Contract Report 537

Ground-Water Investigation at Peoria, Illinois: Central Well-Field Area

by Richard Schicht
Office of Ground-Water Resources Evaluation & Management

Prepared for the
Illinois-American Water Company

September 1992

Illinois State Water Survey
Hydrology Division
Champaign, Illinois

A Division of the Illinois Department of Energy and Natural Resources

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Schicht, Richard
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INTRODUCTION

The Central-Well Field Area in Peoria for this study is defined as the area in all of Township 8N and Range 8E and in Sections 19-21 and 28-33 of Township 9N and Range 8E (figure 1) northwest of the Illinois River and east of Kickapoo Creek. Almost the entire area lies within the city of Peoria corporate limits. Centers of ground-water withdrawal are located at Dodge Street, Griswold Street and just southwest of the recharge pits in Section 17 T8N R8E. The area's ground-water resources have been important to area industries since 1840. Development of area ground-water for municipal use was initiated in 1946 at the Dodge Street well-field site.

Because of declining ground-water levels, local municipal and industrial officials asked the Illinois State Water Survey to investigate ground-water resources in the Peoria area in 1940. The Survey has been collecting ground-water information in the study area ever since.

Two Water Survey reports describe the ground-water resources in the Peoria area: Horberg, Suter, and Larson (1950) and Marino and Schicht (1969). A third Water Survey report by Suter and Harmeson (1960) describes the construction and operation of artificial recharge pits designed to use river water to replenish the aquifer during periods of low ground-water levels. The pits are located in Section 16, T8N R8E.

The main purpose of this report was to determine the ground-water availability in the area north of Griswold Street. The need to meet maximum peak demands forecast for the year 2005 was of particular concern.

Acknowledgments

Funding for this study was provided by the Illinois-American Water Company, an investor-owned utility that provides water to the City of Peoria and other municipalities in the Peoria vicinity. The Illinois State Water Survey appreciates having been involved in this study. The information obtained on aquifer hydraulic properties and the riverbed materials are invaluable in furthering our knowledge of the ground-water resources of this area.

This report was prepared under the supervision of Adrian Visocky, Director, Office of Ground-Water Resources Evaluation & Management. Special thanks go to Survey employees Andy Buck and Scott Meyer, who conducted the aquifer tests. Also appreciated are the efforts of former Survey employee Thad Wilson, who worked on the equipment designed to obtain water levels.

Pam Lovett prepared the final manuscript. John Brother and Linda Hascall prepared the illustrations. Eva Kingston edited the report. Special thanks also go to

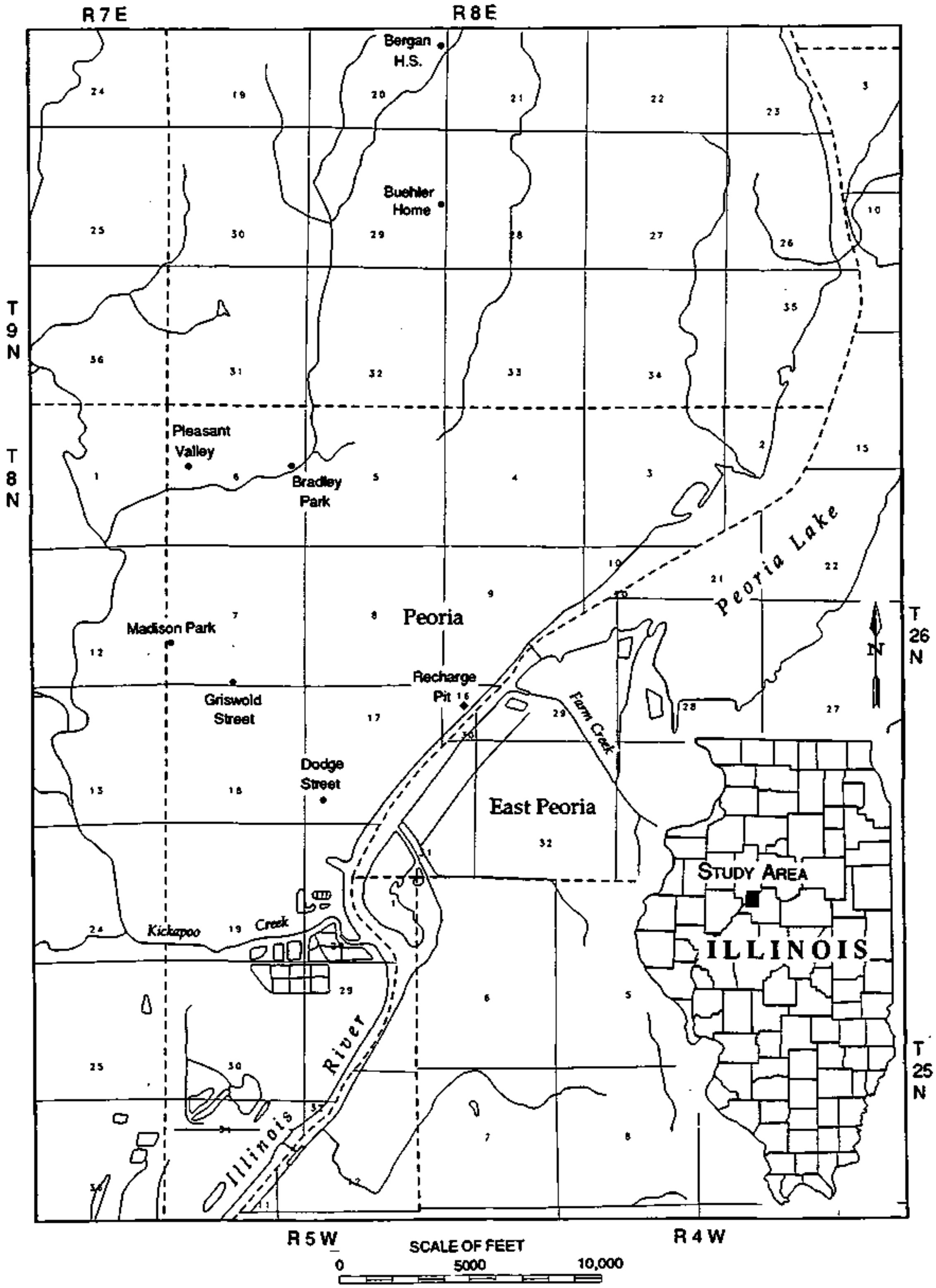

Figure 1. Location of study area

Becky Bennett for assistance in financial matters relative to the contract with the Illinois-American Water Company.

GEOLOGY AND HYDROLOGY

For a detailed discussion of the geology and hydrology of the study area, the reader is referred to Horberg et al. (1950). The following section is based largely upon this report.

The study area is covered with glacial drift, which probably reaches thicknesses exceeding 300 feet in the bedrock valley in the north. The glacial drift is underlain by bedrock formations of Pennsylvanian age, which do not constitute an important aquifer because of their low permeability and poor water quality with depth. Large water supplies, available from older bedrock formations, are unsatisfactory because of poor water quality.

In a large part of the study area, the glacial drift contains a thick deposit of glacial sand (Sankoty sand) (figure 2), except along the Illinois River where the bedrock is overlain by glacial drift consisting of stratified sand and gravel deposits (glacial outwash). The Sankoty sand is also missing on bedrock uplands in the western part of the study area. A contour map showing the topography of the bedrock surface is shown in figure 3. The bedrock topography doesn't differ from that shown by Horberg et al., except for the area along the Illinois River in the southeastern part of the study area where 10-foot contours are shown. This 10-foot contour interval indicates the existence of a bedrock high.

The main feature of the bedrock is the bedrock valley extending from north to south along the western part of the study area. The valley contains thick deposits of Sankoty sand.

Figure 2 shows ground-water conditions in the study area. A large part of the area underlain by Sankoty sand provides favorable conditions for development of large ground-water supplies except where the Sankoty thins in the northeast. Conditions are also favorable for development of large supplies in the glacial outwash materials along the river except where the outwash thins towards Peoria Lake. Ground-water is under water-table conditions in most of the study area.

Drillers logs of key wells north of Griswold Street are shown in Table 1.

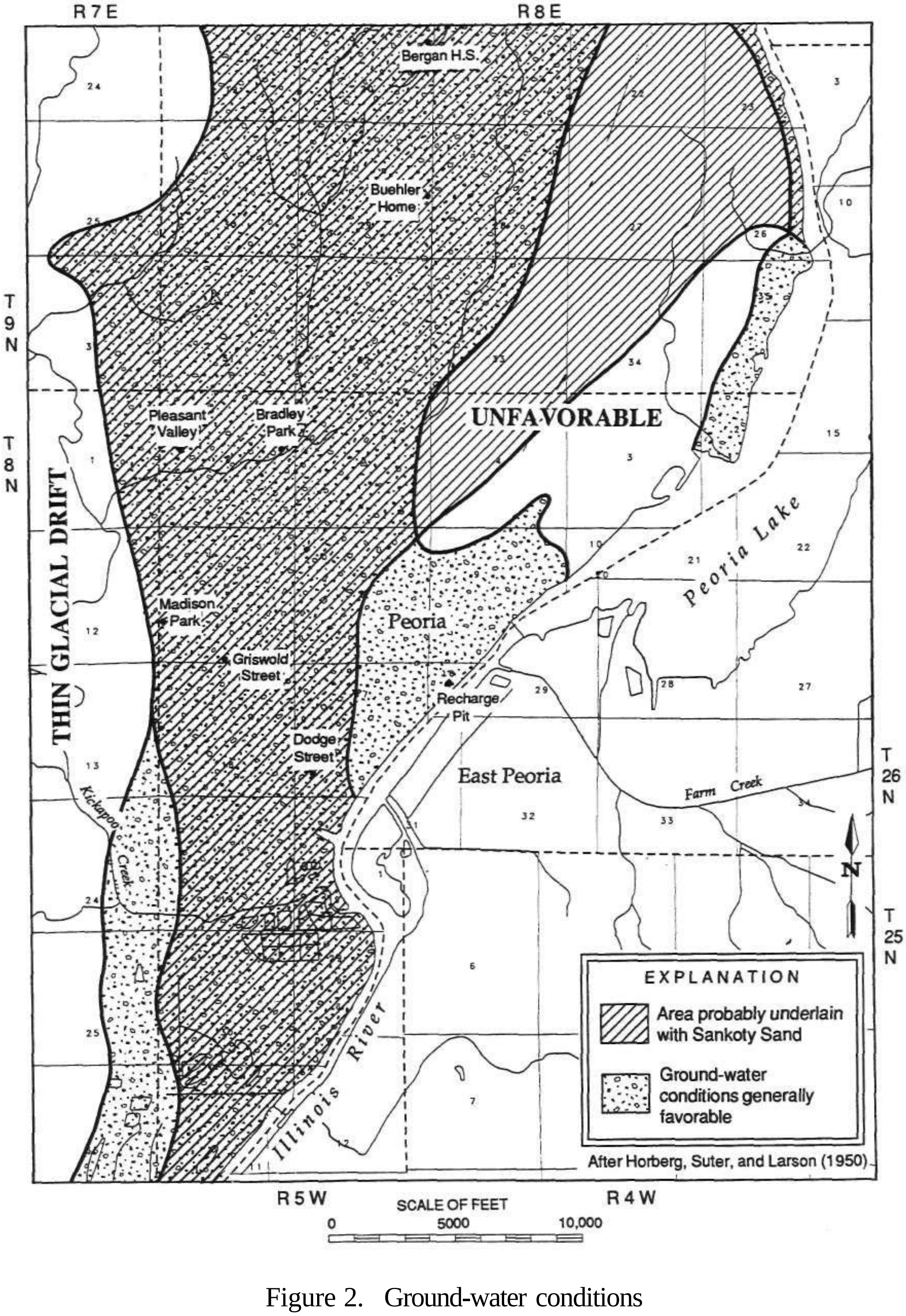

Figure 2. Ground-water conditions

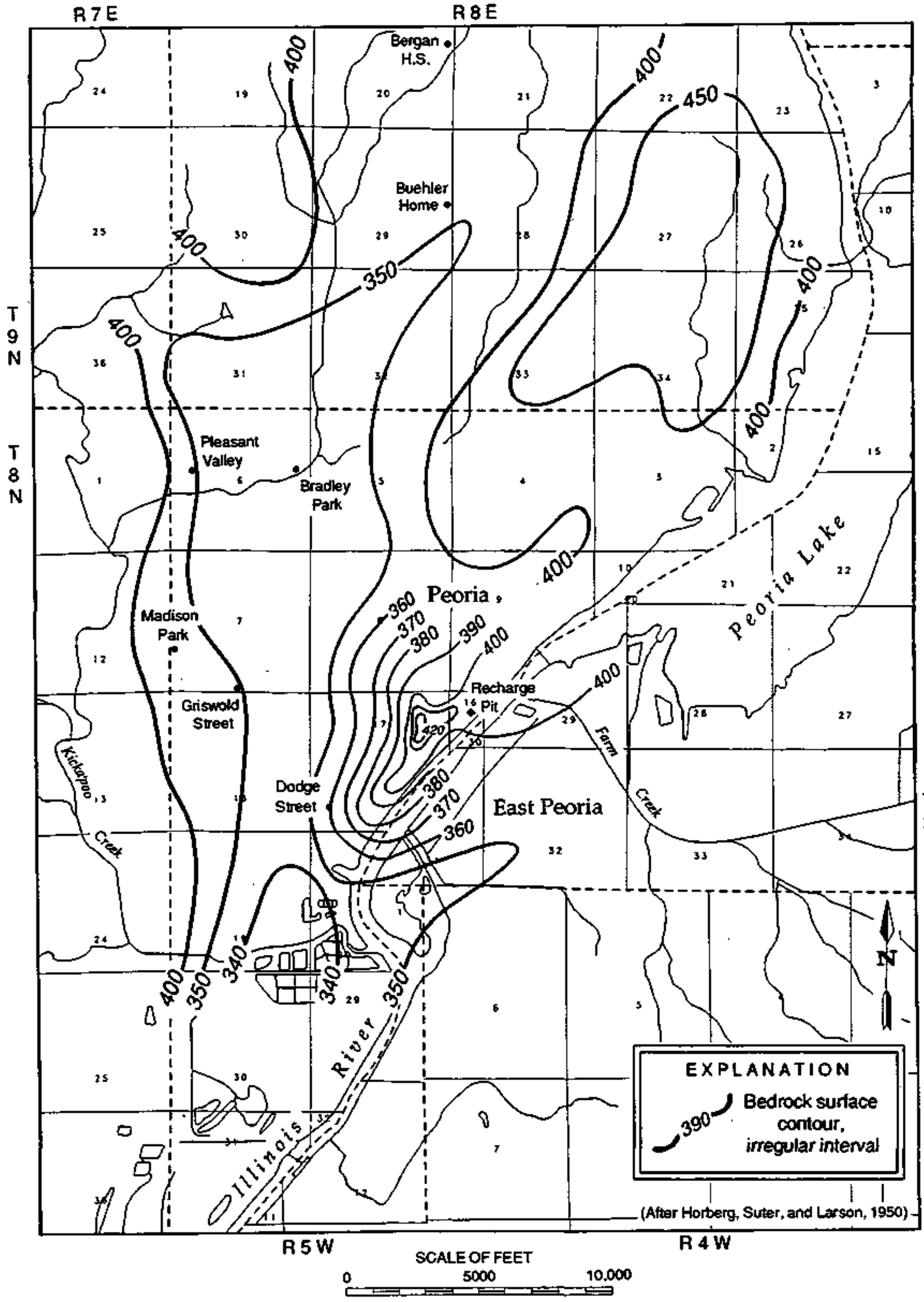

Figure 3. Bedrock surface map

Table 1. Drillers Logs of Key Wells North of Griswold Street

Griswold St.
LSD 514

Formation	*Depth (ft)*
Black top soil	0-5
Sand	5-20
Yellow clay w/stones	20-31
Blue till	31-34
Coarse gravel and stones	34-59
Rocks (as large as 20")	59-62
Sand and gravel and stones	62-100
Sand	100-105
Sand, gravel, stones	105-111
Gravel and cobble stones	111-119
Coarse sand w/stones up to 3")	119-162
Blue clay	162-163
Very fine sand	163-165
Bedrock-shale	165

Madison Park
LSD 460

Formation	*Depth (ft)*
Brown clay	0-18
Dirty sand	18-30
Coarse sand	30-47
Very fine sand	47-65
Medium sand	65-77
Coarse sand	77-86
Medium-coarse sand	86-107

Pleasant Valley #4
LSD 510+

Formation	*Depth (ft)*
Clay, sand, gravel, brick	0-6
Brown fine sand to coarse gravel	6-21
Brown fine to coarse sand	21-40
Brown medium sand to small gravel, some coarser layers (boulder at 50)	40-121
Brown fine to coarse sand	121-124
Brown medium sand to coarse gravel	124-130

Bradley Park
LSD 509

Formation	*Depth (ft)*
Sand, pebbly dirty	0-15
Sand, medium, yellow clean	15-30
Sand, gray	30-50
Sand and gravel	50-70
Sand, medium, slightly pinkish	70-85
No sample	85-90
Sand, pebbly, gray to brown	90-95
Sand, slightly pebbly, pink, highly polished	95-162
Bedrock	162

Buehler Home
LSD 665

Formation	*Depth (ft)*
Clay, yellow	0-15
Clay, blue	15-100
Clay, sandy blue (gas)	100-105
Clay, sandy, blue	105-150
Sand, coarse	150-245
Sand, very fine	245-269
Sand, coarse	269-274
Sand, very fine	274-312
Shale	Shale

Bergan High School
LSD 720

Formation	*Depth (ft)*
Top soil	0-5
Yellow clay, some gravel	5-14.5
Gray clay, traces sand and gravel	14.5-106
Gray clay, traces sand and gravel, also water	106-145
Gray clay, traces sand and gravel	145-204
Dirty dry sand	204-280
Sankoty sand, some gravel	280-315
Fine medium sand	315-322
Medium-coarse sand, few stones	322-341

HYDRAULIC PROPERTIES

The principal hydraulic properties that influence well yields and water-level response to pumpage in the water-table aquifer in the study area are the transmissivity and the storage coefficient.

The capability of an aquifer to transmit water is expressed as transmissivity, which is defined as "the rate of flow of water through a unit width of the aquifer under a unit hydraulic gradient." Associated with transmissivity is the hydraulic conductivity, which is defined as "the rate of flow through a unit cross-sectional area of the aquifer under a unit hydraulic gradient." The hydraulic conductivity is derived by dividing the transmissivity by the total saturated thickness of the aquifer as shown by the following equation. Dimensions that will be used in this report are also given below.

$$k = T/m \qquad (1)$$

where k = hydraulic conductivity, gallons per day per square foot (gpd/ft^2)
T = transmissivity, gallons per day per foot (gpd/ft)
m = saturated aquifer thickness, feet (ft)

The storage properties of the water-table aquifer are expressed by the storage coefficient, which is defined as "the volume of water that is released from or taken into storage per unit of surface area of the aquifer per unit change in head normal to that surface." The storage coefficient is represented in equations by "S" and is dimensionless. Typical values for water-table storage coefficients in unconsolidated materials similar to those in the study area range from 0.01 to 0.3.

The hydraulic properties of aquifers are commonly determined from water-level and pumpage data collected during specific capacity or aquifer tests. Specific capacity tests, the more common type, measure the effect of pumping at a known constant rate in the pumped well only. Aquifer tests measure the effect of pumping at a known constant rate, both in the pumped well and in observation wells penetrating the aquifer.

The specific capacity or yield in gallons per minute per foot of drawdown (gpm/ft) cannot be used to determine aquifer properties with the same degree of accuracy as can aquifer test data. The specific capacity varies with the effective radius of the well, which may be difficult to determine for wells finished in sand and gravel aquifers. More importantly the specific capacity can be significantly affected by partial penetration, drawdown due to turbulent flow in the well (well loss), aquifer dewatering, and geohydrologic boundaries. Still, the analysis of specific capacity data is important since it may be the only data available to estimate aquifer transmissivity and hydraulic conductivity. Specific capacity data is not normally used to estimate the storage coefficient.

Specific Capacity Data

A summary of aquifer property data determined from specific capacity data collected in the study area is given in Table 2. Methods of analysis outlined by Walton (1962) were used.

Aquifer Test Data

Test 1

A major effort of the study involved collecting and analyzing data from two aquifer tests. The first test was conducted at the Water Company's Dodge Street site to verify aquifer hydraulic property values determined from specific capacity data given in Table 2 for glacial outwash materials along the Illinois River.

Three production wells (Wells 1,3, and 4) are located at the Dodge Street site (figure 4) in close proximity to each other (Well 2 is abandoned). Wells 1,3, and 4 were drilled from 1946 to 1950 and are reported to be equipped with pumps with capacities of 2, 2, and 4 million gallons per day (mgd), respectively. To meet demand, one or more wells need to be in operation at any given time. Well construction features are described in Water Survey files. A correlated drillers log from the Illinois State Geological Survey follows.

Depth (ft)	Formation
0-15	Till, sand, gravel and clay
15-56	Sand, gravel, rocks
56-100	Sand, gravel, stones, rocks, clay
100-118.5	Sand, gravel and stones

Boulders taken from Well 2 are shown in figure 5.

It was proposed that three observation wells be drilled to measure drawdown during a 24-hour pumping test. Because of difficulty in obtaining easements, however, it was possible to drill only one observation well on Water Company property. The observation well, located 134 feet from Well 3 and 121 feet from Well 4 (figure 4), was drilled by Layne-Western Company, Aurora, Illinois, in December 1991. It is 110 feet deep and is constructed of 2-inch polyvinyl chloride (PVC) pipe with 20 feet of slotted (20 slot) PVC pipe at the bottom.

The drillers log for the observation well follows.

Table 2. Results of Specific Capacity Tests

Well location	Well #	Well owner	Well depth (ft)	Test date	Saturated thickness (ft)	Static level (ft)	Pump rate (gpm)	Observed specific capacity (gpm/ft)	Trans-missivity (gpd/ft)	Hydraulic conductivity (gpd/ft^2)	Test length (hr)
08N 07E											
1.1A	3	Pleasant Valley Water Dist.	128	1969	80	48.00	510	12.4	81,000	1,010	2.0
1.1D	2	Pleasant Valley Water Dist.	98	1959	58	40.00	239	17.1	38,000	660	0.5
1.4E	1	Pleasant Valley Water Dist.	70	1954	42	28.00	265	15.6	25,000	600	0.5
08N 08E											
6.7E	4	Pleasant Valley Water Dist.	130	1985	91	39.00	600	19.1	120,000	1,320	3.0
7.5A		Peoria Water Works Co.(Gl)	166	1949	79	87.75	1430	164.9	245,000	3,110	7.5
7.5A		Peoria Water Works Co.(G2)	162	1953	71	91.00	1470	183.8	320,00	4,520	8.0
9.3F		Block and Kuhl Co.	59		26	33.00	1300	325.0	740,000	28,400	3.0
17. IE	1	Hiram Walker and Sons	57	1934	27	29.00	3700	528.6	700,000	25,900	
17. IF	2	Hiram Walker and Sons	53	1934	23	30.00	1327	884.7'	1,150,000	50,000	0.3
17.1F	3	Hiram Walker and Sons	56	1934	21	30.00	2500	625.0	1,100,000	52,400	
17.2E	4	Hiram Walker and Sons	53	1934	27	27.00	3000	375.0	560,000	20,800	2.0
17.7B		Peoria Water Works Co.(Dl)	118	1944	59	63.25	1030	120.0	5,100,000	86,500	1.8
17.7B		Peoria Water Works Co.(D2)	113	1946	61	62.00	1390	560.0	2,900,000	47,000	
17.7B		Peoria Water Works Co.(D3)	124	1948	66	57.87	1600	800.0	1,520,000	23,200	7.0
20.7F		Peoria Sanitary Dist.	104		72		500	400.0	2,100,000	29,200	24.0
20.8E	4	Com. Sol. Corp.	94	1936	58	35.86	1800	236.8	400,000	6,900	24.0
19.1A	7	Com. Sol. Corp.	93	1985	82	11.00	1623	115.9	500,000	6,100	24.0
19.IB	1	Com. Sol. Corp.	106	1985	92	14.00	1381	125.5	450,000	4,900	24.0
19.2B	9	Com. Sol. Corp.	97	1985	87	10.00	1566	82.4	440,000	5,100	24.0

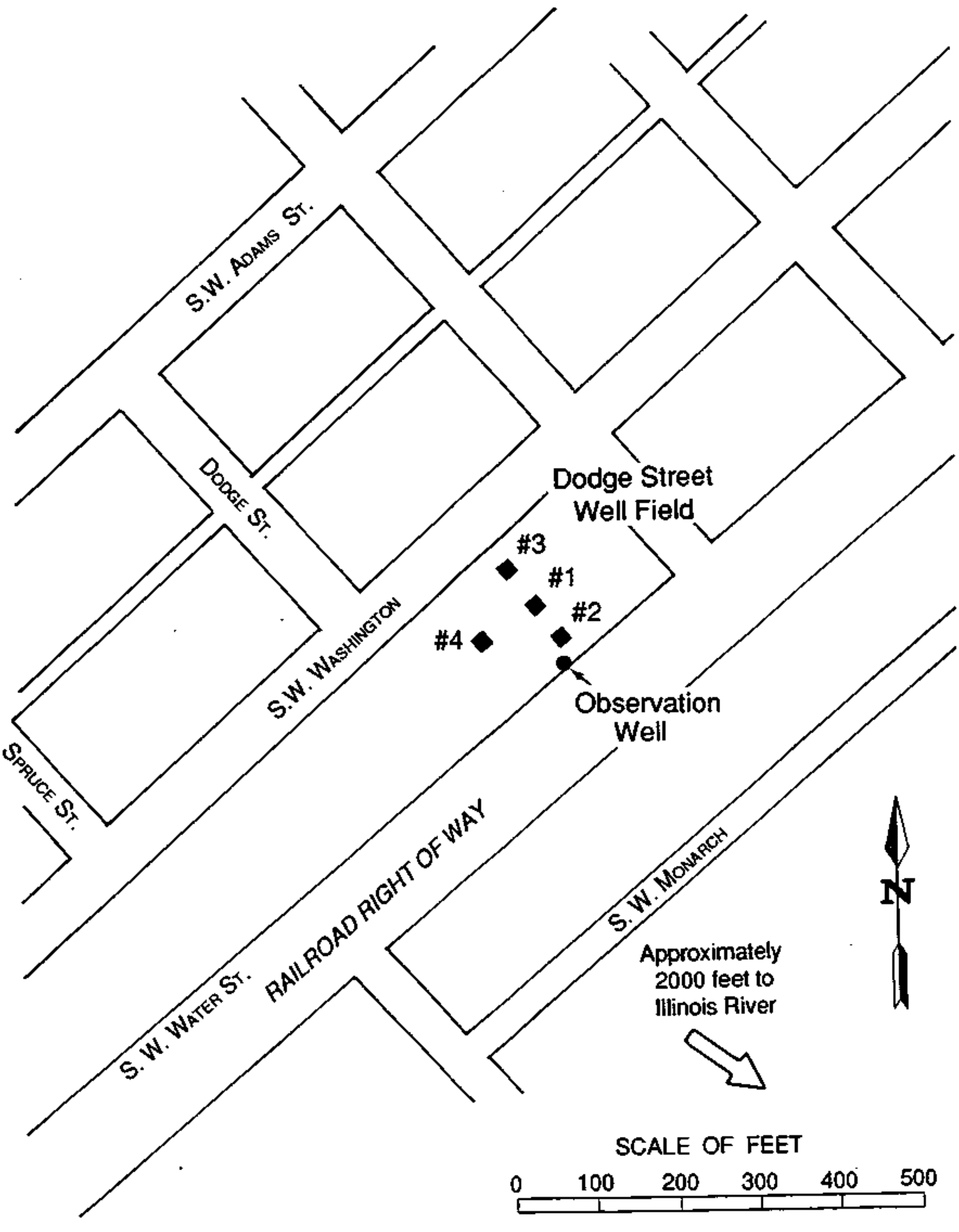

Figure 4. Location of wells at Dodge Street

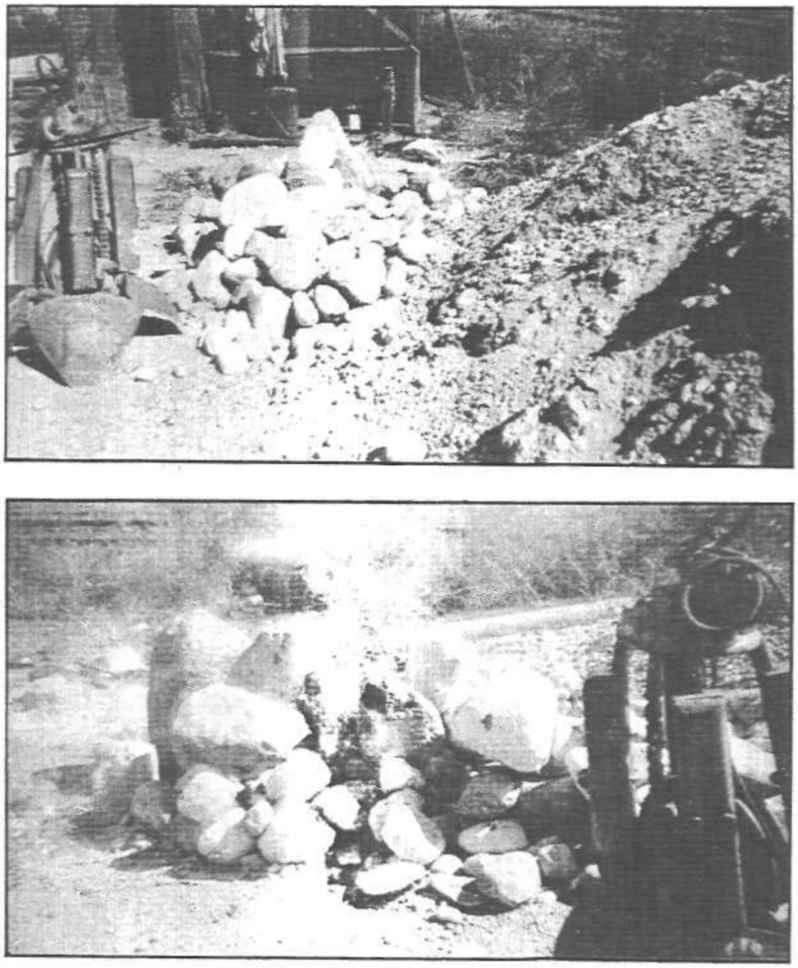

Figure 5. Boulders taken from Dodge Street Well No. 2

Depth (ft)	Formation
0-3	Black top soil
3-15	Sand and gravel (bricks, etc.)
15-20	Sand and gravel
20-110	Coarse sand and gravel
	Occasional boulder and sand seams
110-120	Fine sand-lime ledges

To conduct the aquifer test on February 12 and 13, 1992, it was necessary to change the normal pumping schedule. To meet required water demand, it was planned to pump Well 4 continuously for 24 hours before and during the test. Well 1 would be off during this period. Well 3 would be turned on the first day and pumped continuously for 24 hours. A computer-driven water-level measuring device would measure the effects of pumping Well 3 on ground-water levels in the observation well. The device was installed in the observation well on February 11 to measure water-level trend prior to starting the pump in Well 3. A summary of the pumping sequence is given below.

Date	Time	Pump status	Observation well depth to water from land surface (ft)
2/11		Well 1 off	
	11:00 a.m.		41.78
	11:20 a.m.	Well 4 on	
	11:22 a.m.	Well 3 off	
2/12	10:37 a.m.		41.89
	11:01 a.m.	Well 3 on	
2/13	9:58 a.m.		42.17
	10:30 a.m.	Well 3 down	
	10:51 a.m.	Well 4 down	
	11:10 a.m.	TEST OVER	

Analyses of water-level drawdown data collected in the observation well 11:01 a.m., February 12, to 10:30 a.m., February 13, and recovery water-level data from 10:30 to 10:51 a.m. and 10:51 to 11:10 a.m., February 13, were made using methods outlined by Prickett (1965) for aquifer tests under water-table conditions.

Results of the analyses indicate the aquifer transmissivity in the vicinity of the Dodge Street wells is about 1,500,000 gpd/ft. The storage coefficient is in the water-table range, about 0.15 to 0.20. The hydraulic conductivity based on a saturated thickness of 78 feet is about 19,000 gpd/ft^2 and compares well with hydraulic conductivities from specific capacity analyses given in table 2 and figure 6.

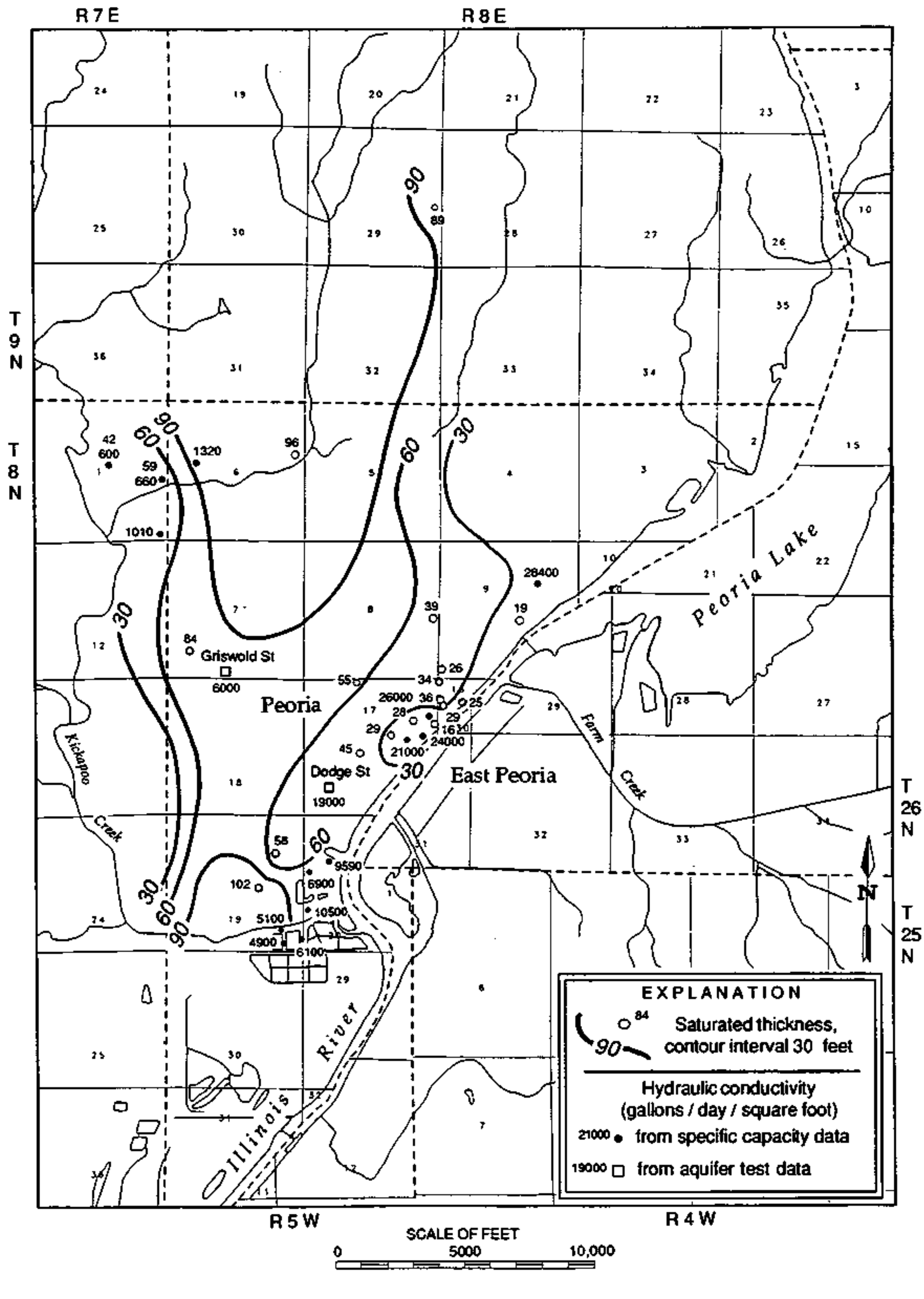

Figure 6. Aquifer hydraulic conductivity and saturated thickness

Test 2

The second test was conducted at the Griswold Street site (figure 7). The purpose of this test was to determine aquifer properties for the Sankoty sand and the existence of a possible aquifer boundary west of the Griswold Street site.

Two production wells, Wells 1 and 2, are located at the Griswold Street site (figure 7) about 150 feet apart. Well 1 was drilled in 1949, and Well 2 was drilled in 1953. The wells are reported to be equipped with 2 mgd pumps. Well construction features are described in Water Survey files. A drillers log for Well 1 is given below.

Depth (ft)	Formation
0-5	Black top soil
5-20	Sand
20-31	Yellow clay with stones
31-34	Blue till
34-59	Coarse gravel and stones
59-62	Rocks (as large as 20 in.)(blasted with dynamite)
62-100	Sand, gravel, stones
100-105	Sand
105-111	Sand, gravel, stones
111-119	Gravel and cobblestones (slow drilling)
119-162	Coarse sand with stones up to 3 in. diameter
162-163	Blue clay
163-165	Very fine sand
at 165	Bedrock-shale

Three observation wells to observe water-levels during a proposed aquifer test were drilled by Layne-Western Company, Aurora, Illinois, in December 1991 and February 1992 south of the Griswold Street wells at approximate distances of 130, 380, and 670 feet from Griswold Street Well 1 (figure 7). The wells were constructed with 2-inch PVC casing and 20 feet of No. 20 slot PVC screen. Observation Wells 1, 2, and 3 were 130, 130, and 160 feet deep, respectively. Drillers logs were comparable to the log for Griswold Street Well 1.

A seven-day aquifer test was conducted April 2-9, 1992. It was planned to pump Griswold Street Well 1 continuously for seven days after both Wells 1 and 2 had been shut down for several days allowing water levels to stabilize. Well 2 would not be pumped during the seven-day period. Water-level measurements were to be made in the observation wells with automatic water-level measuring equipment during and 24 hours prior to the test to detect any water-level trend. Unfortunately, emergency needs for water necessitated frequent use of Well 1 prior to the test and intermittent use of Well 2 about 45 hours after the start of the test. It was possible to correct water levels measured during the 45 hours for trend based on water-level

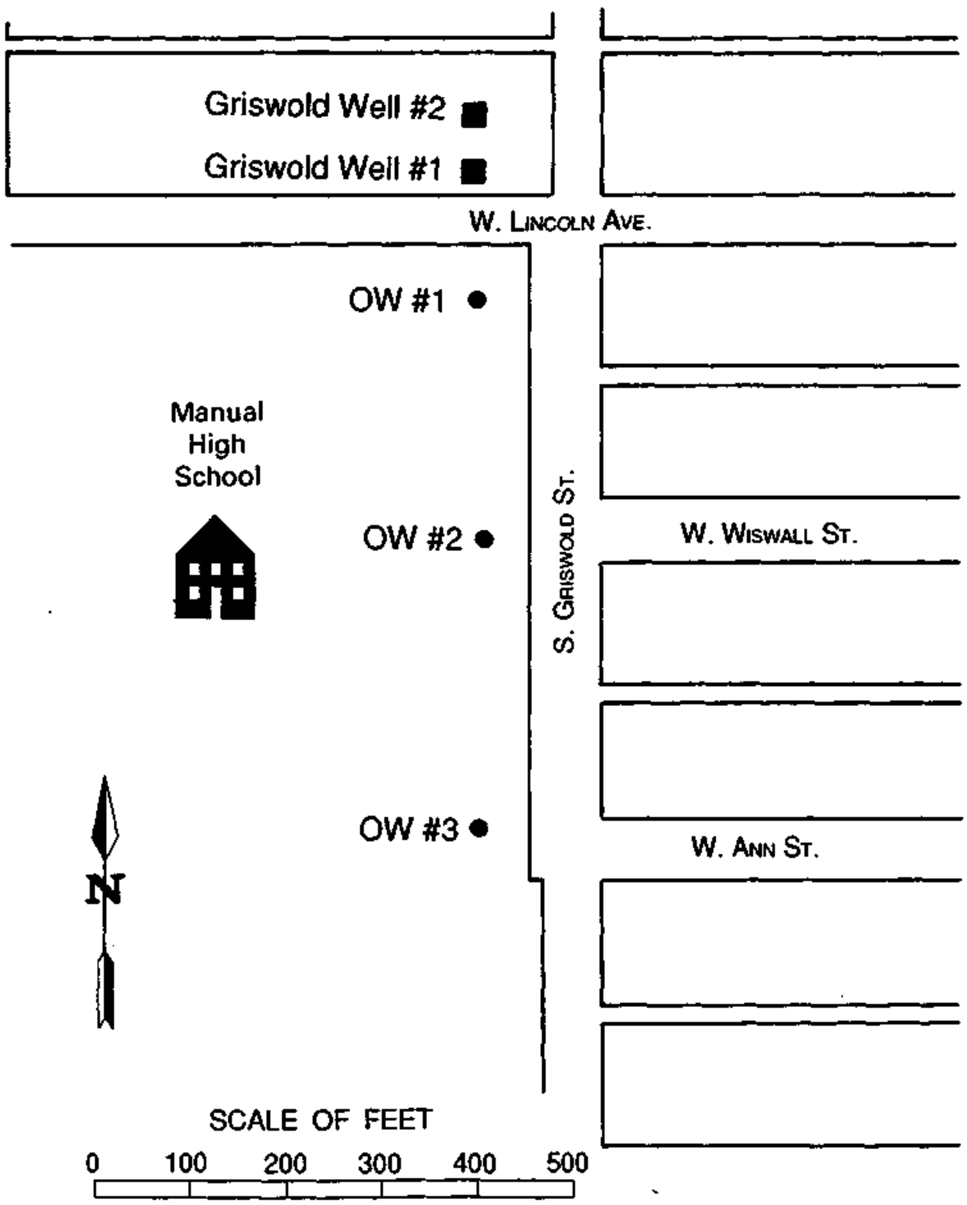

Figure 7. Location of wells at Griswold Street

measurements made prior to the test. It was not possible to correct for changes in water levels due to pumping Well 2 after 45 hours into the test.

Ground-water-level data collected during the 45-hour test and corrected for trend were adequate to estimate aquifer hydraulic properties at the Griswold Street site. Analysis of adjusted water-level data using methods described by Prickett (1965) indicate the aquifer transmissivity at the site approximated 527,000 gpd/ft. Based on a saturated thickness of 88 feet, aquifer hydraulic conductivity was estimated to be about 6,000 gpd/ft^2, a reasonable value acording to examination of aquifer properties for the Sankoty sand in Water Survey files. Estimates made for the coefficient of storage were in the water-table range.

Examination of water-level data collected during the seven-day test indicated it was unlikely that an aquifer boundary was encountered.

Aquifer properties were used to compute theoretical drawdown in Griswold Street Well 1. A computed drawdown (6.9 feet), including estimates for well loss, partial penetration, and dewatering, agrees closely with a measured airline drawdown (7 feet). (Theoretical drawdown wasn't computed for Dodge Street wells. Since drawdown couldn't be measured in the pumped wells at Dodge Street, a comparison of actual versus theoretical drawdown wasn't possible.)

WATER LEVELS

Water-Table Contour Maps

Water-table contour maps are available from Horberg et al. (1950) and Marino and Schicht (1969). For preparation of water-table contour maps for this study, it was decided to use Marino and Schicht's data for August 1966 and April 1967. Water-level data used by Horberg et al. are not available. Field checks by Water Survey staff for a ground-water quality study in the Peoria-Pekin area indicated that many of the wells measured by Marino and Schicht have been abandoned and filled (Burch, personal communication, 1992).

Since pumpage patterns today are similar to those for 1966 and 1967, and a larger number of observation wells were available for measurement, it was concluded that the data collected by Marino and Schicht are the best available for preparation and study of water-table contour maps.

The water-table contour maps for August 1966 and April 1967 are shown in figures 8 and 9, respectively. Features of the two maps are generally the same. There is a trough of low ground-water levels along the Illinois River from about Peoria Lake to just northeast of Kickapoo Creek. The trough was due to ground-water withdrawals at the Dodge Street site by the water company and northeast of the

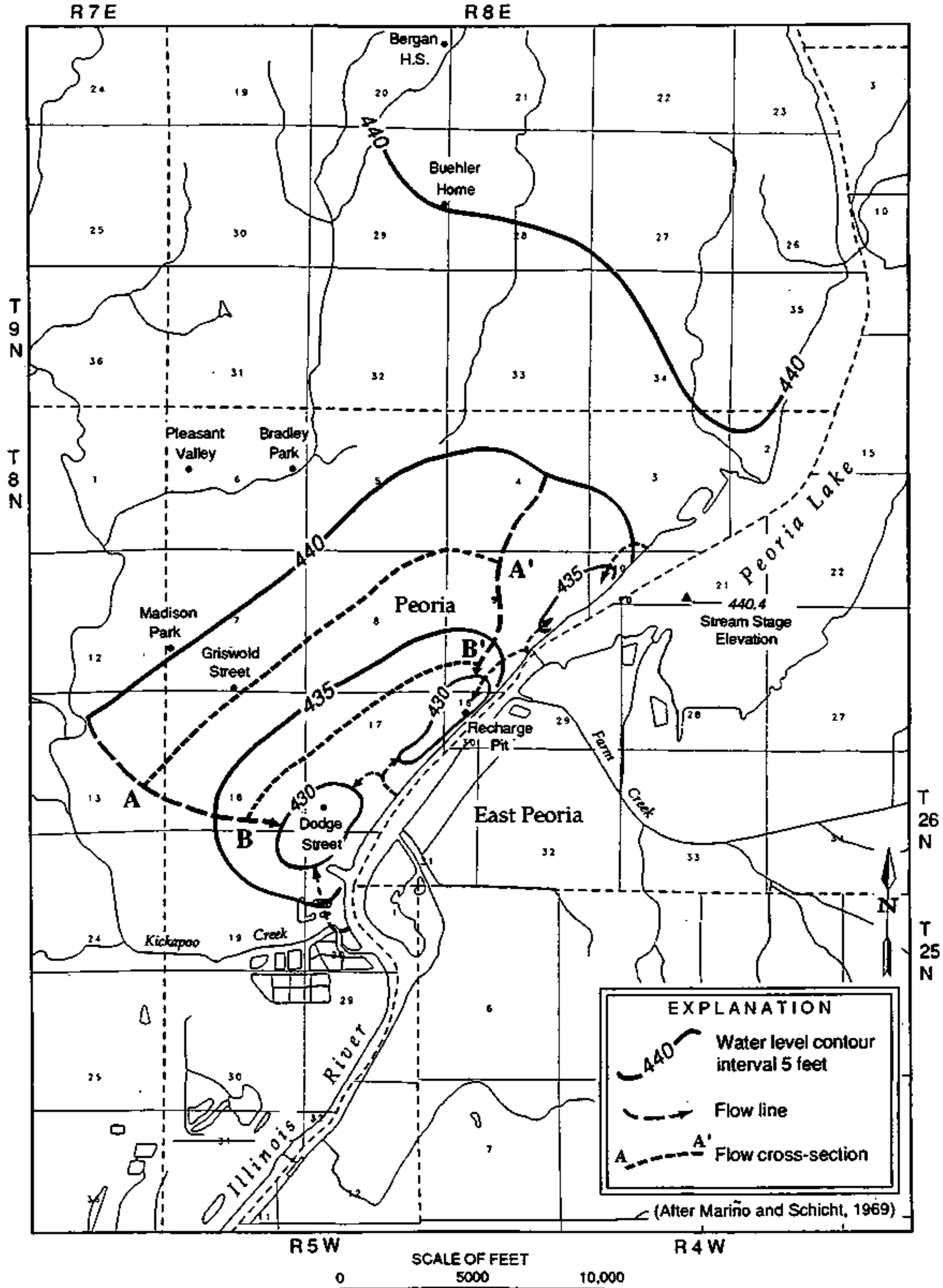

Figure 8. Water-table contour map, August 1966

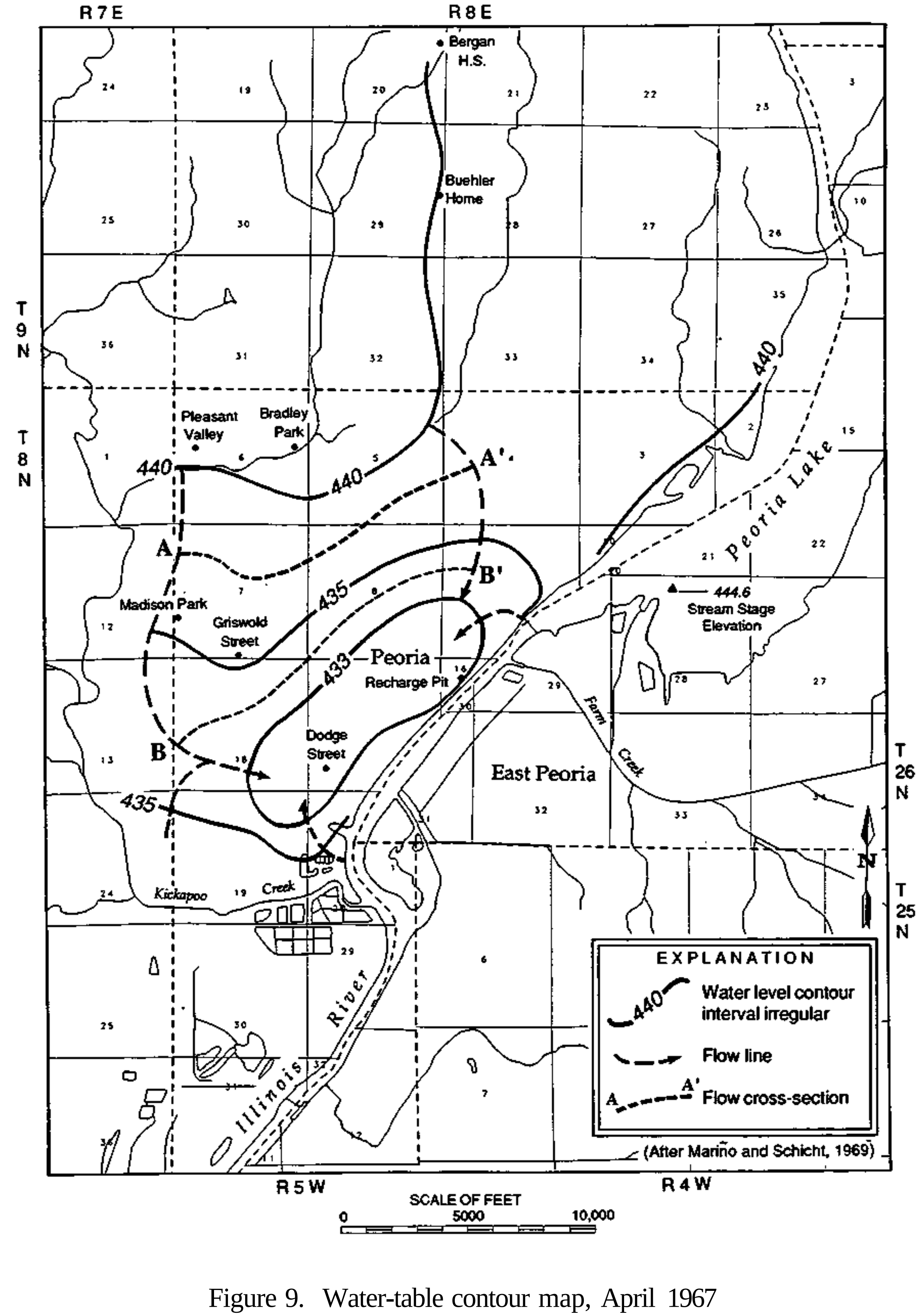

Figure 9. Water-table contour map, April 1967

Dodge Street site by industry. Although there were significant withdrawals of ground water at the Griswold Street site, water-levels were not available at that site.

The general pattern of flow in 1966 and 1967 in the study area was slow movement of water toward the trough of low water levels. The lowering of water-levels along the river that accompanied ground-water withdrawals established hydraulic gradients from the Illinois River toward the pumping center.

Water Levels in Observation Wells

Water levels in the area have been measured periodically for more than 40 years. Marino and Schicht (1969) described the effects of precipitation, the Illinois River, ground-water pumpage, and artificial recharge on ground-water levels.

In the area just north of the Griswold Street site, ground-water levels have been measured continuously in the Bradley Park observation well since 1942. The water-level hydrograph for this well is shown in figure 10. A major factor affecting ground-water levels in the vicinity of this well is recharge from precipitation because the well is located in an area favorable for this (figure 11). As shown in figure 11, fine-grained materials overlying the aquifer that would impede recharge are thin or missing in this area. A comparison of ground-water levels and precipitation at Peoria Airport (figure 11) gives a general indication of how water levels are affected by precipitation at this site. For example, ground-water levels were high during the period 1981-1986 when precipitation was above the 1961-1990 average of 36.25 inches. During 1987-1989, however, precipitation was considerably below average and ground-water levels responded with a significant decline. Only a general comparison can be made between ground-water levels and precipitation since the timing and intensity of the precipitation is a factor in the quantity of precipitation reaching the water table.

No long-term continuous water-level records are available for the area north of Griswold Street in the area designated as unfavorable for recharge from precipitation. Thick deposits of fine-grained material overlie the aquifer in this area, inhibiting recharge. A limited record of water levels is available for the Buehler Home observation well (figure 12). Inspection of the Buehler well water-level hydrograph indicates that seasonal fluctuations of water levels are almost non-existent.

GROUND-WATER PUMPAGE AND ARTIFICIAL RECHARGE

Data on ground-water pumpage and artificial recharge in the study area is available from the Water Survey Water Inventory files. Ground-water withdrawals during 1990 were approximately 16 mgd. About 1.7 mgd were concentrated at the Griswold Street site, with the rest located along the Illinois River.

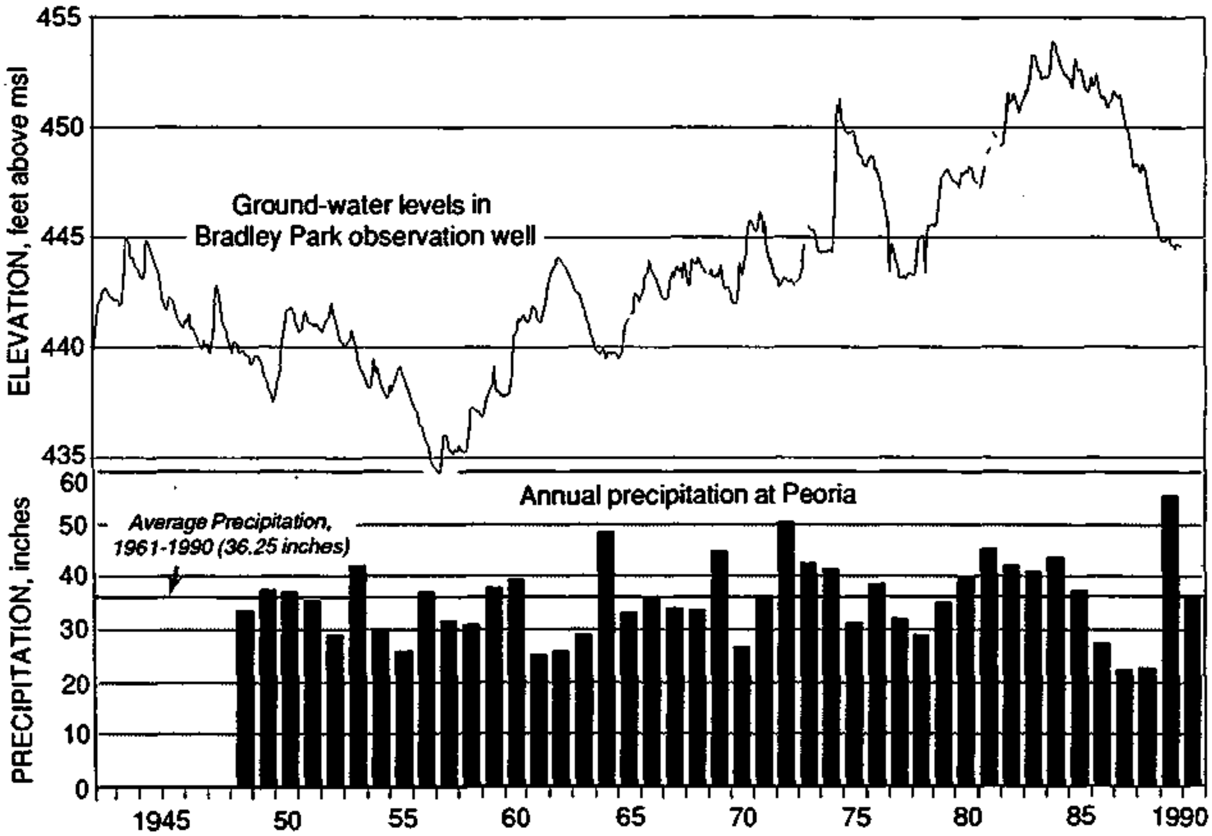

Figure 10. Ground-water levels at Bradley Park well,
and annual precipitation at Peoria Airport

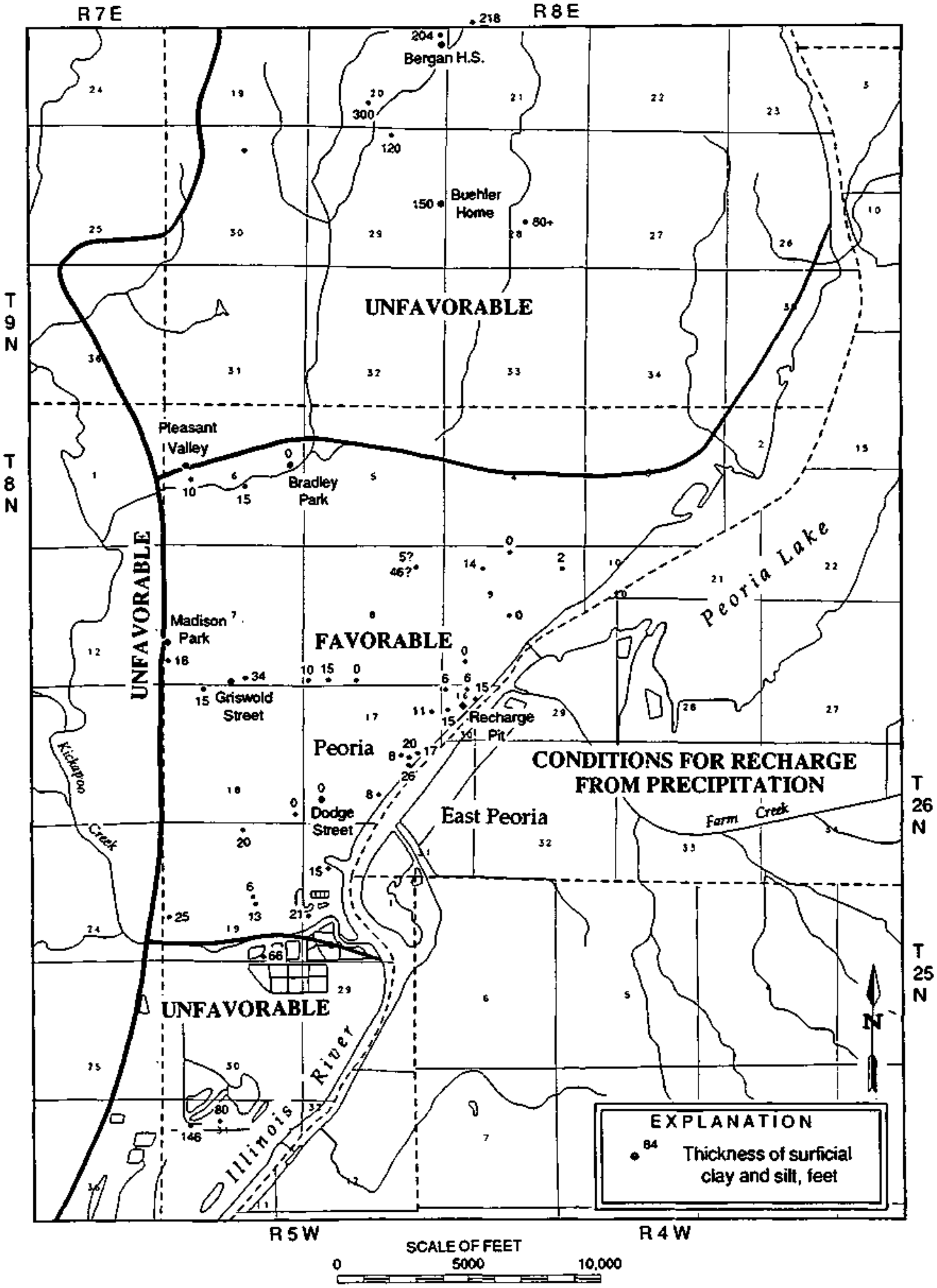

Figure 11. Conditions for recharge from precipitation

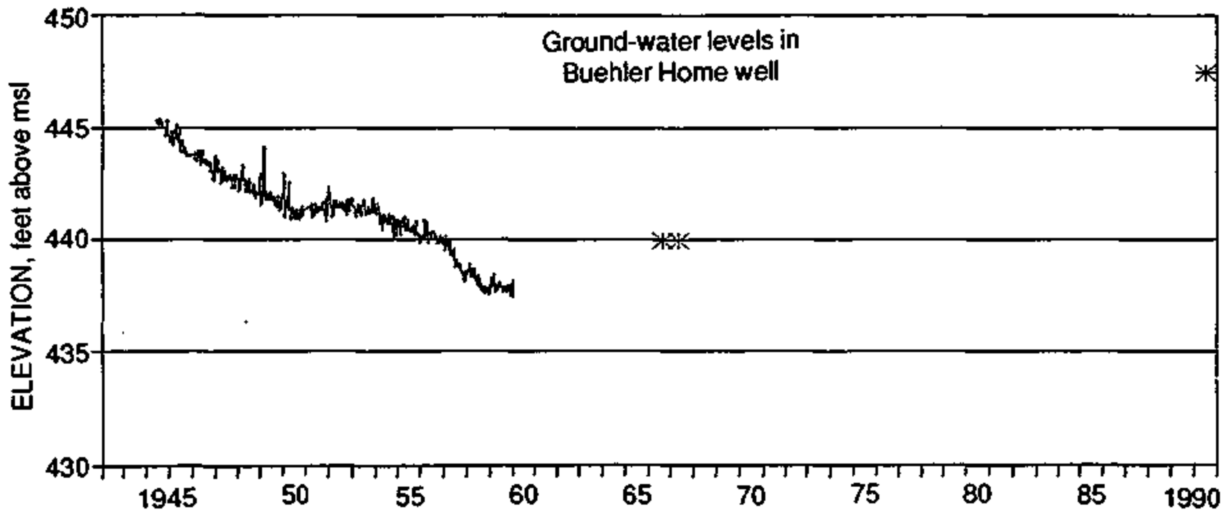

Figure 12. Ground-water levels at Buehler Home well

Artificial recharge of river water occurs at the northern edge of the trough of low water-levels. About 1.9 mgd were recharged in 1990. For information regarding the recharge pits, see Suter and Harmeson (1960).

RECHARGE FROM ILLINOIS RIVER

Because ground-water levels are below the surface of the river, water from the river can percolate through the riverbed and into the aquifer. The quantity of water that can percolate into the river is dependent upon the hydraulic connection between the river and ground water. The hydraulic connection is dependent upon several factors including the vertical hydraulic conductivity of the riverbed, the hydraulic gradient (the difference between the river's surface water elevation and the ground-water stage beneath it), and the riverbed area through which infiltration occurs. These factors are collectively referred to as the infiltration rate. In addition the temperature of the river water impacts the infiltration rate: the lower the temperature, the lower the viscosity of the water, thus the lower the infiltration rate.

Proposed increases in ground-water withdrawals in the area north of Griswold Street (see Introduction) will increase drawdown in the wells along the river and divert water that now flows to the pumping centers along the river by developing new cones of depression and reversing the ground-water gradient. Thus it is important to understand the infiltration rate. Two questions need to be answered:

1. Can the reduction in water moving down gradient from the northwest because of diversion into new cones of depression be balanced by additional water from the river?

2. Is there enough additional drawdown in wells along the river to maintain well yields?

Based on ground-water withdrawals during past years in this area-exceeding 35 mgd in the 1930s and 1940s (Marino and Schicht, 1969) compared with 16 mgd withdrawn in 1990~a high infiltration rate would be expected.

The infiltration rate can be determined through a controlled aquifer test by methods described by Walton (1962). The Dodge Street site is too far from the river to practically estimate the infiltration rate from an aquifer test. Another approach to determine the infiltration rate would be to balance ground-water withdrawals and river recharge. This would require estimates of ground-water levels beneath the river, which are not available.

To qualitatively determine the infiltration rate, it was decided to collect river-bed samples on 15 transects from Peoria Lake to Kickapoo Creek. Two samples were collected on each transect: one near the western shore and one near the river center line. A report describing sampling procedure and sampling results is given in Bogner (1992). Based on the samples collected, there is a high degree of certainty that the hydraulic conductivity of the riverbed is high.

Infiltration rates of streambeds determined from aquifer test data in Illinois, Indiana, and Ohio were summarized by Schicht (1965). Infiltration rates at a temperature of 40° F ranged from 37,500 to 1,010,000 gallons per day per acre-foot (gpd/acre ft) of average head difference between the river surface elevation and ground-water elevations beneath the river.

Maximum infiltration occurs when ground-water levels are at or below the riverbed. To determine if river infiltration could support large ground-water withdrawals, the head difference was computed for a conservative infiltration rate of 100,000 gpd/acre-ft and a pumping rate of 35 mgd. A river reach of 8000 feet and a river width of 800 feet were used in computations.

The head difference was computed to be 2.4 feet—considerably less than the average river depth indicated from sounding data provided by the U.S. Army Corps of Engineers.

Although there is strong evidence that river infiltration will support large quantities of ground-water withdrawals, well yields may be impacted adversely where lowered ground-water levels would significantly reduce the aquifer transmissivity. This would be particularly true where the aquifer thins in the area between the 30-foot saturated thickness contour north of Dodge Street (figure 6).

RECHARGE FROM PRECIPITATION

Ground-water recharge from precipitation is described in various Water Survey reports, including Walton (1965). In general, recharge is greatest in areas where the water table is close to land surface and the materials overlying the aquifer are permeable. The study area was delineated into favorable and unfavorable areas for recharge (figure 11). In the favorable area, fine-grained materials overlying the aquifer are thin or missing. In the unfavorable area, fine-grained materials overlying the aquifer are thick. In the northern part of the study area, the water table is a few hundred feet below land surface. In the area west of Kickapoo Creek, aquifers are essentially missing.

Recharge directly from precipitation can be determined by flow-net analyses of the water-table maps for October 1966 and April 1967. The rate of recharge directly from precipitation can be estimated on the basis of the difference in discharge through successive flow cross-sections with the following equation (Walton, 1962):

$$R = [(Q_2-Q_1) \pm \Delta h_t \, SA_1(2.1x10^8)] \, / \, A_1 \qquad (2)$$

where:

R = rate of recharge, gpd per square mile (gpd/sq mi)
Q_2-Q_1 = difference in discharge or water through successive flow cross sections, gpd
Δh_t = average rate of water-level decline or rise within area between successive flow cross sections, feet per day (fpd)
A_1 = surface area between successive flow cross sections, sq mi
S = coefficient, fraction

The " + " sign is used when there is a water-level rise, and the "-" sign is used when there is a water-level decline.

Flow through cross sections, Q, can be determined by using the following modified form of the Darcy equation (see Ferris, 1959):

$$Q = TIL \tag{3}$$

where:

Q = discharge through flow cross section, gpd
T = transmissivity, gpd/ft
I = hydraulic gradient, ft/mi
L = width of flow cross section, mi

Recharge from precipitation was estimated for the period October 1966-April 1967. Flow lines were drawn at right angles to the water-table contour map for April 1967 to define flow cross sections. Flow was computed through cross sections A-A' and B-B'. Transmissivity was estimated by multiplying the saturated thickness shown in figure 6 along the cross section by an estimated hydraulic conductivity interpreted from data given in figure 6. The hydraulic gradient and width of flow cross sections were determined from the water-table contour map. The average rate of water-level decline within the area defined by the flow cross sections was estimated by subtracting the 1967 map from the 1966 map (figure 13) and dividing the average decline by the number of days from October-April. A coefficient of storage of 0.20 was used in computations.

Accounting for withdrawals from the Griswold Street well field, it was estimated that the average recharge rate for the period October 1966-April 1967 was 400,000 gpd/sq mi. Although this agrees with recharge rates for similar environments in Illinois (Walton, 1965), further estimates would be required for different periods with different rainfall amounts.

Estimates of recharge from the October 1966 water-table map were not made since accurate water-level changes to compute changes in storage were not available.

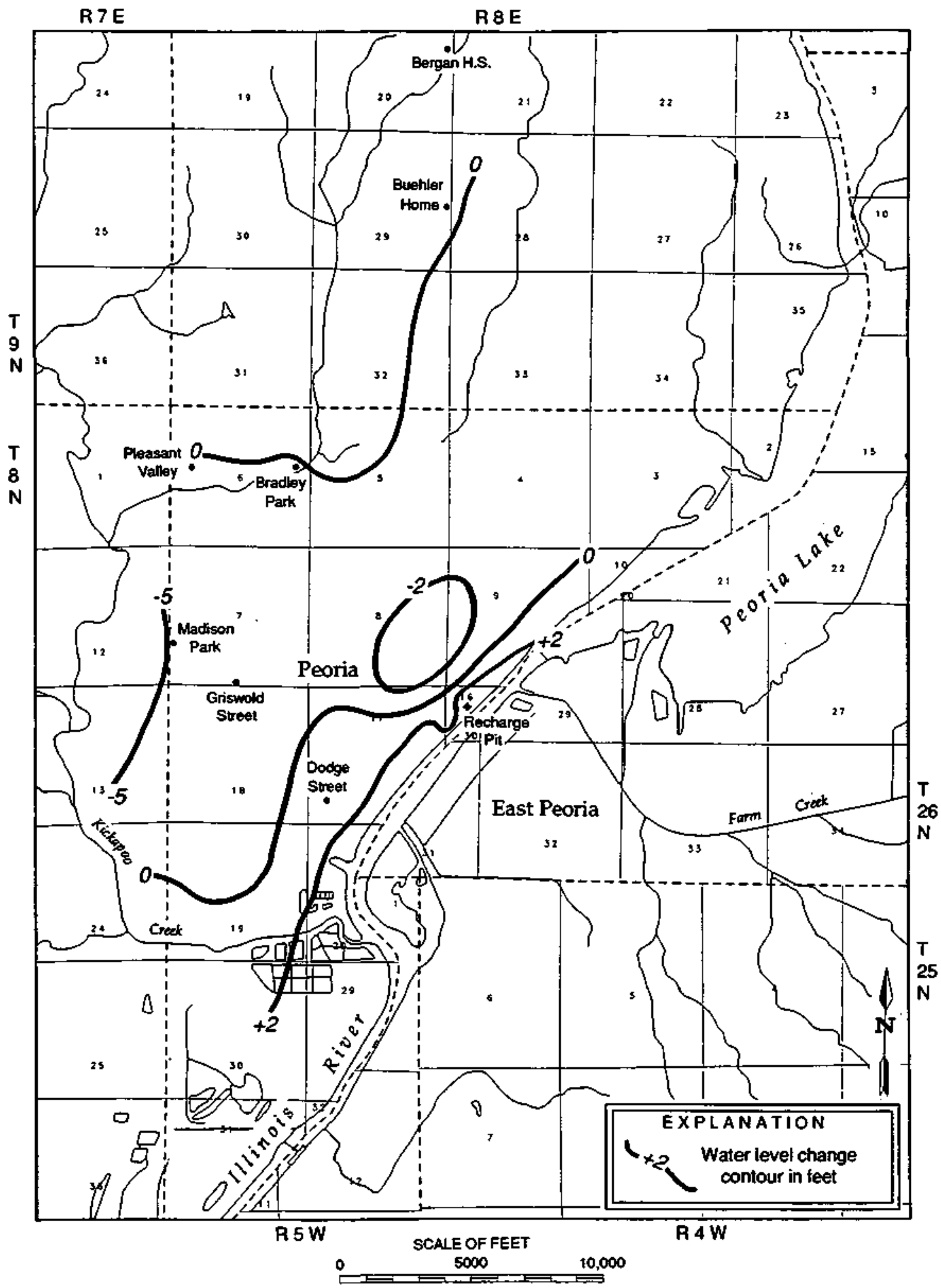

Figure 13. Ground-water level changes, August 1966-April 1967

In summary, although the estimated recharge from precipitation is high, the area designated as favorable for recharge is not large—probably around 6 sq mi. Only about 2.4 mgd would be available from recharge from precipitation.

Attempts to estimate recharge north of cross sections A-A' from the water-table maps were not successful. It was not possible to determine the recharge area and average water-level changes from available data. Based on flow net analysis, significant quantities of water move through cross section A-A'—6.2 mgd in April 1967 and 6.1 mgd in October 1966. Water-table contour maps from Horberg et al. (1950) were used to estimate flow through cross sections at the same general location as those for the 1966 and 1967 maps. Flows were estimated to be 9, 6.4, 8.2, 8.1, and 8.2 mgd from analysis of maps for August 1941, May 1943, and September 1942, 1943, and 1945, respectively.

ESTIMATED WELL-FIELD YIELDS

To estimate well-field yields, it was decided to approximate field conditions with a mathematical aquifer model and solve the equations that describe ground-water conditions analytically. Because of the paucity of geologic and hydrologic data in the area north of Griswold Street simple models were constructed. A numerical model involving a computer would normally be used for more complex data sets.

Ground-water conditions in the area north of Griswold Street were simulated by a model aquifer with straight-line boundaries and an effective aquifer width, length, and thickness. Since water-table conditions exist in the aquifer, aquifer transmissivity and storage coefficient were the only aquifer hydraulic properties needed for computations. Aquifer transmissivity was determined by multiplying a hydraulic conductivity of 6000 gpd/ft^2-determined from the Griswold Street site aquifer test and used regionally to determine a reasonable value of ground-water recharge from flow-net analysis-by aquifer thickness. A storage coefficient of 0.20 was used. Based on the aquifer test, this seems reasonable, particularly for long pumping periods.

Results from two aquifer models are presented. Each model has a barrier boundary located along the north-south township line between R7E and R8E. As shown in figure 2, ground-water conditions are unfavorable west of this line. A barrier boundary also exists in the eastern part of the area where ground-water conditions are unfavorable in a large area in the northeastern part of the study area west of the Illinois River (figure 2). Because of the ground-water divide associated with the trough of low water levels along the river from roughly Peoria Lake to Kickapoo Creek, a no-flow boundary is formed. Collectively these conditions would represent a barrier boundary on the east.

The aquifer models studied were strip aquifers of 8000 and 4000 feet bounded by barriers on the west and east and extending open ended to the north and south. The 8000-foot-wide aquifer represents conditions just north of the Griswold Street site to the township line between T9N and T8N. Since the bedrock valley narrows to the north, an aquifer width of 4000 feet was selected for the area north of the township line (see figure 14).

Since the aquifer narrows in the southern part of the study area, a wedge-shaped aquifer was also considered. Results from the analysis described below for the wedge-shaped aquifer and for the 8000-foot strip aquifer were not much different.

Yields of different well-field configurations for varying pumping periods were determined for each aquifer model. Well fields consisted of multiple wells spaced 200 feet apart, located in the center of the valley, and parallel to the aquifer boundaries. The 200-foot spacing was arbitrary. Long-term drawdown was computed for the middle well for each well-field configuration. When the drawdown approached a critical level—one-half the saturated thickness based on low water-levels experienced during the 1954-1958 drought-it was assumed that the well-field yield had been reached. Initial computations were based on an individual well yield of 1000 gpm, which were increased or decreased depending upon additional drawdown available or computed drawdown exceeding the critical water level. The number of wells was also increased or decreased for some cases.

The components of drawdown included aquifer loss due to the laminar flow of water through the aquifer towards the well. Aquifer loss is computed based on the aquifer properties, the effective well diameter (one foot was used), and the pumping period, using the Theis equation described by Walton (1962). To compute the drawdown component from interference from other production wells in the well-field configuration, the Theis equation was also used. Increases in drawdown due to the effects of the aquifer boundaries were computed based on the image well theory described by Ferris (1959).

Another component of drawdown is well loss due to the turbulent flow of water through the screen and inside the casing to the pump intake. Well loss can be approximated by the following equation (Jacob, 1946):

$$sw = CQ^2 \tag{4}$$

where sw = well loss, feet
C = well-loss constant, sec^2/ft^5
Q = discharge, cubic feet per second (cfs)

A well-loss constant of one was used in computations. For a pumping rate of 1000 gpm, drawdown due to well loss is about 4.9 feet. For an aquifer with a high permeability like the Sankoty sand, a well-loss constant of one is considered high.

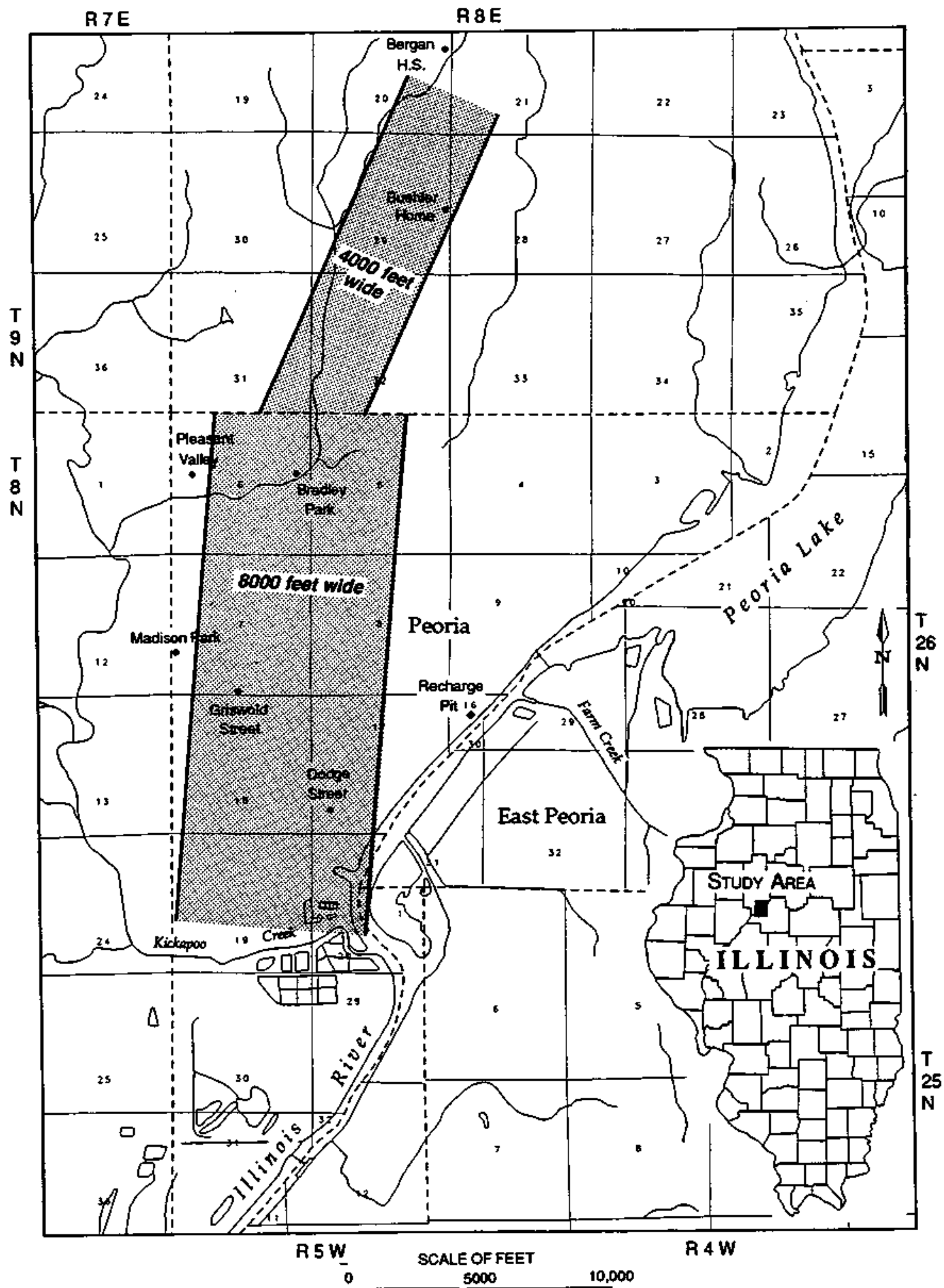

Figure 14. Location of strip aquifer models

Since well loss does not reduce the saturated thickness, it was not considered in well-yield computations unless the addition of the well-loss drawdown component drew water levels below the top of the screen. A screen length of 30 feet was considered adequate for a pumping rate of 1000 gpm. As stated earlier, the available drawdown was assumed to be one-half of the saturated aquifer thickness estimated for the 1954-1958 drought. For the area south of the township line, the saturated thickness was estimated to be 80 feet, thus the available drawdown was 40 feet. For the area north of the township line, the saturated thickness was estimated to be 90 feet, thus the available drawdown was 45 feet. Under water-table conditions, drawdown must be adjusted to account for the decrease in aquifer transmissivity so that equations derived for artesian conditions that assume constant transmissivity can be used. The following equation from Jacob (1944) was used to adjust the available drawdown:

$$s' = s - (s^2/2m) \tag{5}$$

where:

s' = drawdown that would occur in an equivalent artesian aquifer, feet (adjusted drawdown)
s = observed drawdown, feet (available drawdown)
m = initial saturated thickness, feet

Adjusted drawdown south of the township line computed using equation 5 is 30 feet; north of the line, 33.8 feet.

Well-field yields without recharge for pumping periods of 360 days, 1460 days, and 10,000 days for the each strip aquifer are given below.

Pumping Period	4000 ft. Strip *Yield(mgd)*	8000 ft. Strip *Yield (mgd)*
360 days	8.3	11.9
1,460 days	4.4	6.5
10,000 days	1.9	4.8

For an areally extensive shallow aquifer under water-table conditions with a high recharge rate, a pumping period of 180 days is usually selected to determine well yield. It is assumed that 180 days would be the longest period without recharge. For the study area, however, only a small part of the area can be classified as having a high recharge rate, and the recharge rate per square mile for most of the area is not known. By looking at yields for different pumping periods, however, upper and lower limits of well yields can be inferred. For example, the yield for a pumping period of 10,000 days would represent the lower limit: 1.9 mgd for a 4000 foot strip aquifer and 4.8 mgd for an 8000-foot strip. The yields for a pumping period of 1460 days would probably represent the upper limit yield~4.4 mgd for the 4000-foot strip aquifer, 6.5 mgd for the 8000-foot strip aquifer. These yields are in the same order of magnitude of the quantity of water moving down gradient estimated from flow-net analysis and may represent the long-term yield if this quantity of water can be captured.

SUMMARY

Estimates of hydraulic properties from an aquifer test at Griswold Street and from available information in State Water Survey files indicates high hydraulic conductivity for the aquifer north of Griswold Street. Coupled with an aquifer saturated thickness in excess of 80 feet, the aquifer is highly transmissive. A comparison of ground-water levels and the top of the aquifer indicates the aquifer is under water-table conditions. Recharge locally from precipitation is limited because overlying materials in a large part of the area is overlain by fine-grained materials impeding the vertical movement of water. It was estimated that 6 mgd was moving down gradient toward the river from the area north of Griswold Street.

Because of the high infiltration rate of the riverbed, little impact on existing well fields along the river is expected by developing well fields north of Griswold Street.

Two strip aquifer models were studied to estimate well-field yields in the study area: a strip aquifer 8000 feet wide representing conditions in the south and a strip aquifer 4000 feet wide representing conditions in the north. Calculations indicate that well-field yields from 1.9 mgd to 4.4 mgd may be reasonable for the 4000-foot aquifer vs. yields of 4.8 mgd to 6.5 mgd for the 8000-foot aquifer.

Because of the paucity of data in the area of interest, it is recommended that test wells be drilled and a minimum of two long-term aquifer tests be conducted to verify the assumptions made in this report.

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