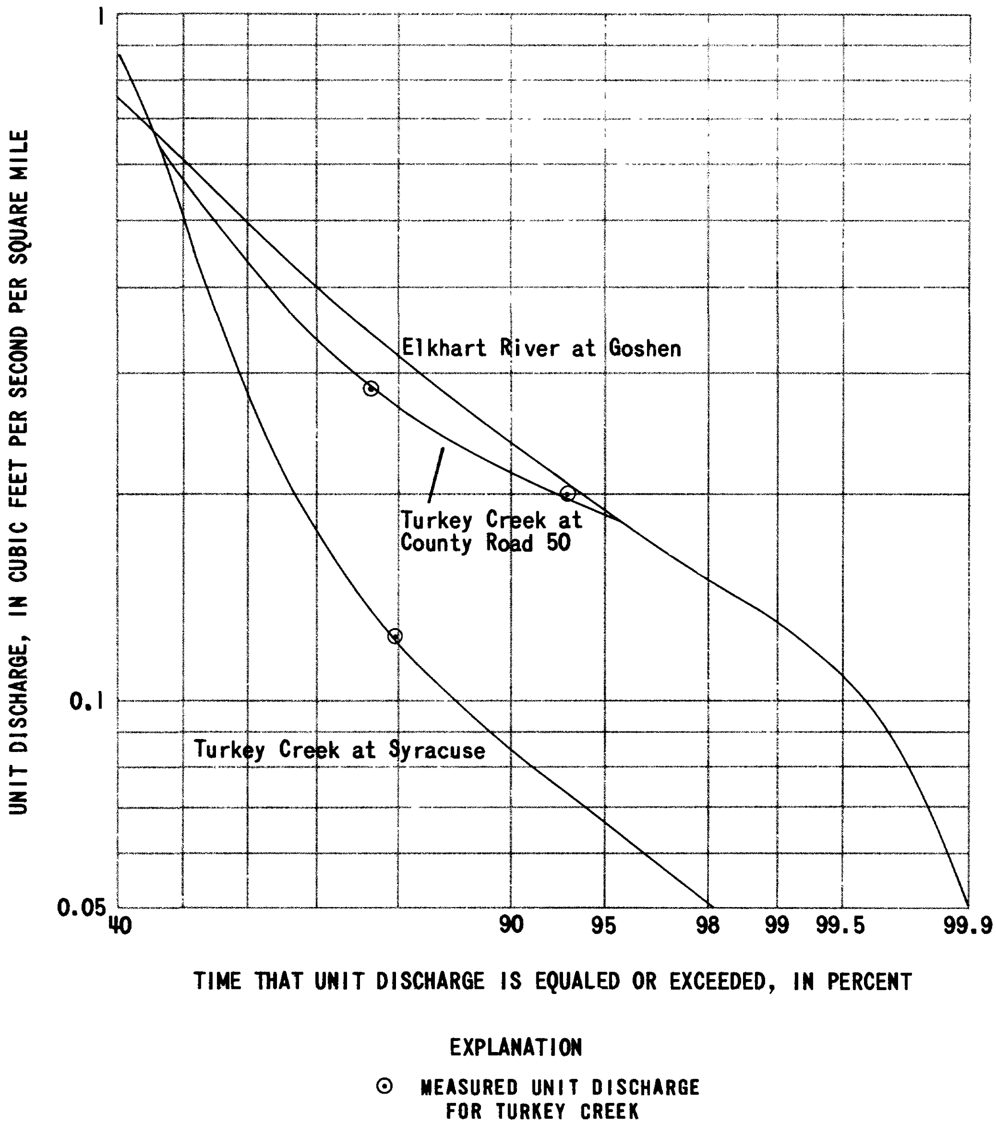


Figure 15.-- Flow-duration curves for Elkhart River at Goshen, Turkey Creek at Syracuse, and Turkey Creek at county road 50.

Comparison of the hydrographs for Preston Miles and Kieffer ditches also reveals a difference in flow characteristics between the two streams (fig. 17). The range of flow represented in the hydrograph of mean-daily discharges during June through August 1982 is greater for Kieffer ditch than for Preston Miles ditch. The lower minimum flows for Kieffer ditch are attributed to less ground water seeping into the channel during periods when streamflow is maintained by ground water.

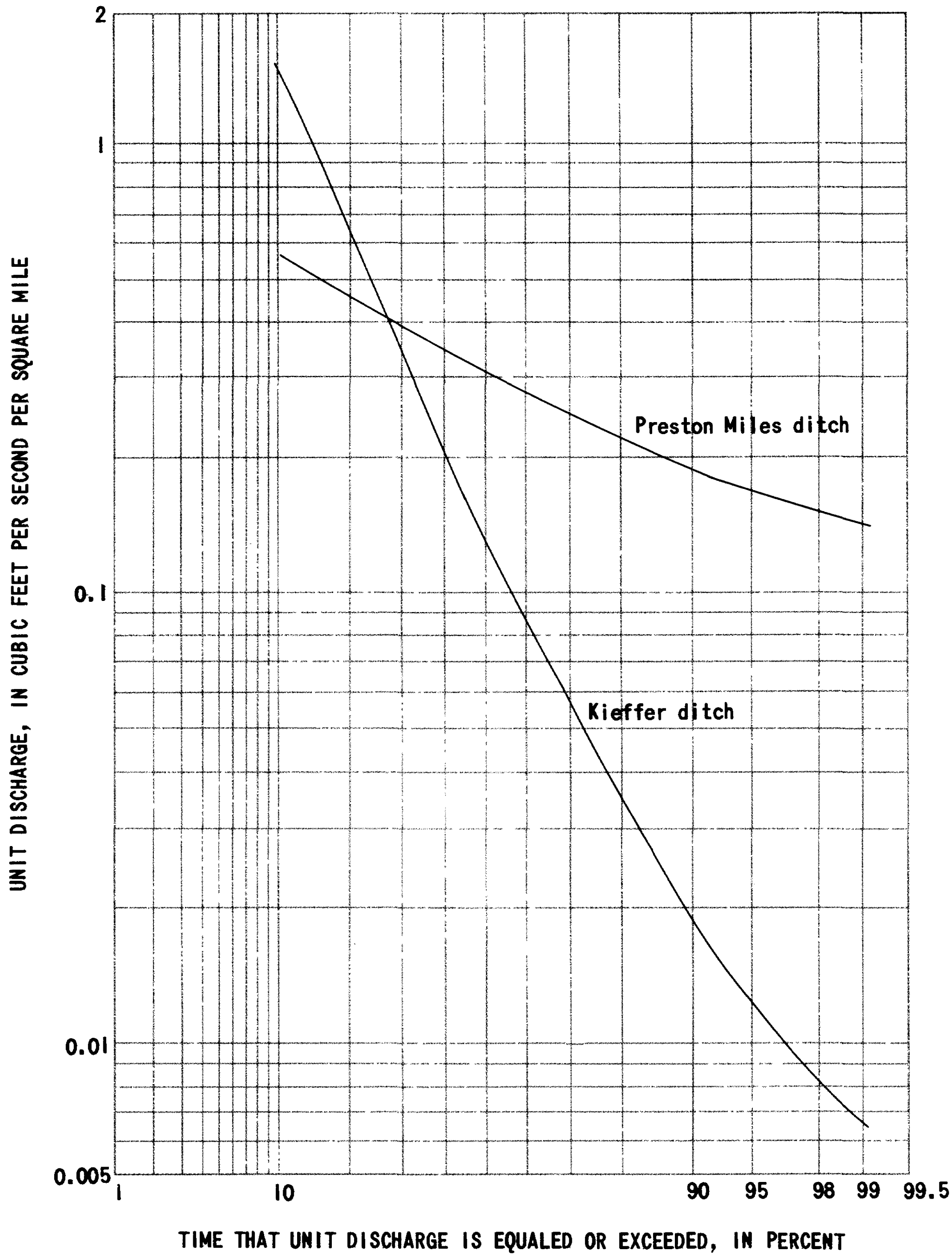
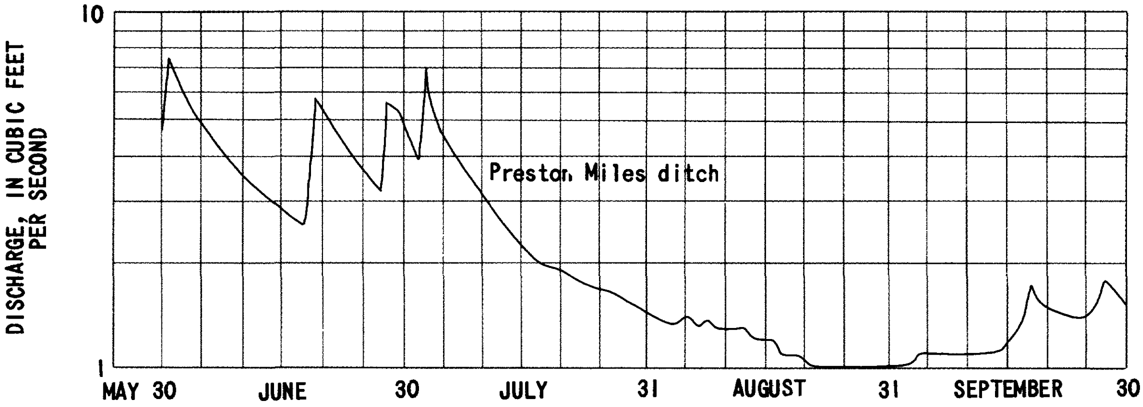


Figure 16.-- Flow-duration curves for Preston Miles and Kieffer ditches.



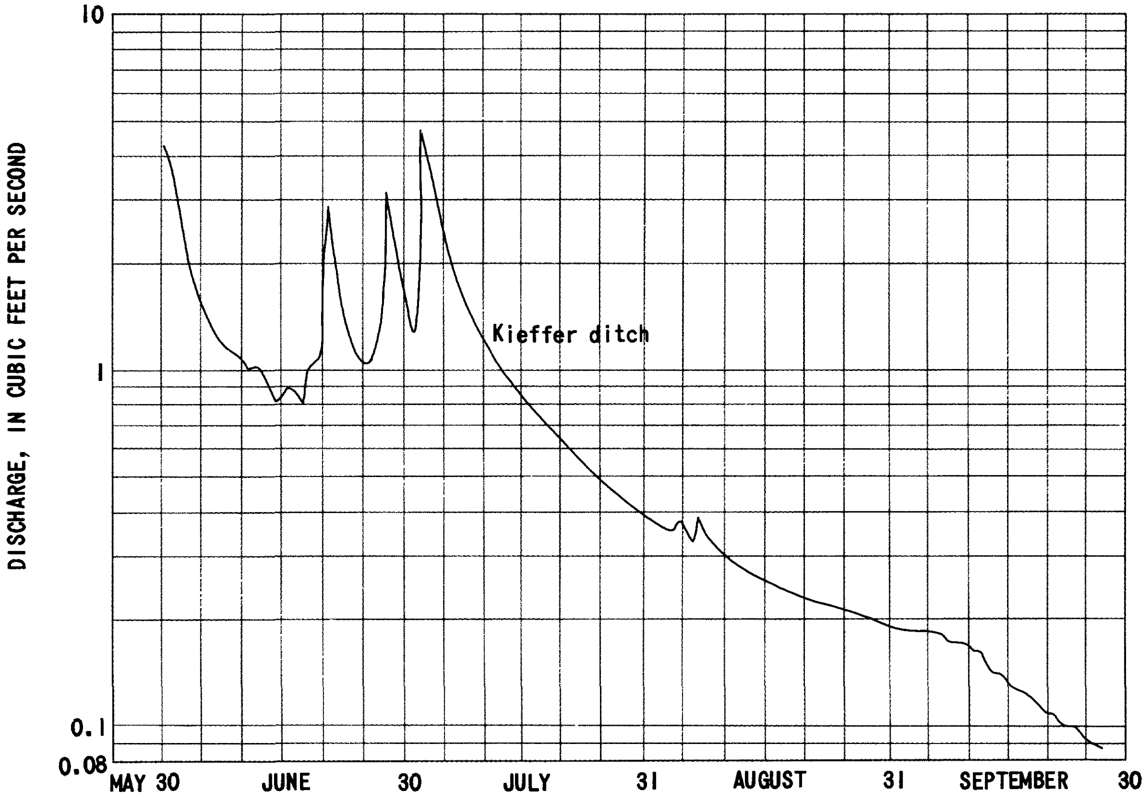


Figure 17.-- Hydrographes for Preston Miles and Kieffer ditches, June-August 1982.

Water-level fluctuations

Long-term water-level data were available for observation well Elkhart-4 in southern Elkhart County (fig. 1). The well is screened in a sand and gravel aquifer similar to that in the study area. Water-level changes in Elkhart-4 well seem to be unrelated to irrigation.

The hydrograph consists of cyclic seasonal fluctuations over the ten-year period, 1973-82 (fig. 18). Monthly water levels are generally highest in spring and lowest in autumn. In the 1982 water year, water levels were higher than normal, owing to higher-than-normal rates of spring snowmelt.

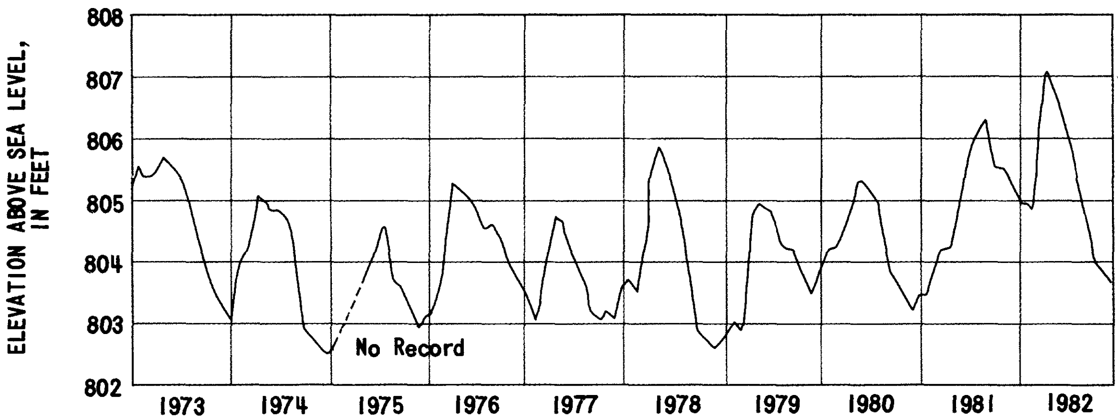


Figure 18.-- Hydrograph of maximum monthly water levels in well Elkhart-4, 1973-82.

Water levels were recorded continuously at observation well 101-5 (fig. 5) during the 1982 water year. The increase in March water levels in the hydrographs for wells Elkhart-4 and 101-5 (fig. 19) are attributed the recharge from snowmelt in March. The subsequent recession was faster in well 101-5 than in well Elkhart-4, owing, in part, to ground-water pumping from well 101-1, which is 800 ft to the west. During the summer, ground-water pumping from well 101-1 temporarily lowered water levels in well 101-5 nearly .1 ft below nonpumping levels (fig. 19).

Recharge

Minimum recharge to aquifer 1 in the Milford area for the 1982 water year was estimated by assuming that recharge equals precipitation minus potential evapotranspiration. Surface runoff was assumed to be negligible because of the

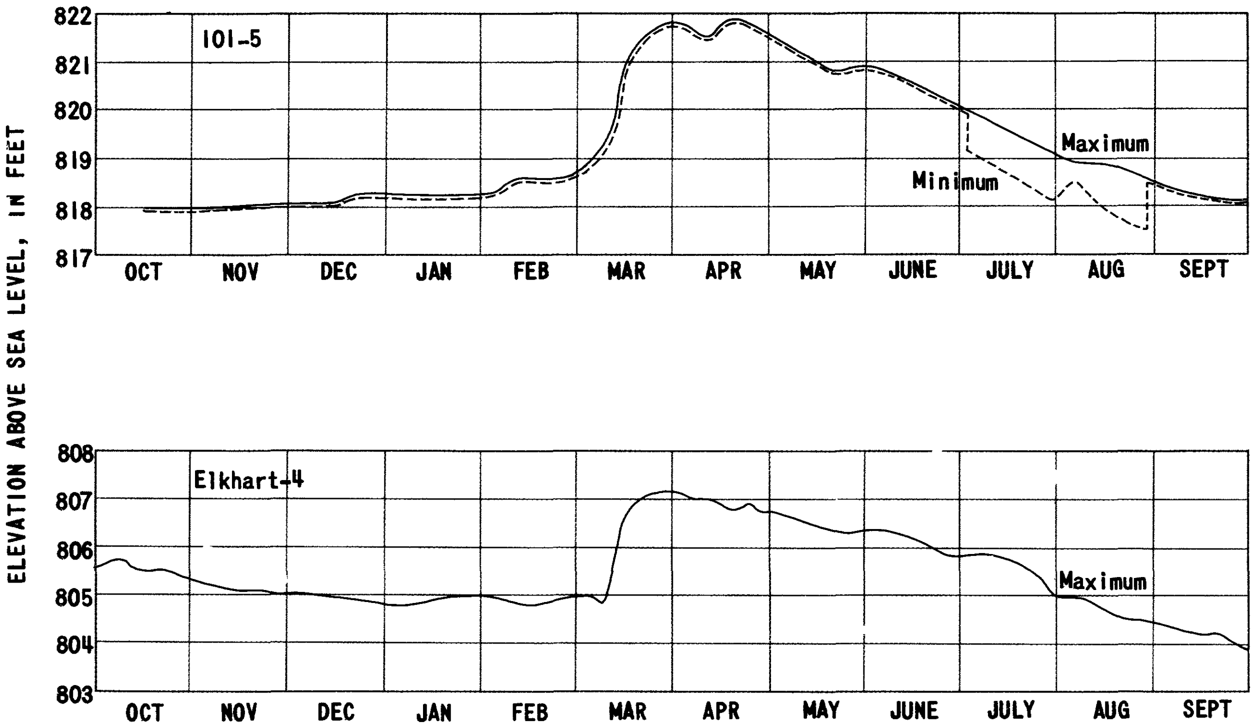


Figure 19.-- Hydrographes of maximum and minimum daily water levels in well 101-5 and maximum daily water levels in well Elkhart-4. 1982 water year.

high permeability of the soil over most of the area. Precipitation at Goshen and Warsaw weather stations (fig. 1) averaged 32 inches for water year 1982. Potential evapotranspiration for water year 1982 was estimated to be 26 in. by using methods described by Thornthwaite (1948). Thus, an estimate of minimum recharge to the surficial aquifer in water year 1982 is 6 in.

Recharge rates to various aquifers in the St. Joseph River basin were estimated by Pettijohn (1968, p. 24). According to Pettijohn, unconfined outwash aquifers receive an average of 10.5 in/yr (fig. 4); stratified drift deposits, such as kames, receive 6.3 in/yr; and confined outwash aquifers overlain by till receive 4.2 in/yr.

Ground-water Flow

Ground water flows in the direction of decreasing head perpendicular to lines of equal potential. Water levels measured in autumn 1982 were used to draw water-level contours for aquifers 1 and 2 (figs. 20 and 21). Regional flow generally parallels Turkey Creek. Ground water flows into the area from the east and south and out of the area at the northern boundary. Flow was

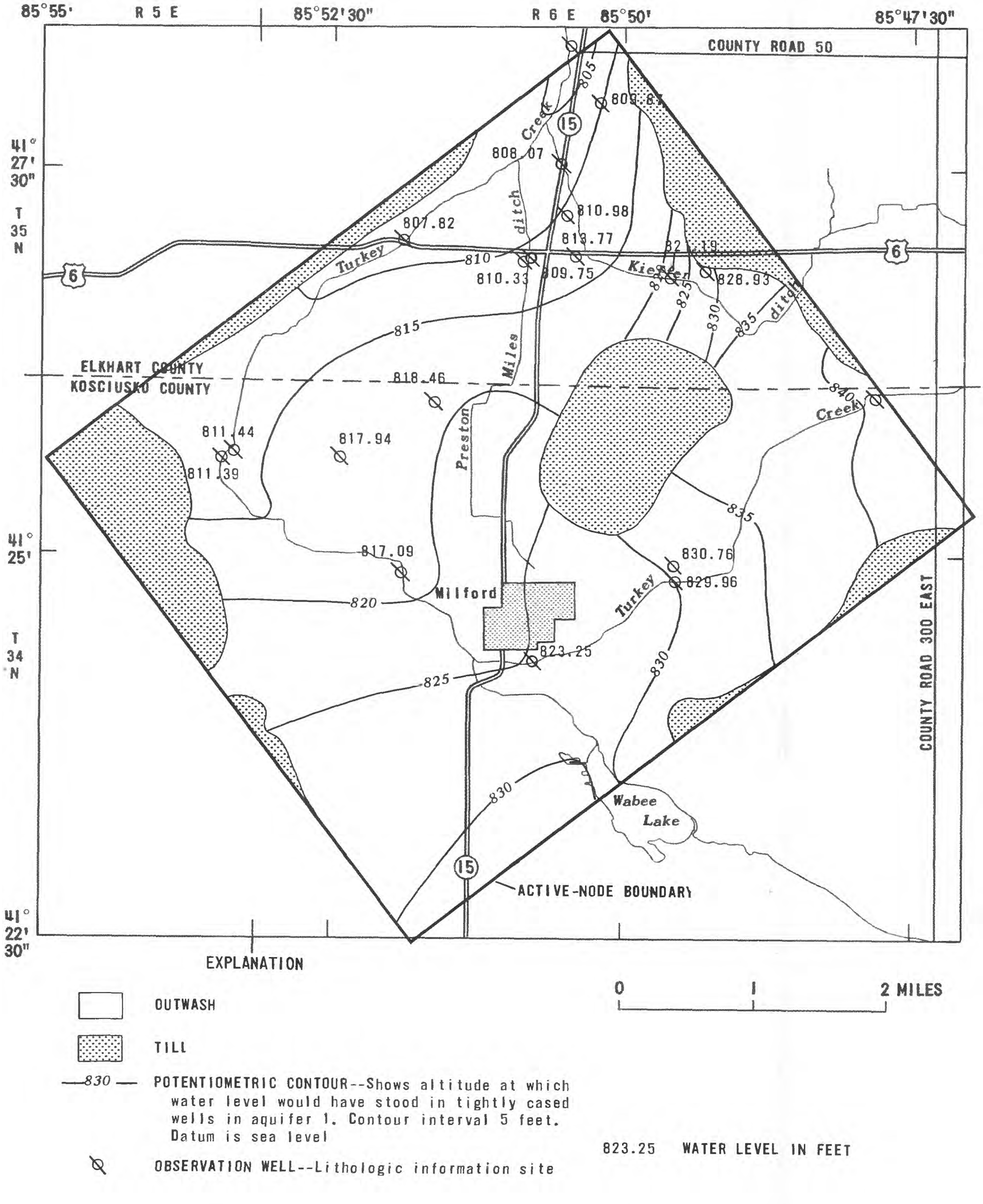
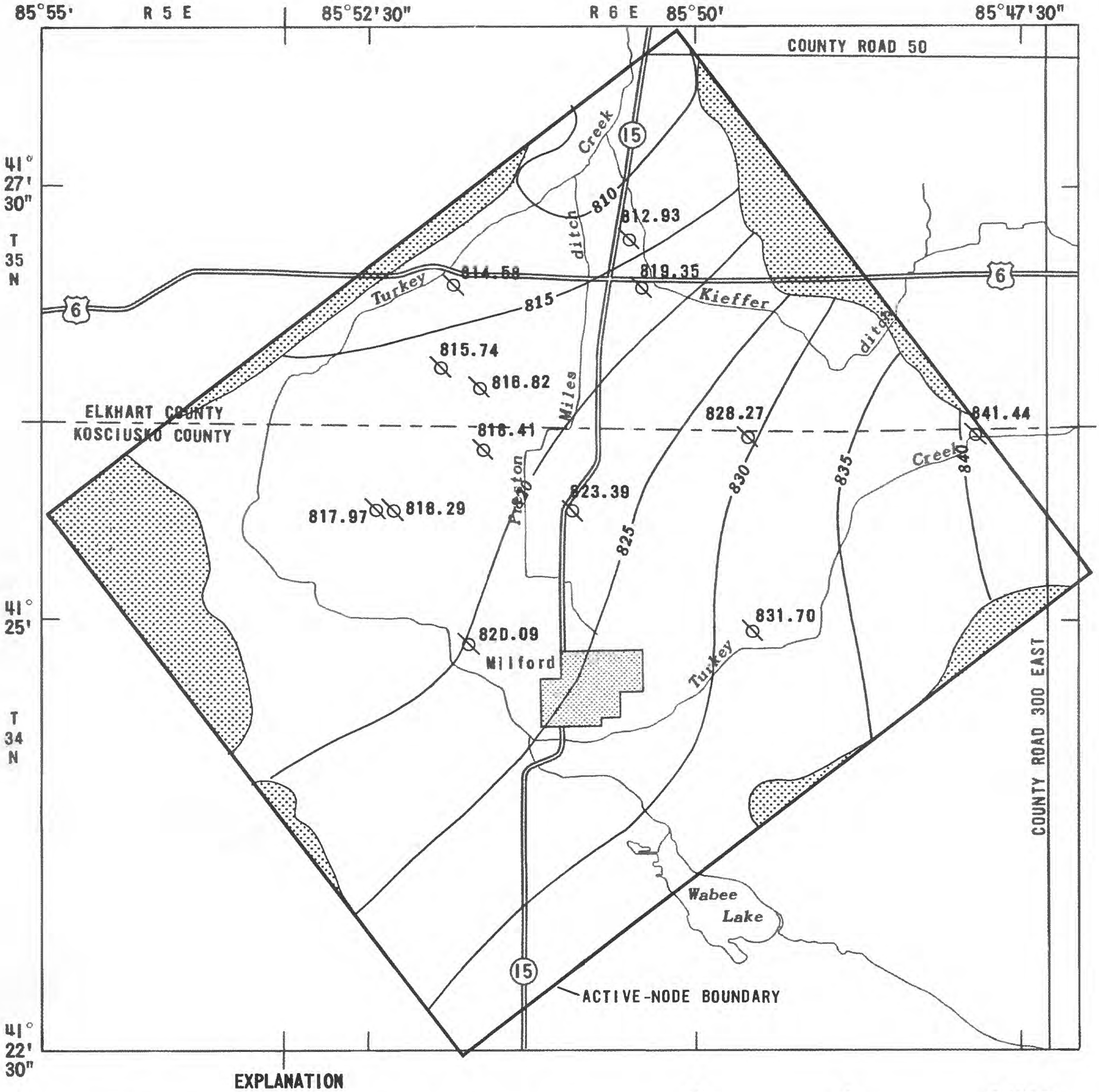


Figure 20.-- Water levels in wells screened in aquifer 1 autumn 1982.



EXPLANATION

0 1 2 MILES

OUTWASH

TILL

—830— POTENTIOMETRIC CONTOUR--Shows altitude at which water level would have stood in tightly cased wells in aquifer 2. Contour interval 5 feet. Datum is sea level

820.09 WATER LEVEL IN FEET

OBSERVATION WELL--Lithologic information site

Figure 21.-- Water levels in wells screened in aquifer 2, autumn 1982.

assumed to be horizontal in the aquifers except near stream channels (fig. 6). Vertical flow was especially noticeable near Turkey Creek where water levels in paired, shallow and deep observation wells differed by as much as 6 ft.

HYDROLOGIC EFFECTS OF WATER WITHDRAWAL

Estimation of Effects of Surface-water Withdrawals

The effect of water withdrawal on streamflow depends on whether water is pumped directly from the stream channel or from ground water near the stream. In the first case, the effect of withdrawal is immediate. The amount of water removed equals reduction in streamflow if the increased seepage of ground water into the channel in response to lowered streamstage is ignored. However, when water is pumped from an aquifer that is hydraulically connected to a stream, the effect on streamflow is delayed, and the amount of water pumped from the well will usually be greater than the reduction in streamflow caused by the pumping. An analysis of the effects of ground-water pumping on reduction of streamflow is presented in the section "Streamflow reduction". The discussion in this section is limited to reduction by direct withdrawal of water from the stream channel.

Predicting the effect of direct withdrawal on streamflow for a given flow condition requires an estimate of natural flow in the channel at the point of withdrawal. A simple method of estimating flow at an ungaged site is to determine a unit discharge for the reach of stream being considered and to multiply it by the drainage area at the withdrawal point. If unit discharge is not available for a given stream, it can be determined by using a flow-duration curve from a gaged stream with drainage characteristics similar to those of the stream being considered. Drainage areas have been determined for streams in Indiana draining areas greater than 5 mi^2 (Hoggatt, 1975). After the natural stream discharge at the withdrawal point is estimated, the pumping rate is subtracted to estimate the streamflow immediately downstream from the withdrawal point.

Example:

Problem.--Assume that when the flow in Turkey Creek at the Milford wastewater treatment plant is less than 6 ft^3/s, the natural assimilative capacity of the stream is insufficient to maintain acceptable water quality downstream from the outfall of the plant. Three irrigators upstream from the plant (fig. 22) pump 5 (gal/min)/acre from Turkey Creek when rainfall does not provide adequate moisture for their corn crops. Irrigators 1, 2, and 3 irrigate 80, 80, and 120 acres or a total of 280 acres. What minimum natural flow in Turkey Creek is needed for irrigation and assimilation of treated sewage?

Solution.--Water needed for irrigation is:

$$280 \text{ acres} \times 5(\text{gal/min})/\text{acre} \times .00223 \frac{ft^3/s}{\text{gal/min}} = 3 \ ft^3/s$$

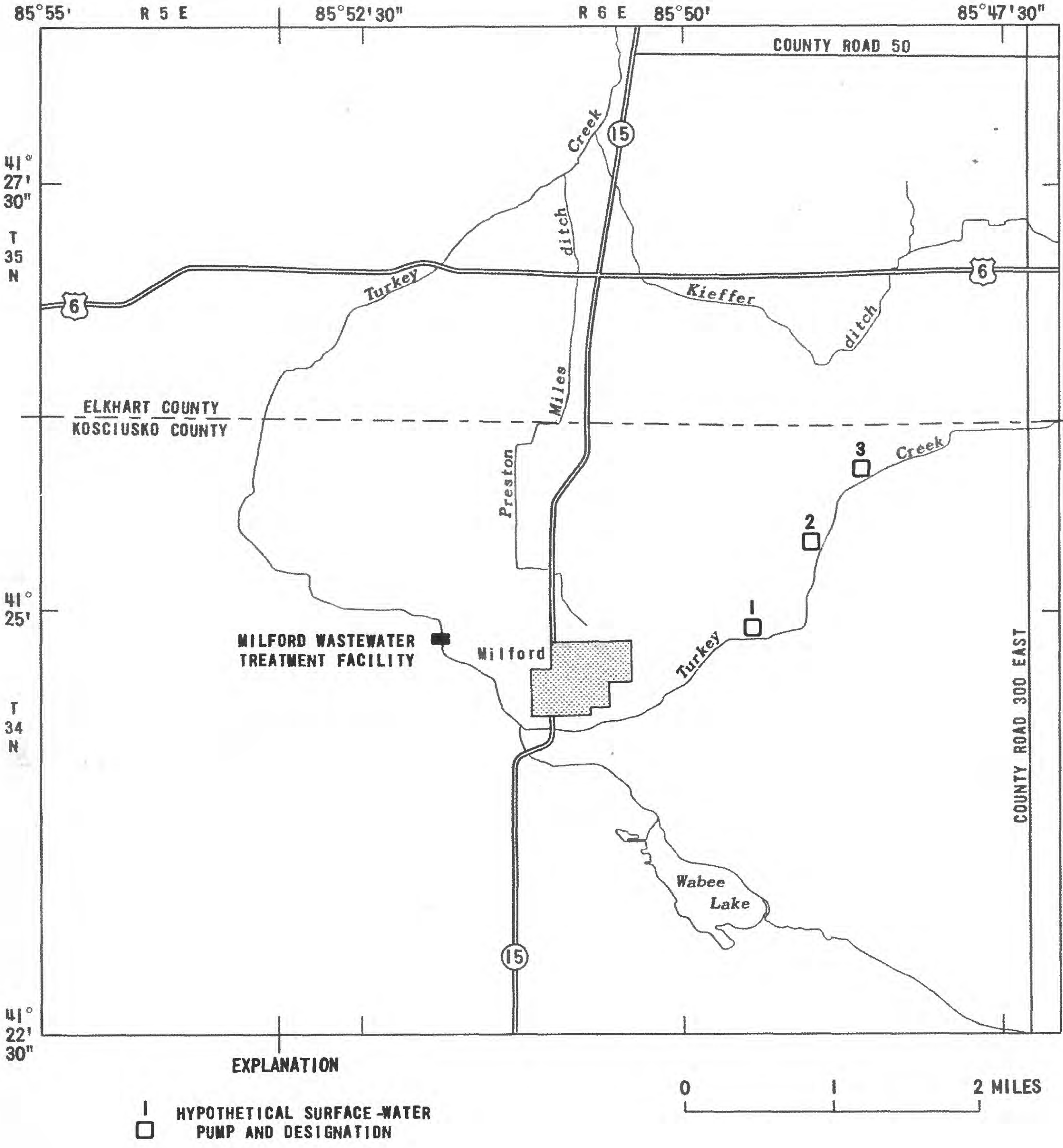


Figure 22.-- Locations of three hypothetical surface-water pumps and the Milford wastewater treatment facility.

Thus, total natural streamflow needed at the treatment facility is:

$$3 \text{ ft}^3/\text{s} + 6 \text{ ft}^3/\text{s} = 9 \text{ ft}^3/\text{s}.$$

The drainage area at the outfall is about 75 mi^2, and the unit discharge for the required flow is:

$$\frac{9 \text{ ft}^3/\text{s}}{75 \text{ mi}^2} = 0.12 \text{ (ft}^3/\text{s)/mi}^2.$$

Flow at gaged sites on other streams in the area could be used to monitor flow in Turkey Creek. For example, streamflow of the Elkhart River at Goshen, where the drainage area is 594 mi^2 and the unit discharge of 0.12 $(ft^3/s)/mi^2$, is:

$$594 \text{ mi}^2 \times 0.12 \text{ (ft}^3/\text{s)/mi}^2 = 71 \text{ ft}^3/\text{s}$$

Thus, when flow at Elkhart River at Goshen is near 71 ft^3/s, the flow in Turkey Creek at the outfall would be near 9 ft^3/s without irrigation or 6 ft^3/s if the three irrigators were pumping from the stream channel. Additional simultaneous measurements of flow at the two sites would be needed to improve the accuracy of prediction.

As mentioned previously, this analysis does not account for ground water seeping into the stream because of reduced streamstage downstream from pumping withdrawals or for effects of irrigation from ground water. These two factors have opposite but not necessarily offsetting effects on flow in the stream.

Simulation of Effects of Ground-water Withdrawals

Model Assumptions

The three-dimensional, finite-difference ground-water flow model by McDonald and Harbaugh (1984) was used to simulate the hydrologic system of the Milford area. The following assumptions were made for the model:

1. Flow is horizontal in the aquifers and vertical through the semi-confining units between aquifers.
2. The shale bedrock is an impermeable boundary to flow.
3. The aquifer material within each node is isotropic and homogeneous.
4. Areal recharge is uniform over each type of surficial geologic material.
5. Streambeds are 1 ft thick and are of lower hydraulic conductivity than the outwash.
6. Intermittent streams can be ignored as points of ground-water discharge.
7. Effects of storage in the clay between aquifers can be ignored.
8. The ground-water system was near steady state at the time of water-level measurements.

Local deviations from these assumptions can cause localized differences between model-calculated and observed ground-water conditions, but the overall model analysis will not be adversely affected by these deviations.

Conceptual Model

The model for the Milford area consists of three aquifer layers and two semi-confining units separating the layers. Aquifer 1 was modeled as layer 1, aquifer 2 as layer 2, and aquifer 3 as layer 3. The flow till between layers 1 and 2 and the clay till beneath layer 2 are modeled as semi-confining units. Five of the six irrigational wells were screened in layer 2, so ground-water development was simulated in layer 2.

Stream stage of Turkey Creek changed less than 10 percent in response to the range of flows used in the study and was held constant in the model.

Model Construction

A rectangular grid was drawn to cover the area of outwash and was extended past Turkey Creek to no-flow till boundaries to minimize boundary effects near pumping wells (figs. 23-24). The dimensions of the grid are 4.4 mi by 5.1 mi; it is divided into 40 rows and 46 columns of 587-foot square grid blocks. Each block represents an area of approximately 0.01 mi^2 (344,569 ft^2). The modeled area was not used for any purpose other than constructing the model. All areal determinations such as land use and irrigational potential were based on the study area as presented in figure 2.

In general, linear interpolation was used to calculate values of aquifer characteristics for nodes between locations of field measurements. The locations of field measurements are indicated in the various figures (for example, fig. 20). Layer 1 is assumed to be unconfined. Layer 2 and layer 3 are assumed to be confined.

Necessary data for each node in layer 1 included the altitude of the bottom of the aquifer, initial water levels, and hydraulic conductivity. The bottom of the aquifer (fig. 8) is defined by the top of the underlying flow till. Initial water levels (fig. 21) were based on those measured in observation wells from August 31 through September 2, 1982 (fig. 20). The average hydraulic conductivity 280 ft/d was used for all active nodes in layer 1.

Turkey Creek, Preston Miles ditch, and Kieffer ditch were modeled as river nodes shown in figure 23. Stream stage, elevation of the streambed, and streambed conductance were required for each river node. Stream stage was that measured at selected sites shown in figure 3 during gain/loss measurements, August 31 through September 2, 1982. A uniform gradient between measured sites was assumed. Elevation of the streambed was calculated by subtracting measured depths of the streams from the stage at each node.

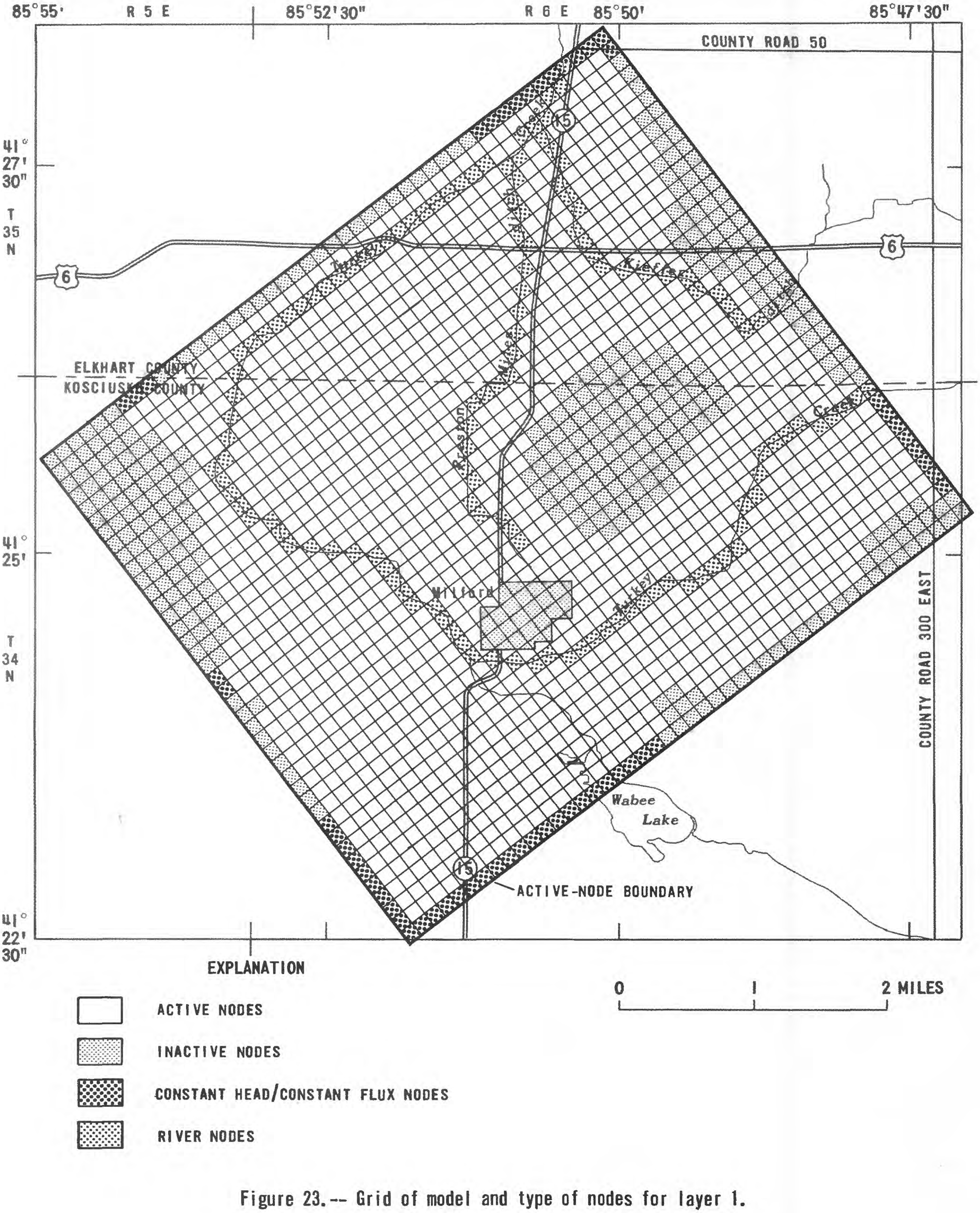


Figure 23.-- Grid of model and type of nodes for layer 1.

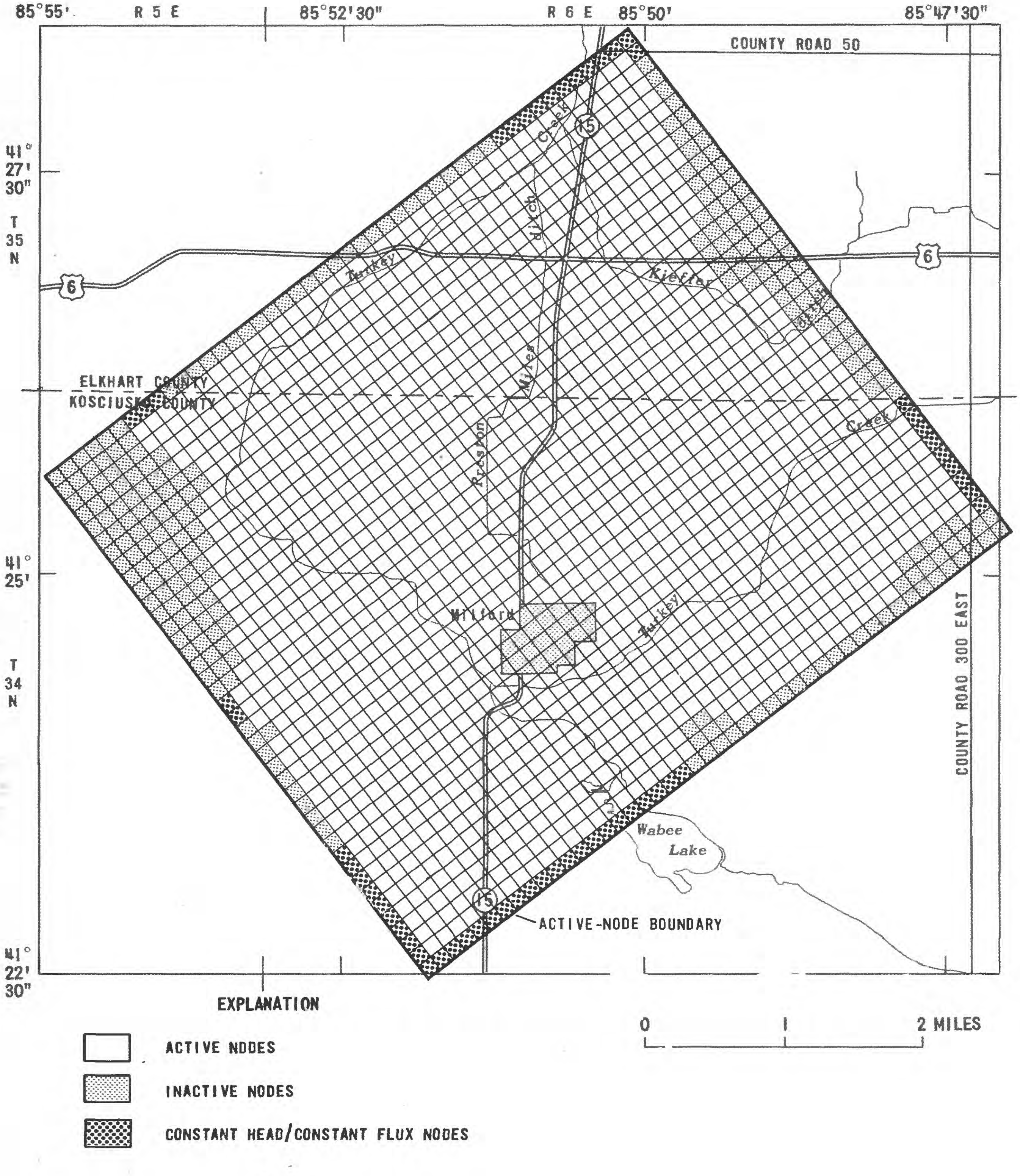


Figure 24.-- Grid of model and type of nodes for layers 2 and 3.

Streambed conductance, in ft^2/d, was calculated using the formula:

$$conductance = KA/B,$$

where

K is vertical hydraulic conductivity of the streambed, in square feet per day;
A is area of the stream within the node, in square feet; and
B is streambed thickness, in feet.

Vertical hydraulic conductivity was initially and arbitrarily assumed to be 1 ft/d. The area of the stream within each node was calculated as the product of measured stream widths and node lengths. Streambed thickness was arbitrarily assumed to be 1 ft.

Recharge was applied mainly to layer 1. Where layer 1 was absent (northeastern part of the modeled area), recharge was applied directly to layer 2.

Vertical leakage between layers 1 and 2 through the flow till was calculated by dividing the vertical hydraulic conductivity of the till by the thickness of the till (fig. 9). Values of vertical hydraulic conductivity ranged from 0.01 to 0.1 ft/d.

The data required by the model for layer 2 included initial water levels and transmissivity. Initial water levels (fig. 21) were based on those measured August 31 through September 2, 1982. Transmissivity was calculated by multiplying the average hydraulic conductivity (180 ft/d) by the thickness of the aquifer; the product is shown in parenthesis in figure 10.

Vertical leakage between layers 2 and 3 through the till was calculated by dividing the assumed vertical hydraulic conductivity of the till (0.01 ft/d) by the thickness of the till (8 ft). This resulted in the constant vertical leakage (0.00125 d^{-1}) for the lower till.

The data required by the model for layer 3 included estimates of initial water levels and transmissivity. Initial water levels were assumed to be the same as for layer 2. Transmissivity was calculated by multiplying a hydraulic conductivity (55 ft/d) by the thickness of the layer (fig. 11).

Model Boundaries

The top surface of the model is the water table, and the bottom is the bedrock surface, which serves as a no-flow boundary. At the edges of the modeled area, no-flow boundaries were assumed where outwash is in contact with till or kames (figs. 23 and 24), because the till and kame deposits have much lower hydraulic conductivities than the outwash. No extensive sand and gravel lenses were identified within the till.

Constant-head boundaries were assumed where outwash extended beyond the study area. This assumption was necessary so that the model would calculate the lateral fluxes into and out of the area.

The map of surficial geology (fig. 4) was used to help define the lateral boundaries for layer 1 (fig. 23). Layer 1 is absent in the northeast. Layer 2 was assigned the same lateral boundaries as layer 1, except in the northeast where layer 2 was assumed to extend beneath the surficial till. Layer 3 was assumed to have the same lateral boundaries as layer 2.

Model Calibration

The steady-state model was calibrated by varying the values of parameters in the model until results approximated water levels and ground-water discharge to streams in autumn of 1982 when there was no irrigational pumping. A close match between modeled and measured water levels and discharge does not necessarily suggest a valid conceptual model because several combinations of values of parameter and boundaries can produce identical results by the model. For this reason, hydrologic judgement is required to develop the best model possible from available information. Information should be obtained by measuring as many parameters as is practical and insuring that values of estimated parameters are varied within reasonable limits during calibration.

Values of the following parameters were adjusted during calibration of the model: areal recharge, hydraulic conductivities of the aquifer layers, vertical leakage between layers, and streambed conductivity. Each parameter was varied within a reasonable range of values defined by field measurements and (or) published data.

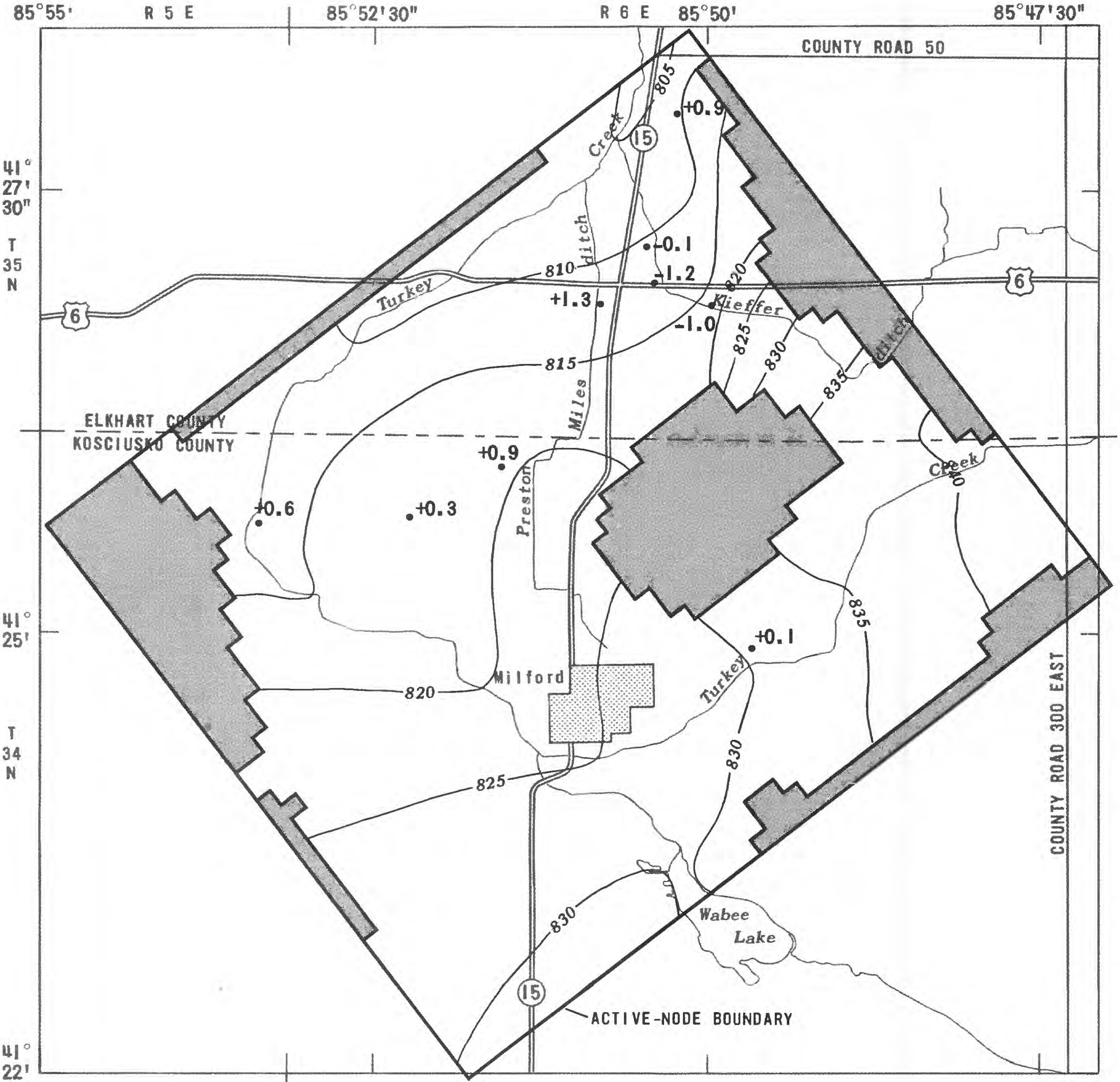
Water levels calculated by the model for each layer are shown in figures 25-27. The observation wells used for calibration and the difference between calculated and measured water levels are indicated. Most water levels in the calibrated model match those measured in layers 1 and 2 within 1 ft.

Net seepage to the streams in the modeled area was calculated by the model to be 18.2 ft^3/s. Measured seepage during autumn 1982 was 15.3 ft^3/s.

Varying recharge by factors ranging from 0.5 to 2 times the initial values had little effect on water levels or seepage to streams. Values in figure 4 were used in the final calibrated model.

Values of hydraulic conductivity for layers 1 and 2 were varied independently from approximately 20 to 3,000 ft/day. The model was sensitive to small changes in conductivity. The initial values of 280 and 180 ft/d for layers 1 and 2 provided the best results. Changing the conductivity of layer 3 from 5.5 to 550 ft/d had little effect on measured water levels in the two upper layers and on discharge to streams.

In the area between the isolated till and the till boundary to the northeast, calibration was improved when the hydraulic conductivity of layer 1 was decreased to 0.03 ft/d which suggests that the kames may cover more area than shown in figure 4.



EXPLANATION

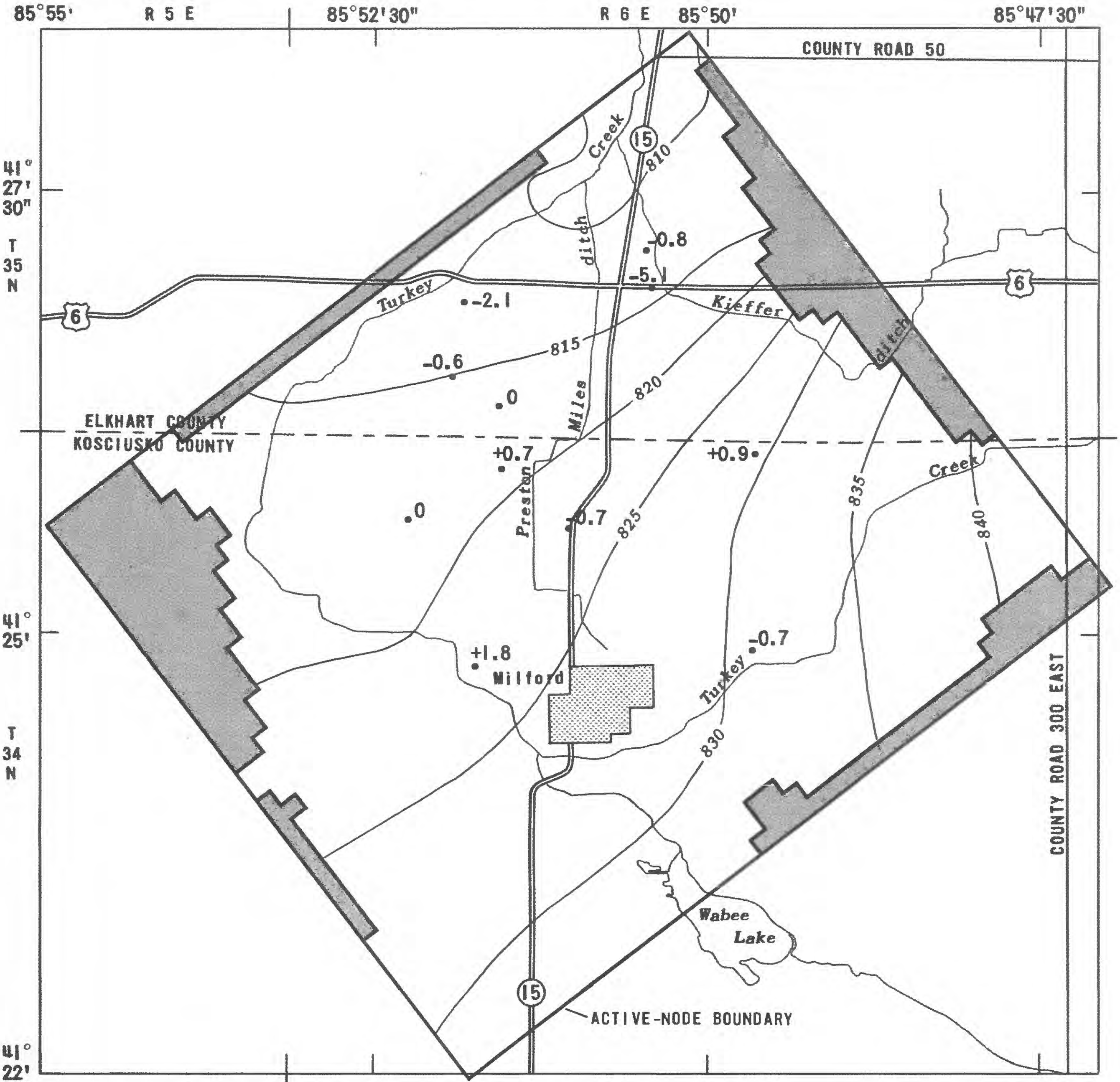
AREA OF NO FLOW

—*830*— POTENTIOMETRIC CONTOUR--Shows altitude of simulated potentiometric surface of layer 1. Contour interval 5 feet. Datum is sea level

+0.3 POINT OF MEASURED WATER LEVEL AND VALUE--Value equals the measured elevation subtracted from the calculated elevation

0 1 2 MILES

Figure 25.-- Water levels in layer 1 calculated by the calibrated model for autumn 1982.



EXPLANATION

0 1 2 MILES

AREA OF NO FLOW

—830— POTENTIOMETRIC CONTOUR--Shows altitude of simulated potentiometric surface of layer 2. Contour interval 5 feet. Datum is sea level

+1.8 POINT OF MEASURED WATER LEVEL AND VALUE-- Value equals the measured elevation subtracted from the calculated elevation

Figure 26.-- Water levels in layer 2 calculated by the calibrated model for autumn 1982.

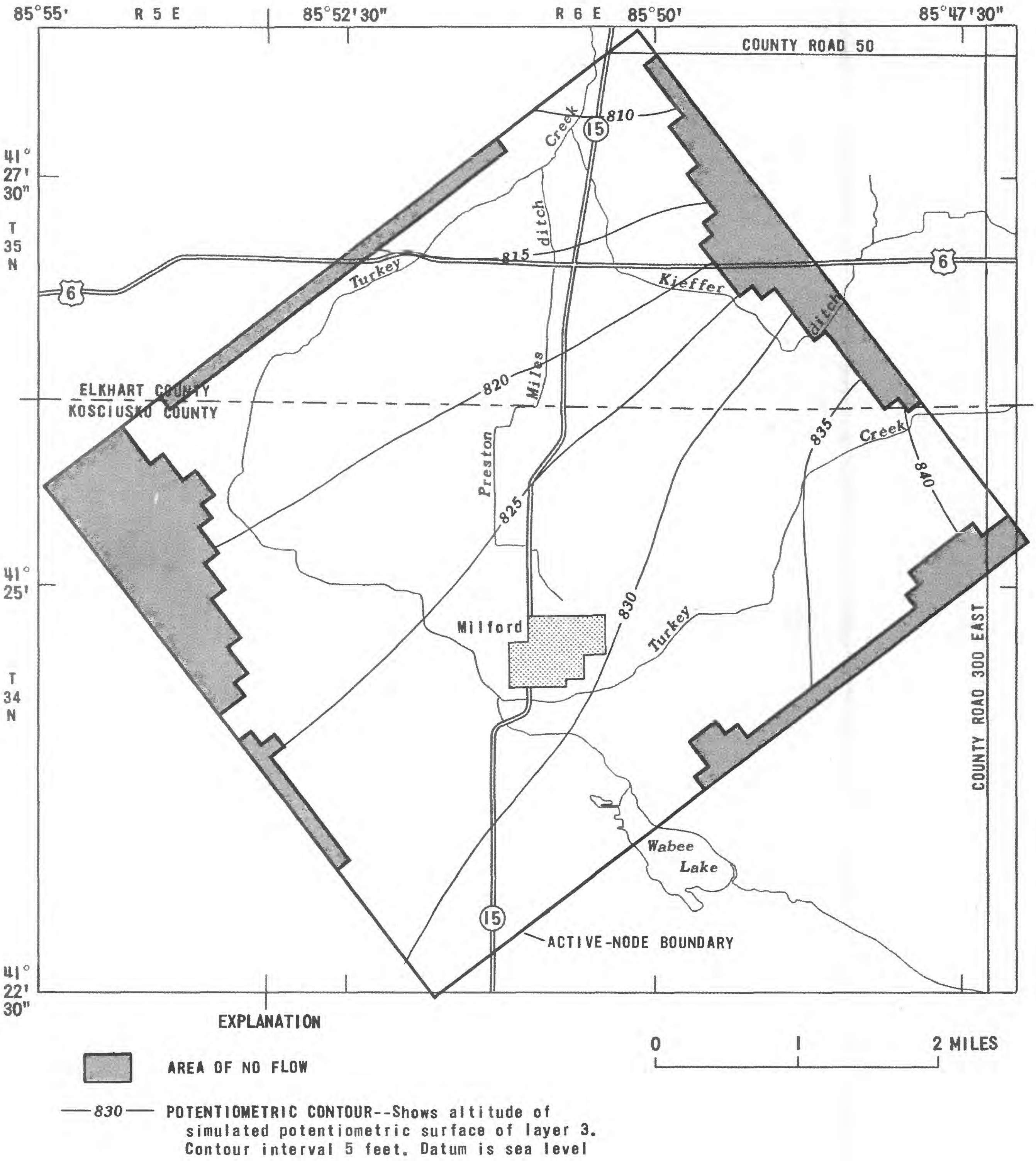


Figure 27.-- Water levels in layer 3 calculated by the calibrated model for autumn 1982.

The vertical hydraulic conductivity of the flow till between layers 1 and 2 was varied from 0.001 to 1.0 ft/day. Varying the conductivity caused large changes in water levels and discharge to streams. Values of 0.01 ft/d for most nodes and 0.1 ft/d for nodes representing the more sandy till in the west-central part of the area were used for final calibration.

The values of vertical hydraulic conductivity in the clay separating layers 2 and 3 were varied from 0.0004 to 1.0 ft/d. Varying the values had no measurable effect on either seepage to streams or water levels in layer 2. A value of 0.01 was arbitrarily chosen for final calibration.

The vertical conductivity of the streambed was adjusted from 1.0 ft/d to 0.001 and 10.0 ft/d over the entire area. For final calibration, the initial value of 1.0 ft/d was used for Turkey Creek, Preston Miles ditch and Kieffer ditch downstream from the gage on Kieffer ditch. A value of 0.1 ft/d was used upstream from the gage because the channel bottom was more clayey in this area. Varying the vertical conductivity of the streambed had a greater effect on discharge to streams than on water levels.

Based on the water budget for the calibrated model, there is a net loss of regional ground-water flow through the area (table 4). The change in boundary flux of about 4 ft^3/s is presumably lost to streamflow. This loss probably results from the funneling of the outwash toward the northern, down-gradient edge of the area (fig. 4). According to the water budget, only about 10 percent of ground water would leave the area through the northern boundary. Most of it would flow from the area as seepage to Turkey Creek from layer 1.

Table 4.--Ground-water budget calculated by the calibrated model for autumn 1982

Water-budget terms	Flow (ft^3/s)	Percentage of total
Sources:		
Boundary flux	6.56	31
Areal recharge	13.75	66
Streambed seepage	.54	3
Total	20.85	100
Discharges:		
Boundary flux	2.11	10
Streambed seepage	18.74	90
Total	20.85	100

Sensitivity Analyses

The sensitivity of the model to changes in recharge, vertical conductivity of confining layers, aquifer transmissivity, and streambed conductance was analyzed by using root-mean-square error (RMSE). RMSE is calculated by

$$\mathrm{RMSE} = \sqrt{\frac{\sum_{i=1}^{N} \left(h_i^m \quad h_i^c\right)^2}{N}}$$

where

N indicated the number of observations,

h_i^m is the measured water level

h_i^c is the calculated water level

(Bailey and Imbrigiotta, 1982, p. 44).

The smaller the value of RMSE, the better the agreement between measured and calculated water levels. Analysis is presented for the calibrated, steady-state model with constant-flux boundaries. When constant-head boundaries were used, the model was much less sensitive to changes in values of parameters than when constant flux boundaries were used. This reduced sensitivity presumably resulted from the variable amounts of water moving across constant-head boundaries (Bailey and Imbrigiotta, 1982, p. 45-46).

For the analysis, values of parameters used in the calibrated model were varied by multiples of the calibrated values. Values of the multiples for recharge ranged from 0.1 to 5.0 and for the other three parameters ranged from 0.1 to 10.0. The values of the multiples and corresponding values of RMSE were plotted on semilogarithmic paper (figs. 28-31).

The RMSE value for the calibrated model was 1.8 ft, which means that, on the average, measured water levels differed from calculated water levels by 1.8 ft. In all of the four sensitivity analyses, lower values of RMSE were obtained for multiple values other than 1.0, the value corresponding to the value of the parameter used for calibration. The reason for this apparent discrepancy is that seepage to the streams was also used as a criterion in the calibration procedure. This criterion is not accounted for in the calculation of RMSE. The largest decrease in RMSE below the value for calibration was for the sensitivity of leakage between aquifers. The decrease was 0.3 ft or 17 percent of the calibration value.

The model was most sensitive to recharge at values greater than two times those used for calibration and was not as sensitive at values of recharge less than those used for calibration (fig. 28). Increases in streambed conductivity produced very little increase in RMSE, but decreases below a multiple value of 0.3 produced a sharp increase in RMSE (fig. 29). The shape of the sensitivity curve for leakance of the confining beds (fig. 30) is similar to the one for streambed conductivity--that is, at multiple values greater than 1.0, sensitivity is low. At values less than 1.0, sensitivity is greater. Changes in values of hydraulic conductivity above or below the calibrated value, significantly changed the values of RMSE (fig. 31).

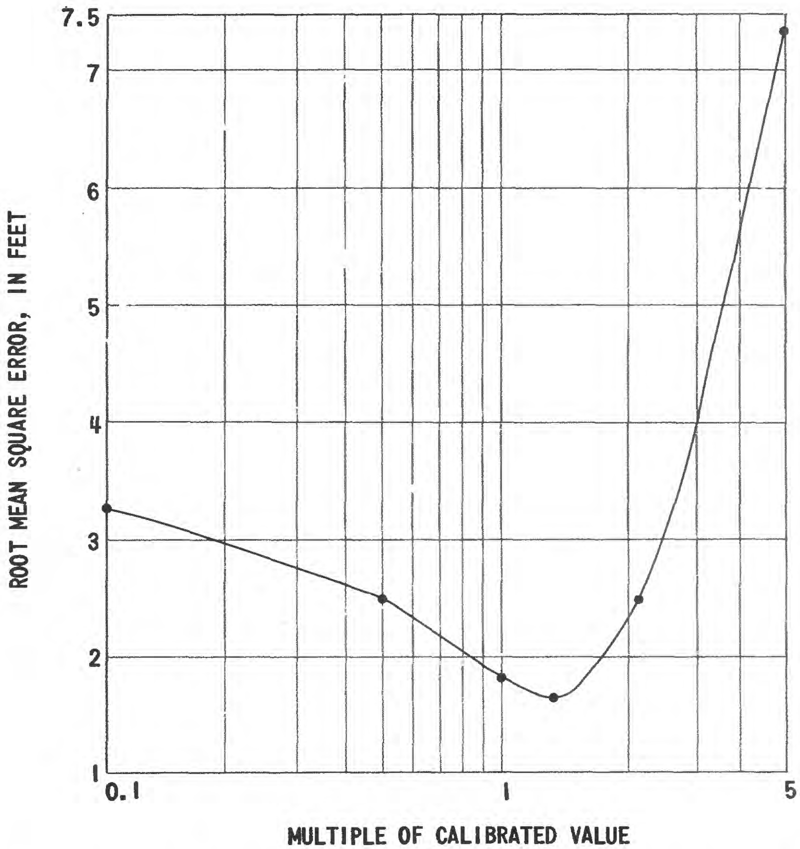


Figure 28.-- Sensitivity of the calibrated model to adjustments in recharge to aquifer.

The sensitivity graphs are helpful in evaluating to what degree errors in estimation of parameters affect the accuracy of the calculated water levels. For example, if improvements in the estimate of streambed conductivity indicated that a better value is 10 times greater than the one used for calibration, the model would still be useful because it is not sensitive to a ten-fold increase in leakance (fig. 29). However, if the improved conductivity value is found to be one-tenth the value used for calibration, a new calibration would be suggested because the RMSE would be increased considerably by the change.

Simulation of Pumping Strategies

Methods

The calibrated model was used to study the potential for increased development of ground water and surface water in the Milford area. Five hypothetical pumping plans were developed (table 5). The first four were

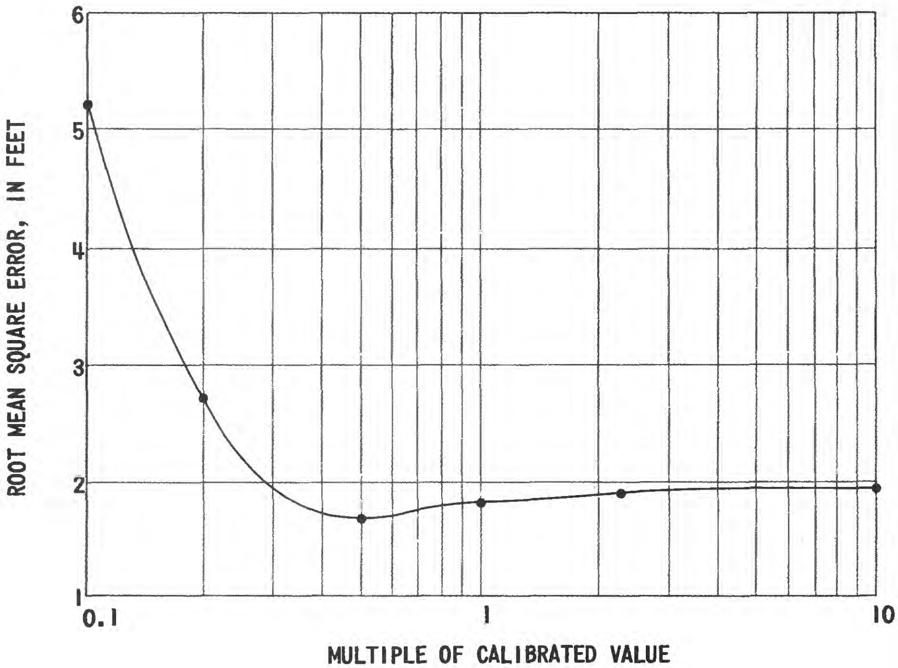


Figure 29.-- Sensitivity of the calibrated model to adjustments in conductivity of streambed.

designed to simulate increases in irrigation. A fifth pumping plan was designed to simulate high pumping rates maintained for an extended period of time to evaluate the effects of maximum water use on water supply. Plan 5 was not intended to simulate withdrawal for agricultural irrigation alone, and pumpage was simulated year-round.

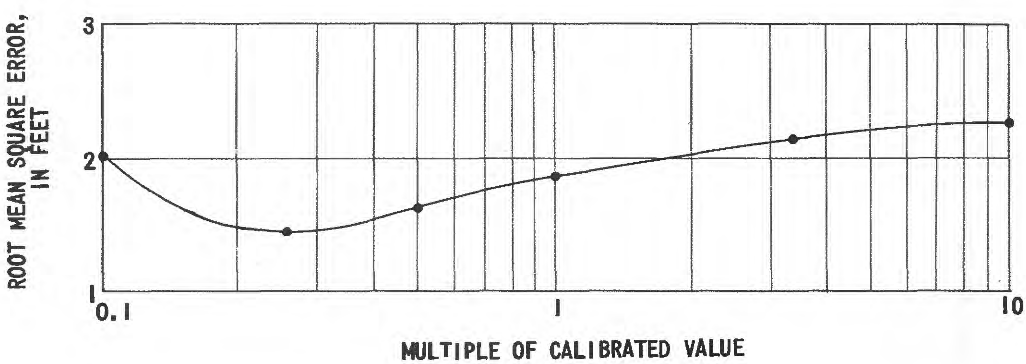


Figure 30.-- Sensitivity of the calibrated model to adjustments in leakance of confining beds.

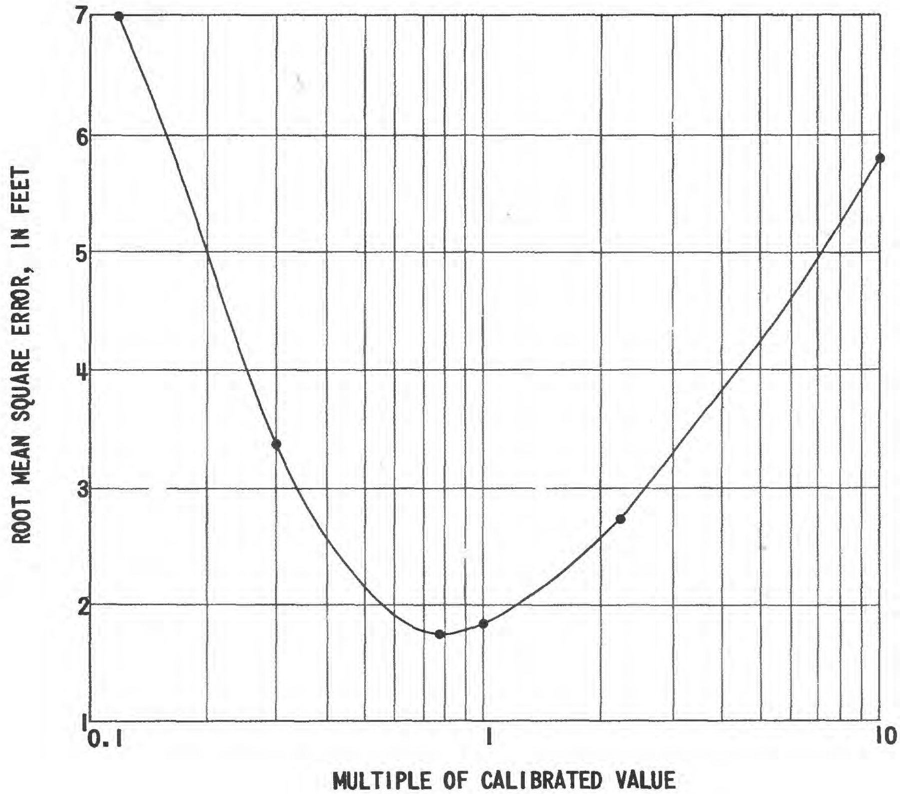


Figure 31.-- Sensitivity of the calibrated model to adjustments in hydraulic conductivity of aquifers.

For the four pumping plans that simulated irrigational pumping, three applications of water were used: (1) 2 in., the average application in 1982; (2) 7.2 in., an estimate of the application needed for an irrigational season with normal precipitation; and (3) 9.7 in., the estimate of the application needed for an irrigational season with below-normal precipitation. The application requirements for normal and dry years are based on probability of precipitation and on water requirements of corn for use in methods described by Chelf (1983).

The amount of land irrigated in plans 1 and 2 was 975 acres--the amount irrigated in 1982 (table 5). For plans 3 and 4, the land irrigated was assumed to be 3,225 acres--the maximum agricultural land available for irrigation (fig. 32). The determination of irrigable land was based on soil permeability and current land use (Chelf, 1983). In designing a hypothetical pumping network to irrigate the increased land, the following guidelines were used: (1) The simulated pumping locations were selected independently of the 1982 pumping locations, (2) the minimum size for an irrigated parcel of land would be 40 acres, (3) the maximum area irrigated by a single pump would be 160 acres, and (4) irrigable land within one half mile of Turkey Creek would be irrigated with water from the Creek. All other irrigable land would be irrigated by wells.

Table 5.--Five hypothetical pumping plans for simulations by the model

Pumping plan	Condition of precipitation	Irrigated land (acres)	Model type
1	Above normal	975 (Same as 1982)	Transient
2	Below normal	975 (Same as 1982)	Transient
3	Normal	3225 (Maximum available)	Transient
4	Below normal	3225 (Maximum available)	Transient
5	Below normal	--[1]	Steady state

[1] Value not applicable because irrigation was not simulated.

Using these guidelines, the authors calculated that 26 wells and 12 surface-water pumps (fig. 33) were needed to irrigate 3,225 acres. By multiplying the land irrigated by each pump, in acres, by 5 (gal/min)/acre, a pumping rate, in gal/min, was calculated for each pump (fig. 33). With this procedure, 9.0 days of continuous pumping were needed to apply 2.0 in. of water in plan 1. In plan 3, 27.3 days were needed to apply 7.2 in. of water. In plans 2 and 4, 36.7 days were needed to apply 9.7 in. (table 6).

Normally, irrigational pumps operate intermittently and independently of each other throughout the summer. However, for the four transient pumping plans, the assumption was made that all pumps operate continuously throughout shortened pumping periods. This assumption was made to simplify modeling and to produce the largest effect on drawdown and streamflow reduction for a given volume of pumpage. Thus, though the volume of pumpage for plan 1 was the same as that during the 1982 irrigational season, the values of drawdown and rates of streamflow reduction predicted by the model will be greater than those that could be attributed to actual irrigation in 1982.

Five of the six wells used for irrigation in 1982 tapped aquifer 2. The exception was well 103-1 which tapped aquifer 1. This well actually connected two 4-in. diameter casings screened at a depth of 40 ft below the surface--a well construction not likely to be repeated for future wells. For simulation, all wells were assumed to tap layer 2.

Unlike the calibrated model, a model of the four irrigational pumping plans required that water levels change with time. To convert the calibrated model from one in which water levels are constant (steady state) to one in which

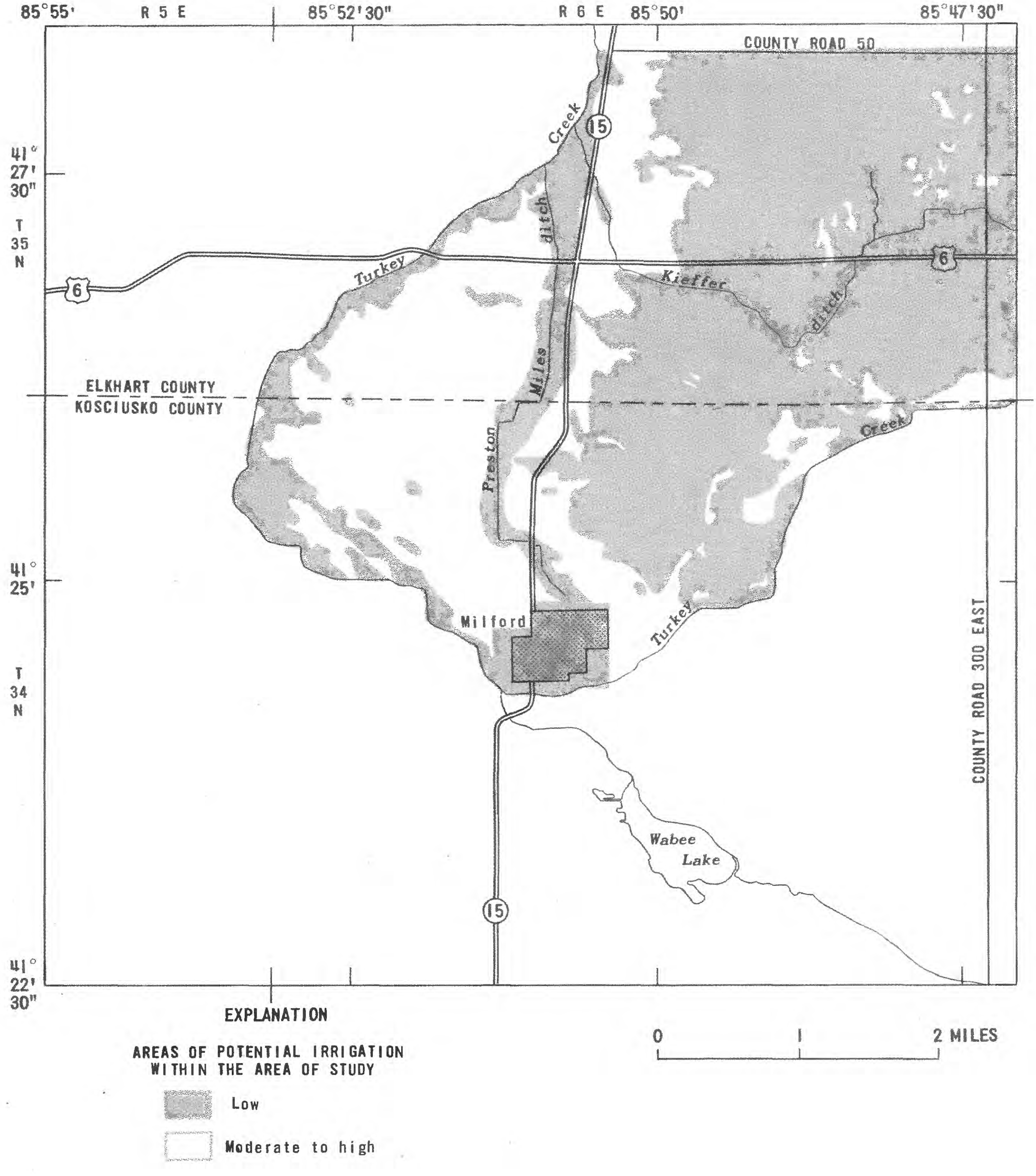


Figure 32.-- Areas of potential irrigation.

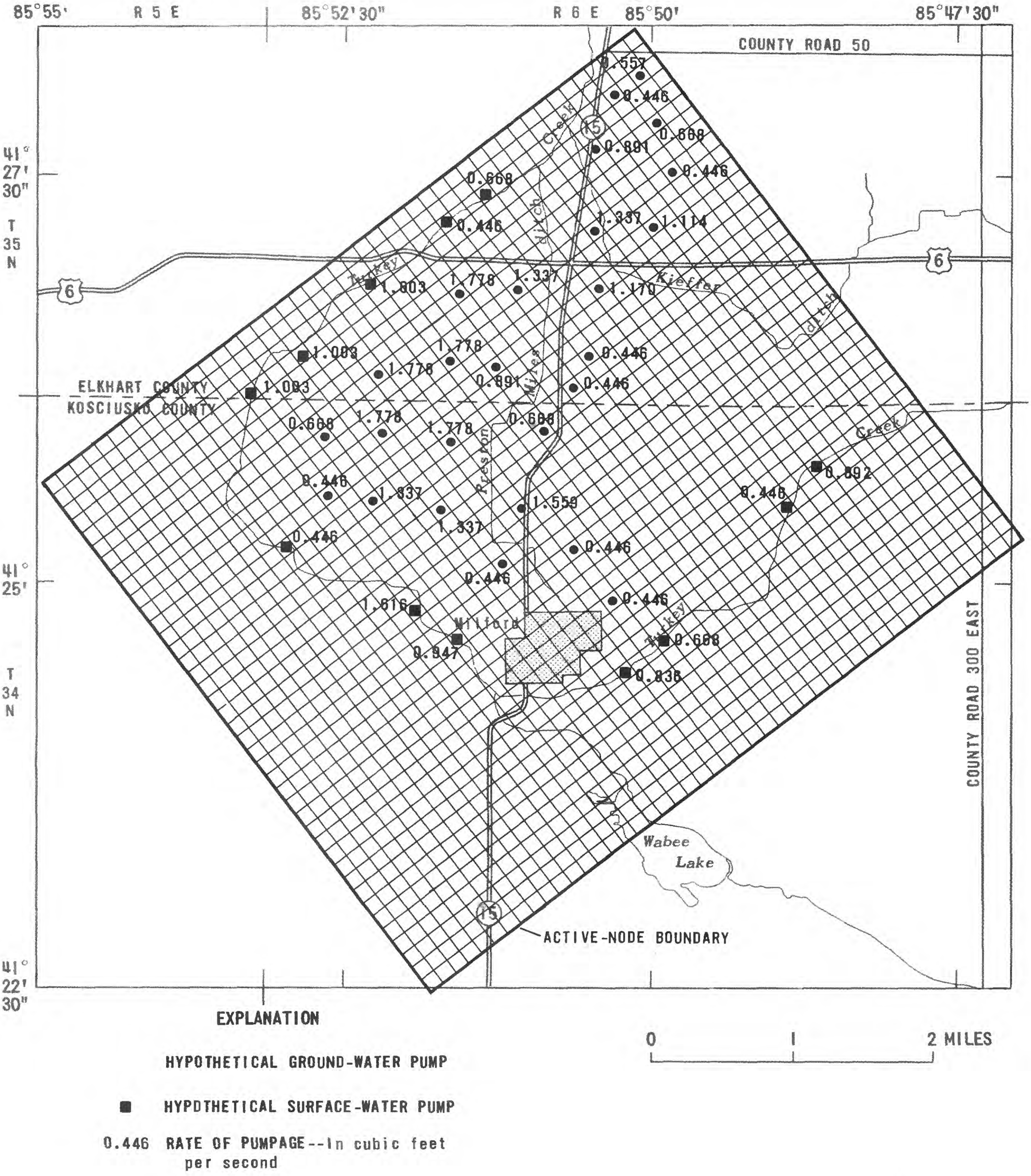


Figure 33.-- Hypothetical pump locations and pumping rates used for pumping-plans 3, 4, and 5.

Table 6.--Conditions for simulating five hypothetical pumping plans

Pumping plan	Application			Irrigated land, (acres)	Duration of pumping (days)	Number of pumps		Rate of pumpage (ft^3/s)	
	Depth[1] (in)	Ground water ($ft^3 x 10^6$)	Surface water ($ft^3 x 10^6$)			Ground water	Surface water	Ground water	Surface water
1	2.0	7.4	1.0	975	9.0	6	1	9.5	1.3
2	9.7	30.2	4.1	975	36.8	6	1	9.5	1.3
3	7.2	61.3	23.6	3,225	27.3	26	12	26.0	10.0
4	9.7	82.7	31.8	3,225	36.8	26	12	26.0	10.0
5	--[2]	--[2]	--[2]	--[2]	--[2]	26	12	26.0	10.0

[1]The depth of water applied uniformly over the irrigated land.
[2]Values not applicable because pumping was to steady state and irrigation was not simulated.

water levels are changing (transient) required two modifications. First, aquifer storage needed to be considered. For the unconfined aquifer (layer 1), the specific yield used was 0.15. For the confined aquifers (layers 2 and 3), the storage coefficient used was 0.0003. Second, constant-head boundary nodes were changed to constant-flux nodes in order to limit the amount of water passing through the boundaries during pumping. Because the McDonald-Harbaugh model does not have a constant-flux option for boundary nodes, constantly discharging (or recharging) wells were used in the model to simulate constant flux at appropriate nodes. The boundary fluxes used for the transient model were those calculated by the steady-state model.

Each pumping plan was simulated with constant-head and constant-flux boundaries. Constant-flux boundaries limit the flow into and out of the study area to the amounts computed by the calibrated model. Therefore, simulated drawdown and streamflow depletion are maximums under constant-flux conditions. The constant-head boundaries allow water to flow into and out of the model in the amount necessary to maintain the constant head. Therefore, water-level changes and streamflow reduction are minimums under constant-head conditions. If the boundaries are far enough away from the pumping wells, modeling results for the two boundary conditions will be the same.

Ideally, the transient model would be calibrated using water-level data collected during periods of pumping in order to verify the values of storage. However, adequate data for a transient calibration were not available so that the accuracy of the results could not be determined. Therefore, the transient simulations indicate how the hydrologic system may respond to pumping stress but cannot be used for precise prediction.

Results

For all four transient pumping simulations, the difference in results between using constant-flux and constant-head boundary conditions was negligible, so only the results of the more conservative constant-flux simulations are discussed in detail. The distribution of drawdown for layer 3 was similar to that of layer 2, although the values of drawdown for layer 3 were usually less than those for layer 2. Because of this similarity, drawdown maps for layer 3 are not presented.

Results of the five model simulations are discussed in terms of potentiometric drawdown in the three layers, reduction in streamflow in Turkey Creek, and ground-water budgets.

Drawdown.--After 9 days of simulated pumping in plan 1, the maximum nodal drawdown in layers 1 and 3 was calculated by the model to be less than 0.5 ft. Evidently, the pumping period was too short for predicted leakage through the two semiconfining layers to reduce the head appreciably in either of the two layers. However, drawdown in layer 2 (the source aquifer) was more pronounced. Maximum drawdown at pumping nodes ranged from about 5 to 15 ft (fig. 34). Calculated drawdown was greatest at the west-central part of the area where two wells are within one-half mile of each other.

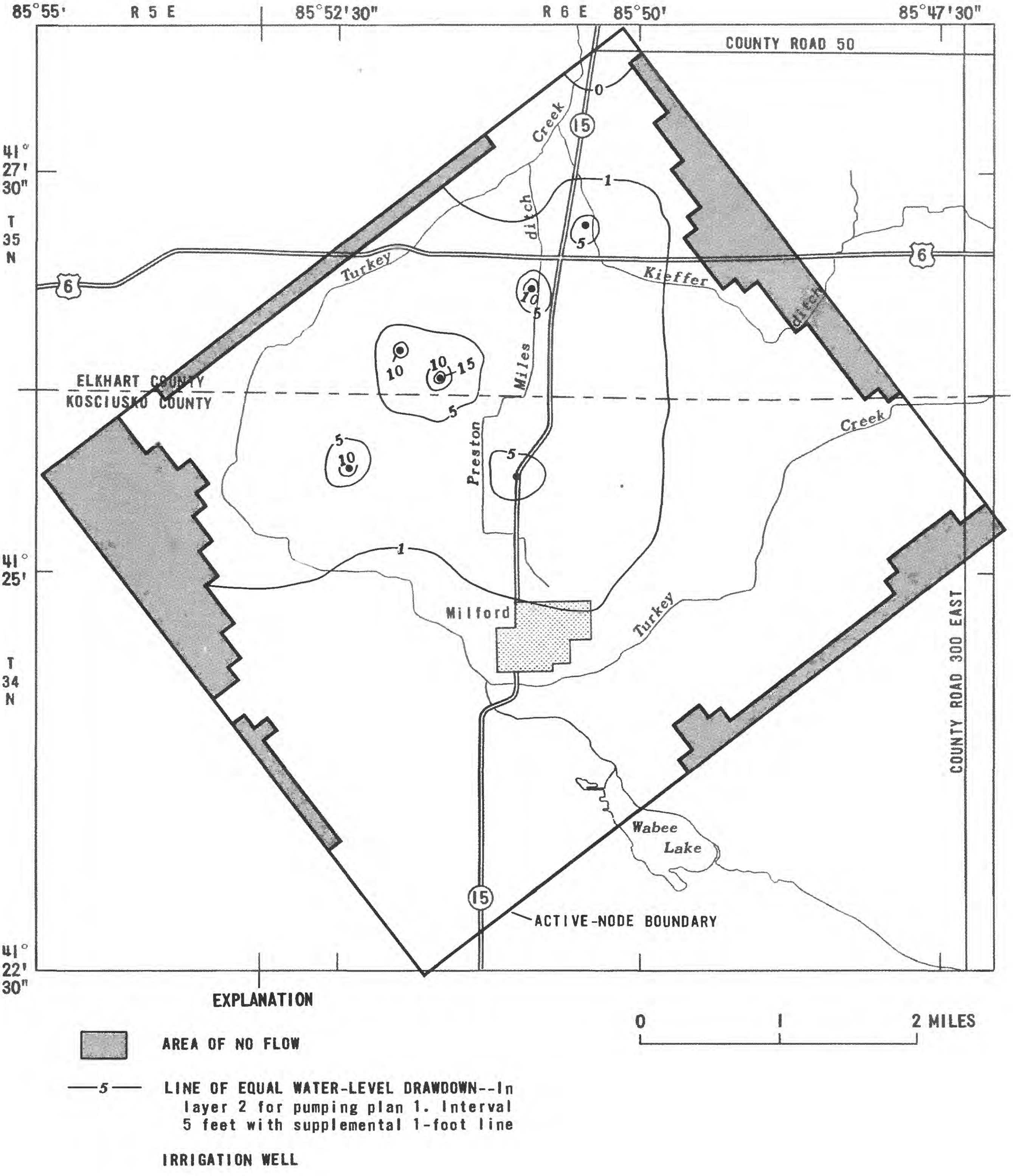


Figure 34.-- Drawdown in layer 2 calculated by the transient model for pumping-plan 1.

The maximum drawdown measured at well 101-5 during the 1982 irrigational season was about 1 ft. (fig. 19). The model indicates that predicted drawdown in the node representing well 101-5 would be about 5 ft. (fig. 34). This difference results from the assumption of continuous pumping used in the model as discussed on page 51.

For pumping plan 2, the model indicates that the 1982 distribution of irrigational wells tapping layer 2 during a dry year would produce drawdowns in layer 1 of less than 2 ft at nodes representing the three western-most irrigational wells (fig. 35). The maximum drawdown calculated was 1.7 ft in a node where the total available drawdown was 37.8 ft (table 7). For layer 1, the effect of pumping was greatest in the west-central part of the outwash, not only because this area is near three pumping wells, but also because the flow till between layers 1 and 2 is thin in this area (fig. 9). In layer 2, drawdowns at the pumping nodes generally ranged from more than 5 to 15 ft (fig. 36). The maximum nodal drawdown (17.3 ft) was about 20 percent of the total available drawdown (table 7). For layer 3, the model calculated a maximum nodal drawdown of 2.7 ft.

For pumping plan 3, maximum irrigational development during a summer of normal precipitation was simulated (table 5). The drawdown in layer 1 was less than 3 ft in the immediate area of pumping (fig. 37). Drawdown was as much as 2 ft beneath both Preston Miles and Kieffer ditches but was less than 1 ft beneath Turkey Creek. In layer 2, calculated drawdown generally ranged from 1 ft to 15 ft (fig. 38). The maximum drawdown (19 ft) is less than one quarter of the saturated thickness of the aquifer (table 7). Calculated drawdown at the northwest constant-flux boundary was nearly 5 ft.

Maximum irrigation development during a period of below-normal precipitation was simulated with plan 4 (table 5). Drawdown maps for layers 1 and 2 (figs. 39 and 40) are similar to those for plan 2. This similarity results from the small increase in ground-water pumping for plan 3 compared to plan 2. Maximum drawdowns in all three layers were also similar for plans 2 and 3 (table 7).

The same pump network and pumping rates used in plan 4 were used in plan 5 (table 6), but the pumping period was extended until steady-state was reached. Drawdowns in all three layers were greater than those in plan 4. Largest drawdowns in layers 1 and 2 were in the vicinity of highest pumping concentration and at the no-flow boundary at the northeast part of the area (figs. 41 and 42). The maximum drawdown in layer 1 nearly equaled the entire saturated thickness and in layer 2 was more than one-third of the total available drawdown.

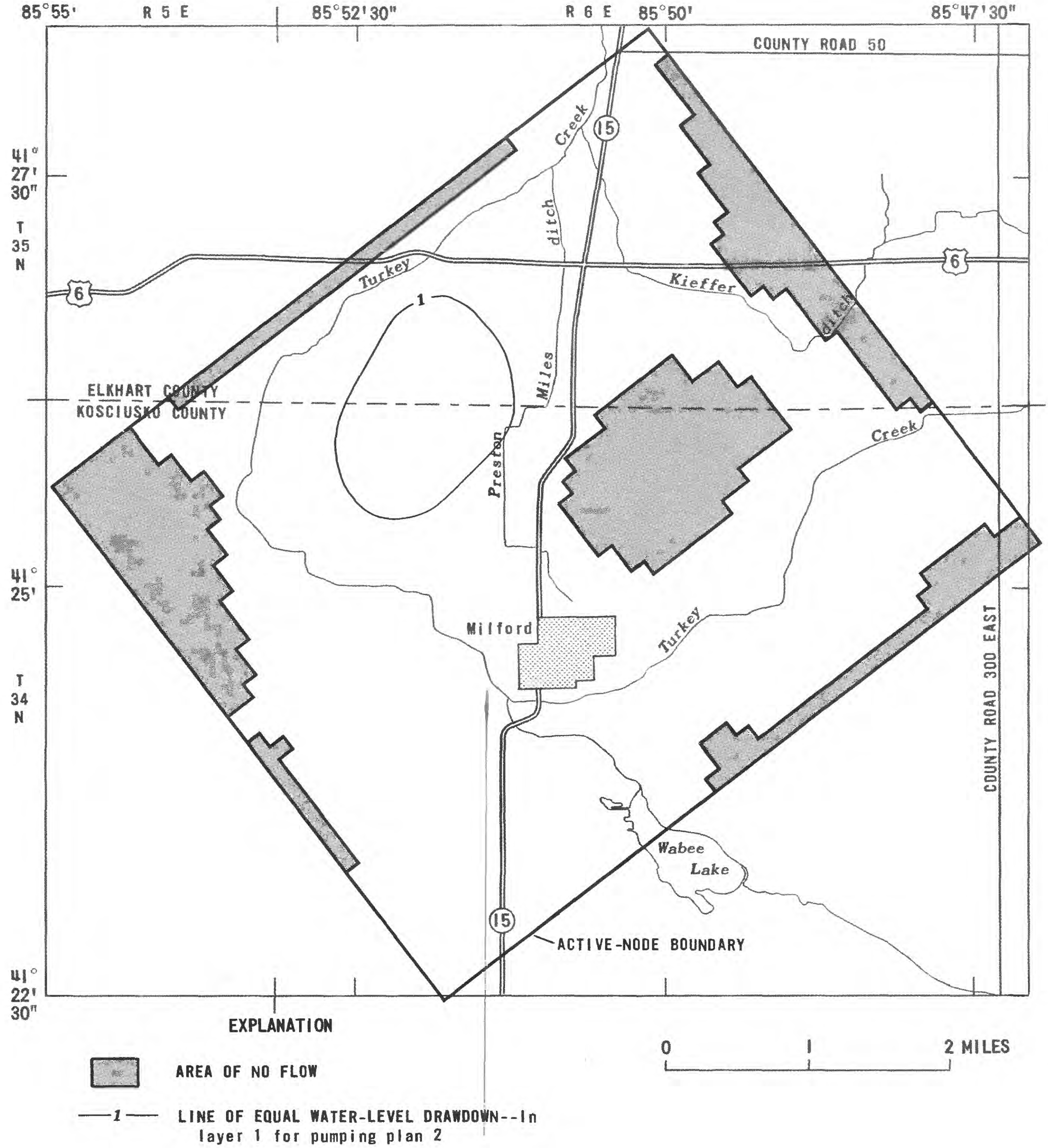


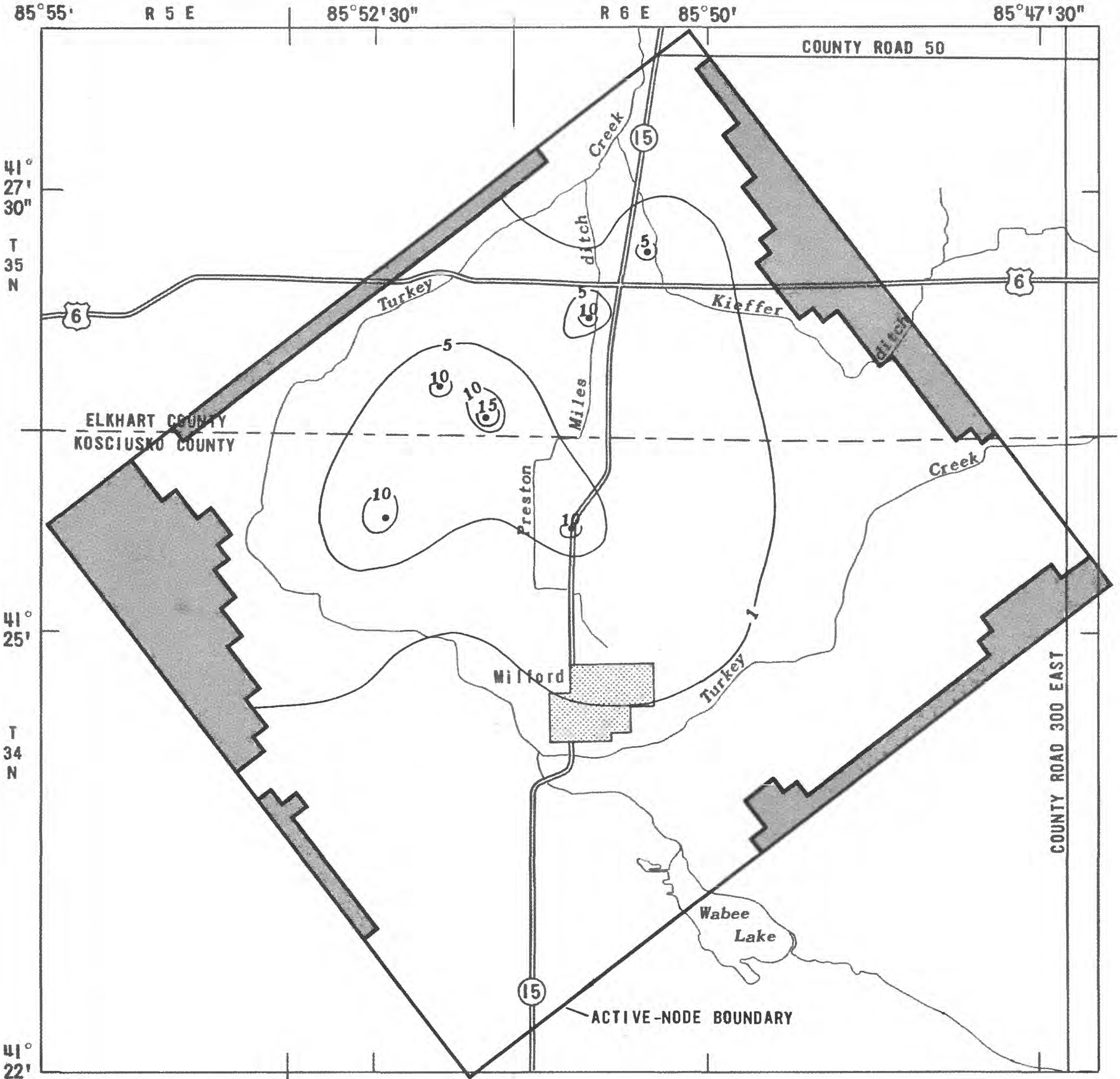
Figure 35.-- Drawdown in layer 1 calculated by the transient model for pumping-plan 2.

Table 7.--Maximum nodal drawdown and total available drawdown for five hypothetical pumping plans

Pumping plan	Layer	Maximum nodal drawdown[1], (ft)	Total available drawdown at node of maximum drawdown[2], (ft)
1	1	0.5	36
	2	16.9	83
	3	1.5	--
2	1	1.7	37
	2	17.3	83
	3	2.6	--
3	1	2.9	36
	2	18.7	84
	3	8.9	--
4	1	3.6	35
	2	20.7	84
	3	9.4	--
5	1	26.0	30
	2	30.6	82
	3	30.0	--

[1] Nodal drawdown is the average drawdown within the node and does not represent drawdown at the pumping well.

[2] Total available drawdown for layer 1 (unconfined) is total saturated thickness before pumping and for layer 2 (confined) is the initial water-level altitude minus the altitude of the bottom of layer 2. Available drawdown for layer 3 is unknown.



EXPLANATION

AREA OF NO FLOW

—5— LINE OF EQUAL WATER-LEVEL DRAWDOWN--In layer 2 for pumping-plan 2. Interval 5 feet with supplementel 1-foot line

IRRIGATION WELL

Figure 36.-- Drawdown in layer 2 calculated by the transient model for pumping-plan 2.

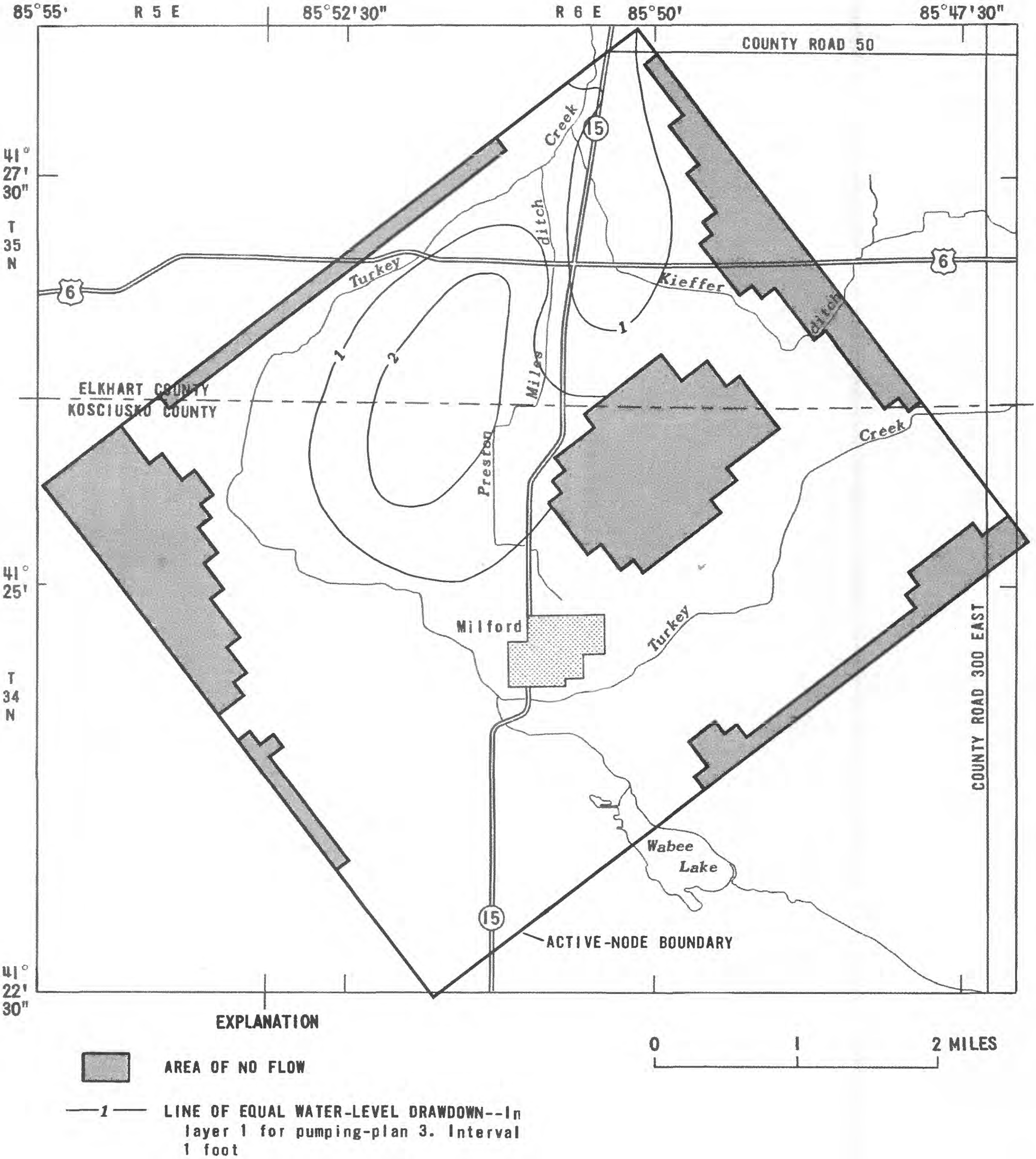
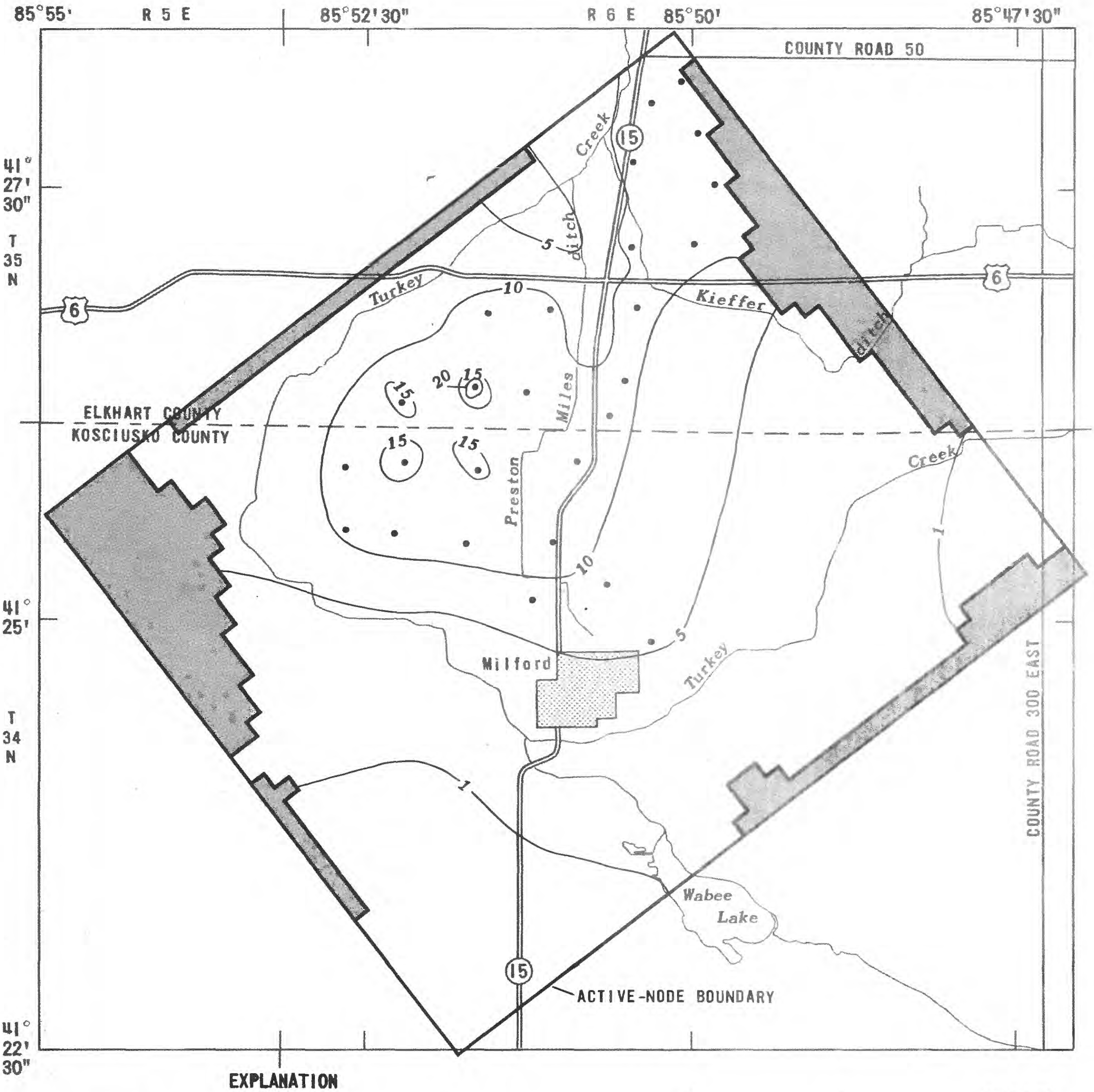


Figure 37.-- Drawdown in layer 1 calculated by the transient model for pumping-plan 3.



EXPLANATION

AREA OF NO FLOW

—5— LINE OF EQUAL WATER-LEVEL DRAWDOWN--In layer 2 for pumping-plan 3. Interval 5 feet with supplemental 1-foot line

• IRRIGATION WELL

Figure 38.-- Drawdown in layer 2 calculated by the transient model for pumping-plan 3.

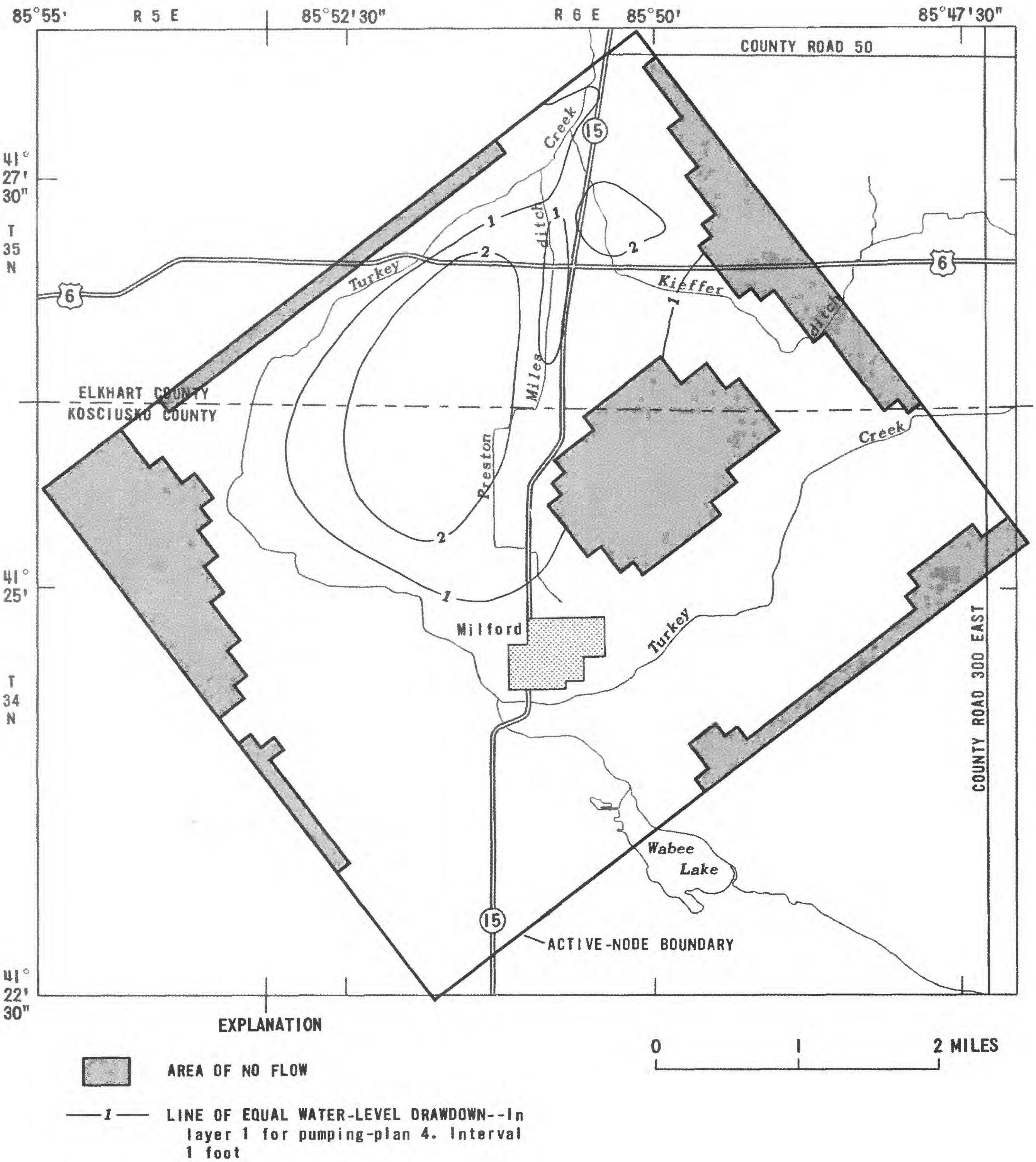


Figure 39.-- Drawdown in layer 1 calculated by the transient model for pumping-plan 4.

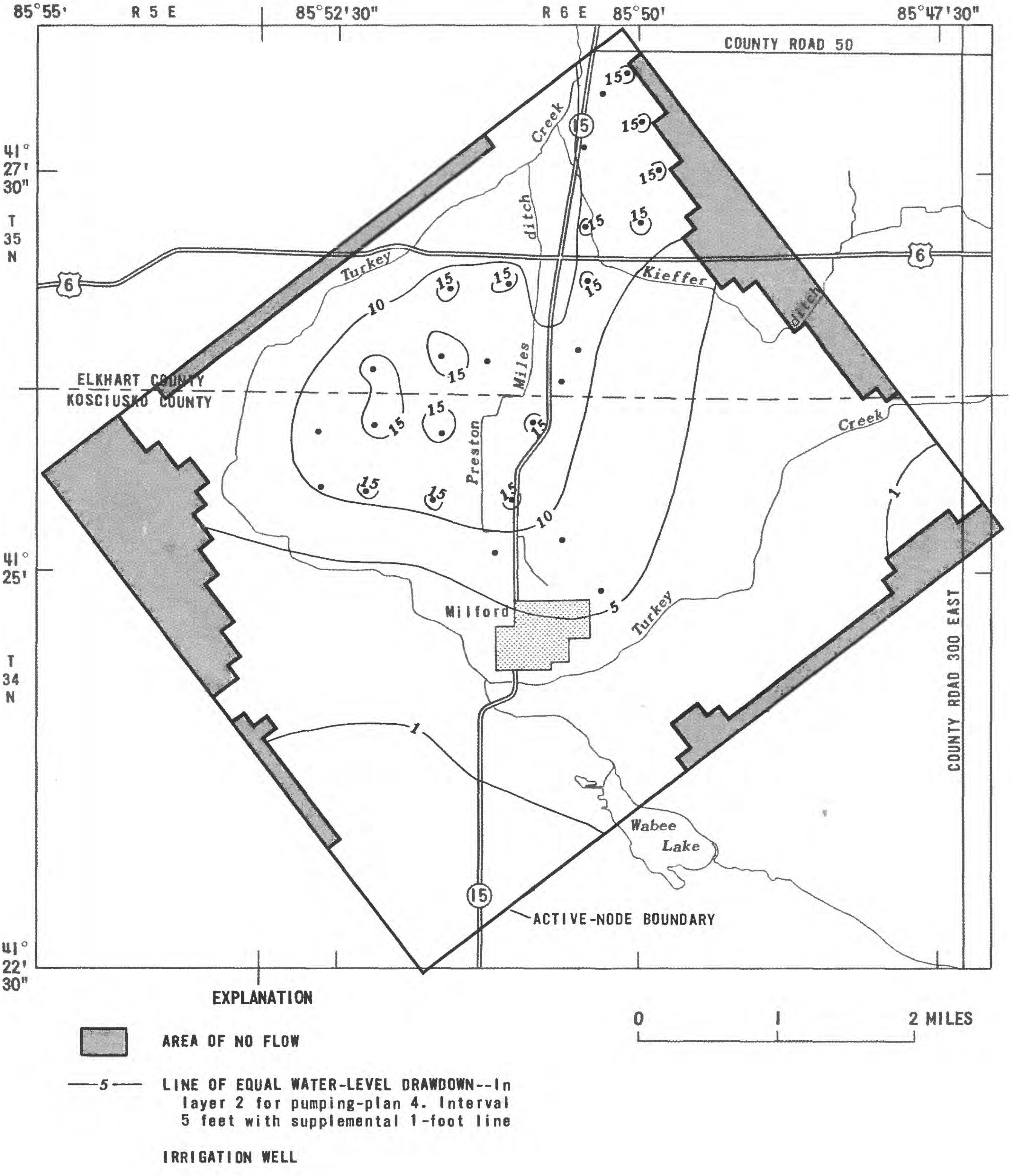
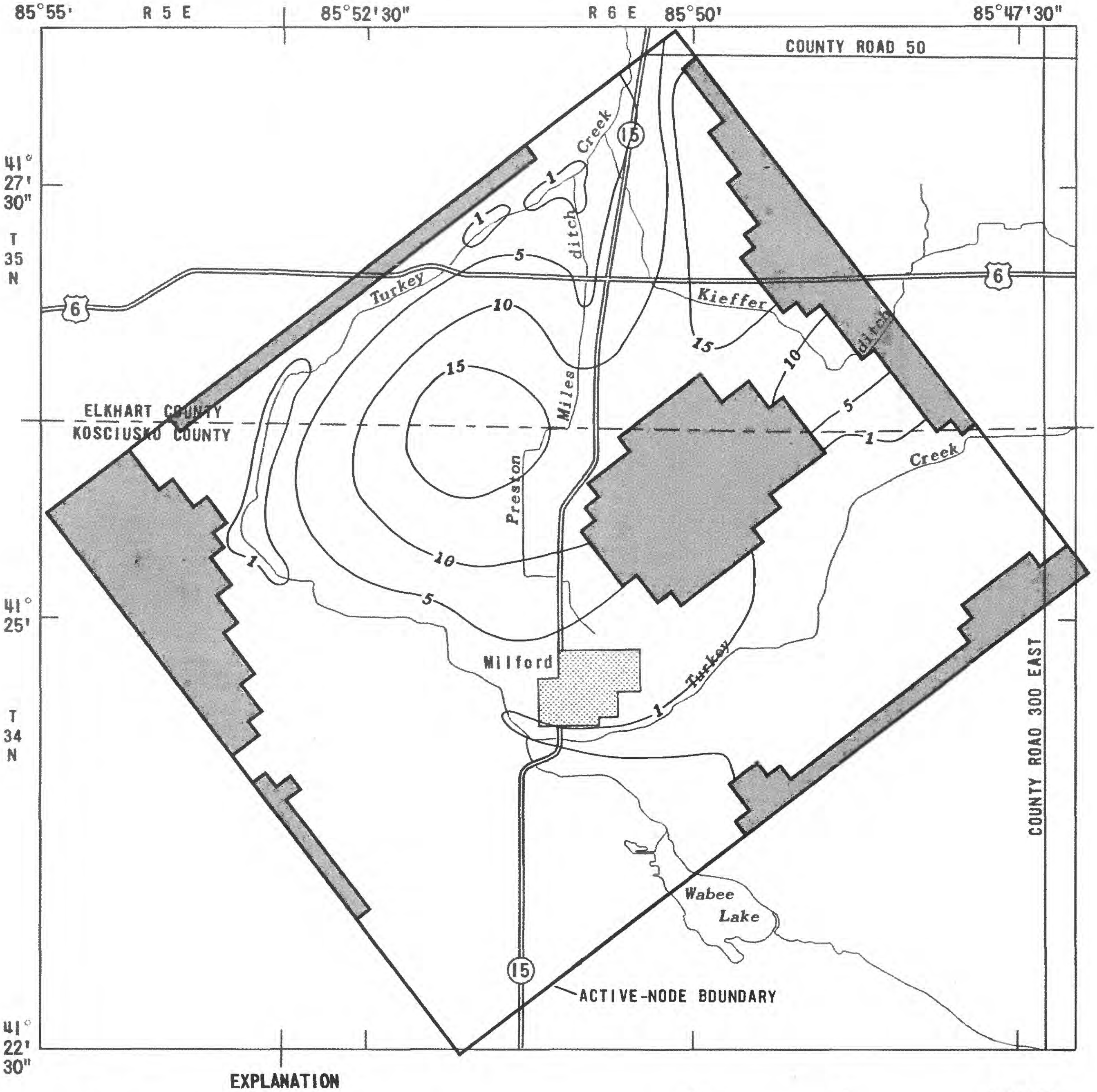


Figure 40.-- Drawdown in layer 2 calculated by the transient model for pumping-plan 4.



EXPLANATION

AREA OF NO FLOW

—5— LINE OF EQUAL WATER-LEVEL DRAWDOWN--In layer 1 for pumping-plan 5. Interval 5 feet with supplemental 1-foot line

0 1 2 MILES

Figure 41.-- Drawdown in layer 1 calculated by the steady-state model for pumping-plan 5.

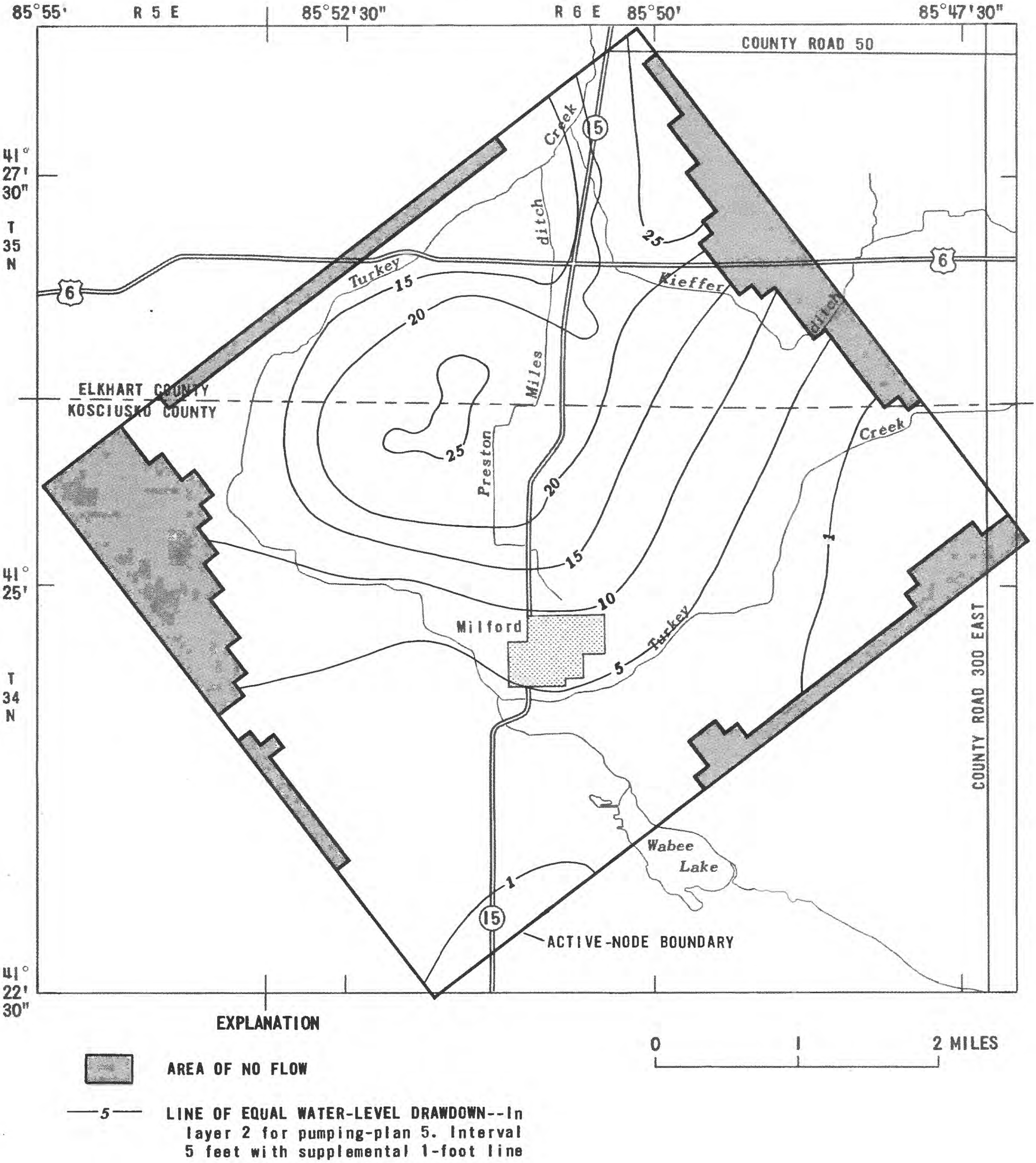


Figure 42.-- Drawdown in layer 2 calculated by the steady-state model for pumping-plan 5.

The reason for the large increase in drawdown in layer 1 for plan 5 is the extended pumping period. Calculated net leakage between layers 1 and 2 in plan 5 was 23 ft^3/s (table 8). Net leakage for plans 3 and 4 was calculated to be 22 ft^3/s, but, because of their shorter pumping periods, these two plans produced much less drawdown in layer 1 than did plan 5.

Table 8.--Net leakage through semiconfining layers calculated by model for calibration and five hypothetical pumping plans

[All values are for constant-flux boundaries except the values for steady-state calibration for which constant-head boundaries were used]

Type of model	Leakage between layers 1 and 2 (ft^3/s)	Leakage between layers 2 and 3 (ft^3/s)
Calibration	4	1
Plan 1	5	1
Plan 2	6	1
Plan 3	22	1
Plan 4	22	1
Plan 5	23	1

Streamflow reduction for Turkey Creek.--Streamflow reduction includes water removed directly from the stream or reduction in ground-water seepage to the stream. Before the effects of the pumping plans on flow in Turkey Creek could be evaluated, the natural (unaffected) streamflow for the hydrologic conditions of each pumping plan had to be estimated. Criteria analogous to those used to estimate precipitation were used to estimate streamflow--that is, streamflow during an irrigational season with normal and below-normal precipitation was assumed to be at 50- and 80-percent flow duration (fig. 15). Also, the authors assumed that without pumpage, ground-water seepage to Turkey Creek would remain at 16.8 ft^3/s (the value calculated during steady-state calibration) regardless of the amount of precipitation.

The results of the analysis of streamflow reduction for Turkey Creek are present in table 9. Information from model calibration is included for comparison. Values of ground-water seepage to Turkey Creek (col. 2) were calculated by the model. Streamflow reduction by ground-water pumping (col. 3) was obtained by subtracting values in column 2 from 16.8 ft^3/s, the amount of natural seepage calculated from field measurements with no pumpage. Values of streamflow reduction from surface-water pumping are presented in column 4. Column 5 is the sum of values in columns 3 and 4. Column 6 is the difference between values in columns 1 and 5.

Table 9.--Streamflow reduction in Turkey Creek calculated by the model for calibration and five hypothetical pumping plans

	Column					
	1	2	3	4	5	6
Type of model	Natural streamflow (ft^3/s)	Streambed seepage (ft^3/s)	Streamflow reduction by ground-water pumping (ft^3/s)	Streamflow reduction by surface-water pumping (ft^3/s)	Total streamflow reduction (ft^3/s)	Net streamflow (ft^3/s)
Calibration	[1]42.1	16.8	--	--	--	42.1
Plan 1	[1]42.1	15.8	1.0	1.3	2.3	39.8
Plan 2	[2]40	14.6	2.2	1.3	3.5	36.5
Plan 3	[3]80	12.3	4.5	10.0	14.5	65.5
Plan 4	[2]40	11.1	5.7	10.0	15.7	24.3
Plan 5	[2]40	[4]-6.2	23.0	10.0	33.0	7.0

[1]Flow measured at county road 50, September 1, 1982.
[2]Flow based on 80 percent flow duration (see fig. 15).
[3]Flow based on 50 percent flow duration (see fig. 15).
[4]Negative number indicates seepage induced from stream to aquifer.

The rate of streamflow reduction by ground-water pumping in plan 2 increased by 1.2 ft^3/s over the rate for plan 1. This increase resulted from extending the pumping period from 9.0 d to 36.8 d. Natural streamflow in plan 2 was also lower than that in plan 1 because plan 2 simulated below-normal precipitation. The combination of these two factors resulted in a net streamflow for plan 2 of 3.3 ft^3/s less than that for plan 1.

For pumping plan 3, calculated total streamflow reduction increased about fourfold compared to plan 2. This increase resulted mainly from the large increase in surface-water pumping. However, because the natural streamflow in Turkey Creek would be greater for plan 3 than for plan 2, the remaining flow for plan 3 would also be greater.

For plan 4, the combination of low natural streamflow and high streamflow reduction resulted in a predicted streamflow of 24.3 ft^3/s. This value was lowest among the transient pumping simulations.

The steady-state simulation for plan 5 resulted in high streamflow reduction--about double the value for plan 4. Flow in Turkey Creek, predicted by plan 5, was about 18 percent of the natural flow. A unique consequence of plan 5 was that it predicted a net seepage loss from Turkey Creek. This loss nearly equaled the remaining streamflow at county road 50.

The effect of ground-water pumping on Preston Miles and Kieffer ditches was evaluated by applying model-calculated seepage to that part of each ditch within the modeled area (table 10). For Preston Miles ditch, that part represented the entire length of channel. Thus, the seepage to Preston Miles ditch represented total flow in the stream channel. The low seepage predicted to both ditches is of questionable accuracy. However, the model detected a slight effect of pumping on seepage to both ditches and, for plans 3, 4, and 5, seepage approached zero.

Table 10.--Ground-water seepage to Preston Miles and Kieffer ditches calculated by the model for calibration and five hypothetical pumping plans

Type of model	Seepage (ft^3/s)	
	Preston Miles ditch	Kieffer ditch[1]
Calibration	1.3	<0.1
Plan 1	.6	< .1
Plan 2	.4	< .1
Plan 3	< .1	< .1
Plan 4	< .1	< .1
Plan 5	< .1	< .1

[1] Does not include seepage to the channel upstream from the modeled area.

A flow commonly used for streamflow assessment is the 7-day, 10-year low flow ($Q_{7,10}$), which is the minimum flow for 7 consecutive days that occurs, on the average, once every 10 years. The $Q_{7,10}$ value for Turkey Creek at county road 50 was estimated to be 12 ft^3/s. This value was obtained by using a $Q_{7,10}$ estimate for Turkey Creek calculated by Stewart (1983, p. 272) and then adjusting this estimate for drainage area.

The transient pumping simulations (plans 1 through 4) predicted that the flow in Turkey Creek remaining after subtracting streamflow reduction (column 6 in table 9) would be well above $Q_{7,10}$. However, plan 5 predicted that the residual flow would be reduced to about 60 percent of $Q_{7,10}$.

Ground-water budgets.--The ground-water budgets calculated by the model for the five pumping simulations are presented on table 11. The budget for the calibration is included for comparison. Streamflow reduction (sum of reduction from all streams) attributed to ground-water pumping was calculated by algebraically summing the source (+) and discharge (-) terms for streambed seepage (table 11), then subtracting the result from the value of seepage unaffected by pumping. This unaffected value is 19 ft^3/s and is the calculated seepage value from the calibration. As an example, net streambed seepage for plan 4 is (+)1 ft^3/s + (-)11 ft^3/s = (-)10 ft^3/s. By summing (-)10 ft^3/s with (+)19 ft^3/s, a streamflow-reduction rate of (+)9 ft^3/s is obtained.

By assuming that recharge and boundary fluxes are constant for all five pumping plans, the only variables in the budgets are storage, streambed seepage, and pumping.

As pumping time increases the percentage of water pumped from wells that comes from storage decreases and correspondingly, the percentage of water coming from streamflow reduction increases. When the pumping period is sufficiently long, steady state is reached. In this case, the percentage of water coming from storage approaches zero and all the water pumped by the wells comes from streamflow reduction. Thus, the percentage of water coming from storage in plans 1, 4 and 5 is 89, 65 and zero percent, and the rates of ground-water pumping and streamflow reduction both equal 26 ft^3/s for plan 5 (table 11). Following pumping, the effect of the pumping on streamflow continues but the magnitude of the effect decreases with time. Streamflow reduction approaches zero as water levels approach prepumping levels and as the storage requirement in the aquifers becomes satisfied (Jenkins, 1970).

Table 11.--Summary of water budgets and streamflow reduction calculated by the model for calibration and five hypothetical pumping plans

Water budget terms	Calibration		Plan 1		Plan 2		Plan 3		Plan 4		Plan 5	
	Flow (ft^3/s)	Percent of total	Flow (ft^3/s)	Percent of total	Flow (ft^3/s)	Percent of total	Flow (ft^3/s)	Percent of total	Flow (ft^3/s)	Percent of total	Flow (ft^3/s)	Percent of total
Sources												
Aquifer storage	--	--	8	28	6	22	19	47	17	44	--	--
Boundary flux	7	33	7	24	7	26	7	18	7	18	7	23
Areal Recharge	14	67	14	48	14	52	14	35	14	35	14	47
Streambed seepage	0	0	0	0	0	0	0	0	1	3	9	30
Total	21	100	29	100	27	100	40	100	39	100	30	100
Discharges												
Boundary flux	2	10	2	7	2	7	2	5	2	5	2	7
Streambed seepage	19	90	18	62	16	60	12	30	11	28	2	7
Pumping wells	--	--	9	31	9	33	26	65	26	67	26	86
Total	21	100	29	100	27	100	40	100	39	100	30	100

SUMMARY AND CONCLUSIONS

The water-resources and the potential for their development were studied in a 16.5 square-mile area near Milford, Indiana (fig. 1). Ninety percent of the land use is agricultural. The well-drained, sandy soils in most of the area are derived from glacial outwash and respond favorably to irrigation, which represented 46 percent of the water use in 1982.

Surface drainage is to Turkey Creek primarily by way of two tributaries--Preston Miles ditch and Kieffer ditch (fig. 2).

The outwash-aquifer system comprises three aquifers separated by two continuous confining beds. Aquifer 1 comprises surficial sand and gravel and averages about 40 ft in thickness. The average hydraulic conductivity of the aquifer is estimated to be 280 ft/d, and the transmissivity is estimated to range from 5,600 to 15,400 ft^2/d. The specific yield is estimated to be 0.15.

Aquifer 2, below aquifer 1 and separated from it by a continuous flow till, is also about 40 ft in thickness. The average hydraulic conductivity is estimated to be 180 ft/d and the transmissivity is estimated to range from 3,600 to 10,800 ft^2/d. The storage coefficient is estimated to be 0.0003.

A clay till at least 8 ft thick separates aquifer 2 from aquifer 3. On the basis of scant data, the thickness of aquifer 3 ranges from 20 to 150 ft. No estimates of hydraulic conductivity of the aquifer were available, but the higher clay content of the aquifer suggests that it has a lower conductivity than either aquifers 1 or 2.

The flow till between aquifers 1 and 2 generally ranges in thickness from 5 to 40 ft. The vertical hydraulic conductivity was estimated to range from 0.01 to 0.1 ft/d. In the west-central part of the area, where the flow till is thin and sandy, leakage through this layer is especially evident.

The thickness and the hydraulic conductivity of the clay between aquifers 2 and 3 were assumed to be 8 ft and 0.01 ft/d, respectively, based on very limited information.

Unit discharge, flow duration, and streamflow hydrographs for Turkey Creek indicate a good hydraulic connection between the stream and the surficial aquifer.

Estimated recharge to the ground-water system ranges from 4.2 in./yr for confined aquifers overlain by till to 10.5 in/yr for the surficial outwash aquifer. Ground-water flow is generally horizontal in the aquifer and parallels the flow in Turkey Creek. Vertical flow upward to Turkey Creek is indicated near the channel.

A calibrated three-dimensional, ground-water flow model was used to simulate ground- and surface-water development with five hypothetical pumping plans.

1. Land irrigated in 1982 and above-normal precipitation.
2. Land irrigated in 1982 and below-normal precipitation.
3. Maximum acreage irrigated and normal precipitation.
4. Maximum acreage irrigated and below-normal precipitation.
5. Maximum year-round water use.

Transient pumping was assumed in the first four plans. Pumping to steady state was assumed in plan 5. Water was pumped from aquifer 2 and from Turkey Creek. Recharge and ground-water flow across boundaries of the model were assumed to be constant. Thus, water to wells was assumed to come from streamflow reduction and, for transient pumping, from aquifer storage.

Under the preceding constraints, simulated irrigational pumping for plans 1 through 4 would cause minimal streamflow reduction in Turkey Creek. However, flow in the two smaller streams--Preston Miles and Kieffer ditches--might cease. Most of the water to the wells during transient pumping (nearly 70 percent for plan 4) came from aquifer storage. Thus, the effect of ground-water withdrawals on streamflow would continue after pumping stopped. Drawdown in layer 2 was predicted to be as much as 20.7 ft for plan 4 and represented 25 percent of available drawdown.

All water withdrawn during pumping plan 5 was assumed to come from streamflow reduction. Based on streamflow reduction in plan 5, the flow in Turkey Creek would be reduced to less than $Q_{7,10}$. Drawdown in aquifer 2 was as much as 30.6 ft, which is about one-third of the total available drawdown. Part of aquifer 1 was nearly dewatered. Plan 5 was designed to explore the results of a maximum stress rather than to simulate possible future development.

The droughty soils developed on outwash require irrigation in order to optimize crop yields. However, because these soils overlie sand and gravel aquifers of high transmissivity and high rates of recharge, and because surface water is readily available, this area can support present water use and substantial future growth. Similar results were reported for an outwash system in the Howe study area, Lagrange County. Throughout the St. Joseph River basin, the areas of outwash that respond best to irrigation are also those areas that probably have the best water supply to meet irrigational needs.

Unfortunately, this favorable relationship between need and supply is not found in all areas of the State. In two northwestern counties, irrigation from fractured limestone has caused drawdowns great enough to affect the water supply of nearby domestic wells. The contrast between pumping from the outwash and pumping from bedrock demonstrates that similar hydrologic stress can produce dissimilar effects when applied to differing hydrologic systems. The contrast illustrates the need to account for differences in hydrogeology when developing statewide water-management policies.

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