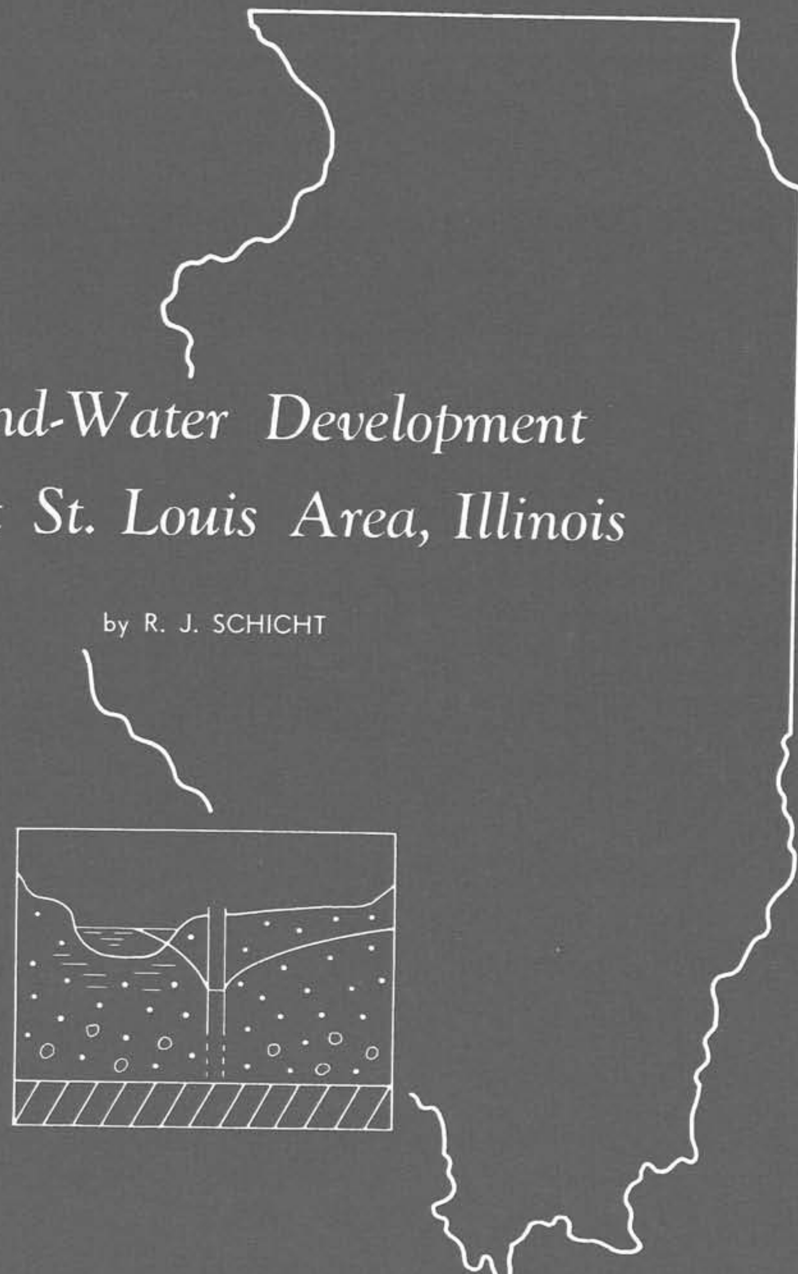
REPORT OF INVESTIGATION 51

STATE OF ILLINOIS

DEPARTMENT OF REGISTRATION AND EDUCATION

Ground-Water Development in East St. Louis Area, Illinois

by R. J. SCHICHT



ILLINOIS STATE WATER SURVEY

URBANA

1965

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STATE WATER SURVEY DIVISION
William C. Ackermann, Chief

URBANA

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Ground Water Development in East St *Louis Area, Illinois*

by R. J. Schicht

ABSTRACT

The East St. Louis area extends along the valley lowlands of the Mississippi River in southwestern Illinois and covers about 175 square miles. Large supplies of ground water chiefly for industrial development are withdrawn from permeable sand and gravel in unconsolidated valley fill in the area. The valley fill composed of recent alluvium and glacial valley-train material has an average thickness of 120 feet. The coefficient of permeability of the valley fill commonly exceeds 2000 gallons per day per square foot (gpd/sq ft); the coefficient of transmissibility ranges from 50,000 to 300,000 gallons per day per foot (gpd/ft).. The long-term coefficient of storage of the valley fill is in the water-table range.

Pumpage from wells increased from 2.1 million gallons per day (mgd) in 1900 to 110.0 mgd in 1956 and was 105.0 mgd in 1962. Of the 1962 total pumpage, 91.1 percent was industrial; 6.4 percent was for public water supplies; 2.3 percent was for domestic uses; and 0.2 percent was for irrigation. Pumpage is concentrated in five major pumping centers: the Alton, Wood River, Granite City, National City, and Monsanto areas.

As the result of heavy pumping, water levels declined about 50 feet in the Monsanto area, 40 feet in the Wood River area, 20 feet in the Alton area, 15 feet in the National City area, and 10 feet in the Granite City area from 1900 to 1962. From 1957 to 1961 water levels in the Granite City area recovered about 50 feet where pumpage decreased from 31.6 to 8.0 mgd. Pumping of wells and draining of lowlands have considerably reduced ground-water discharge to the Mississippi River, but have not reversed at all places the natural slope of the water table toward that stream. In the vicinity of some pumping centers, the water table has been lowered below the river and other streams, and induced infiltration of surface water is occurring.

Recharge directly from precipitation based on flow-net analysis of piezometric maps varies from 299,000 to 475,000 gallons per day per square mile (gpd/sq mi). Subsurface flow of water from bluffs bordering the area into the aquifer averages about 329,000 gallons per day per mile (gpd/mi) of bluff. Infiltration rates of the Mississippi River bed according to the results of aquifer tests range from 344,000 to 37,500 gallons per day per acre per foot (gpd/acre/ft). Approximately 50 percent of the total pumpage in 1962 was derived from induced infiltration of surface water.

An electric analog computer consisting of an analog model and excitation-response apparatus was constructed for the East St. Louis area so that the consequences of further development of the aquifer could be forecast. The accuracy and reliability of the analog computer were established by comparing actual water-level data with piezometric surface maps prepared with the analog computer.

The analog computer was used to estimate the practical sustained yields of existing pumping centers. Assuming that critical water-levels will occur when pumping water levels are below tops of screens and/or more than one-half of the aquifer is dewatered, the practical sustained yields of all existing pumping centers exceed present withdrawals. Pumpage in the Monsanto area probably will exceed the practical sustained yield by 1966; the practical sustained yield of other pumping centers probably will not be reached until after 1980. The analog computer was also used to describe the effects of a selected scheme of development and to determine the potential yield of the aquifer under an assumed pumping condition.

INTRODUCTION

The East St. Louis area has been one of the most favorable ground-water areas in Illinois. It is underlain at depths of 170 feet or less by sand and gravel aquifers that have been prolific sources of water for more than 50 years. The available ground-water resources have promoted industrial expansion of the area and also facilitated urban growth.

The tremendous industrial growth in the East St. Louis area has brought about local problems of water supply. Heavy concentrated pumpage in the Granite City area caused water levels to decline to critical stages during an extended dry period (1952-1956). As a result, an industry was forced to abandon its well field and construct a pipe line to the Mississippi River for its water supply.

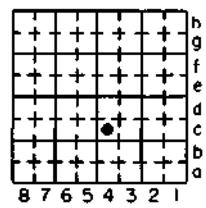
This report presents a quantitative evaluation of the ground-water resources of the East St. Louis area and is based on all data on file at the State Water Survey and in other published reports. The geohydrologic characteristics of the ground-water reservoir are given along with an analysis of past, present, and probable future development of ground-water resources. Basic geologic, hydrologic, and chemical data, maps, and interpretations applicable to local problems and to regional and long-range interpretations are presented to provide a basis for water-resource planning and a guide to the development and conservation of ground water in the area.

Although this report summarizes present-day knowledge of ground-water conditions in the East St. Louis area, it must be considered a preliminary report in the sense that it is part of a continuing study of the East St. Louis ground-water resources. The conclusions and interpretations in this report may be modified and expanded from time to time as more data are obtained.

The State Water Survey accelerated its program of ground-water investigation in the East St. Louis area in 1941 after alarming water-level recessions were observed by local industries especially at Granite City. Water-level data for the period 1941 through 1951 were summarized and the ground-water withdrawals in 1951 were discussed by Bruin and Smith (1953). The ground-water geology of the area has been described by the State Geological Survey (Bergstrom and Walker, 1956). Ground-water levels and pumpage in the area during the period 1890 through 1961 were discussed by Schicht and Jones (1962). Other reports pertaining to the ground-water resources of the East St. Louis area are listed in the references at the end of this report.

Well-Numbering System

The well-numbering system used in this report is based on the location of the well, and uses the township, range, and section for identification. The well number consists of five parts: county abbreviation, township, range, section, and coordinate within the section. Sections are divided into rows of ⅛-mile squares. Each ⅛-mile square contains 10 acres and corresponds to a quarter of a quarter of a quarter section. A normal section of 1 square mile contains 8 rows of ⅛-mile squares; an odd-sized section contains more or fewer rows. Rows are numbered from east to west and lettered from south to north as shown in the diagram.



St. Clair County

T2N, R10W

Section 23

The number of the well shown is: STC 2N10W-23.4c. Where there is more than one well in a 10-acre square they are identified by arabic numbers after the lower case letter in the well number.

There are parts of the East St. Louis area where section lines have not been surveyed. For convenience in locating observation wells, normal section lines were assumed to exist in areas not surveyed.

The abbreviations for counties discussed in this report are:

Madison MAD Monroe MON St. Clair STC

In the listing of wells owned by municipalities, the place-name is followed by V, T, or C in parentheses to indicate whether it is a village, town, or city, except where the word City is part of the place-name.

Acknowledgments

This study was made under the general supervision of William C. Ackermann, Chief of the Illinois State Water Survey, and Harman F. Smith, Head of the Engineering Section. William C. Walton, formerly in charge of ground-water research in the Engineering Section, aided in interpretation of hydrologic data and reviewed and criticized the final manuscript. E. G. Jones, field engineer, collected much of the water-level, pumpage, and specific-capacity data, and aided indirectly in preparing this report.

Many former and present members of the State Water Survey assisted in the collection of data, wrote earlier special reports which have been used as reference material, or contributed other indirect assistance to the writer. Grateful acknowledgment is made, therefore, to the following engineers: G. E. Reitz, Jr., R. R. Russell, Sandor

Csallany, W. H. Walker, T. A. Prickett, Jack Bruin, J. P. Dorr, R. E, Aten, H. G. Rose, and O. E. Michaels. J. W. Brother prepared the illustrations.

This report would have been impossible without the generous cooperation of officials of municipalities and industries, consulting engineers, water well contractors, and irrigation and domestic well owners who provided information on wells, water levels, and pumpage.

GEOGRAPHY

The East St. Louis area, known locally as the "American Bottom," is in southwestern Illinois and includes portions of Madison, St. Clair, and Monroe Counties. It encompasses the major cities of East St. Louis, Granite City, and Wood River, and extends along the valley lowlands of the Mississippi River from Alton south beyond Cahokia as shown in figure 1. The area covers about 175 square miles and is approximately 30 miles long and 11 miles wide at the widest point. Included is an area south of Prairie Du Pont Floodway containing Dupo and East Carondelet.

Topography and Drainage

Most of the East St. Louis area lies in the Till Plains Section of the Central Lowland Physiographic Province (Fenneman, 1914; and Leighton, Ekblaw, and Horberg, 1948). The extreme southwestern part of St. Clair County and the western part of Monroe County lie in the Salem Plateau Section.

Much of the area lies in the flood plain of the Mississippi River; the topography consists mostly of nearly level bottomland. Along the river channel the flood plain slopes from an average elevation of 415 feet near Alton to 405 feet near Dupo. In the northern part of the area, terraces stand above the flood plain. A terrace that extends from East Alton to Roxana is at an elevation of 440 to 450 feet or about 25 to 35 feet above the flood plain. North of Horseshoe Lake much of the area is above the flood plain at elevations ranging from 420 to 435 feet.

The elevation of the land surface near the eastern bluff is 30 to 50 feet higher than the general elevation of the valley bottom. The bluff, along the eastern edge of the valley bottom, rises abruptly 150 to 200 feet above the lowland. The topography immediately east of the bluff consists of rather rugged uplands.

Monks Mound, which rises 85 feet above the flood plain, is the largest of a group of mounds just east of Fairmont City. The shape of the mounds indicates an artificial origin; however, some of them may be remnants of an earlier higher flood plain (Bergstrom and Walker, 1956).

Drainage is normally toward the Mississippi River and its tributaries; Wood River, Cahokia Diversion Channel, Cahokia Canal, and Prairie Du Pont Floodway. The tributaries drain much of the flood plain and the uplands bordering the flood plain. The valley bottom is protected from flooding by a system of levees that fronts the Mississippi River and the Chain of Rocks Canal and flanks the main tributaries. However, flooding does occur in parts of the area because drainage facilities which convey and store major flood runoff from the flood plain and the upland watersheds are inadequate (Illinois Division of Waterways, 1950). The southeastern part of the area near Cahokia, Centreville, and Grand Marais State Park is particularly affected by flooding. Figure 1 shows areas flooded after heavy rainfall on May 5, 6, 7, 8, and 19, 1961.

Prior to settlement of the East St. Louis area, floodwaters from the Mississippi River and its tributary streams, Wood River, Cahokia Creek, Canteen Creek, Schoenberger Creek, and Prairie Du Pont Creek, frequently inundated large sections of the valley bottom. The water table was near the surface and poorly drained areas were widespread. Development of the area led to a system of drainage ditches, levees, canals, and channels. According to Bruin and Smith (1953) the natural lake area between 1907 and 1950 was reduced by more than 40 percent and 40 miles of improved drainage ditches were constructed during the same period; this had an effect of lowering ground-water levels by an estimated 2 to 12 feet.

The present drainage system is shown in figure 2. Much of the flow from the upland areas east of the bluff is diverted into four channels that traverse or flank the valley bottom, thence flow to the Mississippi River. The four channels are Wood River, Cahokia Diversion Channel, Prairie Du Pont Floodway, and Canal No. 1.

Wood River carries flow from the confluence of the East and West Forks of Wood River north of East Alton south-southwest to the Mississippi River. Much of the channel of Wood River is leveed.

The Cahokia Diversion Channel intercepts flow from Cahokia and Indian Creeks in sec 7, T4N, R8W, Madison County, and diverts it westward to the Mississippi River.

Prairie Du Pont Floodway is a relocated and improved channel of Prairie Du Pont Creek and conveys runoff from Canal No. 1 and Prairie Du Pont Creek near Stolle westward to the Mississippi River. In addition it carries flow from the valley bottom drainage area north of Prairie Du Pont Creek and from Harding Ditch.

Canal No. 1 intercepts flow from several small upland

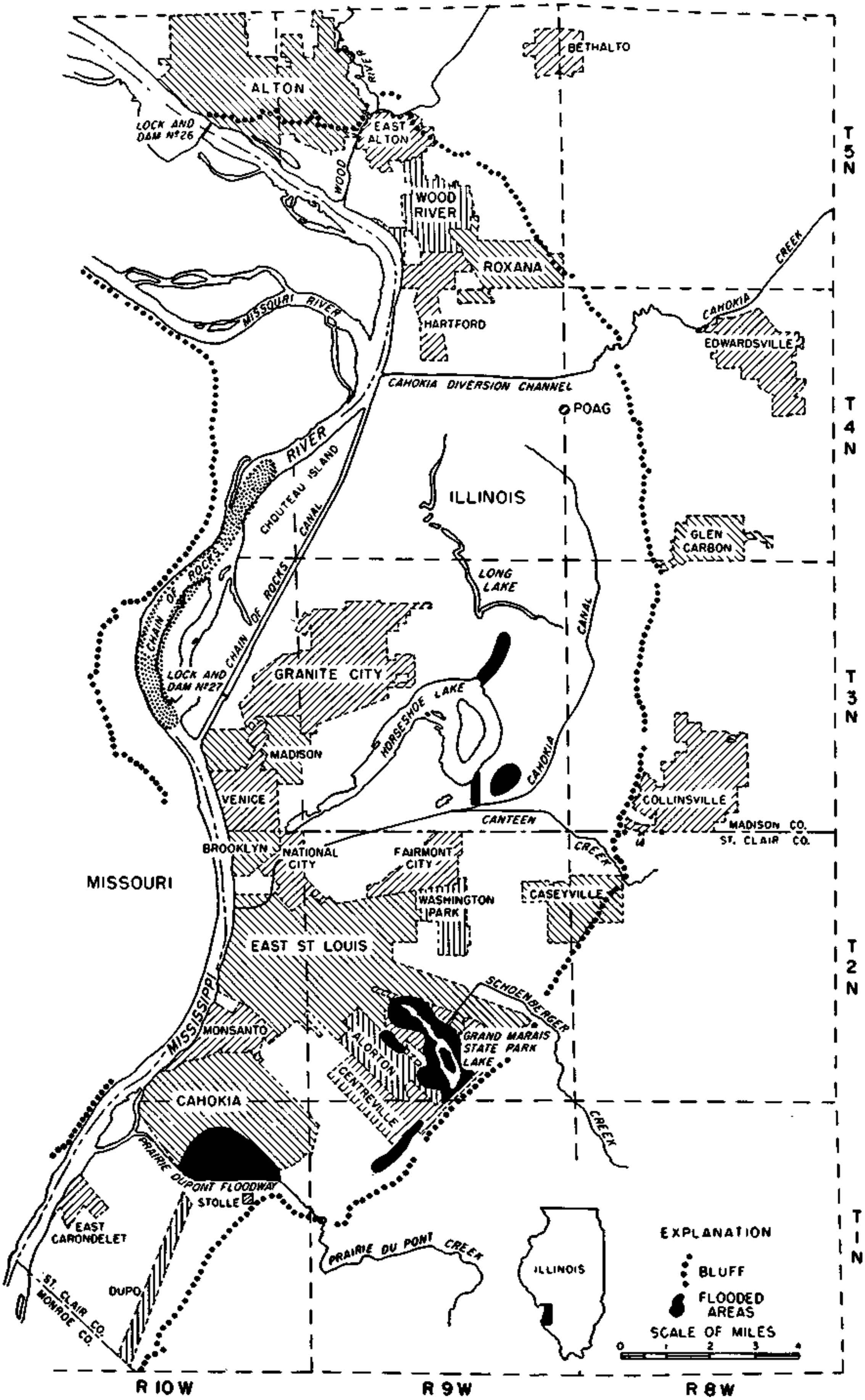


Figure I. Location of Easf St. Louis area showing areas flooded during May 1961

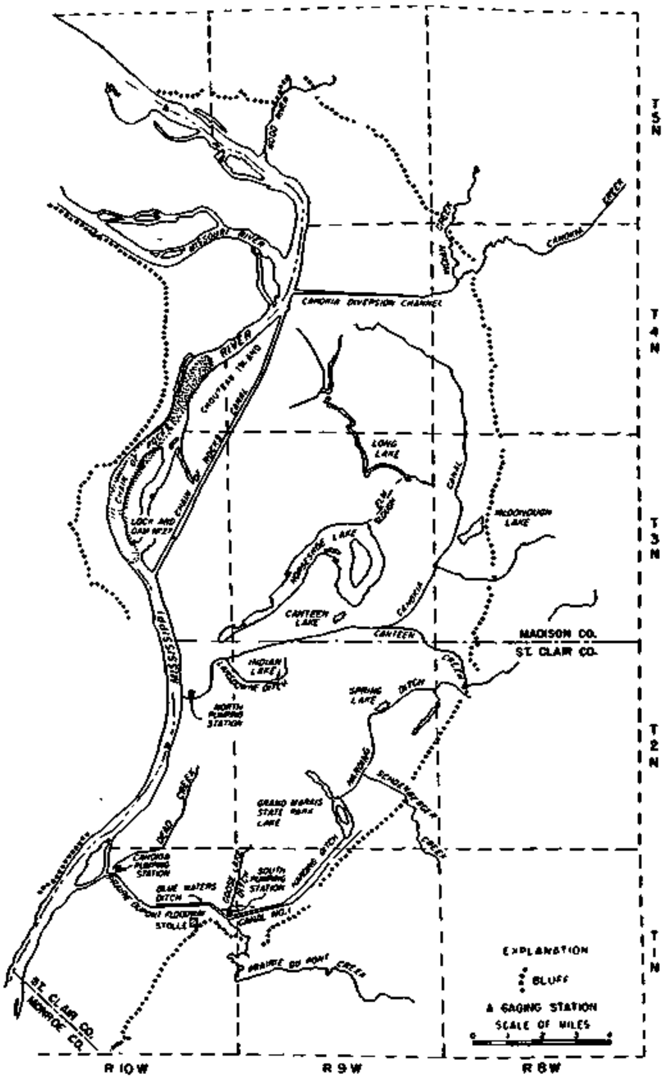


Figure 2. Drainage system and locations of stream-gaging stations

streams between Prairie Du Pont Floodway and the southern edge of Centreville and discharges the flow into the floodway.

The valley bottom is drained through Indian Creek, several small ditches north of the Cahokia Diversion Channel, Long Lake, Cahokia Canal, Lansdowne Ditch, Harding Ditch, the Blue Waters-Goose Lake Ditch system, and the Dead Creek-Cahokia drainage system. In addition, closed storm sewer systems drain much of the urban areas within the valley bottom.

Long Lake drains much of the area to the north of Horseshoe Lake. During periods of overflow it drains into Horseshoe Lake through Elm Slough.

The Cahokia Canal consists of an improved and leveed channel along the old course of Cahokia Creek. The canal begins in sec 14, T4N, R9W, flows southeasterly to sec 31, T4N, R8W, and then southwesterly around the southern end of Horseshoe Lake, through National City and the northwestern corner of East St. Louis to the Mississippi River. Discharge to the Mississippi River is by gravity flow during periods when the stage of the Mississippi River is low; when the river is at flood stage, water is pumped from Cahokia Canal to the river at the North Pumping Station. Runoff in excess of the storage capacity of Cahokia Canal or of the pumping station is stored temporarily in Indian and Horseshoe Lakes until it can be discharged into the river. The principal tributaries to the canal are Long Lake (by way of Horseshoe Lake), Lansdowne Ditch, Canteen Creek, and several small streams to the east.

Harding Ditch begins at Caseyville and flows southwesterly to Park Lake in Grand Marais State Park, which acts as a regulating reservoir, thence to Prairie Du Pont Floodway. Discharge to the Mississippi River is either by gravity flow or pumps at the South Pumping Station.

The Dead Creek-Cahokia drainage system drains most of the Monsanto and Cahokia areas. The outlet of the system is to the Prairie Du Pont Floodway at the Cahokia Pumping Station.

The Blue Waters-Goose Lake Ditch system drains the area east of Cahokia, southwest of Centreville, and northwest of Harding Ditch and Prairie Du Pont Floodway. Goose Lake Ditch discharges into Blue Waters Ditch near Harding Ditch. Blue Waters Ditch can discharge into Prairie Du Pont Floodway or Harding Ditch when the floodway is at low stage; when the stage of the floodway is high, runoff is stored temporarily in Blue Waters Ditch and adjacent low areas.

Numerous lakes were formed in the flood plain by the meandering of the Mississippi River. Many of the lakes have been drained and the original lake bottoms are now being cultivated. Table 1 gives data on the more important lakes now in existence.

Table I. Areas and Water-Surface Elevations of Lakes*

Lake	Approximate surface area when full (acres)	Approximate water surface elevation when full (ft above msl)
McDonough	75	404
Long	85	415
Horseshoe	2500	402
Canteen	105	403
Park	990	405.5
Spring	10	410

**From Illinois Division of Waterways (1950)*

The average gradient of the Mississippi River from Alton to Dupo is about 6 inches per mile. The average gradients of Wood River, Cahokia Diversion Channel, Cahokia Canal, and Prairie Du Pont Floodway are given in table 2. The gradients of streams draining the uplands east of the bluff are much greater, ranging from about 6 feet per mile for Cahokia Creek to about 30 feet per mile for Schoenberger Creek.

The Chain of Rocks Canal was constructed to bypass the reach of the Mississippi River known as Chain of

Table 2. Average Gradients of Tributaries to Mississippi River

Tributary	Gradient (ft per mi)
Wood River	5
Cahokia Diversion Channel	2
Cahokia Canal	1.7
Prairie Du Pont Floodway	1.6

Rocks Reach (figure 1), which was difficult to navigate because the velocity of the river sometimes exceeded 12 feet per second. In addition, the navigable depth in Chain of Rocks Reach was reduced to 5.5 feet when the stage of the river was low. The canal, which was opened to river traffic on February 7, 1953, is 300 feet wide at the bottom and about 550 feet wide at the top, and has a total length of 8.4 miles. In the vicinity of Granite City the canal was widened, for a distance of 6750 feet, to a bottom width of 700 feet. A depth of slightly less than 15 feet at minimum low water stage is provided at the lower end of the canal downstream from Lock No. 27. At the upstream entrance of the canal, a minimum depth of 10.4 feet is provided.

The locations of stream gages in the East St. Louis area are shown in figure 2. The U. S. Geological Survey measures the discharge of the Mississippi River at Alton, and at St. Louis. The discharges of Indian Creek near Wanda and Canteen Creek near Caseyville are also measured by the U.S. Geological Survey, and the discharge of Long Lake near Stallings was measured from December 1938 to December 1949. Extremes and average discharges of streams are given in table 3.

During the 1952 to 1956 drought the average discharge of Indian and Canteen Creeks was reduced considerably. The average daily discharge was 6.23 cubic feet per second (cfs) in Indian Creek at Wanda and 5.81 cfs in Canteen Creek near Caseyville. There was no flow in these streams during many days in the summer and fall months of the drought period.

The flow of the Mississippi River in the East St. Louis area is affected by many reservoirs and navigation dams in the upper Mississippi River Basin and by many reservoirs and diversions for irrigation in the Missouri River Basin. Along the reach of the Mississippi River from Alton to Dupo the flow of the river is affected by Lock and Dam No. 26 at Alton, the Chain of Rocks Canal, and Lock and Dam No. 27 at Granite City on the canal. There is a low water dam on the Mississippi River south of the northern end of Chain of Rocks Canal.

Floodwaters from the Missouri River enter the Mississippi River above the gaging station at Alton when levees along the Missouri River are overtopped. Overflow from the Missouri River was estimated by the U. S. Geological Survey and is given in table 4.

Mississippi River stages in the East St. Louis area are measured daily at Lock and Dam No. 26 at Alton; at Hartford, Illinois; Chain of Rocks, Missouri; Lock No. 27 at Granite City, Illinois; Bissell Point, Missouri; St. Louis, Missouri; and the Engineer Depot, Missouri. The elevation of the maximum river stage at Alton was estimated to be 432.10 feet and occurred in June 1844; the elevation of the minimum stage was 390.50 feet on January 27, 1954, The elevation of the maximum river stage

Table 3. Streamflow Records

Stream	Drainage area (sq mi)	Location of gaging station	Maximum discharge (cfs) and date of occurrence	Minimum discharge (cfs) and date of occurrence	Average discharge (cfs) and length of record	Average discharge (cfs) during 1952-1956 drought
Mississippi River	171,500	At Alton, mile 202.7 upstream from Ohio River	437,000 May 24, 1943	7,960 November 7, 1948	93,130 33 years	
Mississippi River	701,000	At St. Louis mile 180.0 upstream from Ohio River	1,300,000* **June 1844**	18,000 December 21-23, 1863	174,700 99 years	
Indian Creek	37	At Wanda, SE ¼ NW ¼ sec 31, T5N, R8W	9,340 August 15, **1946**	0†	24.8 21 years	6.23
Long Lake	5	At Stallings, NW ¼ NW ¼ sec, 12, T3N, R9W	121 August 18, 1946	0†	2.31 12 years	
Canteen Creek	23	At Caseyville, N ½ NW ¼ sec 8, T2N, R8W[r]	10,200 June 15, 1957	0†	17.5 22 years	5.81

***Estimated**
†Zero flow occurred during several periods in drought years

at St. Louis was 421.26 feet and occurred on June 27, 1844; the elevation of the minimum stage was 373.33 feet on January 16, 1940.

Table 4. Overflow from Missouri River

Period	Overflow for period (ac-ft)	Maximum: Date of occurrence	Maximum: Overflow (cfs)
May 21-June 4, 1943	1,075,000	May 24, 1943	90,000
April 29-May 13, 1944	891,000	April 30, 1944	90,000
June 29-July 19, 1947	687,000	July 2, 1947	65,000
July 5-31, 1951	2,534,000	July 20, 1951	110,000

Climate

The East St. Louis area lies in the north temperate zone. Its climate is characterized by warm summers and moderately cold winters.

According to the Atlas of Illinois Resources, Section 1 (1958), the average annual precipitation in the East St. Louis area is about 38 inches. Precipitation has been measured at St. Louis since 1837. Graphs of annual and mean monthly precipitation collected by the U. S. Weather Bureau at Lambert Field near St. Louis (1905 to 1962) and at Edwardsville (1930 to 1962) are given in figures 3 and 4, respectively. According to the records at Edwardsville, the months of greatest precipitation (exceeding 3.5 inches) are March through August; December is the month of least precipitation having 2.07 inches.

In addition to precipitation records available for Edwardsville, St. Louis, and Lambert Field, records for different periods are available for the gaging stations given in table 5 within and near the East St. Louis area.

The annual maximum precipitation amounts occurring on an average of once in 5 and once in 50 years are 45 and 57 inches, respectively; annual minimum amounts expected for the same intervals are 31 and 25 inches, respectively. Amounts are based on data given in the Atlas of Illinois Resources, Section 1 (1958).

The mean annual snowfall is about 17 inches. On the average, about 16 days a year have 1 inch or more, and

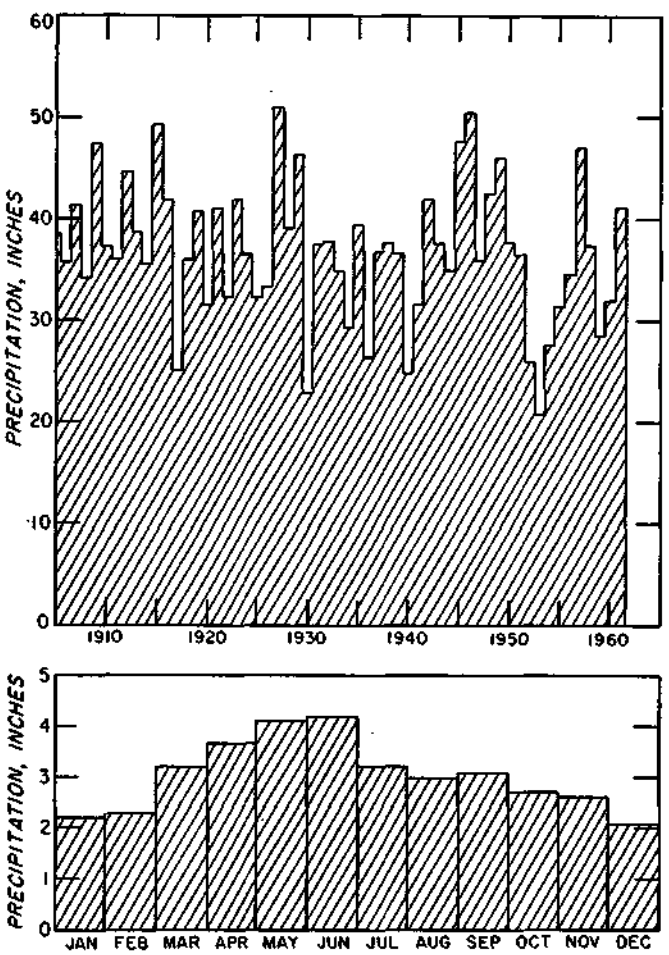


Figure 3. Annual and mean monthly precipitation at Lambert Field

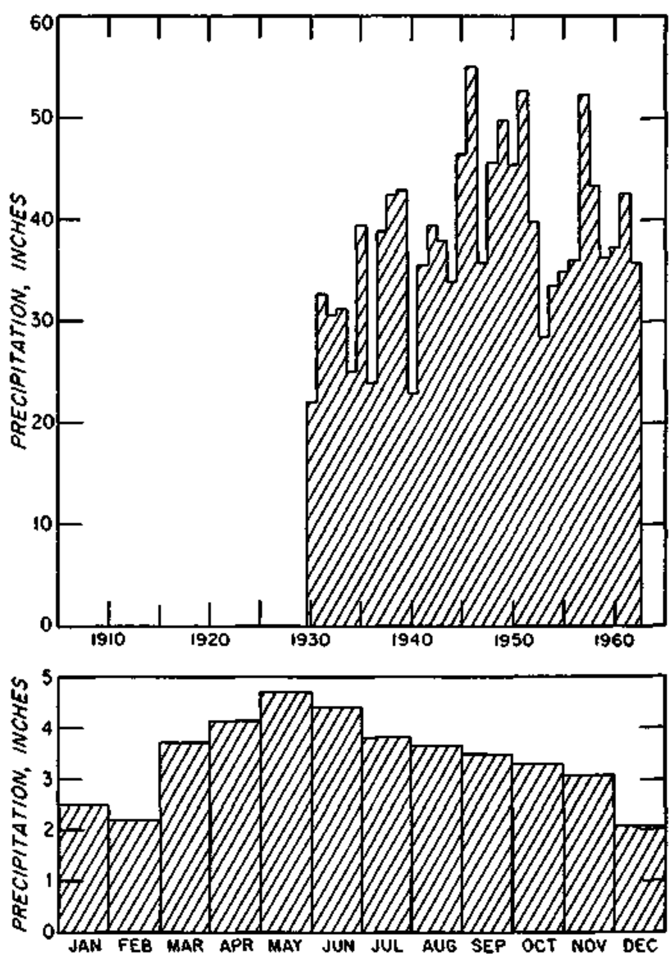


Figure 4. Annual and mean monthly precipitation at Edwardsville

about 8 days a year have 3 inches or more, of ground snow cover.

Based on records collected at Lambert Field, the mean annual temperature is 56.4 F. June, July, and August are the hottest months with mean temperatures of 75.2, 79.6, and 77.8 F, respectively. January is the coldest month with a mean temperature of 32.1 F. The mean length of the growing season is 198 days.

A large part of central and southern Illinois, including the East St. Louis area, experienced a severe drought beginning in the latter part of 1952 (Hudson and Roberts, 1955). For the period 1953 through 1956, cumulative deficiency of precipitation at Edwardsville and Lambert Field was about 22 and 34 inches, respectively.

An intense rainstorm, exceeding 16 inches in 12 hours at places, occurred June 14 and 15, 1957. The storm is discussed in detail by Huff et al. (1958). A Heavy rainstorm also occurred August 14-15, 1946, when over 11 inches were recorded at East St. Louis.

Table 5. Precipitation Gaging Stations

Owner	*Location of gage*
Shell Oil Co.	Wood River
East St. Louis and Interurban Water Co.	Chouteau Island
East Side Levee and Sanitary Dist.	Centreville
East Side Levee and Sanitary Dist.	Collinsville
East Side Levee and Sanitary Dist.	Edgemont
East Side Levee and Sanitary Dist.	Millstadt
Standard Oil Co.	Wood River
Illinois State Water Survey	Lakeside Airport
American Smelting and Refining Co.	Alton
Olin Mathieson Chemical Co.	East Alton
U. S. Weather Bureau	Collinsville
U. S. Weather Bureau	Belleville, Scott Air Force Base
U. S. Weather Bureau	Alton Dam 26
U. S. Weather Bureau	East St. Louis, Parks College

GEOLOGY AND HYDROLOGY

Large supplies of ground water chiefly for industrial development are withdrawn from permeable sand and gravel in unconsolidated valley fill in the East St. Louis area. The valley fill is composed of recent alluvium and glacial valley-train material and is underlain by Mississippian and Pennsylvanian rocks consisting of limestone and dolomite with subordinate amounts of sandstone and shale. The valley fill has an average thickness of 120 feet and ranges in thickness from a feather edge, near the bluff boundaries of the area and along the Chain of Rocks Reach of the Mississippi River, to more than 170 feet near the city of Wood River. The thickness of the valley fill exceeds 120 feet (figure 5) in places near the center of a buried bedrock valley that bisects the area as shown in figure 6.

According to Bergstrom and Walker (1956) recent alluvium makes up the major portion of the valley fill in most of the area. The alluvium is composed largely of fine-grained materials; the grain size increases from the surface down. Recent alluvium rests on older deposits including valley-train materials in many places. The valley-train materials are predominantly medium-to-coarse sand and gravel, and increase in grain size with depth. The coarsest deposits most favorable for development are commonly encountered near bedrock and often average 30 to 40 feet in thickness. Logs of wells in cross section A—A' in figure 7 and in table 6 show that the valley fill commonly grades from clay to silt to sand and gravel interbedded with layers of silt and clay with increasing depth.

The valley fill is immediately underlain by bedrock formations of Mississippian age in the western part of the area and bedrock formations of Pennsylvanian age in the eastern part of the area. Because of the low permeability of the bedrock formations and poor water quality with depth, the rocks do not constitute an important aquifer in the area.

Soils

The soils of the East St. Louis area were divided into three groups by the University of Illinois Agricultural Experiment Station as follows: bottomland soils, silty terrace soils, and sandy terrace soils. The bottomland soils in St. Clair County were divided into seven soil types by Smith and Smith (1938) as follows: Beaucoup clay loam, Drury fine sandy loam, River sand, Newart silt loam, Gorham clay loam, Dupo silt loam, and Riley fine sandy loam.

Drury fine sandy loam extends in a very narrow strip along the Mississippi River. It is a grayish-yellow to yellow, light brown, medium-to-coarse sand with variable thickness, usually 7 feet. The subsurface and subsoil are not well developed. Surface drainage is slow to rapid and permeability is rapid.

Beaucoup clay loam, Newart silt loam, Gorham clay loam, and Dupo silt loam cover much of the area. They are generally dark gray to grayish brown clay loams to silty clay loams 6 to 15 inches thick. The subsurface var-

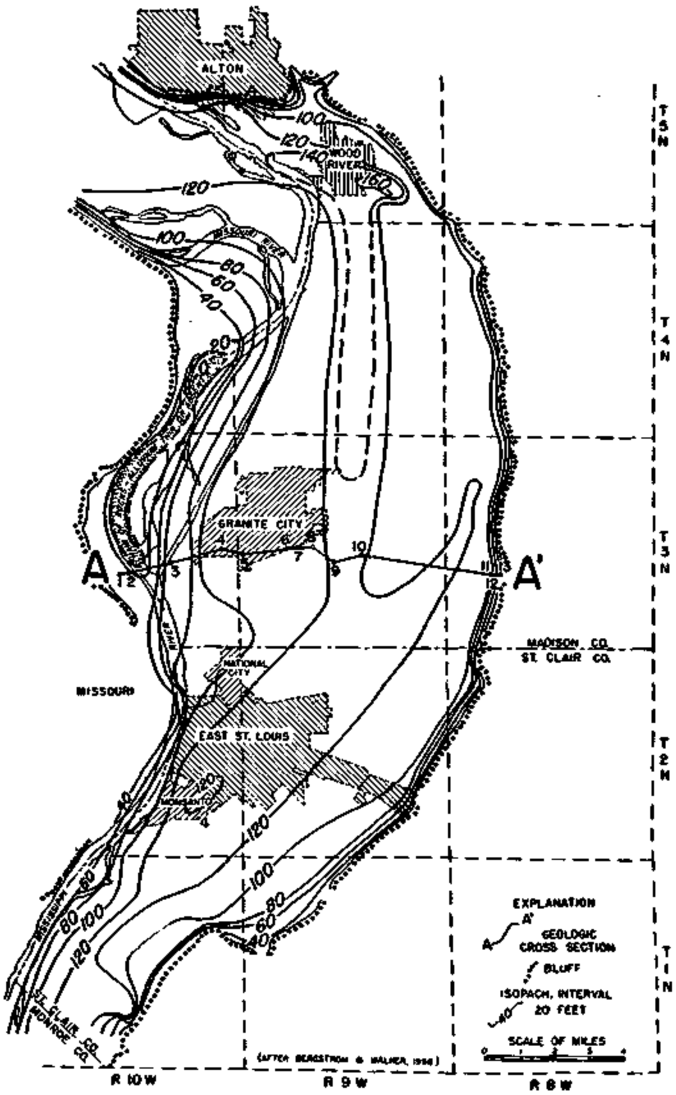


Figure 5. Thickness *of* the valley (ill

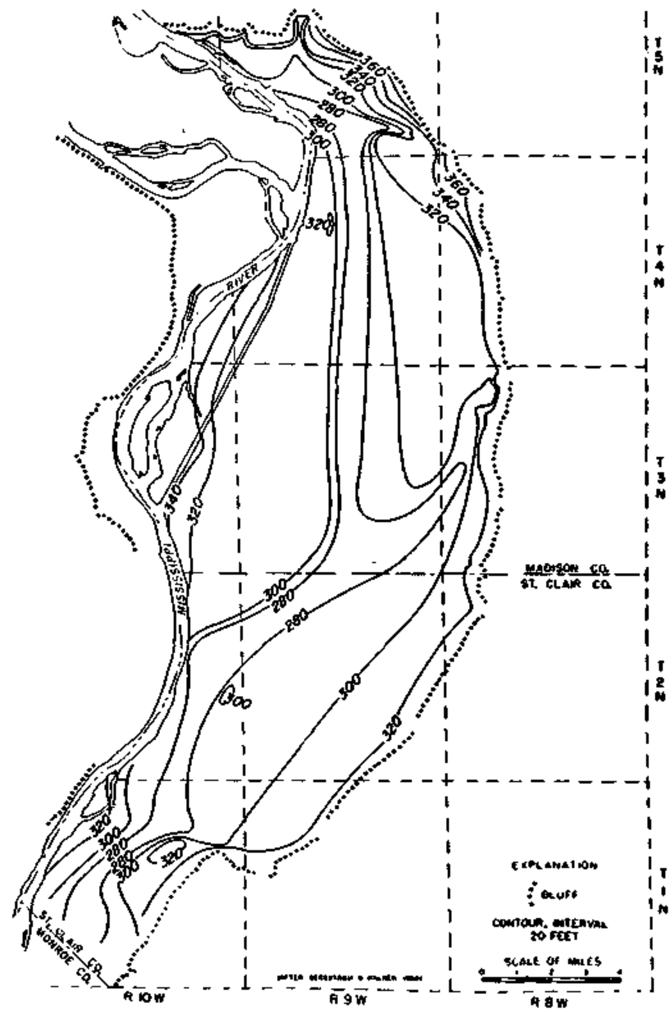


Figure 6. Bedrock topography

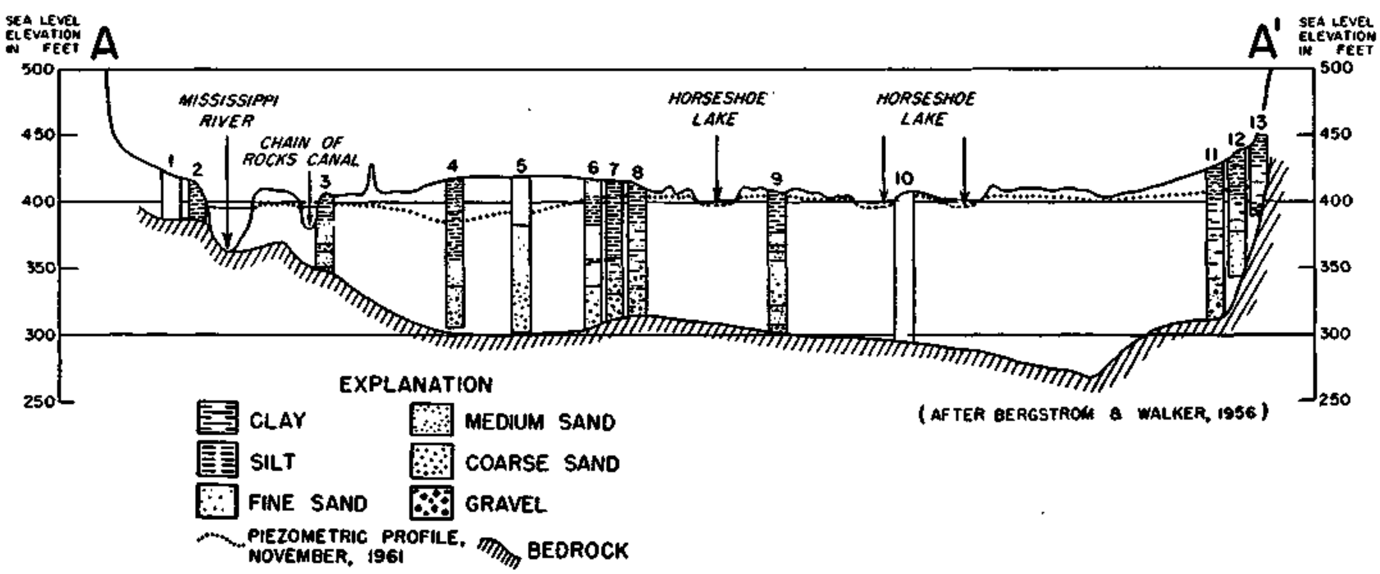


Figure 7. Geologic cross section and piezometric profile of the valley fill

Table 6. Logs of Selected Wells*

Illinois Slate Geological Survey lest hole 3 (1954)—Roxana Water Works, SE ¼ NE ¼ SE¼ SE¼ sec 27, T5N, R9W, Madison Co. Samples studied by R. E. Bergstrom. Est. elev. 445 feet.

	Thickness (ft)	Depth (ft)
Pleistocene Series		
Wisconsin or older Pleistocene		
Clay and silt, yellowish brown, noncalcareous	10	10
Silt and clay, with fine sand, yellowish brown, lumps of pink clay, slightly calcareous	5	15
Sand, fine, dirty, dark reddish brown, calcareous, pink-stained quartz grains	15	30
No samples	5	35
Sand, medium, light reddish brown, calcareous, subrounded grains, rhyolite porphyry, feldspar, gray-wacke, milky chert	15	50
Sand, medium to coarse, as above	20	70
Sand, fine to very coarse, light brown, dirty, gray silt, coal, mica	20	90
Sand, medium to coarse, light reddish brown, subrounded to subangular grains, abundant feldspar, reddish siltstone and rhyolite porphyry	15	105
Sand, coarse to medium, as above	10	115
Sand, very coarse, as above	5	120
Sand, very coarse, with granule gravel, subangular to angular grains, chert, reddish siltstone, granite, gray-wacke	5	127
Pennsylvanian System		
Shale, gray and brown	9.5	136.5

Sinclair OH Company well 2 (1952)—150 feet If, 1750 feet E of SW corner sec 34. T5N, R9W, Madison Co. Samples studied by R. E. Bergstrom. Est. elev. 431 feet.

	Thickness (ft)	Depth (ft)
Pleistocene Series		
Recent alluvium		
No samples	35	35
Sand, very fine, well sorted, olive gray, mollusk shell fragments, abundant mica, coal, wood	35	70
Silt and clay, with fine sand and small gravel, pebbles to % inch, mollusk shell fragments, calcareous	5	75
Wisconsin or older Pleistocene		
Sand, medium to coarse, yellowish brown, dry sample has pinkish cast, grains subrounded to rounded, slightly calcareous	40	115
Sand and pebble gravel, pebbles to 1.5 inches in diameter, abundant chert, limestone, gray-wacke, rhyolite	7.5	122.5

Illinois Geological Survey test hole 2 (1954)—Lutton farm: 4300 feet S of 80° 32'30" N, 5200 feet E of 90° 15' W, Cahokia Quadrangle, St. Clair Co. Studied by R. E. Bergstrom. Est. elev. 405 feet.

	Thickness (ft)	Depth (ft)
Pleistocene Series		
Recent and older alluvium		
Silt and clay, dark brownish gray	5	5
Silt and clay, with fine sand, dark brownish gray, calcareous, mica	10	15
Sand, fine to medium, dirty, dark olive gray, mica, wood fragments, coal, tiny calcareous spicules, shell fragments	30	45
Sand, coarse to very coarse, with granule gravel, abundant feldspar, granite, gray-wacke, chert, and dolomite granules	30	75
Gravel, granule size, with coarse to very coarse sand, quartz, granite, chert, dolomite granules (driller reports boulders)	20	95
Gravel, granule size with broken limestone rock, chert (pebble count of 50 pebbles—15 gray-wacke and fine-grained basic igneous rock; 12 chert, brown, reddish, and cream-colored; 11 quartz; 3 feldspar; 4 limestone; 4 granite; 1 dolomite); broken rock consists of sharp angular limestone, granite, rhyolite porphyry, and chert	10	105
Broken rock (limestone rubble above solid bedrock?) and granule gravel	7.5	112.5

Union Starch and Refining Company (1952)—950 feet S of 38°42'30" N, 2350 feet E of 90° 10' W, J3N, R10W, Madison Co. Illinois Geological Survey sample set 23406. Studied by R. E. Bergstrom. Est. elev. 4i2 feet.

	Thickness (ft)	Depth (ft)
Pleistocene Series		
Recent and older alluvium		
Soil, clay, and silt, dark gray	10	10
Sand, fine to coarse, subangular grains, abundant feldspar, tiny calcareous spicules, coal	30	40
Sand, medium, with granule gravel, as above, mollusk shell fragments	10	50
Sand, fine, with granule gravel, poor sorting, calcareous spicules, abundant dark grains of igneous rocks, ferromagnesium minerals, coal	10	60
Gravel, granule size, with coarse sand, granules mainly igneous rocks and feldspar	10	70
No samples	10	80
Sand, medium to fine, calcareous spicules, subangular grains, coal	10	90
No samples	5	95

Table 6 (Continued)

	Thickness (ft)	Depth (ft)
Sand, very coarse to coarse, with granule gravel, pinkish cast, abundant pink-stained quartz grains, subangular to subrounded grains	15	110
Sand, medium, well sorted, pink, subrounded to subangular grains, abundant pink feldspar	5	115

****From Bergstrom and Walker (1956)***

ies from silty loam to clay and is generally 2 to 3 feet thick. The subsoil is not well developed. The permeability and surface drainage is generally slow; the permeability of Newart silt loam is moderate.

Riley fine sandy loam covers much of the area near Monsanto, Cahokia, and Centreville. It is a light brown, fine sandy loam 8 to 10 inches thick. The subsurface is a loamy fine sand 8 to 12 inches thick, and the subsoil is a fine sandy loam with occasional clay lenses. Surface drainage is moderate to rapid and permeability is moderately rapid.

Drury fine sandy loam is a brownish yellow to yellowish silt loam to very fine sandy loam and is variable in thickness. It extends along the bluff in strips varying in width from a few feet to several miles. The subsurface is a silt loam to sandy loam about 3 feet thick. The subsoil is not well developed. Surface drainage is rapid and permeability is moderately rapid.

The soils in the East St. Louis area in Madison County have not been divided into soil types. According to McKenzie and Fehrenbacher (1961) bottomland soils predominate; however, silty terrace soils extend in a narrow strip along the bluffs just south of Cahokia Creek to the Madison-St. Clair County line, and in an area that extends from just south of Wood River southeast through Roxana and terminates a few miles southeast of Roxana. Sandy terrace soils extend in a strip a few miles wide from East Alton to Wood River and in a narrow strip southeast of Poag to about 3 miles northwest of Glen Carbon; sandy terrace soils also occur in an area southeast of Roxana.

The bottomland soils in Madison County exhibit a wide range of characteristics similar to those of the soil types in St. Clair County. The silty terrace and sandy terrace soils have moderately good to good drainage and moderately rapid to rapid permeability.

Occurrence of Ground Water

Ground water in the valley fill occurs under leaky artesian and water-table conditions. Leaky artesian conditions exist at places where fine-grained alluvium, consisting of silt and clay with some fine sand that impedes or retards the vertical movement of water, overlies coarser alluvium and valley-train deposits; water in these deposits is under artesian pressure. Under leaky artesian conditions, water levels in wells rise above the top of the valley-train and coarse alluvium deposits to stages within the finer grained alluvium. Water-table conditions prevail at many places where alluvium is missing and the upper surface of the zone of saturation is in valley-train deposits or the coarser alluvium, and at places within deep cones of depression created by heavy pumping where water levels in wells rise to stages within the valley-train deposits or the coarser alluvium and water is unconfined.

As shown in figure 8, leaky artesian conditions prevail in most of the area. Water-table conditions prevail in a wide belt from East Alton through Poag where alluvium is missing and heavy pumping in the vicinity of Wood River has lowered water levels below the base of the finer grained alluvium. Water-table conditions also prevail in: 1) the Monsanto and National City areas where heavy pumping has lowered water levels to stages within the valley-train deposits and coarser alluvium; 2) an area

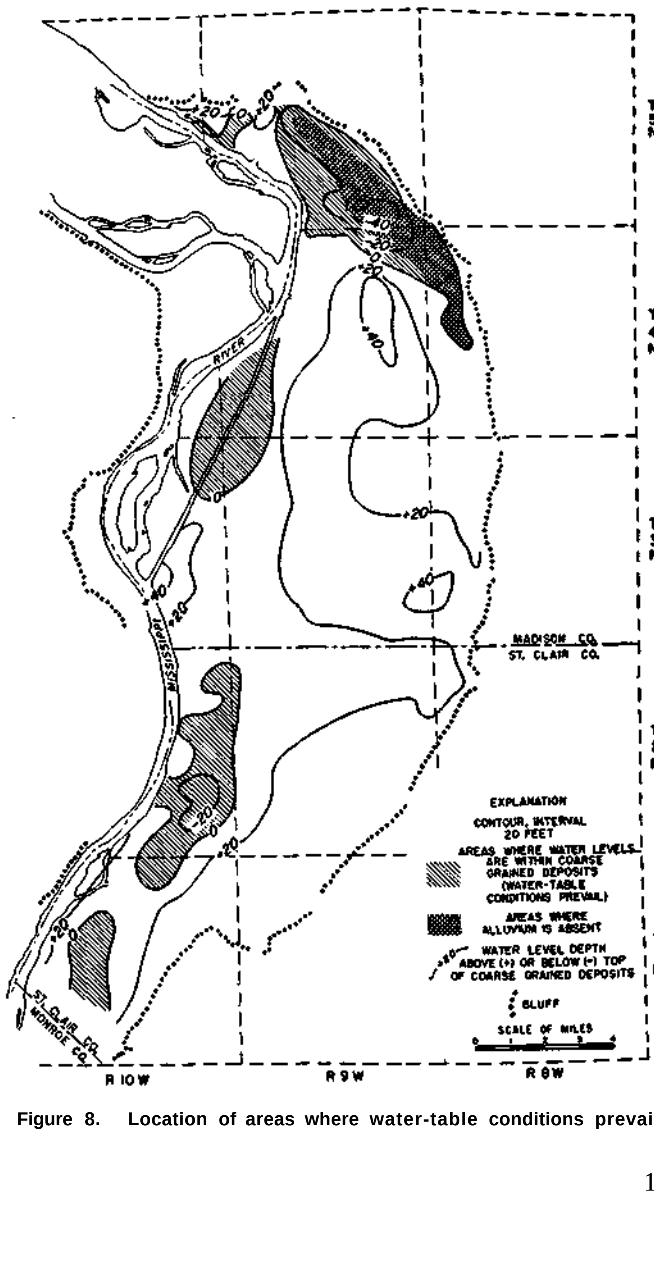


Figure 8. Location of areas where water-table conditions prevail

through Dupo and along the northern reach of the Chain of Rocks Canal where the finer grained alluvium is thin and water levels are in the coarser deposits; and 3) locally in the vicinity of well fields in the Granite City area and other areas where the saturated thickness of the finer grained alluvium is not great. The saturated thickness of the finer grained alluvium is greatest west of Poag near the center of T4N R9W, along the Mississippi River near Venice, and in an area 4 miles northwest of Collinsville.

Because water occurs most commonly under leaky artesian conditions, the surface to which water rises, as defined by water levels in wells, is hereafter called the piezometric surface.

HYDRAULIC PROPERTIES

The principal hydraulic properties of the valley fill and alluvium influencing water-level declines and the yields of wells in the East St. Louis area are the coefficients of transmissibility, or permeability, and storage. The capacity of a formation to transmit ground water is expressed by the coefficient of transmissibility, *T*, which is defined as the rate of flow of water in gallons per day, through a vertical strip of the aquifer 1 foot wide and extending the full saturated thickness under a hydraulic gradient of 100 percent (1 foot per foot) at the prevailing temperature of the water. The coefficient of transmissibility is the product of the saturated thickness of the aquifer, *m*, and the coefficient of permeability, *P*, which is defined as the rate of flow of water in gallons per day, through a cross-sectional area of 1 square foot of the aquifer under a hydraulic gradient of 100 percent at the prevailing temperature of the water. The storage properties of an aquifer are expressed by the coefficient of storage, *S*, which is defined as the volume of water released from storage per unit surface area of the aquifer per unit change in the water level.

Aquifer Tests

The hydraulic properties of the valley fill and alluvium may be determined by means of aquifer tests, wherein the effect of pumping a well at a known constant rate is measured in the pumped well and at observation wells penetrating the aquifer. Graphs of drawdown versus time after pumping started, and/or drawdown versus distance from the pumped well, are used to solve equations which express the relation between the coefficients of transmissibility and storage and the lowering of water levels in the vicinity of a pumped well.

The data collected during aquifer tests can be analyzed by means of the nonequilibrium formula (Theis, 1935). Further, Walton (1962) describes a method for applying the Theis formula to aquifer test data collected under water-table conditions, and gives equations for compensating observed values of drawdown for decreases in the saturated thickness of an aquifer.

Six controlled aquifer tests were made during the period 1952 to 1962. The results of the tests are summarized in table 7.

Table 7. Results of Aquifer Tests

Owner	Location of test site	Date of test	Duration of test (days)	Pumping rate (gpm)	Coefficient of transmissibility (gpd/ft)	Saturated thickness (ft)	Coefficient of permeability (gpd/sq ft)	Coefficient of storage	Method of analysis*
Olin Mathieson Chemical Corp.	Madison County, T5N, R9W, sec 19	May 29-Jun 1, 1956	3	760	95,600	90	1060	0.135	D-D
City of Wood River	Madison County, T5N, R9W, sec 28	Nov 20-21, 1962	1	491	134,000	60	2240	0.155	D-D
Shell Oil Co.	Madison County, T5N, R9W, sec 33	Mar 3-6, 1952	3	510	210,000	100	2100	0.002	D-D
Southwestern Campus of IU, Edwardsville	Madison County, T4N, R8W, sec 20	Dec 13-17, 1960	4	308	131,000	84	1560	0.020	T-D
Mobil Oil Co.	St. Clair County, T2N, R10W, sec 25	Oct 25-26, 1961	1	630	212,000	73	2900	0.100	T-D
Monsanto Chemical Corp.	St. Clair County, T2N, R10W, sec 27	Aug 4-8, 1952	4	1100	210,000	75	2800	0.082	T-D

**D-D, distance-drawdown; T-D., time-drawdown*

An aquifer test was made October 25 and 26, 1961, at the Mobil Oil Company Refinery near Monsanto by the State Water Survey in cooperation with the company. The test site was located in an area about 2600 feet north and 3500 feet west of the intersection of T2N, R10W and T1N, R9W. The effects of pumping well 19 were measured in test well 8, well 6, and well 20, The locations of wells used in the test (test 1) and test wells for which drillers logs are available are shown in figure 9. Pumping was

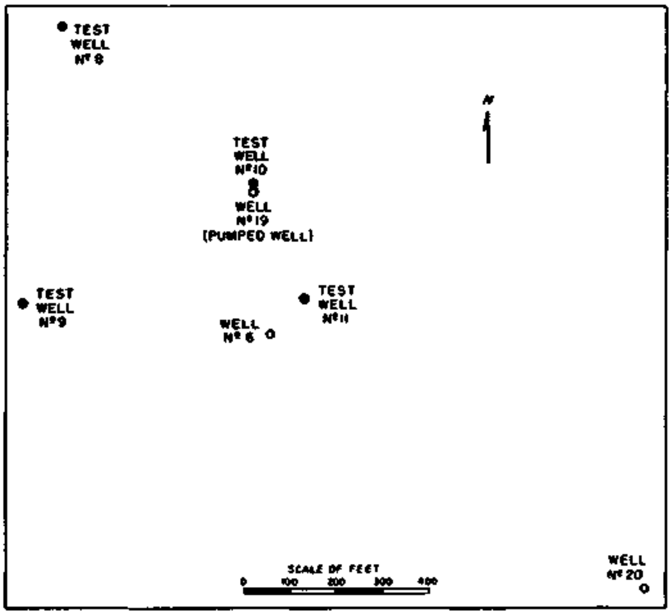


Figure 9. Location of wells used in aquifer test I

started at 9 a.m. October 25 and continued for 24 hours at a constant rate of 630 gpm. Pumping was stopped at 9 a.m. October 26 and water levels were allowed to recover for 1 hour, after which a step-drawdown test was conducted. Water levels were measured continuously with a recording gage in well 6, and periodically with a steel tape in well 20 and test well 8.

Well 19 is 16 inches in diameter, was drilled to a depth of 114 feet, and is equipped with 35 feet of No. 50 continuous slot Johnson Everdur screen between the depths of 79 and 114 feet. The well is an artificial pack well with a pack thickness of about 9 inches. Well 6 is 16 inches in diameter, 115 feet deep, and is screened at the bottom with 30 feet of 16-inch diameter Johnson Everdur screen with varying continuous slot sizes of 40, 50, 70, and 90. The thickness of the pack is not known. Well 20 is 24 inches in diameter and is 107 feet deep; there is 35 feet of 24-inch diameter Johnson Everdur screen at the bottom. The lower 17.5 feet of the screen is No. 100 slot and the upper 17.5 feet is No. 60 slot. The pack thickness is 9 inches. Test well 8 is 8 inches in diameter and 105 feet deep. The screen and casing are constructed of wood. The screen is 53 feet long with $\frac{3}{16}$ by 3-inch slots. The thickness of the pack is 5 inches. The logs of wells are given in table 8.

A time-drawdown field data graph (figure 10) for well 6 was superposed on the nonequilibrium type curve devised by Theis and described by Jacob (1940). The Theis (1935) nonequilibrium equations were used to determine coefficients of transmissibility and storage of the aquifer for data on the first and third segments of the time-drawdown graph. The coefficient of storage computed from the first segment of the time-drawdown curve is in the artesian range and cannot be used to predict long-term declines of the water table. The coefficient of storage (0.10) computed from the third segment is in the water-table range. The coefficient of transmissibility computed from the third segment is 212,000 gpd/ft.

An aquifer test (test 2) was made December 13-17, 1960, by Warren and Van Praag, Inc., Layne-Western Company, and the State Water Survey in cooperation with the Southwestern Campus of Southern Illinois University near Edwardsville. The test site is located west of Edwardsville in section 20, T4N, R8W. Three wells as shown in figure 11 were used. Pumping was started at 1:45 p.m. December 13, and was continued at a constant rate of 308 gpm until 12:30 p.m. December 17. Pumping was then stopped and water levels were allowed to recover for 1 hour. At 1:30 p.m. pumping was resumed at successive rates of 200, 300, 400, and 500 gpm, each maintained for 30 minutes. Water levels were measured periodically in the observation wells and pumped well during the test.

Observation well 1 was 2 inches in diameter and 94 feet deep, and the bottom 5 feet of pipe was slotted. Observation well 2 was 2 inches in diameter, 89 feet deep, and the bottom 6 feet of pipe was slotted. The pumped well was 10 inches in diameter and was drilled to a depth of 95 feet; 20 feet of screen was installed at the bottom. The well was an artificial pack well with a pack thickness of 3.5 inches. Logs of wells are given in table 9.

A time-drawdown field data graph (figure 12) for observation well 2 was superposed on the nonequilibrium type curve. The Theis (1935) equations were used to determine coefficients of transmissibility and storage of the aquifer for data on the third segment of the time-drawdown curve. The coefficient of transmissibility was computed to be 131,000 gpd/ft. The coefficient of storage (0.020) is in the water-table range.

An aquifer test (test 3) was made November 20 and 21, 1962, by Warren and Van Praag, Inc., Layne-Western Company, and the State Water Survey in cooperation with the city of Wood River. The test site was located in sec. 28, T5N, and R9W. Six wells as shown in figure 13 were used. Pumping was started at 9:45 a.m. November 20 and was continued at a constant rate of 491 gpm until 8:15 a.m. November 21. Pumping was then stopped and water levels were allowed to recover for 50 minutes. At 9:10 a.m. pumping was resumed and a step-drawdown test was conducted. Recording gages were installed in

Table 8. Drillers Logs of Wells Used in Aquifer Test I

Formation	From (It)	To
Test Well 8		
Clay fill	0	7
Fine sand	7	24
Fine to medium gray sand	24	37
Medium to coarse sand	37	41
Fine sand	41	55
Medium to coarse sand	55	64
Medium sand	64	65
Medium to coarse sand	65	73
Very coarse sand	73	80
Coarse to medium sand with cobbles	80	84
Coarse to medium sand	84	85
Medium to fine sand with gravel	85	90
Medium sand with gravel	90	95
Fine sand with gravel at 103.5 feet	95	103.5
Fine to coarse sand with gravel and cobbles	103.5	104
Coarse sand to heavy gravel with cobbles	104	106.9
(unable to drill beyond 106.9 feet because of heavy cobbles)		

Formation	From (ft)	To
Test Well 9		
Fill, clay, gravel	0	5
Silt and sandy silt	5	33
Medium gray sand	33	40
Fine sand, gray	40	45
Medium sand, gray	45	50
Coarse sand, gray, trace of clay	50	52
Very coarse sand with gravel	52	56
Medium coarse sand with gravel	56	58
Coarse gravel sand with gravel	58	62
Very coarse sand with gravel	62	63
Coarse sand and very coarse sand	63	72
Coarse to medium sand	72	79
Fine to medium sand with cobbles	79	79.5
Fine sand	79.5	80
Fine to medium sand	80	82
Medium coarse sand with gravel	82	83
Very coarse sand with gravel and lignite, cobbles at 88 feet	83	89.5
Coarse to medium sand	89.5	98
Very fine sand	98	100
Fine sand with gravel, cobbles at 102 feet	100	102.5
Coarse sand with gravel	102.5	104
Coarse sand with gravel and cobbles	104	113

Formation	From (It)	To
Test Well 10		
Gravel fill, gumbo	0	10
Dark silt	10	32
Fine gray sand	32	33
Dark fine silt	33	36
Medium fine sand	36	43
Fine sand	43	47

Formation	From (ft)	To
Test Well 10 (Continued)		
Dark gray silty sand	47	55
Fine sand	55	57
Medium fine sand	57	74
Very coarse sand with pea gravel and lignite	74	74.5
Very coarse sand with cobbles	74.5	80
Very coarse sand	80	87
Medium coarse sand with cobbles from 89 to 91 feet	87	90
Fine sand	90	95
Very fine sand	95	99
Very fine sand with cobbles at 100.5 feet	99	100
Coarse sand with cobbles	100	102
Coarse sand with cobbles	102	103
Coarse sand	103	104
Coarse sand with gravel	104	114

Formation	From (ft)	To
Test Well 11		
Mixture of clay, fill, silt, fly ash	0	10
Fine gray silt	10	15
Very fine gray sand	15	20
Fine gray sand	20	25
Fine to medium gray sand	25	40
Medium gray sand	40	45
Medium to fine gray sand	45	52
Medium to coarse sand	52	55
Fine to medium sand	55	61
Medium to fine sand	61	73
Medium sand	73	76
Coarse sand	76	80
Very coarse sand with cobbles	80	87
Medium to coarse sand with cobbles	87	95
Fine to medium sand	95	95.5
Medium to coarse sand with ½-inch gravel	95.5	100
Very coarse sand and gravel with boulders	100	115

Formation	From (ft)	To
Well 20		
Silty sandy gray clay	0	15
Medium gray sand	15	20
Fine gray sand	20	25
Coarse gray sand	25	30
Fine to medium gray sand	30	35
Very fine sand	35	53
Very coarse sand with 2-inch gravel	53	55
Medium sand with gravel	55	57.5
Medium to coarse sand with gravel	57.5	60
Very coarse sand with pea gravel	60	65
Medium to coarse sand with gravel	65	75
Coarse sand	75	80
Very coarse sand with pea gravel	80	83.5
Medium to coarse sand	83.5	85
Very coarse sand with 2-inch gravel	85	107

relief wells 137 and 139. Water levels were measured periodically with a steel tape in the pumped well, test hole 5, test hole 4, and relief well 140.

The pumped well was 10 inches in diameter and was drilled to a depth of 84 feet; 20 feet of 8-inch slotted pipe was installed at the bottom. The well is an artificial pack well with a pack thickness of 4 inches. Test holes 4 and 5 were 2 inches in diameter and 70 and 66.5 feet in

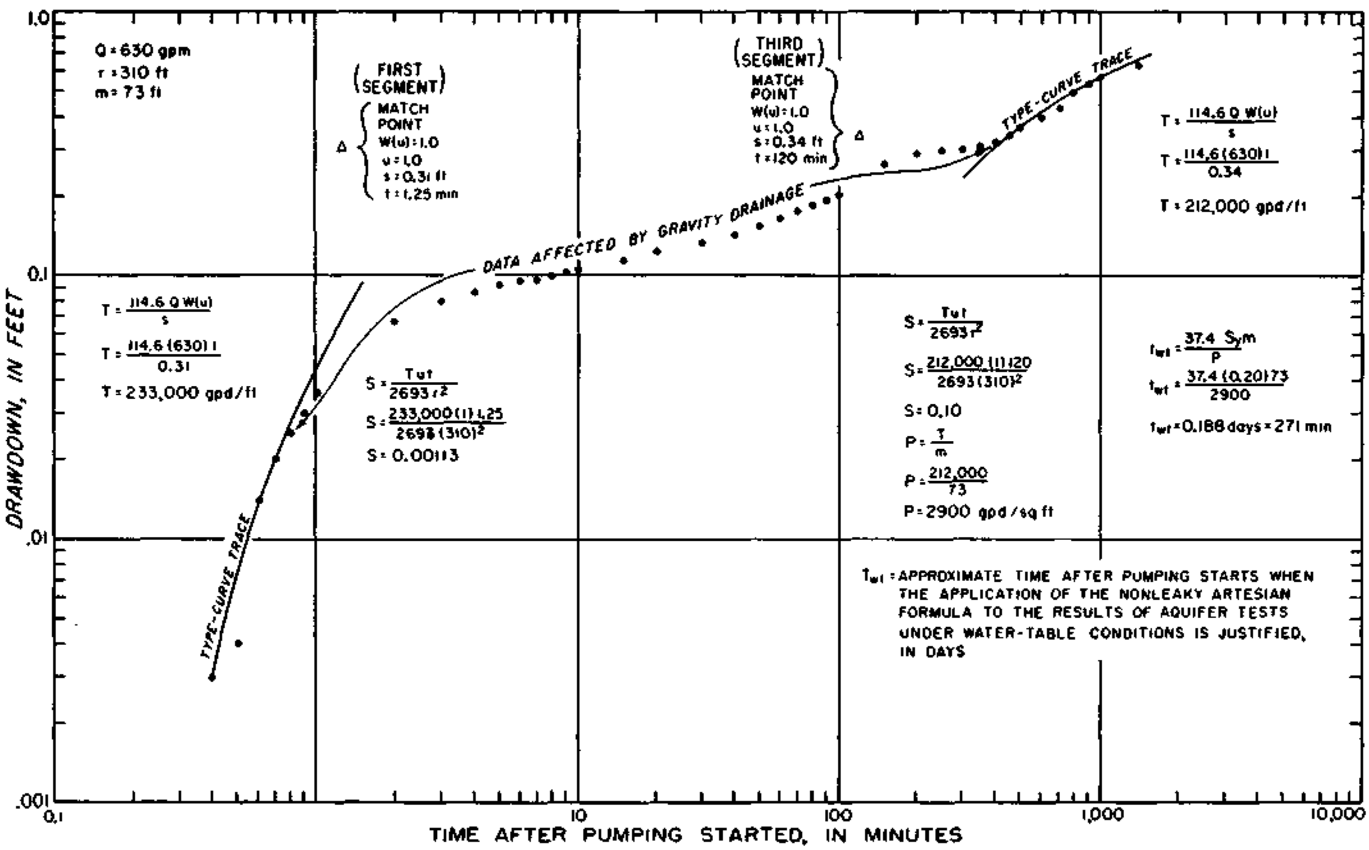


Figure 10. Time-drawdown data for well 6, aquifer test I

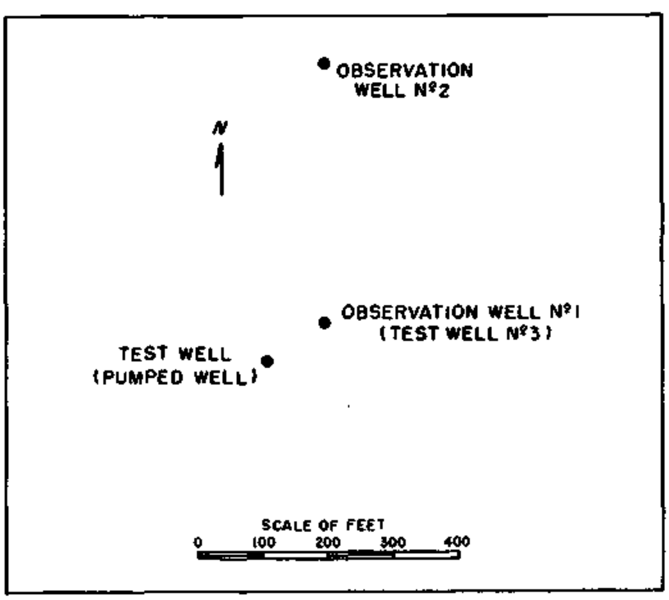


Figure I I. Location of wells used in aquifer test 2

Table 9. Drillers Logs of Wells Used in Aquifer Test 2

Formation	From (ft)	To (ft)
Test Well (Pumped Well)		
Sandy clay	0	14
Fine brown sand	14	50
Coarse gray sand	50	75
Fine-to-medium brown sand	75	90
Medium gray sand	90	95
Fine brown sand	95	98
Observation Well 1		
Brown clay	0	14
Fine brown sand, clay streaks	14	50
Medium gray sand, loose	50	75
Coarse gray sand, some gravel, loose	75	90
Fine sand	90	100
Light gray shale	100	130
Limestone		130
Observation Well 2		
Brown clay	0	14
Fine red sand, clay streaks	14	65
Medium gray sand, little gravel, few clay balls	65	90
Fine sand	90	100

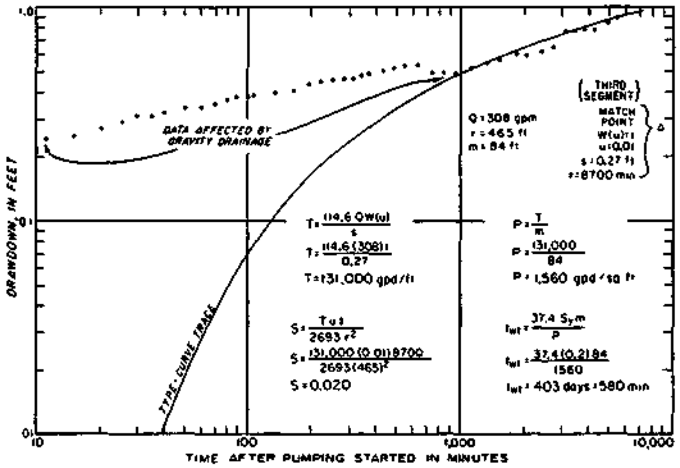


Figure 12. Time-drawdown data for observation well 2, aquifer test 2

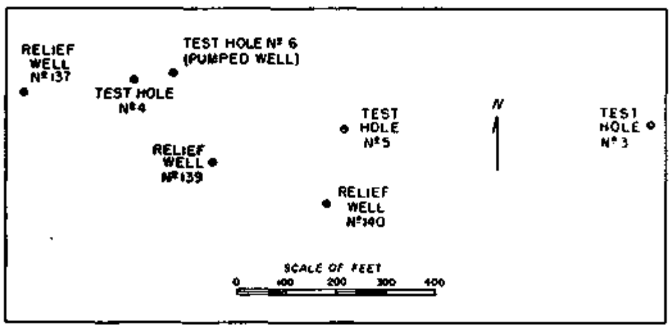


Figure 13. Location of wells used in aquifer test 3

depth, respectively. The lower 6.4 feet of casing in each test hole was slotted. The logs of test holes are given in table 10.

A distance-drawdown field data graph (figure 14) prepared with water-level data collected in the observation wells after a pumping period of 1335 minutes was superposed on the nonequilibrium type curve. The Theis (1935) equations were used to determine coefficients of transmissibility and storage of the aquifer. The coefficient of transmissibility was computed to be 134,000 gpd/ft. The coefficient of storage (0.155) is in the water-table range.

The cone of depression created by pumping a well near a river that is hydraulically connected to the aquifer is distorted. The hydraulic gradients between the river and the pumped well will be steeper than the hydraulic gradients on the land side of the well. The flow towards the well will be greatest on the river side of the well, and under equilibrium conditions most of the pumped water will be derived from the river.

When the well is pumped, water is initially withdrawn from storage within the aquifer in the immediate vicinity of the well. If pumping is continued long enough water levels in the vicinity of the river will be lowered and water that under natural conditions would have discharged into the river as ground-water runoff or into the atmosphere as evapotranspiration is diverted toward the pumped well. Water levels are ultimately lowered below all or part of the river bed in the immediate vicinity of the well, and the aquifer is then recharged by the influent seepage of surface water. The cone of depression will continue to grow until sufficient area of the river bed

Table 10. Drillers Logs of Test Holes Used in Aquifer Test 3

Formation	From (ft)	To
Test hole 3		
Brown clay	0	20
Soft blue clay	20	46
Fine sand	46	50
Medium to coarse	50	82
Sand, loose	82	104
Gray clay	104	116
Fine sand, loose	116	120
Red clay	120	
Rock		
Test hole 4		
Brown clay	0	9
Fine sand, clay streaks	9	25
Medium sand, some clay	25	30
Fine tight sand	30	52
Coarse sand and gravel, loose	52	79
Hard gray clay	79	83
Fine sand, clay streaks	83	90
Bedrock		90
Test hole 5		
Brown clay	0	11
Fine sand and clay	11	17
Fine sand	17	55
Coarse sand and gravel, loose	55	83
Gray clay	83	100
Test hole 6 (Pumped Well;)		
Brown clay	0	10
Fine sand and clay	10	18
Fine sand	18	48
Coarse sand and gravel, boulders drilled like rock ledge at 57 feet	48	80
Gray clay	80	84

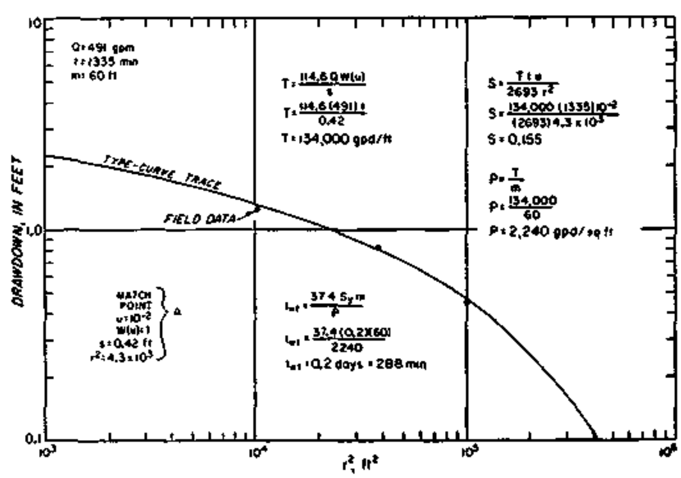


Figure 14. Distance-drawdown data for aquifer test 3

is intercepted and the cone is deep enough so that the induced infiltration balances discharge.

The area of the river bed over which recharge takes place is replaced by a line source. According to the image well theory (Ferris, 1959), the effect of a line source on the drawdown in an aquifer, as a result of pumping from a well near the line source, is the same as though the aquifer were infinite and a like recharging well were located across the line source, and on right angles thereto, and at the same distance from the line source as the real pumping well. Based on the image well theory and the nonequilibrium formula, the drawndown distribution in an aquifer bounded by a line source under equilibrium conditions is given by the following equation:

$$s = [528Q \log_{10} (r_i/r_p)]/T \qquad (1)$$

where:

s = drawdown at observation point, in ft
Q = discharge of pumped well, in gpm
r_i = distance from image well to observation point, in ft
r_p = distance from pumped well to observation point, in ft
T = coefficient of transmissibility, in gpd/ft

In terms of the distance between the pumped well and the line source or recharge boundary, equation 1 was expressed by Rorabaugh (1956) as

$$s = [528Q \log_{10} (\sqrt{4a^2 + r_p^2 - 4a\, r_p \cos \Phi}/r_p)]/T \qquad (2)$$

where:

a = distance from pumped well to recharge boundary, in ft
Φ = angle between a line connecting the pumped well and the image well and a line connecting the pumped well and the observation point

For the particular case where the observation well is on a line parallel to the recharge boundary, equation 2 may be written as follows:

$$s = [528Q \log_{10} (\sqrt{4a^2 + r_p^2}/r_p)]/T \qquad (3)$$

Equations 1 through 3 assume that the cone of depression has stabilized, water is no longer taken from storage within the aquifer, and equilibrium conditions prevail. The pumping period required to stabilize water levels can be computed by using the following equation (see Foley, Walton, and Drescher, 1953):

$$t_e = 3.26a^2 s/[T \varepsilon \log_{10} (2a/r_p)^2] \qquad (4)$$

where:

t_e = time after pumping starts before equilibrium conditions prevail, in days
s = coefficient of storage, fraction
ε = deviation from absolute equilibrium (arbitrarily assumed to be 0.05)

In many cases the stabilization of the cone of depression can be attributed either to the effects of slow gravity drainage, effects of leakage through a confining bed (Walton, 1960a), or effects of induced infiltration if the effects of partial penetration are excluded. Walton (1963) gave methods for proving whether or not water levels stabilize because of the effects of induced infiltration.

According to Walton (1963) the coefficient of transmissibility can often be determined from distance-drawndown data for observation wells on a line parallel to the recharge boundary. Provided the wells are not too distant from the pumped well and not too close to the recharge boundary, the effects of induced infiltration on drawdowns in the wells is approximately equal because the wells are for practical purposes equidistant from the image well associated with the recharge boundary. A plot of maximum drawdowns in the observation wells versus the logarithm of distance from the pumped well will yield a straight-line graph. The slope of the straight line is substituted in the following equation (Cooper and Jacob, 1946) to compute the coefficient of transmissibility:

$$T = 528Q/\Delta s \qquad (5)$$

where:

T = coefficient of transmissibility, in gpd/ft
Q = discharge of pumped well, in gpm
Δs = drawdown difference per log cycle as determined from distance-drawdown graph, in ft

If T is known, the distance from the pumped well to the recharge boundary, a, can be computed with maximum drawdowns in each observation well on a line parallel to the stream and the following equation:

$$\log_{10} \sqrt{4a^2 + r_p^2}/r_p = Ts/528Q \qquad (6)$$

where:

s = drawdown, in ft
a = distance from pumped well to recharge boundary, in ft
r_p = distance from pumped well to observation well, in ft
Q = discharge of pumped well, in gpm
T = coefficient of transmissibility, in gpd/ft

The maximum drawdowns in the observation wells are much less because of the effects of recharge than they would be if the aquifer were infinite; thus, the coefficient of storage cannot be determined from the distance-drawdown graph.

The nonequilibrium formula (Theis, 1935) and computed values of T and a can be used to determine the coefficient of storage. Several values of the coefficient of storage are assumed, and maximum drawdowns in each observation well are computed taking into consideration the effects of the image well associated with the recharge boundary and the pumped well. The computed drawdowns in each observation well are then compared with actual drawdowns, and the coefficient of storage that provided computed drawdowns equal to actual drawdowns is assigned to the aquifer.

Three aquifer tests under induced infiltration conditions were made during the period 1952 to 1956. The results of the tests are summarized in table 7.

An aquifer test (test 4) was made March 3-6, 1952, on property owned by the Shell Oil Company along the Mississippi River in sec. 33, T5N, R9W. The test was conducted for the Shell Oil Company by Ranney Method Water Supplies, Inc. Seven wells, grouped as shown in figure 15, were used. Four wells were approximately

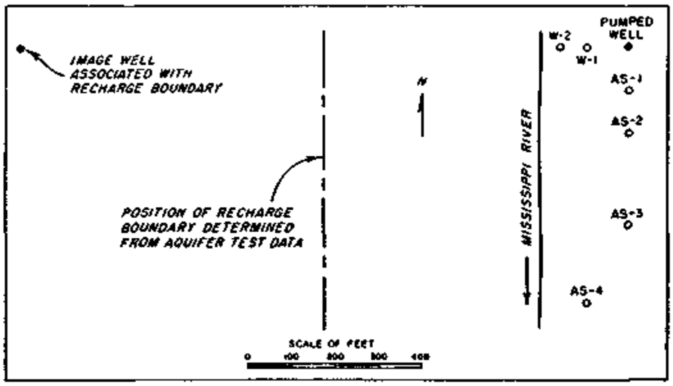


Figure 15. Location of wells used in aquifer test 4

parallel to and about 200 feet east of the Mississippi River. Pumping was started at 9:25 a.m. and was continued at a constant rate of 510 gpm for three days. Pumping was stopped at 9:25 a.m. March 6, and water levels were allowed to recover.

Observation wells AS-1, AS-2, and AS-3 were reported to be 7 inches in diameter and averaged 60 feet in depth; wells AS-4, W-1, and W-2 were 7 inches in diameter and were drilled to depths of 119, 112, and 55 feet respectively. The pumped well was 12 inches in diameter and 100 feet deep. Data on lengths of screens were not available. Recording gages were installed on the six observation wells and the Mississippi River. Logs of wells used in the test are given in table 11.

Values of drawdown in wells AS-1, AS-2, and AS-3 at a time 1800 minutes after pumping started were plotted on semilogarithmic paper against values of distance from the pumped well as shown in figure 16. A straight line was drawn through the points. The slope of the straight line per log cycle and the pumping rate were substituted into equation 5 and the coefficient of transmissibility was computed to be 210,000 gpd/ft.

The distance from the pumped well to the recharge boundary was determined by substituting the computed value of T, the measured rate of pumping, and values of drawdowns in the observation wells into equation 6 and solving for the distance a. The average distance a was found to be about 700 feet.

The coefficient of storage was determined to be 0.002 by using the computed values of T, a, the drawdowns in observation wells, and the nonequilibrium formula. Fine-grained alluvial deposits (see table 11) occur in the portion of the aquifer unwatered by pumping.

An aquifer test (test 5) was made May 29 through June 1, 1956, by Ranney Method Water Supplies, Inc., for the Olin-Mathieson Chemical Corporation. E. G. Jones, Water Survey field engineer, assisted in making the test. The test site was just southeast of the confluence of Wood River and the Mississippi River in sec. 19, T5N, R9W. Eight wells, grouped as shown in figure 17 were used. The wells were arranged in a ‘T’ pattern with four wells parallel to and 350 feet north of the Mississippi River. Pumping was started at 1:30 p.m. on May 29 and stopped at 1:30 p.m. on June 1. The pumping rate during the test was held constant at a rate of 760 gpm.

Table 11. Drillers Logs of Wells Used in Aquifer Test 4

Formation	From	To
	(ID	
Well AS-1		
Brown silty sand	0	19
Blue clay	19	33
Fine gray sand	33	41
Coarse sand and sand and small gravel	41	60
Well AS-2		
Brown silty clay	0	19
Blue clay	19	32
Fine gray sand	32	42
Coarse gravel and small and medium gravel	42	62
Well AS-3		
Brown silty clay	0	19
Blue clay	19	34
Fine gray sand	34	42
Coarse gravel and small and medium gravel	42	60
Well AS-4		
Brown clay	0	5
Dirty fine gray sand	5	37
Fine gray sand	37	51
Coarse sand and gravel	51	71
Fine red sand	71	92
Medium sand and gravel	92	112
Medium sand and gravel	112	119
Well W-l		
Brown clay	0	4
Soft blue clay	4	26
Fine sand	26	37
Sand and gravel	37	116
Hard blue clay	116	118
Bedrock		
Well W-2		
Clay	0	3
Gray silt	3	28
Fine gray sand	28	40
Coarse sand and gravel	40	55

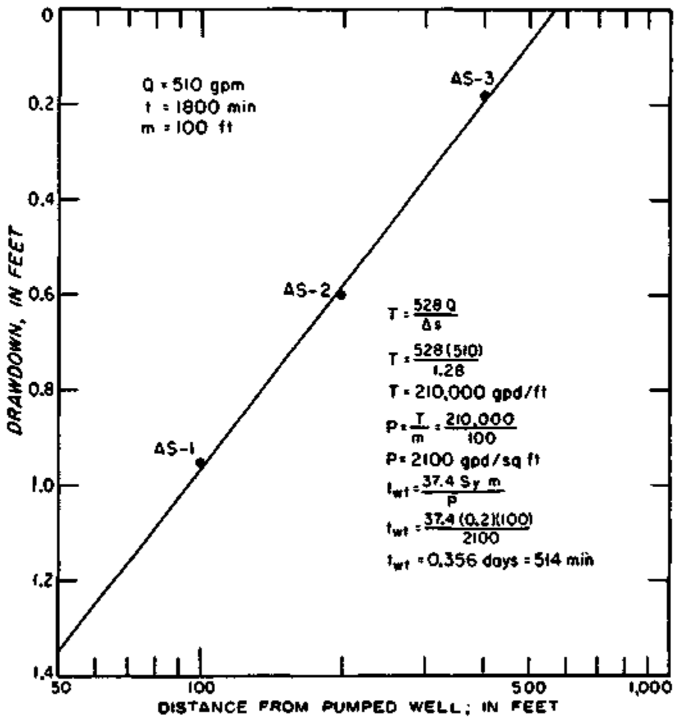


Figure 16. Distance-drawdown data for aquifer test 4

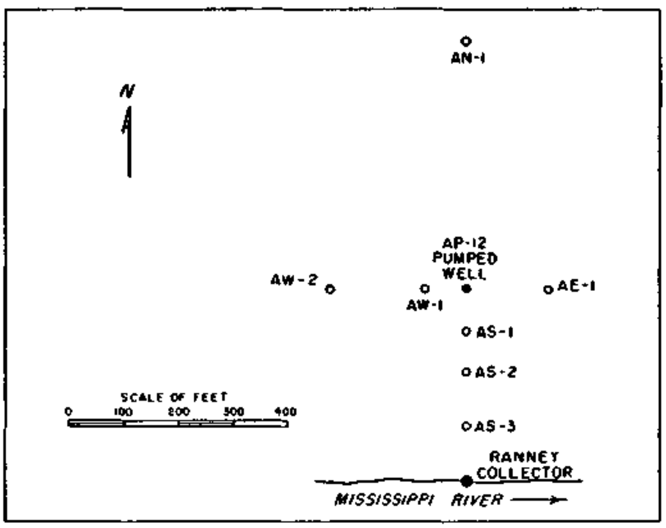


Figure 17. Location of wells used in aquifer test 5

The pumped well was 12 inches in diameter and 88 feet deep; the lower 10 feet of the well was screened. Observation wells AS-1, AS-2, AN-1, AW-1, AW-2, and AE-1 were 6 inches in diameter and averaged about 90 feet in depth. Well AE-3 was 6 inches in diameter and 124 feet in depth. Drillers logs of wells are given in table 12. Recording gages were installed on the observation wells and the Mississippi River. Values of drawdown in wells AS-1, AW-1, AE-1, AS-2, AS-3, and AW-2 at a time 1830 minutes after pumping started were plotted on semilogarithmic paper against values of distances from the pumped well as shown in figure 18. A straight line was drawn through the points. The slope of the straight line per log cycle and the pumping rate were substituted into equation 5 and the coefficient of transmissibility was computed to be 95,600 gpd/ft. The slope of the straight line per log cycle from distance-drawdown data on a line perpendicular to the river and on a line parallel to the river are approximately the same suggesting that the effects of induced infiltration on drawdowns were negligible. The coefficient of storage, S, was computed from the following equation (Cooper and Jacob, 1946):

$$S = Tt/4790r_o^2 \qquad (7)$$

where:

S = coefficient of storage, fraction
t = time after pumping started, in min
T = coefficient of transmissibility, in gpd/ft
r_o = intercept of straight line with zero drawdown axis, in ft

The coefficient of storage (0.135) is in the water-table range.

The distance *a* was found to be 100 feet from the river's edge, as determined from water-level data collected during a production test February 13-19, 1959, using the collector well constructed at the site of aquifer test 5, hydraulic properties of the aquifer determined from the aquifer test May 29 - June 1, 1956, and equation 6. Pumping from the collector well was started at 8 a.m. on February 13 and continued at a constant rate of 7000 gpm until 3:15 p.m. February 17 when the pumping rate was increased to 8400 gpm. The pumping test continued at a rate of 8400 gpm until 8:15 p.m. February 19 when pumping was stopped and water levels were allowed to recover. Recording gages were installed on observation wells AS-3, AE-1, and AN-1. Frequent water-level measurements were made with a steel tape in well AS-2. In addition, recording gages were installed on the Mississippi River, on the collector well, and on an observation well immediately outside the collector well.

An aquifer test (test 6) was made August 4-8, 1952, by Ranney Method Water Supplies, Inc., for the Monsanto Chemical Corporation. The test site is located east of Monsanto, along the Mississippi River in sec. 27, T2N, R10W. Seven wells, grouped as shown in figure 19 were used. The wells were arranged in a ‘T’ pattern with four wells parallel to and 515 feet east of the Mississippi River and three wells perpendicular to the river. Pumping was started at 6 p.m. August 4 and was continued at a constant rate of 1100 gpm until 6 p.m. August 8 when pumping was stopped and water levels were allowed to recover.

Observation wells S-1, W-1, N-1, S-2, W-2, and W-3 were 7 inches in diameter and were drilled to depths of about 100 feet. The pumped well was 12 inches in diameter and was drilled to a depth of 99 feet; 10 feet of screen was installed at the bottom. Available logs of wells are given in table 13. Recording gages were installed on the

Table 12. Drillers Logs of Wells Used in Aquifer Test 5

Formation	From (ft)	To (ft)
Well AP-12 (Pumped Well)		
Fine brown sand, silty	0	15
Fine brown sand, silty, scattered gravel	15	28
Medium to pea gravel, fine sand with scattered clay balls, gray	28	40
Fine sand, scattered gravel	40	60
Very fine sand	60	78
Medium to coarse gravel, fine sand with scattered clay balls	78	81
Medium to pea gravel, medium sand	81	85
Medium to pea gravel, coarse sand	85	88
Gray clay	88	(Total depth)
Well AS-1		
Fine brown sand, silty	0	27
Fine sand, scattered gravel, clay balls	27	30
Medium to pea gravel, fine sand, clay	30	37
Very fine gray sand	37	73
Medium to pea gravel, medium to coarse sand	73	89
Clay balls	89	
Well AS-2		
Fine brown sand, silty	0	28
Fine brown sand, clay balls	28	30
Very fine gray sand	30	37
Medium to coarse gravel, fine sand	37	73
Medium to pea gravel, fine sand	73	89
Clay balls	89	
Well AS-3		
Very fine brown sand, silty	0	22
Medium to pea gravel, fine sand	22	34
Fine gray sand	34	70
Medium to pea gravel, fine sand	70	75
Medium to pea gravel, medium sand	75	90
Gray clay	90	96
Medium sand, scattered gravel, clay balls	96	110
Medium sand, scattered pea gravel	110	124
Sandstone rock	124	
Well AN-1		
Fine sand, brown, silty	0	25
Fine gray sand	25	35
Medium sand, scattered gravel	35	56
Medium sand, scattered gravel, clay balls	56	59
Medium sand, scattered gravel	59	72
Medium to pea gravel, coarse sand	72	80
Medium to pea gravel, medium sand	80	82
Clay balls and boulders	82	83
Medium to fine sand, scattered gravel, clay balls	83	89
Well AW-1		
Fine brown sand, silty	0	20
Medium sand, clay balls	20	31
Fine gray sand, scattered gravel	31	37
Very fine gray sand	37	76
Medium to pea gravel, medium sand	76	87
Gray clay	87	88
Well AW-2		
Fine brown sand, silty	0	38
Very fine gray sand	38	55
Very fine gray sand, scattered gravel	55	57
Very fine gray sand	57	84
Medium to fine sand, scattered gravel	84	89
Clay balls	89	
Well AE-1		
Fine brown sand, silty	0	28
Fine gray sand, clay balls	28	32
Very fine sand	32	75
Medium to pea gravel, fine sand	75	86
Medium to coarse gravel, medium sand	86	90
Clay balls	90	

observation wells; Mississippi River stages were available from the river gage at St. Louis.

A time-drawdown field data graph (figure 20) for well S-2 was superposed on the nonequilibrium type curve. The Theis (1935) equations were used to determine coefficients of transmissibility and storage of the aquifer for data on the third segment of the time-drawdown curve. The coefficient of transmissibility was computed to be 210,000 gpd/ft. The coefficient of storage (0.082) is in the water-table range. Drawdowns deviated from the type-curve trace during the latter part of the test because of the effects of induced infiltration. The distance to the image well associated with the recharge boundary was computed to be 1790 feet from the following equation (see Ingersoll, Zobel, and Ingersoll, 1948):

$$r_i = r_p\sqrt{t_i/t_p} \qquad (8)$$

where:

r_i = distance from image well to observation well, in ft

r_p = distance from pumped well to observation well, in ft

t_p = time after pumping started, before the boundary became effective, for a particular drawdown to be observed, in min

t_i = time after pumping started, after the boundary became effective, when the divergence of the time-drawdown curve from the type-curve trace under the influence of the image well is equal to the particular value of drawdown at t_p, in min

Specific-Capacity Data

The yield of a well may be expressed in terms of its specific capacity, which is defined as the yield in gallons per minute per foot of drawdown (gpm/ft) for a stated pumping period and rate. Walton (1962) gave an equation for computing the theoretical specific capacity of a well discharging at a constant rate in a homogeneous, isotropic, artesian aquifer infinite in areal extent.

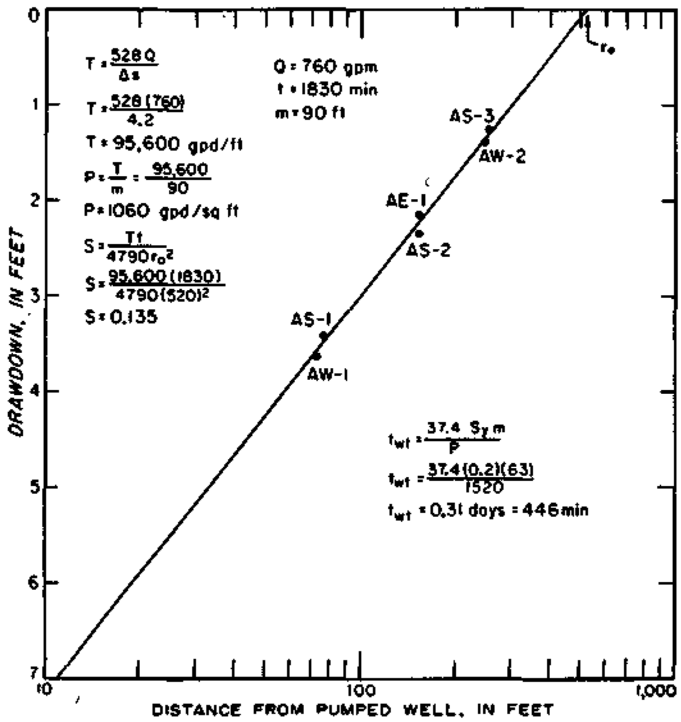


Figure 18. Distance-drawdown data for aquifer test 5

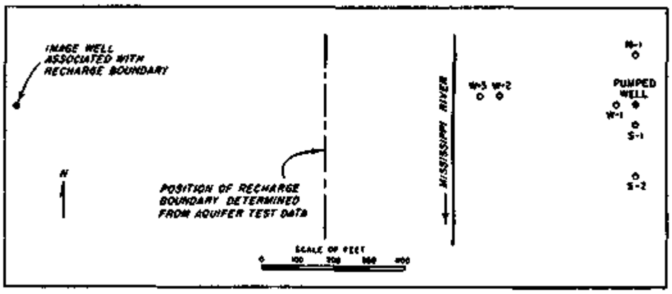


Figure 19. Location of wells used in aquifer test 6

Table 13. Drillers Logs of Wells Used in Aquifer Test 6

Formation	From	To
Well S-1		
Gray sandy clay	0	30
Gray fine sandy clay	30	40
Coarse gray sand, small gravel	40	45
Gray fine sand, scattered fine gravel, brown fine sand	45	66
Brown coarse sand, fine gravel	66	76
Coarse sand and gravel	76	90
Coarse sand, fine to medium gravel	90	100
Bedrock	About	120
Well S-2		
Gray sandy clay	0	30
Gray fine sandy clay	30	40
Coarse gray sand, small gravel	40	45
Gray fine sand, scattered fine gravel, brown fine sand	45	66
Brown coarse sand, fine gravel	66	75
Brown coarse sand, fine gravel, some gray clay	75	76
Coarse sand, small to large gravel	76	90
Brown coarse sand, fine to medium gravel	90	100

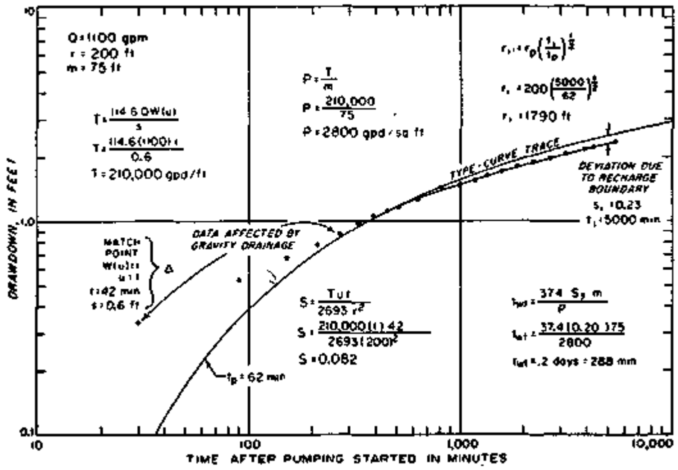


Figure 20. Time-drawdown data for well S-2, aquifer test 6

The specific capacity is influenced by the hydraulic properties of the aquifer, the radius of the well, r_w, and the pumping period, t. The relationship between the theoretical specific capacity of a well and the coefficient of transmissibility is shown in figure 21. A pumping period of 24 hours, a radius of 12 inches, and a storage coefficient of 0.1 were used in constructing the graph.

There is generally a head loss or drawdown (well loss) in a production well due to the turbulent flow of water as it enters the well itself and flows upward through the bore hole. Well loss and the well-loss coefficient may be computed by equations given by Jacob (1946). The computations for the well-loss coefficient, C, require data collected during a step-drawdown test

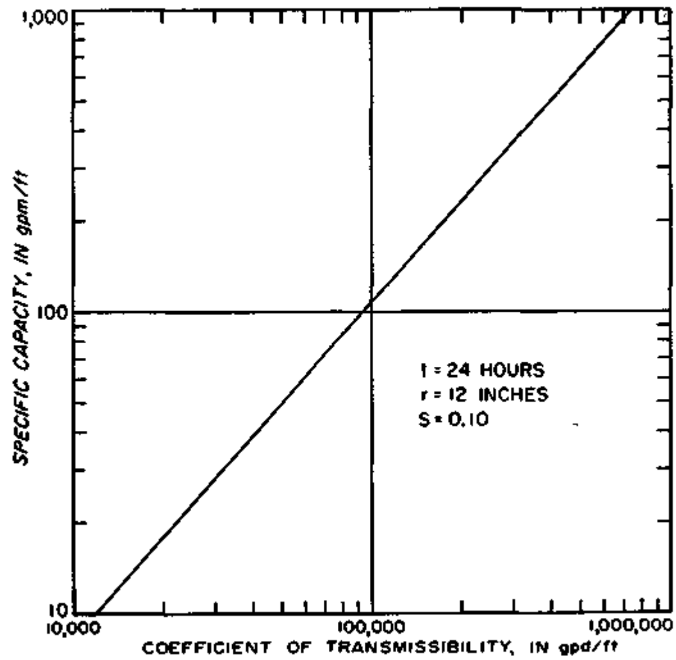


Figure 21. Theoretical relation between specific capacity and the coefficient of transmissibility

in which the well is operated during three successive and equal time periods at constant fractions of full capacity.

Step-drawdown test data are available for nine wells in the East St. Louis area. The results of the step-drawdown tests and construction features of the wells tested are given in table 14. Well-loss constants for wells tested immediately after construction range from 0.2 to 1.0 sec^2/ft^5.

Specific-capacity data collected during well-production tests made on 32 industrial, municipal, and irrigation wells are given in table 15. The well-production tests consisted of pumping a well at a constant rate and frequently measuring the drawdown in the pumped well. Drawdowns were commonly measured with an airline, electric dropline, or steel tape; rates of pumping were largely measured by means of a circular orifice at the end of the pump discharge pipe.

The lengths of tests ranged from 11 minutes to 2 days; pumping rates ranged from 104 to 1905 gpm. Screen diameters ranged from 8 to 32 inches.

Specific-capacity data for 65 selected relief wells are given in table 16. The wells were tested during the period 1952 through 1960 by the U.S. Corps of Engineers. The saturated thickness of the aquifer at well sites was estimated from logs of wells and water-level data. The tests consisted of pumping the wells at a constant rate of 500 gpm for 2 hours and frequently measuring the drawdown in the pumped well.

A coefficient of storage in the water-table range (0.10) estimated from aquifer-test data and several values of t and r_w were used (see Walton 1962) to determine the relationship between specific capacity and the coefficient of transmissibility for various values of r_w^2/t (figure 22). Specific capacities, data concerning the lengths of tests and radii of wells in tables 15 and 16, and figure 23 were used to estimate theoretical coefficients of transmissibility of the aquifer within the cones of depression of production wells. Theoretical coefficients of permeability within the cones of depression were estimated by dividing the coefficient of transmissibility by the average saturated thickness of the aquifer within cones of depression The average saturated thickness of the aquifer within cones of depression was estimated from logs of wells and water-level data. No great accuracy is implied for the coefficients of permeabilities estimated from specific-capacity data because they are based on an estimated coefficient of storage and are not corrected for well-loss and partial penetration losses. However, as shown in table 14, well-loss constants for most newly constructed wells are small. Most wells penetrate completely the more permeable parts of the aquifer. Thus, well and partial penetration losses were probably small and not significant. The data in tables 15 and 16 can be considered only rough approximations of the coefficient of permeability of the aquifer. However, the coefficients of permeability in the Monsanto area estimated from specific-capacity data agree closely with the coefficients of permeability computed from aquifer tests at the Mobil Oil Refinery and the Monsanto Chemical Corporation, indicating that the estimated coefficients of permeability are meaningful.

Water-level and pumpage data for existing pumping centers were used to compute pumping center specific capacities given in table 17. Pumping center specific capacity is here defined as the total pumpage from wells within the pumping center per foot of average drawdown within the pumping center.

Summary of Aquifer-Test Data

A map showing how the coefficient of permeability varies within the East St. Louis area (figure 23) was

Table 14. Results of Step-Drawdown Tests

Owner	Driller	Screen length (ft)	Screen diameter (in)	Screen Material	Date well drilled	Date of test	Well-loss constant (sec^2/ft^5)
Mobil Oil Co.	Luhr Bros.	36	16	Johnson Everdur No. 50 slot	12/59	10/61	2.0
Southwestern Campus of SIU, Edwardsville	Layne-Western	20	10	Slotted pipe	11/60	12/60	0.2
Collinsville (V)	Layne-Western	30	16	Layne No. 4 slot	8/50	8/50	0.7
Thomason	Thorpe		32 X 40	Porous concrete	5/54	5/54	0.2
Amos Bonham	Thorpe		30 X 40	Porous concrete	10/54	4/55	0.45
Herbert Bischoff	Thorpe	60	30 X 40	Porous concrete	1/54	5/54	0.5
V. W. Eckmann	Thorpe	48	30 X 40	Porous concrete	9/54	10/54	1.0
East St. Louis Drainage Dist.	Luhr Bros.			Wood	4/55	5/55	1.0
East St. Louis Drainage Dist.	Luhr Bros.			Wood	4/55	5/55	1.0

Table 15. Specific-Capacity Data for Industrial, Municipal, and Irrigation Wells

Well number	Owner	Depth (ft)	Diameter (in)	Screen length (ft)	Date of test	Length of test (min)	Non-pumping water level (ft below land surface)	Pumping rate (gpm)	Drawdown (ft)	Specific capacity (gpm/ft)	Coefficient of transmissibility (gpd/ft)	Saturated thickness of aquifer (ft)	Coefficient of permeability (gpd/sq ft)
MAD—													
5N10W-													
13.5c	Owens Illinois Glass Co.	68	26	20	3/40	390	9.3	1263	14.5	87	135,000	59	2300
5N9W-													
16.5b2	Olin Mathieson Chemical Corp.	90	26		11/33	60	46	560	9	62.2	62,000	44	1410
21.4h	Olin Mathieson Chemical Corp.	97	26		2/55	1440	56	300	11.7	25.6	42,000	41	1025
19.8h	Alton Boxboard Co.	110	26	35	1/57	480	30	1905	7.17	266	370,000	80	4630
22.1bl	Bethalto (V)	94	32	54	3/42	1440	41.5	320	5.2	62	80,000	53	1510
22.1b2	Bethalto (V)	94	32		3/42	600	41.5	305	4.25	72	100,000	53	1890
22.2c	Bethalto (V)	93	30	48	5/51	375	40	460	7	65	83,000	53	1570
26.8gl	City of Wood River	116	12	40	4/43	385	47.3	730	11	66.4	105,000	69	1520
26.8g2	City of Wood River	112	12	30	4/43	505	45.7	405	6	73.6	115,000	66	1740
26.8g3	City of Wood River	110	16	41	4/56	48	58	925	6	154	200,000	52	3850
26.8g4	City of Wood River	112	16	40	10/57	11	60.9	758	6.5	117	150,000	61	2460
27.1b	Roxana (V)	126	32	72	3/37	1440	48	530	6	88	120,000	78	1540
34.7c	International Shoe Co.	117	12	25	4/51	540	35	1125	17	61	94,000	82	1150
MAD—													
4N9W-													
13.1cl	Edwardsville (V)	112	16	41	2/40	45	25.5	1650	19	87	98,000	96	1020
29.7b	J. Thomason	106	30	60	4/54	100	24	1000	5.48	182	210,000	82	2560
3N8W-													
5.2f2	Glen Carbon (V)	63	30	48	5/56	150	30	104	7	14.9	19,000	33	575
29.3hl	Troy (V)	115	10	20	2/53	2880	25	420	6.35	66	120,000	90	1330
29.3h2	Troy (V)	115	12		11/60	60	25	325	3.1	105	110,000	90	1220
30.7b	V. W. Eckmann	104	30	48	10/54	40	20.3	468	5.38	87	180,000	64	2810
31.2a2	Collinsville (C)	104	26	30	8/58	480	18.8	1150	17.7	68	105,000	103	1020
31.2a3	Collinsville (C)	98	18	30	9/50	30	20.5	627	4.8	130	165,000	77	2150
31.2a4	Collinsville (C)	98	26	25	8/55	255	29.0	1001	11.0	91	130,000	69	1890
3N9W-													
5.8b	Herbert Bischoff	110	30	60	4/54	70	31.0	820	5.1	161	180,000	79	2280
6.3c	Herbert Bischoff	110	30	60	5/54	35	28.3	1120	7.88	140	140,000	82	1710
14.2c	W. Hanfelder	102	12	32	11/56	190	22.1	768	15.5	49.5	70,000	80	875
17.2a	Udell Bischoff	106	30	60	5/54	95	30.6	1150	5.78	199	230,000	75	3170
STC—													
2N8W-													
6.5h	Amos Bonham	106	30		4/55	60	28.4	470	6.55	72	90,000	67	1340
6.8d	E. A. Weissert	105	12	20	9/54	60	27	450	6	75	77,000	78	990
2N9W-													
1.3f	Mounds Public Water Dist.	90	8	10	7/58	900	8	349	10	34.9	60,000	82	730
2N10W-													
1.3a4	National Stock-yards Co.	110	18	40	5/61	360	37	1248	8.18	152.5	200,000	73	2740
1.3a5	National Stock-yards Co.	110	18	40	5/61	300	37	1230	6.55	188	250,000	73	3430
12.6g	Royal Packing Co.	100	12	40	1/59	475	33	475	3	158	190,000	67	2840

Table 16. Specific-Capacity Data for Selected Relief Wells

Well number	Date of test	Specific capacity (gpm/ft)	Coefficient of transmissibility (gpd/ft)	Estimated saturated thickness (ft)	Coefficient of permeability (gpd/sq ft)	Relief well number
Wood River (upper) Drainage District						
MAD—						
5N9W-						
13.2a	8/54	115	135,000	90	1500	41X
13.6d	8/54	238	305,000	60	5100	16
14.1e	9/54	62	67,000	60	1120	1
19.3c	1/55	96	110,000	96	1450	100
19.6e	1/55	156	190,000	96	1980	87XX
Wood River (lower) Drainage District						
5N9W-						
20.5a	10/60	114	134,000	80	1670	105
28.4c	10/60	101	118,000	100	1180	146
28.8e	10/60	89	100,000	100	1000	138
29.4g	10/60	65	72,000	100	720	121

WeU number	Date of test	Specific capacity (gpm/ft)	Coefficient of transmissibility (gpd/ft)	Estimated saturated thickness (ft)	Coefficient of permeability (gpd/sq ft)	Relief well number
East St. Louis (Chain of Rocks) Drainage District						
MAD—						
4N9W-						
20.3g	8/52	94	108,000	85	1270	196
20.4e	6/52	66	72,000	90	800	184
20.5c	5/52	88	100,000	90	1110	175
20.6a	6/52	93	105,000	90	1160	170
29.7g	9/52	68	75,000	85	880	169
29.8d	9/52	79	88,000	80	1100	161
30.1b	9/52	66	72,000	75	960	155
31.2h	8/52	92	104,000	75	1390	150
31.3f	8/52	91	102,000	75	1360	144
31.3g	8/52	56	60,000	75	800	145
31.5c	7/52	91	102,000	70	1460	141
31.6a	7/52	77	86,000	70	1230	126

Table 16 (Continued)

Well number	Date of test	Specific capacity *(gpm/ft)*	Coefficient of trans-missibility *(gpd/ft)*	Esti-mated satu-rated thick-ness *(ft)*	Coeffi-cient of permea-bility *(gpd/ sq ft)*	Relief well number
MAD—3N9W-						
6.7g	7/52	56	60,000	70	857	117
6.8e	8/52	91	103,000	70	1470	108
3N10W-						
1.1c	7/52	22	21,000	70	300	98
12.4f	6/52	103	120,000	70	1720	69
12.6c	6/52	49	52,000	70	743	56
13.8g	4/52	58	62,000	50	1240	38
14.1£	9/52	38	39,000	50	780	33
14.2d	4/52	31	31,000	50	620	26
23.5g	9/52	44	46,000	45	1020	5
East St. Louis Drainage District						
MAD—4N8W-						
7.3a	11/58	172	212,000	80	2660	1
4N9W-						
14.8h	10/58	61	66,000	85	780	3
3N10W-						
22.1a	7/55	15	14,000	30	456	43
23.6c	6/55	41	43,000	40	1070	7
26.6b	7/55	33	33,000	40	825	78
26.7d	7/55	25	24,000	40	600	70
26.8e	7/55	64	70,000	35	2000	64
26.8h	7/55	34	35,000	30	1170	53
35.6f	7/55	39	40,000	45	890	96
35.6h	7/55	36	37,000	45	823	87
STC—2N10W-						
11.4e	10/54	131	156,000	75	2080	131
14.4h	10/54	95	110,000	85	1290	107
23.5h	8/55	156	190,000	90	2110	124
23.6c	8/55	143	175,000	85	2060	129
23.6f	7/55	143	175,000	80	2190	118
23.7a	8/55	139	165,000	80	2060	136
34.5h	8/55	236	300,000	90	3340	159
34.6e	10/54	109	125,000	95	1320	159
34.7c	10/54	151	182,000	95	1910	169
1N10W-						
4.1g	10/54	89	100,000	95	1050	196
4.2e	9/54	113	134,000	90	1490	207
4.3b	11/54	142	175,000	85	2060	237
9.1f	10/58	66	72,000	100	720	262
9.2h	10/58	116	136,000	95	1430	251
10.1c	10/58	125	148,000	80	1850	273
10.4c	10/58	104	120,000	80	1500	263
12.5b	10/58	132	160,000	65	2460	278
13.3h	10/58	126	150,000	60	2500	286
Prairie Du Pont Drainage District						
STC—1N10W-						
4.7b	10/54	126	150,000	70	2140	23
8.2h	10/54	148	180,000	55	3280	28
8.5c	10/54	84	96,000	80	1200	34
8.7a	10/54	103	120,000	65	1850	45
9.4h	10/54	125	150,000	70	2140	15
19.6f	11/54	91	103,000	60	1720	46
30.6h	10/54	130	154,000	85	1810	55

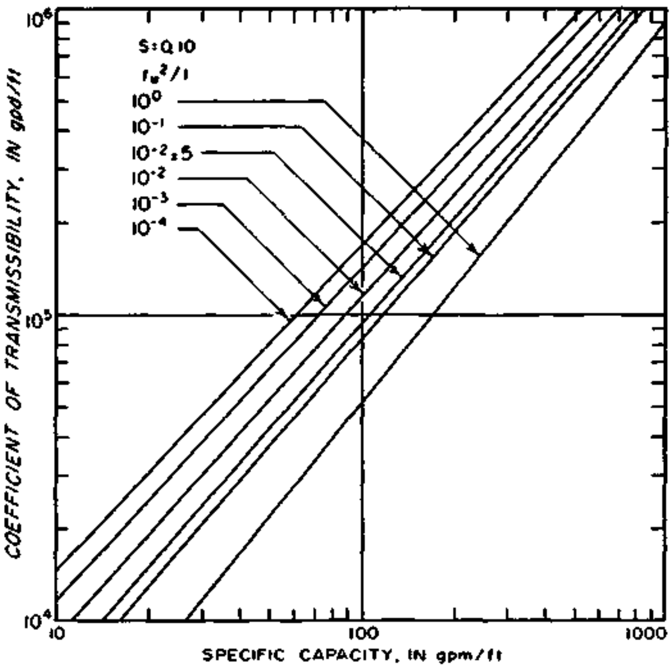


Figure 22. Coefficient of transmissibility versus specific capacity for several values of well radius and pumping period

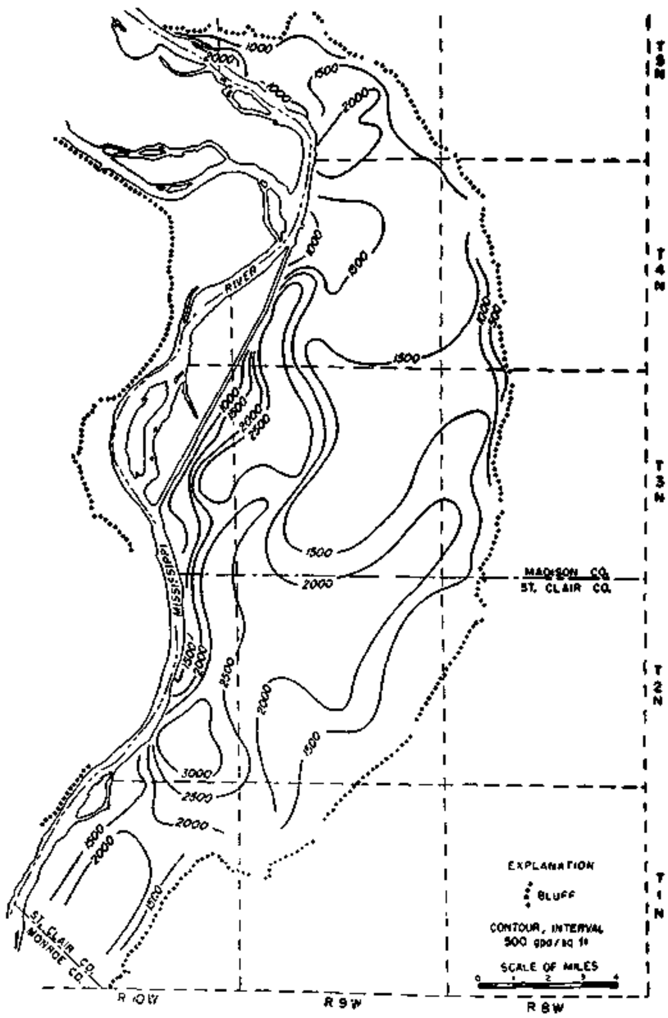


Figure 23. Coefficient of permeability of aquifer

Table 17. Pumping Center Specific-Capacity Data

Pumping center	Pumpage in 1961 (mgd)	Average drawdown (ft)	Specific capacity (epd/ft)
Alton	5.1	20	255,000
Wood River	13.5	40	338,000
Granite City	8.8	15	586,000
National City	10.8	20	540,000
Monsanto	20.5	50	410,000

prepared from data in tables 14, 15, and 16. The coefficient of permeability is high in narrow strips extending from Monsanto north through National City and extending through Granite City northeasterly along the Chain of Rocks Canal. The coefficient of permeability is greatest locally in the Monsanto area, exceeding 3000 gpd/sq ft. The coefficient of permeability is estimated to be greater than 2000 gpd/sq ft south of Alton (along the Mississippi River) in the Wood River area, in a wide area extending from Monsanto northeast to just south of Horseshoe Lake, and in the Dupo area. The coefficient of permeability is less than 1000 gpd/sq ft in an area extending south from near the confluence of the Missouri and Mississippi Rivers to north of Horseshoe Lake. The coefficient of permeability decreases rapidly near the bluffs and west of the Chain of Rocks Canal.

A map showing the saturated thickness of the aquifer (figure 24) was prepared from the bedrock surface map (figure 6), water-level data for November 1961, and a map showing the elevation of the base of the alluvium. The saturated thickness of the aquifer is greatest and exceeds 100 feet in the bedrock valley bisecting the East St. Louis area. It is least along the bluffs and west of Chain of Rocks Canal.

A map showing how the coefficient of transmissibility varies within the East St. Louis area (figure 25) was prepared from figures 23 and 24. The coefficient of transmissibility ranges from less than 50,000 gpd/ft near the bluff and the southern part of the Chain of Rocks Canal to greater than 300,000 gpd/ft near Monsanto.

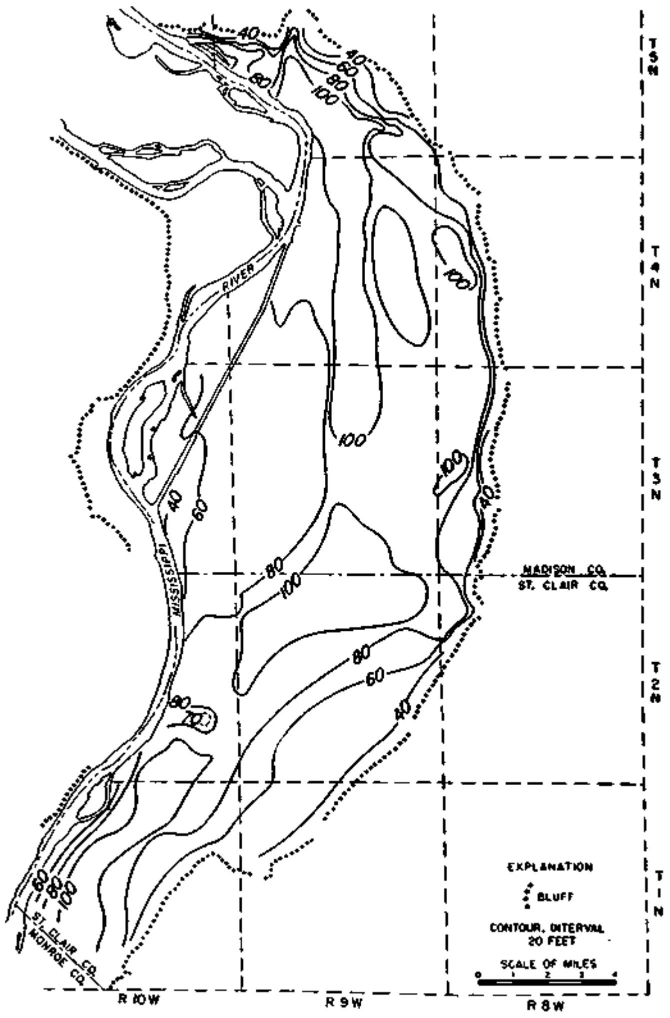


Figure 24. Saturated thickness of aquifer, November 1961

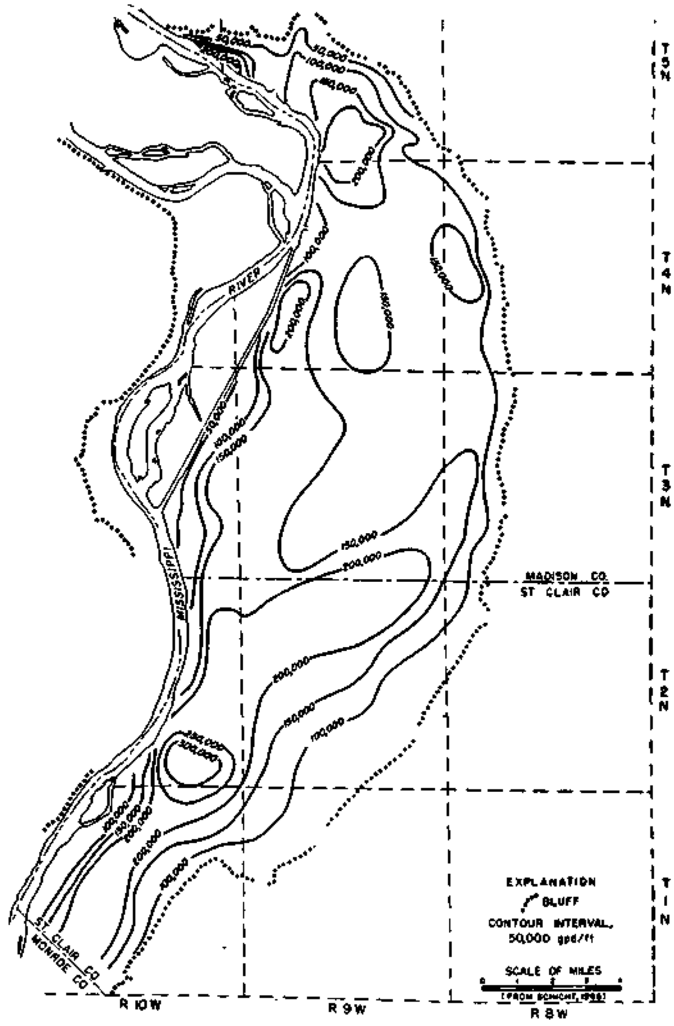


Figure 25. Coefficient of transmissibility of aquifer

Large capacity wells in the East St. Louis area are drilled by the cable tool method, the reverse hydraulic rotary method, or by clam shell type diggers. Collector wells have been constructed in the East St. Louis area by several industries. Most domestic and some irrigation wells are driven; a few dug wells are still used for domestic supplies.

Industrial, municipal, and irrigation wells are usually drilled to bedrock or bit refusal. Several wells just south of Alton terminate at the top of clayey and silty material immediately above bedrock. According to Bergstrom and Walker (1956) the maximum thickness of the clayey and silty material is 25 feet. Production wells are usually cased through the finer alluvial deposits in the upper part of the valley fill and have perforated pipe sections or commercial screens opposite the lower coarser alluvium or valley-train deposits. There are two types of drilled wells in the area: natural pack and artificial pack. Materials surrounding the well are developed in place in the case of the natural pack well; materials having a coarser and more uniform grain size than the natural formation are added around the well in the case of the artificial pack well. As shown in table 18, the thickness of the pack in wells in the area generally ranges from 6 to 11 inches.

Table 18. Construction Features of Selected Wells

			Screen Record				
Depth (ft)	Casing depth (ft)	Casing diameter (in)	Length (ft)	Diameter (in)	Material or manufacturer	Slot number or size (in)	Artificial pack thickness (in)
103	0-73	26	30	26	Everdur Johnson	30	11
110	0-34	26 36	76	26 36	Porous concrete		none
85	0-49	30 40	36	30 40	Porous concrete		none
95	0-47	26 36	48	26 36	Porous concrete		none
120	0-76	16	44	16	Slotted pipe	¼ X 2½	9.5
108	0-73	24	35	24	Everdur Johnson	60 100	6.0
100	0-63	14	37	12	Slotted pipe	¼ X 2½	7.0
105	0-85	12	20	12	Slotted pipe		6.0
111	0-81	16	30	16	Cook	20 30	
114	0-84	16	32	16	Cook	20 40	
111	0-81	16	30	16	Cook	20 40 80	
98	0-78	18	20	18	Layne Shutter	4	
115	0-85	16	30	16	Cook	30	6.0
105	0-89	10	16	10	Cook		none
115	0-100	12	15	12	Johnson	60	none

Several types of well screens have been used in the East St. Louis area. Porous concrete, wood, slotted pipe, and commercial screens are in use. Economic considerations rather than proper well design criteria have governed the types of screens in use. Screen diameters generally vary in diameter from 6 to 30 inches, and screens vary in length from 5 to 76 feet. Screen slot openings vary depending upon the characteristics of the formations encountered or the characteristics of the artificial pack.

Ten collector wells have been constructed in the East St. Louis area, and six are still in use. Four collector wells at the Granite City Steel Company were not in continuous operation in 1962, but were tested periodically and operated occasionally during the summer months. The collector well consists of a large diameter, reinforced concrete caisson from which horizontal screen laterals project radially near the bottom. The standard caisson is 13 feet in diameter. The horizontal screen laterals are fabricated from heavy steel plate, perforated with longitudinal slots, and may be 8 to 24 inches in diameter and 100 to 450 feet in length, depending upon geologic conditions and design of the unit (Mikels and Klaer, 1956).

Thorpe concrete wells are in wide use by municipalities, industries, and irrigation well owners. Thorpe concrete wells consist of a concrete casing and porous concrete screen either 26 or 30 inches in inside diameter with walls 5 inches thick. Lengths of screen vary from 24 to 76 feet. Thorpe concrete wells have been in operation for as long as 35 years. However, in some cases Thorpe concrete wells have been abandoned because of reduction in yield after a few months operation.

Driven wells are usually not greater than 50 feet in depth depending upon the thickness of the alluvium overlying the coarser sand and gravel deposits. The driven wells consist of lengths of 1.25- or 2-inch diameter pipe with a drive (or sand) point at the lower end of the pipe.

About 500 relief wells were drilled in the East St. Louis area by the U.S. Corps of Engineers near and on the land side of levees fronting the Mississippi River to control underseepage beneath levees during floods. Several artificial pack relief wells were also drilled along the Cahokia Diversion Channel. Relief wells in the area range in depth from 47 to 103 feet. Casings and screens are 8 inches in diameter and the pack thickness is about 7 inches. The screens are constructed from redwood or treated Douglas Fir and range in length from 19 to 71 feet. The screens are spiral wound with No. 6 gage galvanized wire and have 18 slots, 3/16 by 3 1/4 inches per spiral.

Slotted pipe screens are widely used in irrigation wells in the East St. Louis area because of their low cost. In comparison, only a few industrial and municipal wells contain slotted pipe screens. Irrigation wells range in diameter from 8 to 16 inches and usually have pack thicknesses of 6 to 8 inches. Lengths of slotted pipe screens range from 10 to 40 feet.

Service Life of Wells and Collector Wells

One of the problems in the East St. Louis area associated with the development of ground-water resources is the short life expectancy of wells. According to a study by Bruin and Smith (1953), the median service life of municipal wells terminating in sand and gravel formations in the East St. Louis area is about half that for similar municipal wells in other parts of the state. Nearly all of the wells retired in the area were taken out of service either because the screens had become partially clogged or the wells had filled with sand.

The results of mechanical analyses presented by Bergstrom and Walker (1956) are shown in figures 26 through 28. According to Bergstrom and Walker the analyses must be accepted with caution because the conditions of collecting most of the samples are not known, and because of the highly variable nature of the valley-fill deposits in the area. A careful examination of the mechanical analysis curves suggests that the valley-fill deposits contain a rather high percentage of fine materials which could, under heavy pumping conditions, migrate toward a screen and partially clog the well wall and screen openings. As indicated by data in the files of industries and municipalities, specific capacities of existing production wells decrease markedly after a few years and in some cases after a few months of operation. Specific capacities are generally determined by the driller after completion of the well by pumping the well at different rates for short periods of time, generally less than 24 hours, and by frequently measuring drawdowns in the pumped well. This method of measuring specific capacity is continued by industrial and municipal personnel periodically.

It is a general practice of industries and municipalities to place a well in operation and pump it at high rates, often about 1000 gpm. As the result of heavy pumping, fine materials migrate towards the well and partially clog screen openings and the voids of the formation surrounding the well. The well-loss constant increases rapidly and, because well loss varies as the square of the discharge rate, drawdown increases rapidly. The relation between well-loss constant and drawdown due to well loss is shown in figure 29. As drawdown increases the specific capacity and, therefore, the yield of the well decreases. Typical decreases in specific capacity due to increases in the well-loss constant are given in table 19.

Theoretical specific capacities of wells with a nominal radius of 15 inches and with 40 feet of screen given in table 19 were determined for values of the coefficient of transmissibility ranging from 100,000 to 300,000 gpd/ft,

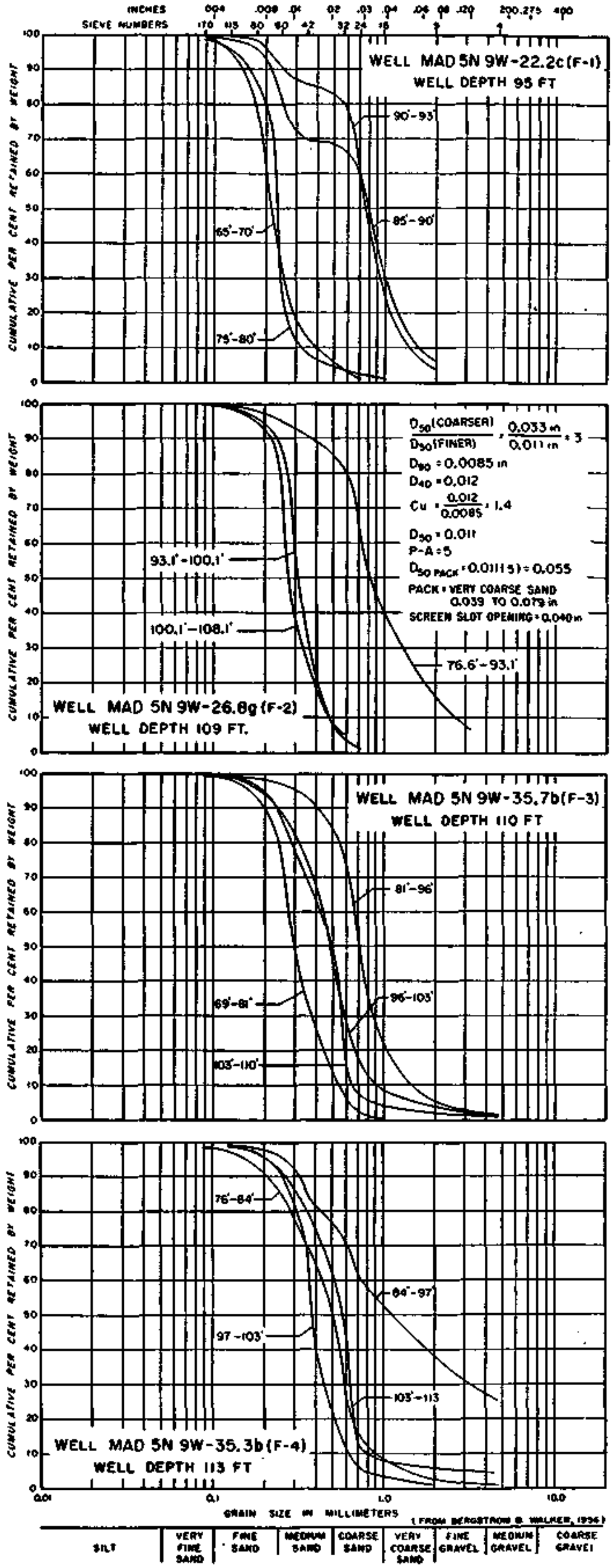


Figure 26. Mechanical analyses of samples from wells

a coefficient of storage of 0.10, a pumping period of 12 hours, pumping rates of 900 or 450 gpm, well-loss constants of 1, 5, and 10 sec^2/ft^5. The effects of dewatering and partial penetration (see Walton, 1962) were taken into consideration in computations.

Computed well-loss coefficients for wells tested immediately after construction (table 14) range from 0.2 sec^2/ft^5 to 1.0 sec^2/ft^5 and meet requirements suggested by Walton (1962) that the value of *G* of a properly developed and designed well should be less than 5 sec^2/ft^5. According to Walton (1962), values of *C* between 5 and 10 sec^2/ft^5 indicate mild deterioration, and clogging is severe when *C* is greater than 10 sec^2/ft^5. It is difficult

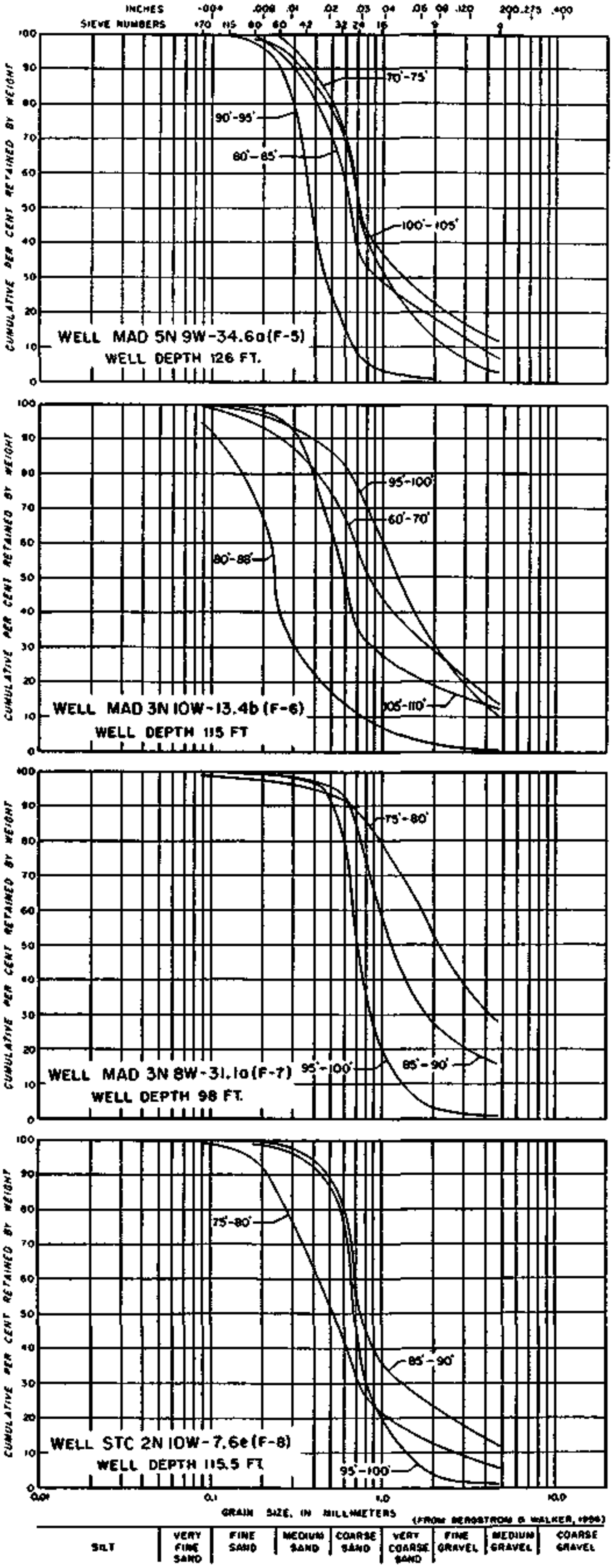


Figure 27. Mechanical analyses of samples from wells

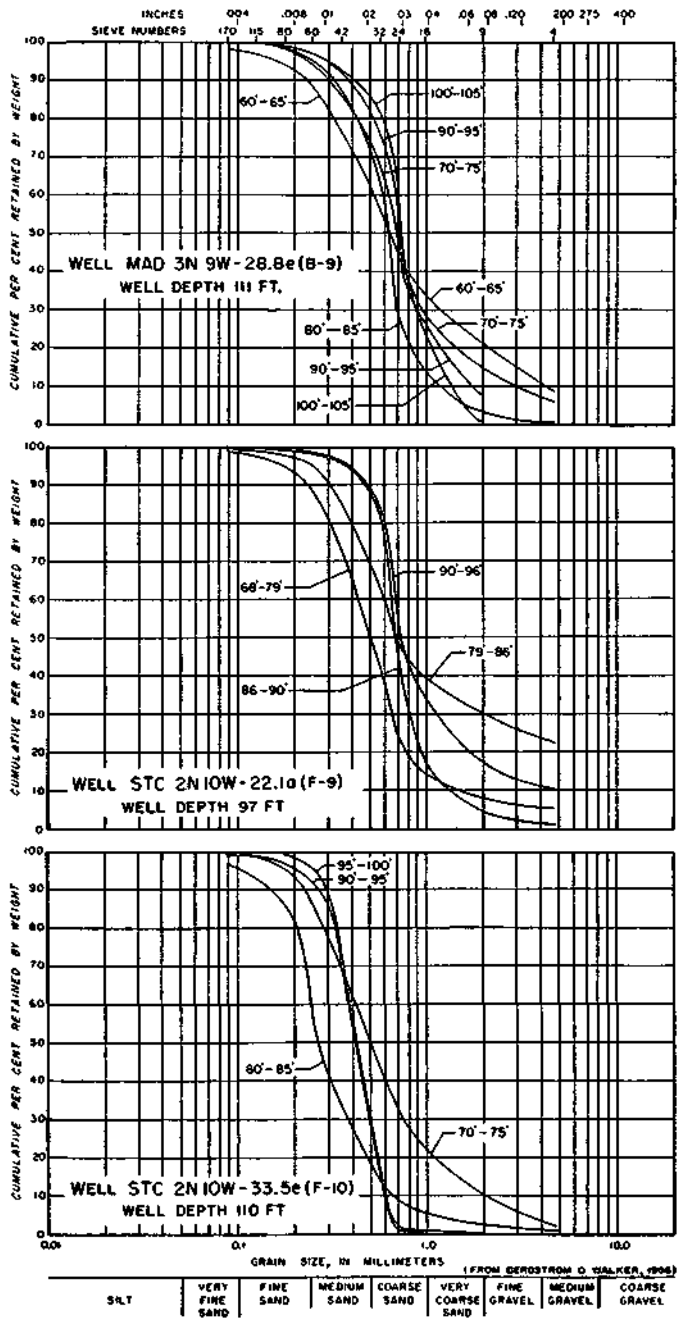


Figure 28. Mechanical analyses of samples from wells

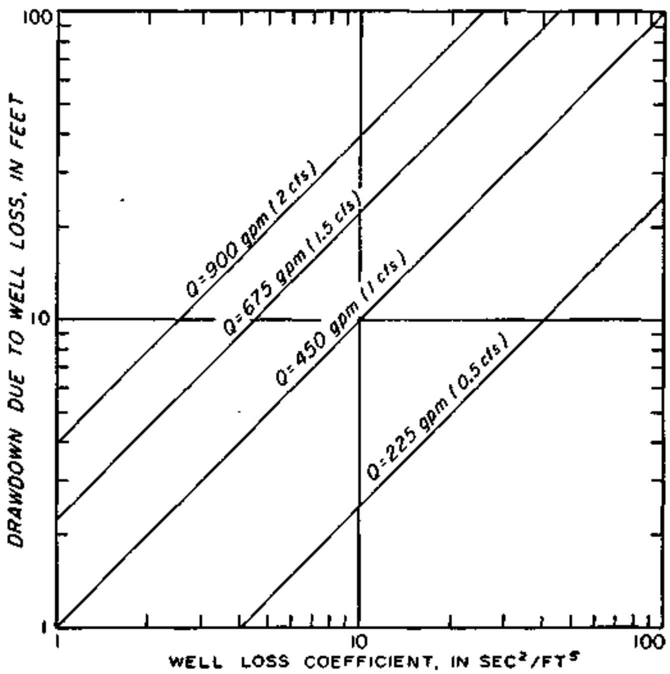


Figure 29. Relation between well-loss constant and drawdown due to well loss

and sometimes impossible to restore the original capacity if the well-loss constant is greater than 40 sec^2/ft^5.

Periodic well treatment by acidizing or other methods has been used successfully to rehabilitate old wells. However, in many cases wells are abandoned as their yields decrease and new wells are drilled nearby.

Based on data for production wells which have been in service a number of years, the average specific capacity of wells in the East St. Louis area is about 30 gpm/ft. An average well yield of 450 gpm can be obtained with a long service life if sufficient screen is provided.

A graph showing the decrease of specific capacity of a collector well owned by the Shell Oil Refinery near the city of Wood River is given in figure 30. The specific capacity of the collector well declined from a peak of 270 gpm/ft in August 1954 to about 50 gpm/ft in March 1963. A part of the decline in specific capacity can be attributed to the partial clogging of the laterals by incrustation and with sand and silt. Mechanical cleaning of one lateral in June 1962 increased the specific capacity from about 50 gpm/ft to 55 gpm/ft.

Table 19. Theoretical Decreases in Specific Capacity Due to Increases in Well-Loss Constant

Coefficient of transmissibility *(gpd/ft)*	Pumping rate *(gpm)*	Well-loss coefficient of 1 sec^2/ft^5 Drawdown* *(ft)*	Specific capacity* *(gpm/ft)*	Well-loss coefficient of 5 sec^2/ft^5 Drawdown* *(ft)*	Specific capacity* *(gpm/ft)*	Well-loss coefficient of 10 sec^2/ft^5 Drawdown* *(ft)*	Specific capacity* *(gpm/ft)*
300,000	900	9.3	96.9	25.3	35.6	45.3	19.9
250,000	900	10.3	87.4	26.3	34.2	46.3	19.4
200,000	900	11.9	75.6	27.9	32.2	47.9	18.8
150,000	900	14.4	62.5	30.4	28.6	50.4	17.9
100,000	900	19.7	45.7	35.7	25.2	55.7	16.1
300,000	450	3.7	122.2	7.7	58.4	12.7	35.4
250,000	450	4.2	110.7	8.2	54.9	13.2	34.1
200,000	450	4.9	91.9	8.9	50.6	13.9	32.4
150,000	450	6.1	73.8	10.1	44.5	15.1	29.8
100,000	450	8.4	53.6	12.4	36.3	17.4	25.9

**Theoretical*

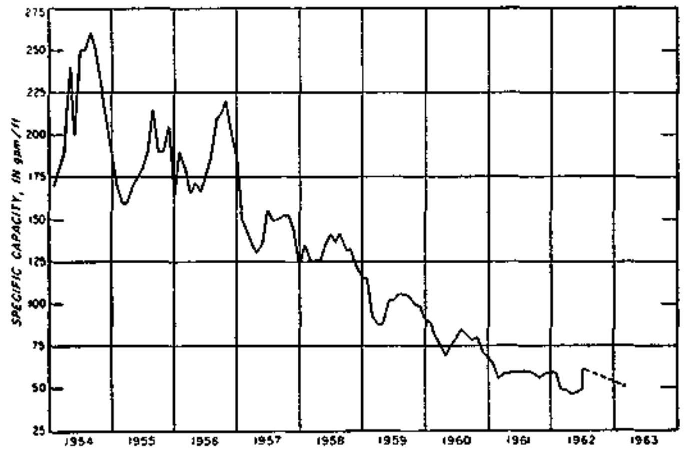


Figure 30. Specific-capacity data for collector well, 1954 to March 1963

Well Design Criteria

Walton (1962) gave criteria for well design in unconsolidated formations in Illinois. Screen design criteria are applicable to industrial, municipal, and irrigation wells. The objective is to design an efficient and economical well with a service life of at least 10 years.

According to Ahrens (1957) artificial pack wells are usually justified when the aquifer is homogeneous, has a uniformity coefficient less than 3.0, and/or has an effective grain size less than 0.01 inch. The uniformity coefficient, C_u, is the ratio of the sieve size that will retain 40 percent of the aquifer materials to the effective size. The sieve size that retains 90 percent of the aquifer materials is the effective size. In addition, an artificial pack is sometimes needed to stabilize well-graded aquifers having a large percentage of fines in order to avoid excessive settlement of materials above the screen or to permit the use of larger screen slots. The uniformity coefficients based on mechanical analyses of samples in figures 26 through 28 are less than 3 and/or the effective grain size is less than 0.01 inch, indicating that an artificial pack well should be constructed at each site.

Selection of the artificial pack is based on the mechanical analysis of the aquifer. A criterion that has been successfully used in Illinois is that the ratio of the 50 percent sizes of the pack and the aquifer (the P-A ratio) be 5 (Smith, 1954). Artificial packs should range in thickness from 6 to 9 inches (Walton, 1962).

To avoid segregation or bridging during placement, a uniform grain size pack should be used. The screen slot opening should be designed so that at least 90 percent of the size fractions of the artificial pack are retained.

A well sometimes encounters several layers of sand and gravel having different grain sizes and gradations. If the 50 percent size of the materials in the coarsest aquifer are less than 4 times the 50 percent size of the materials in the finest aquifer, the slot size and pack, if needed, should be selected on the basis of the mechanical analysis of the finest material (Ahrens, 1957). Otherwise, the slot size and pack should be tailored to individual layers.

One of the most important factors in the design of natural pack well screens is the width or diameter of the screen openings, referred to as slot size. With a uniformity coefficient greater than 6 (a heterogeneous aquifer) and in the case where the materials overlying the aquifer are fairly firm and will not easily cave, the sieve size that retains 30 percent of the aquifer materials is generally selected as the slot size. With a uniformity coefficient greater than 6 and in the case where the materials cave, the sieve size that retains 50 percent of the aquifer materials is selected as the slot size (Walton, 1962). With a uniformity coefficient as low as 3 (a homogeneous aquifer) and in the case where the materials overlying the aquifer are fairly firm and will not easily cave, the sieve size that retains 40 percent of the aquifer materials is selected as the slot size. With a uniformity coefficient as low as 3 and in the case where the materials overlying the aquifer are soft and will easily cave, the sieve size that retains 60 percent of the aquifer materials is selected as the slot size.

The screen length is based in part on the effective open area of a screen and an optimum screen entrance velocity. According to Walton (1962), to insure a long service life by avoiding migration of fine materials toward the screen and clogging of the well wall and screen openings, screen length is based on velocities between 2 and 12 feet per minute (fpm).

The length of screen for a natural pack well is selected from the coefficient of permeability of the aquifer determined from aquifer tests by using table 20 and the following equation (Walton, 1962):

$$L_s = Q/A_e V_c (7.48) \tag{9}$$

where:

L_s = required length of screen, in ft
Q = discharge, in gpm
A_e = effective open area per foot of screen, in sq ft
V_c = optimum entrance velocity, in fpm

On the average about one-half the open area of the screen will be blocked by aquifer materials. Thus, the effective open area averages about 50 percent of the actual open area of the screen.

Table 20. Optimum Screen Entrance Velocities*

Coefficient of permeability *(gpd/sq ft)*	Optimum screen entrance velocities ***(fpm)***
>6000	12
6000	11
5000	10
4000	9
3000	8
2500	7
2000	6
1500	5
1000	4
500	3
< 500	2

**From Walton (1962)*

The results of studies involving the mechanical analyses of samples of the aquifer collected at two sites demonstrate some of the principles involved in the design of sand and gravel wells. Suppose that it is desired to design a 16-inch diameter well based on the mechanical analysis of samples for well MAD 5N9W-26.8g (see figure 26). Since the ratio of the 50 percent grain size of the coarser material from 76.6 to 93.1 feet to the 50 percent grain size of the finer material from 93.1 to 108.1 feet is less than 4, the screen or pack must be designed on the basis of results of analysis of the finer materials. The uniformity coefficient of the finer materials is less than 3 and the effective grain size is less than 0.01 inches, indicating that an artificial pack well should be used. The 50 percent size of the materials of the finest sample is 0.011 inch; thus, with a pack-aquifer, ratio of 5, a very coarse sand pack with particles ranging in diameter from about 0.04 to 0.08 inch is indicated. To retain 90 percent of the size fractions of the pack a slot size of 0.040 inch would be required. An artificial pack thickness of 6 inches is adequate.

For demonstration of the design of a natural pack well, consider the grain-size distribution curves in figure 31. The mechanical analyses are for samples taken from

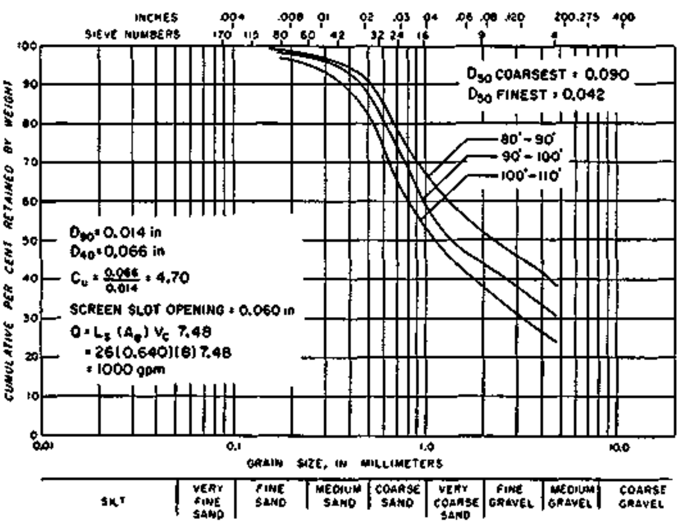


Figure 31. Mechanical analyses of samples for test hole

a test hole near Monsanto. The coefficient of permeability of the aquifer in the vicinity of the test hole was estimated to be 3000 gpd/sq ft from aquifer-test data. The 50 percent size of the materials in the finest sample is less than 4 times the 50 percent size of the materials in the coarsest sample; therefore, the slot size should not be tailored to individual samples but should be based on the mechanical analysis of the finest sample. The effective grain sizes of all three samples are greater than 0.01 and uniformity coefficients are greater than 3. A natural pack well is therefore indicated. The materials overlying the aquifer will not easily cave so the sieve size (0.060 inch) that retains 40 percent of the aquifer materials is selected as the proper slot size.

Suppose a pumping rate of 1000 gpm is desired. Computations made with equation 9, indicate that 26 feet of 16-inch continuous slot screen with a slot opening of 0.060 inches is needed. The effective open area of the screen is estimated to be 0.640 sq ft per foot of the screen. The optimum screen entrance velocity (table 20) is equal to 8 fpm.

Alternate designs to the above example are possible by using a small diameter screen with a longer length or a larger diameter screen with a shorter length.

The following are well diameters that have been used in Illinois (Smith, 1961):

Pumping rate *(gpm)*	Diameter of well *(in)*
125	6
300	8
600	10
1200	12
2000	14
3000	16

Experience has shown that in the case of a multiple well system consisting of more than two wells the proper spacing between wells is at least 250 feet.

GROUND-WATER WITHDRAWALS

The first significant withdrawal of ground water in the East St. Louis area started in the late 1890s. Prior to 1900 ground water was primarily used for domestic and farm supplies; since 1900 pumpage has been mostly for industrial use. The first record of an industrial well in the East St. Louis area is for a well drilled in 1894 by the Big Four Railroad in East Alton (Bowman and Reeds, 1907). The well was 54 feet deep and 8 inches in diameter, and was pumped at an average rate of 75,000 gpd. The water was used primarily in locomotive boilers. The meat packing industry in National City started to pump large quantities of ground water in 1900. According to Schicht and Jones (1962), estimated pumpage from wells in the National City area increased from 400,000 gpd in 1900 to 5.3 mgd in 1910. The first municipal well was drilled in 1899 by Edwardsville at a site near Poag and was pumped at an average rate of 300,000 gpd. The second municipal well was drilled in 1901 by Collinsville at a site about a mile north of Caseyville and was pumped at an average rate of 100,000 gpd. Pumpage from wells in the East St. Louis area from 1890 through 1960 was estimated by Schicht and Jones (1962). Estimated pumpage from wells increased from 2.1 mgd in 1900 to 111.0 mgd in 1956 as shown in figure 32. Pumpage declined sharply from 111.0 mgd in 1956 to 92.0 mgd in 1958 and then gradually increased to 93.0 mgd in 1960. The average rate of pumpage increase for the period 1890 through 1960 was about 1.5 mgd per year.

Pumpage from wells in the East St. Louis area was greatest in 1956, totaling 111.0 mgd. As shown in figure 32 pumpage increased from 93.0 mgd in 1960 to 96.8 mgd in 1961, and increased sharply to 105.0 mgd in 1962.

Pumpage is concentrated in five major pumping centers: the Alton, Wood River, Granite City, National City, and Monsanto areas. Also, there are five minor pumping centers: the Fairmont City, Caseyville, Poag, Troy, and Glen Carbon areas. The distribution of pumpage in 1956 and 1962 are shown in figures 33 and 34 respectively, which also indicate the locations of the pumping centers. As shown in figures 35 and 36, changes in pumpage for the period of record are similar in all major pumping centers. Poor economic conditions are reflected in the decreased pumpage during the years of the late 1920s and early 1930s. The effects of increased production dur-

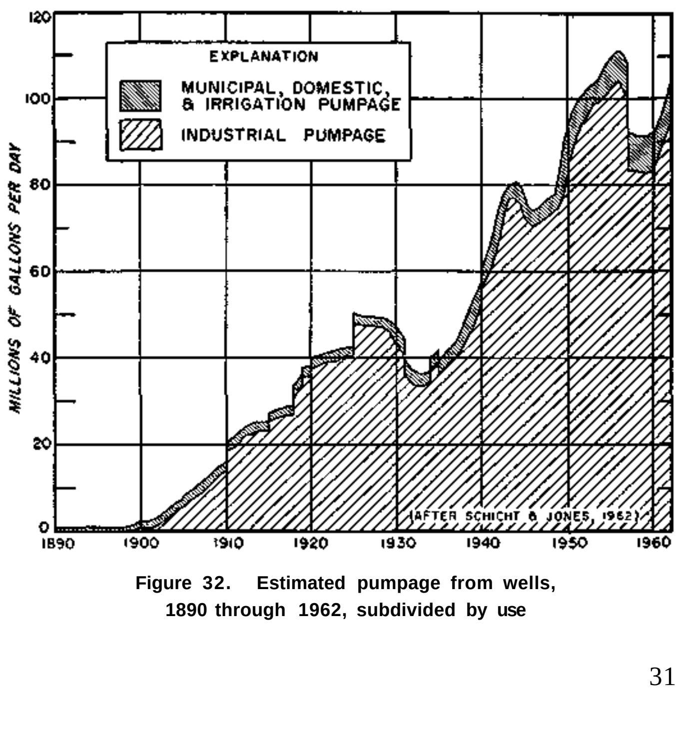


Figure 32. Estimated pumpage from wells, 1890 through 1962, subdivided by use

ing World War II and the post-war reduction in production are evident. There has been a general and gradual increase in pumpage from the five minor pumping centers throughout the period of record as shown in figure 37.

The distribution of pumpage from wells in 1956, 1960, 1961, and 1962 is shown in table 21. The greatest

Table 21. Distribution of Pumpage from Wells

Pumping center	Total pumpage (mgd) 1956	1960	1961	1962
Alton area	9.8	13.6	12.3	13.9
Wood River area	21.1	20.9	24.3	25.5
Granite City area	30.1	7.9	8.8	9.5
National City area	13.8	9.6	10.8	11.6
Monsanto area	30.1	33.2	31.9	35.4
Fairmont City area	2.4	3.2	4.4	4.5
Caseyville area	2.3	2.6	2.4	2.5
Poag area	0.9	1.2	1.2	1.2
Troy area	0.3	0.5	0.4	0.5
Glen Carbon area	0.2	0.3	0.3	0.4
Total	111.0	93.0	96.8	105.0

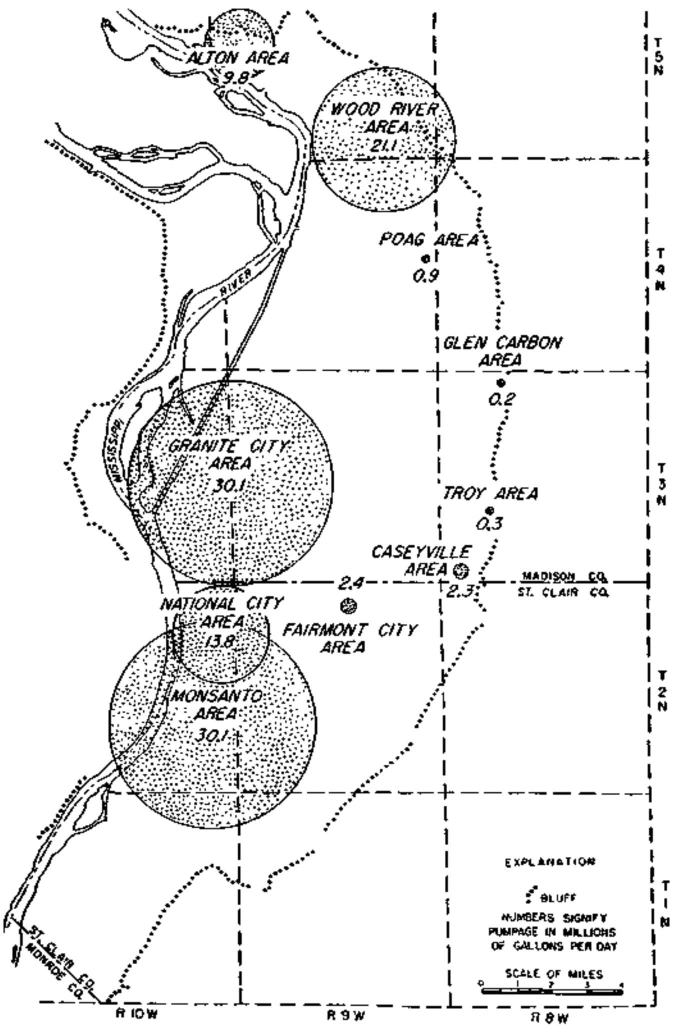


Figure 33. Distribution of estimated pumpage in 1956

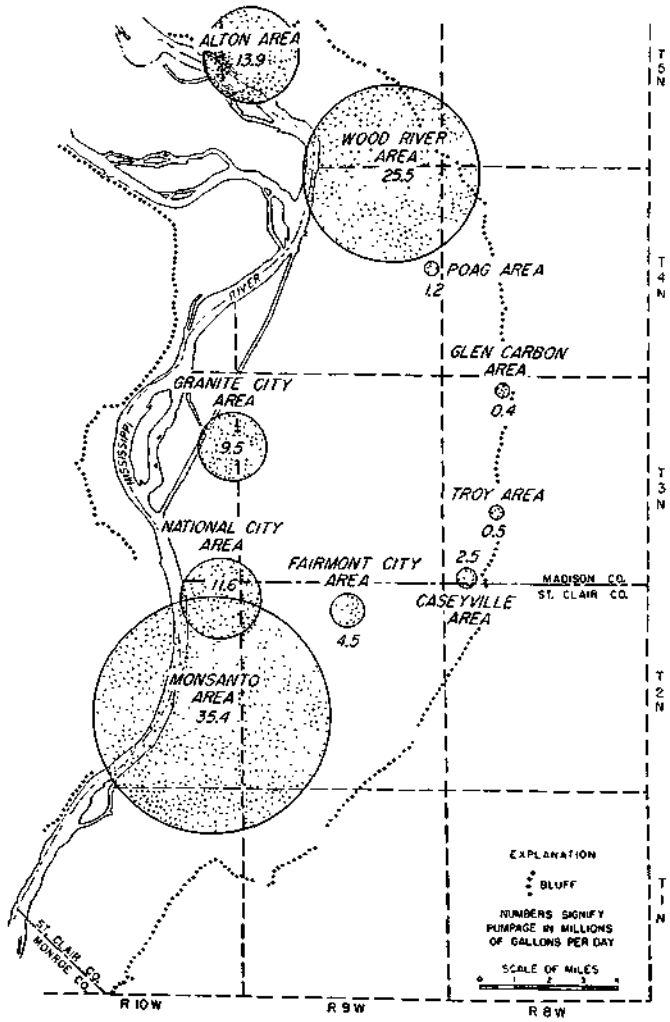


Figure 34. Distribution of estimated pumpage in 1962

change in pumpage from 1956 to 1962 occurred in the Granite City area. Because of a serious decline in water levels caused by heavy pumpage concentrated in a relatively small area and the severe drought during 1952-1956, the Granite City Steel Company abandoned its wells in 1957 and began obtaining water supplies from the Mississippi River. As a result, withdrawals of ground water dropped sharply from 30.1 mgd in 1956 to 7.6 mgd in 1958, and gradually increased to 9.5 mgd in 1962. Pumpage in the National City area in 1962 does not include pumpage necessary to dewater a cut along an interstate highway in construction near National City since this information was not available at the time this report was written.

Of the 1962 total pumpage, withdrawals for public water-supply systems amounted to about 6.4 percent, or 6.7 mgd; industrial pumpage was about 91.1 percent, or 95.7 mgd; domestic pumpage was 2.3 percent, or 2.4 mgd; and irrigation pumpage was 0.2 percent, or 0.2 mgd.

The major industries in the East St. Louis area using ground water are oil refineries, chemical plants, ore re-

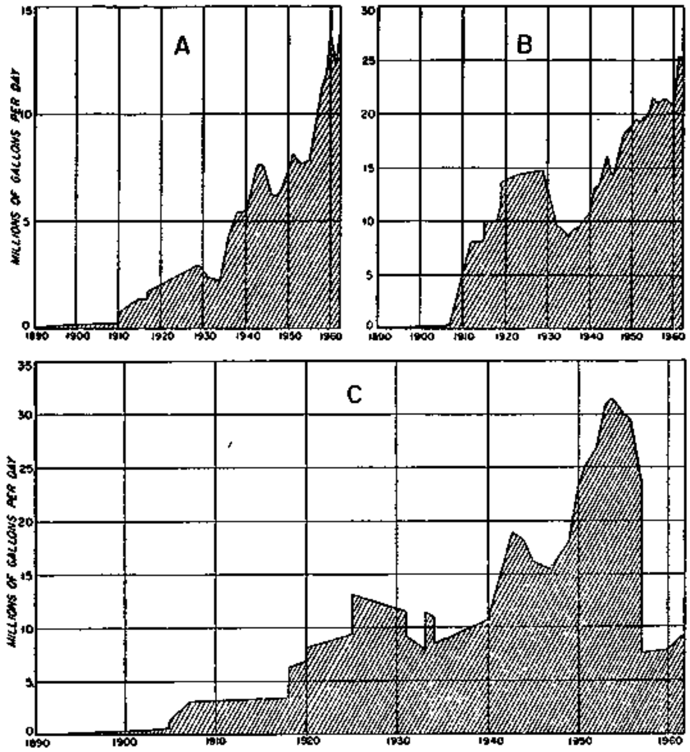


Figure 35. Estimated pumpage, Alton area (A), Wood River area (B), and Granite City area (C), 1890-1962

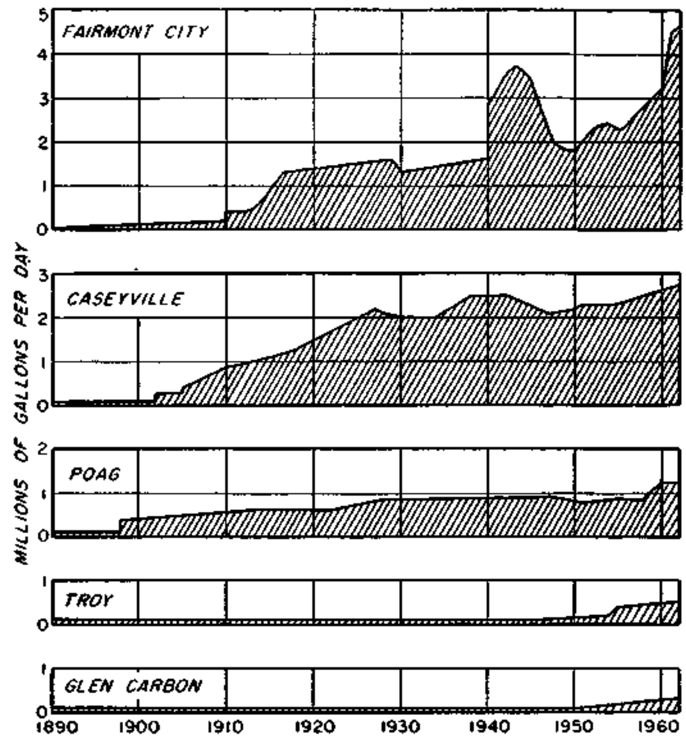


Figure 37. Estimated pumpage, Fairmont City, Caseyville, Poag, Troy, and Glen Carbon, 1890-1962

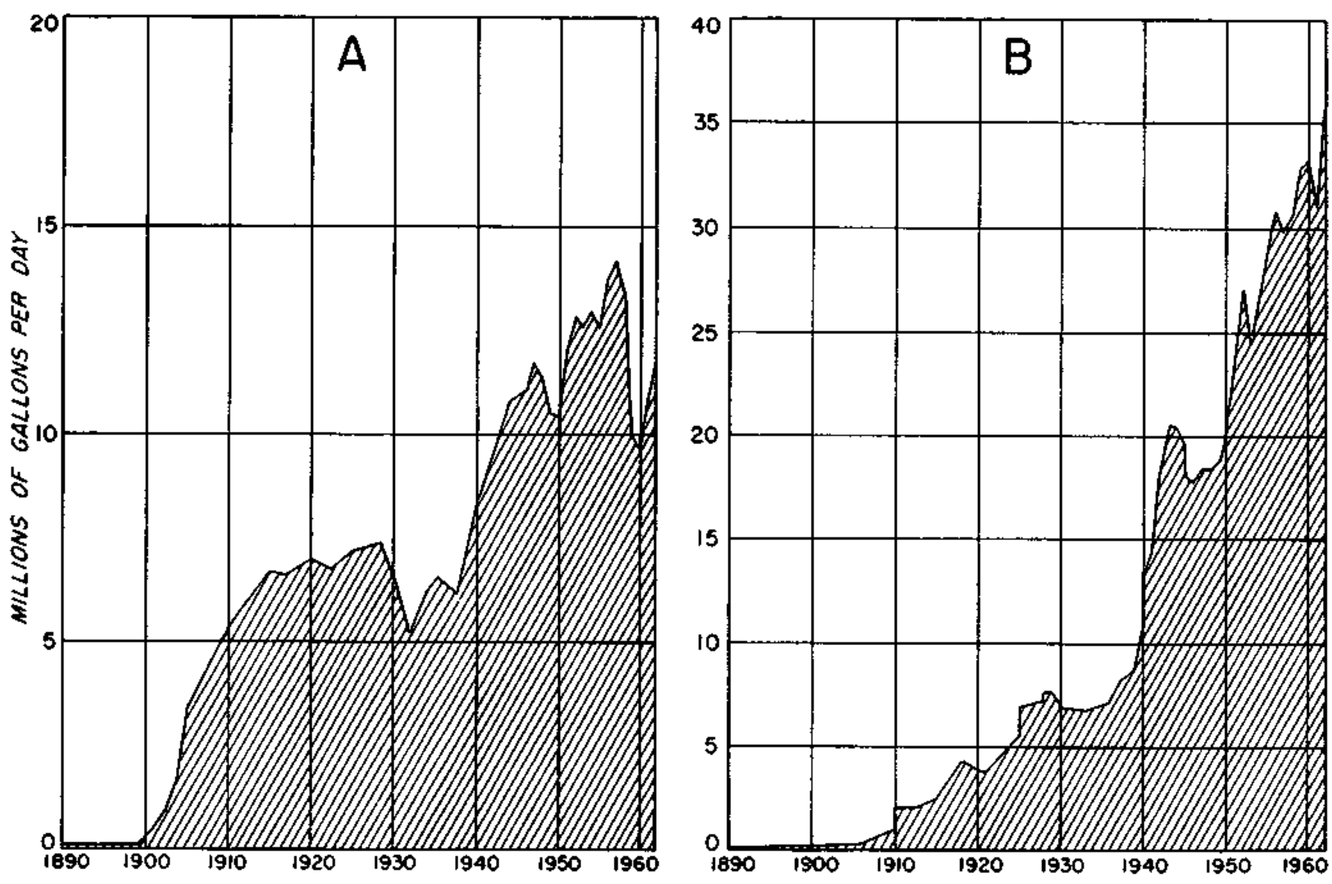


Figure 36. Estimated pumpage, National City area (A) and Monsanto area (B), 1890-1962

fining plants, meat packing plants, and steel plants. Data on industrial pumpage were obtained from 82 plants. Industrial pumpage was 83.5 mgd in 1960, 87.8 mgd in 1961, and 95.7 mgd in 1962. Public supplies include municipal, commercial, and institutional uses. In 1962 there were 10 public water supplies in the East St. Louis area having an estimated total pumpage of 6.7 mgd. Public pumpage was 6.8 mgd in 1960 and 6.6 mgd in 1961. Water pumped by hotels, hospitals, theaters, motels, and restaurants is classified as commercial and institutional pumpage and in 1962 averaged about 400,000 gpd.

Domestic pumpage, including rural farm nonirrigation and rural nonfarm use, was estimated by considering rural population as reported by the U.S. Bureau of the Census and by using a per capita use of 50 gpd. Domestic pumpage was estimated to be 2.4 mgd in 1960, 1961, and 1962.

Development of ground water for irrigation on a significant scale started in 1954 during the drought extending from 1952 through 1956. In 1962 there were 31 irrigation wells in the East St. Louis area. Estimated irrigation pumpage was 300,000 gpd in 1960, 100,000 gpd in 1961, and 200,000 gpd in 1962.

Prior to 1953 pumpage from wells was largely concentrated in areas at distances of 1 mile or more from the Mississippi River. During and after 1953 pumpage from wells at distances within a few hundred feet from the river increased greatly in the Alton, Wood River, and Monsanto areas. Distribution of pumpage from wells near the river during 1956, 1960, 1961, and 1962 is given in table 22. The distribution of pumpage from wells near the river in 1962 is shown in figure 38. During 1962 total pumpage from Alton, Wood River, and Monsanto area pumping centers was 74.8 mgd of which 31.2 mgd or 41.7 percent was withdrawn from wells near the Mississippi River.

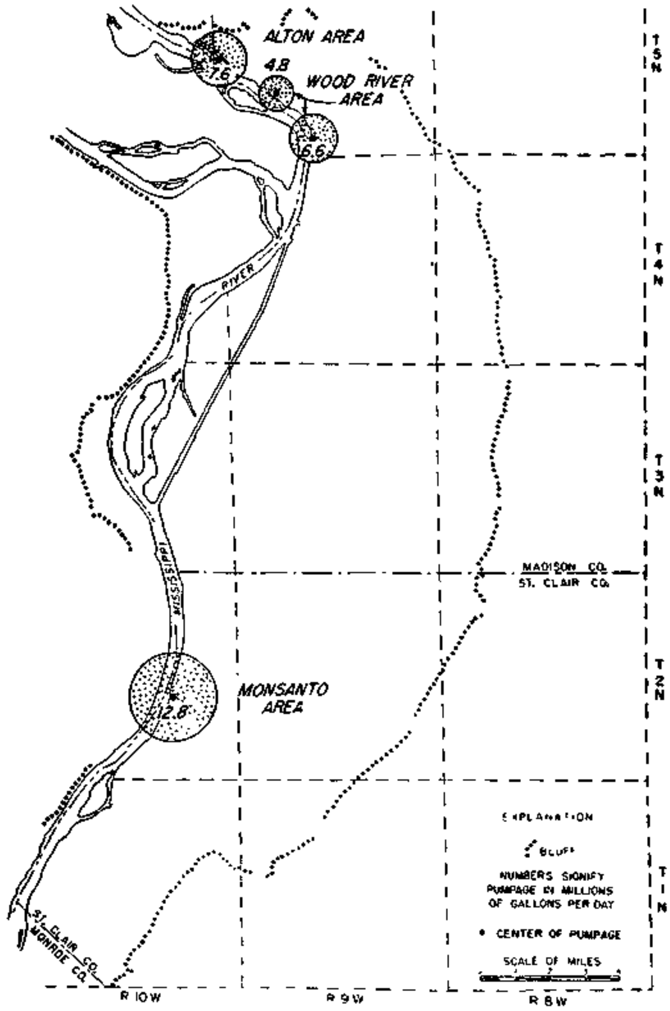


Figure 38. Distribution of estimated pumpage from wells near Mississippi River in 1962

Table 22. Distribution of Pumpage from Wells near Mississippi River

(Pumpage in million gallons per day)

Pumping center	1956 From all wells in center	1956 From wells near river	1960 From all wells in center	1960 From wells near river	1961 From all wells in center	1961 From wells near river	1962 From all wells in center	1962 From wells near river
Alton area	9.8	0	13.6	6.3	12.3	7.2	13.9	7.6
Wood River area	21.1	7.3	20.9	6.8	24.3	10.8	25.5	10.8
Monsanto area	30.1	10.8	33.2	10.5	31.9	11.4	35.4	12.8
Total	61.0	18.1	67.7	23.6	68.5	29.4	74.8	31.2

WATER-LEVEL FLUCTUATIONS

Prior to the settlement of the East St. Louis area, the water table was very near the surface and shallow lakes, ponds, swamps, and poorly drained areas were widespread. Development of the East St. Louis area led to the construction of levees and drainage ditches and subsequent changes in ground-water levels. Bruin and Smith (1953) estimated that these developments caused lowering of ground-water levels by 2 to 12 feet. In addition, industrial and urban expansion and the subsequent use of large quantities of ground water has lowered water levels appreciably in the Alton, Wood River, Granite City, National City, East St. Louis, and Monsanto areas. Lowering of water levels caused by large withdrawals of ground water has also been experienced in the Poag, Caseyville, Glen Carbon, Troy, and Fairmont City areas.

Figure 39 shows the change in water levels in the East St. Louis area during 61 years. The map is based on piezometric surface maps for 1900 and 1961. The greatest declines occurred in the five major pumping centers; 50 feet in the Monsanto area, 40 feet in the Wood River area, 20 feet in the Alton area, 15 feet in the National City area, and 10 feet in the Granite City area. Water levels rose more than 5 feet along Chain of Rocks Canal behind the locks of the canal where the stage of surface water in 1961 was above the estimated piezometric surface in 1900. In areas remote from major pumping centers and the Mississippi River, water levels declined an average of about 5 feet. Water levels have not changed appreciably in the Horseshoe Lake area.

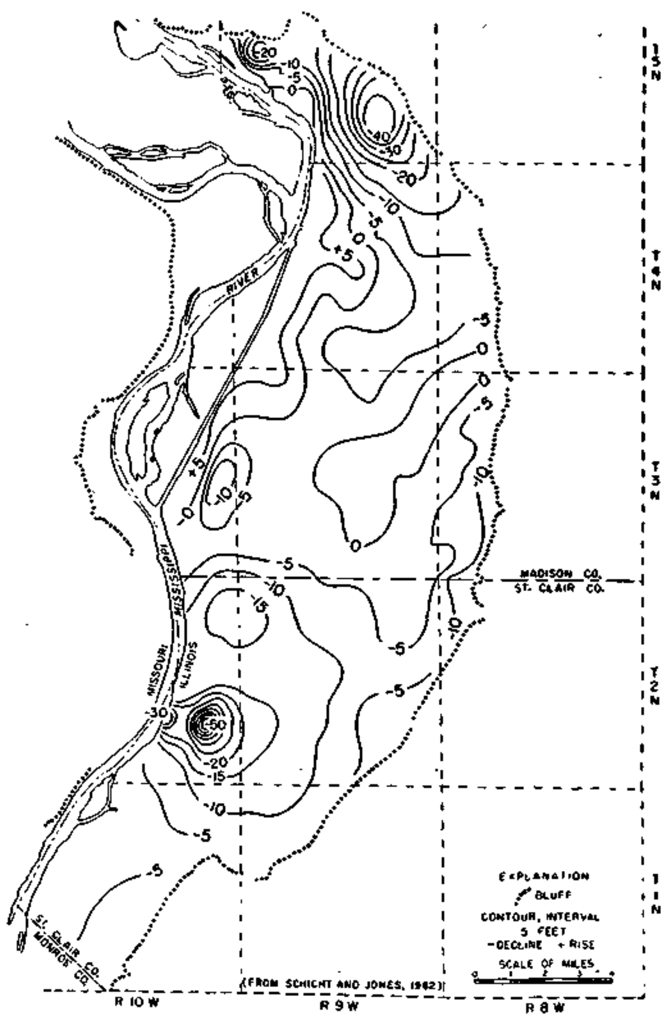


Figure 39. Estimated change in water levels, 1900 to November 1961

The piezometric surface map for December 1956 was compared with the piezometric surface map for November 1961, and figure 40 shows the change in water levels in the East St. Louis area during this time. The greatest rises in water levels, exceeding 50 feet, were recorded in the Granite City area and are due largely to a reduction in pumpage in the area from 31.6 mgd in 1956 to about 8.0 mgd in 1961. Water levels declined slightly in the center of the Monsanto cone of depression because of an increase in pumpage of about 3 mgd from 1956 to 1961. Water levels rose more than 5 feet in other places in the Monsanto area and more than 10 feet in the Alton area. Water levels in the Wood River area declined less than 1 foot near the center of pumping and rose more than 10 feet in other places. Along the Mississippi River west of Wood River water levels rose more than 20 feet; along the Mississippi River west of Monsanto water levels declined slightly in an area affected by an increase in pumpage from wells near the river. In areas remote from major pumping centers and the Mississippi River, water levels rose on the average about 5 feet.

Changes in water levels from June to November 1961 were computed (Schicht and Jones, 1962) and were used to prepare figure 41. The stage of the Mississippi River was higher during November than in June, and as a result ground-water levels rose appreciably along the river especially in areas where induced infiltration occurs. Water levels declined more than a foot at many places in the Granite City and National City areas and along the bluffs north of Prairie Du Pont Creek. Water-level declines averaged about 3 feet south of Prairie Du Pont Creek. Water-level rises exceeded 5 feet in the Alton area and exceeded 7 feet along the Mississippi River west of Wood River. Water levels rose in excess of 4 feet in the Monsanto area. A tongue of water-level rise extended eastward through Monsanto and to a point about 5 miles northeast of Monsanto.

Changes in water levels from June 1961 to June 1962 are shown in figure 42. The stage of the Mississippi River was higher during June 1962 than in June 1961, and as a result ground-water levels rose appreciably in most places along the Mississippi River and Chain of Rocks Canal. Water levels declined more than a foot near Monsanto along the Mississippi River as a result of heavy pumping. Water levels declined less than a foot in the Horseshoe Lake area and in places along the bluffs; water levels also declined in a strip west of Dupo. Water levels rose in excess of 5 feet along the Mississippi River in the Alton and Wood River areas and along the northern reach of Chain of Rocks Canal. Immediately east of Dupo water levels rose in excess of 4 feet.

Changes in water levels from November 1961 to June 1962 are shown in figure 43. Ground-water levels rose appreciably in most places because Mississippi

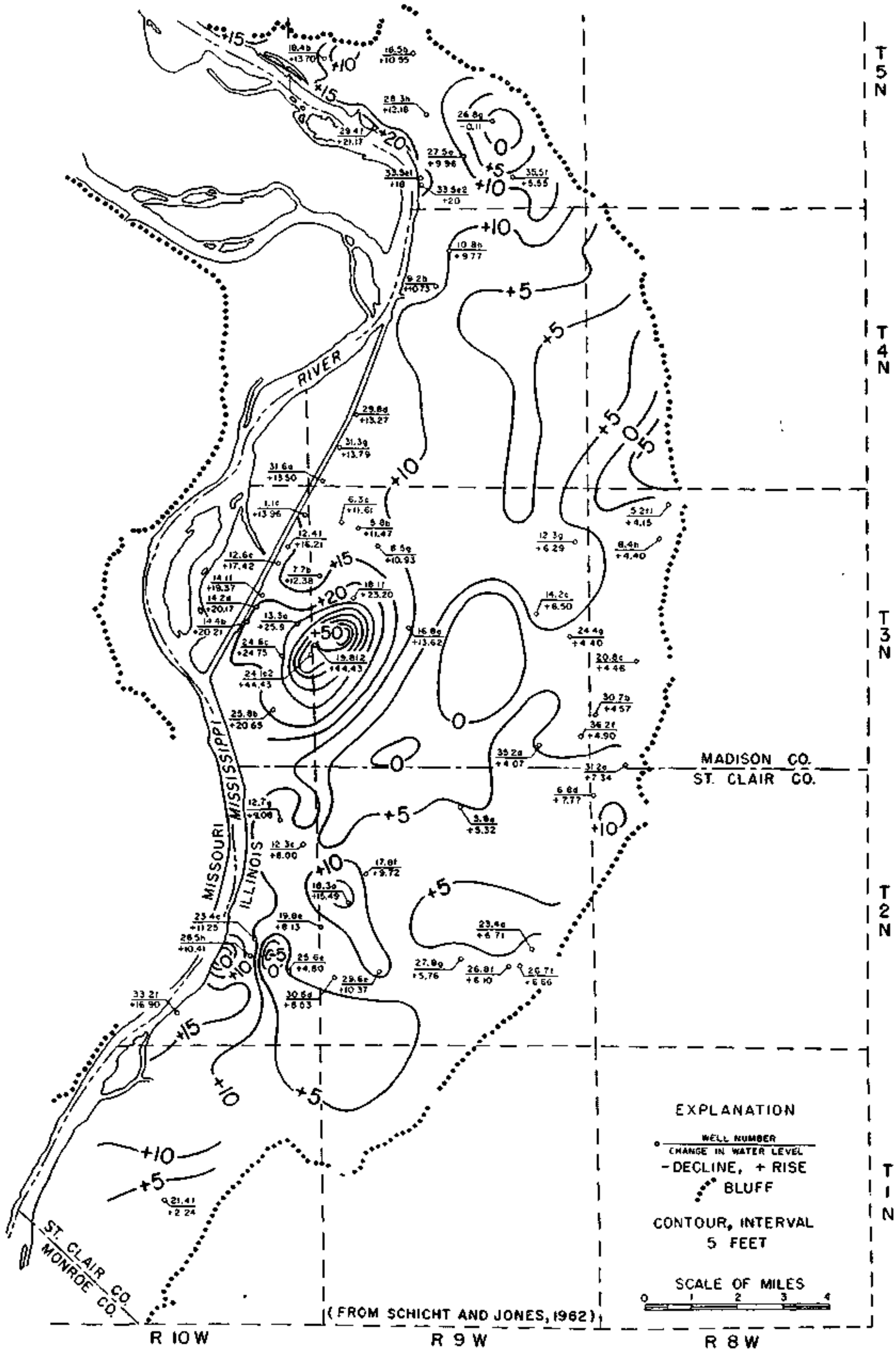


Figure 40. Estimated change in water levels, December 1956 to November 1961

River stages were higher in June 1962 than in November 1961. During the winter and early spring months, conditions were favorable for the infiltration of rainfall to the water table. Ground-water levels rose appreciably along the bluffs, the rise exceeding 7 feet in places. Ground-water level rises along the Mississippi River exceeded 5 feet east of Wood River and east of National City; ground-water level rises exceeded 5 feet at the northern end of Long Lake and near Dupo. Water levels declined less than 1 foot around Horseshoe Lake and between 1 and 2 feet in a small area near Monsanto.

Examples of fluctuations in water levels in the East St. Louis area are shown in figures 44-49. The locations of observation wells for which hydrographs are available are given in figure 50. As illustrated by the hydrographs for wells remote from major pumping centers in figure 44, water levels generally recede in the late spring, summer, and early fall when discharge from the ground-water reservoir by evapotranspiration, by ground-water runoff to streams, and by pumping from wells is greater than recharge from precipitation and induced infiltration of surface water from the Mississippi River and other

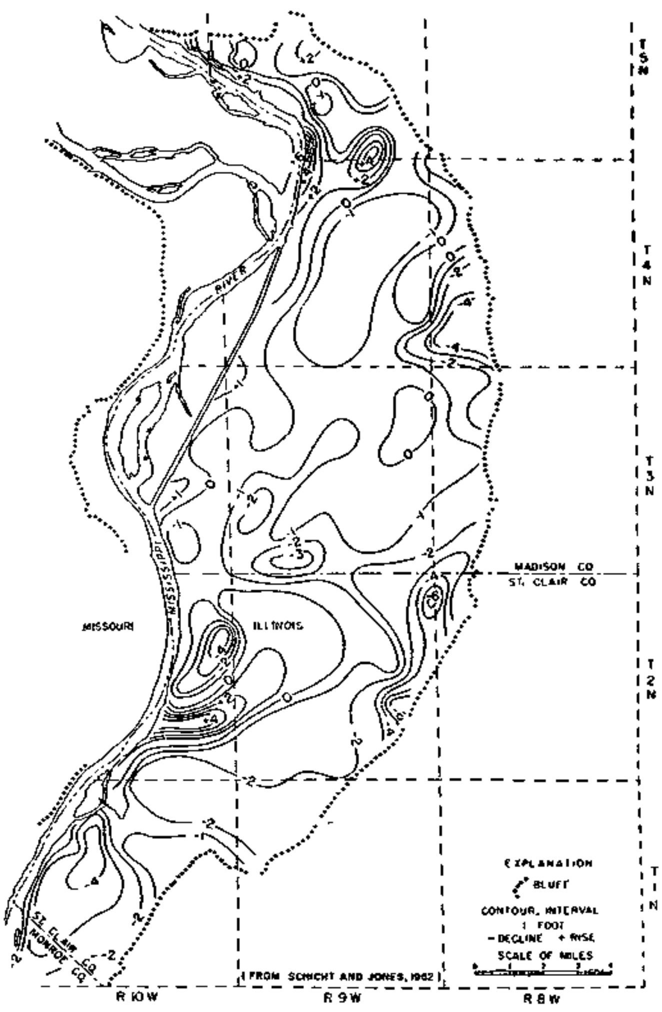


Figure 41. Estimated change in water levels, June to November 1961

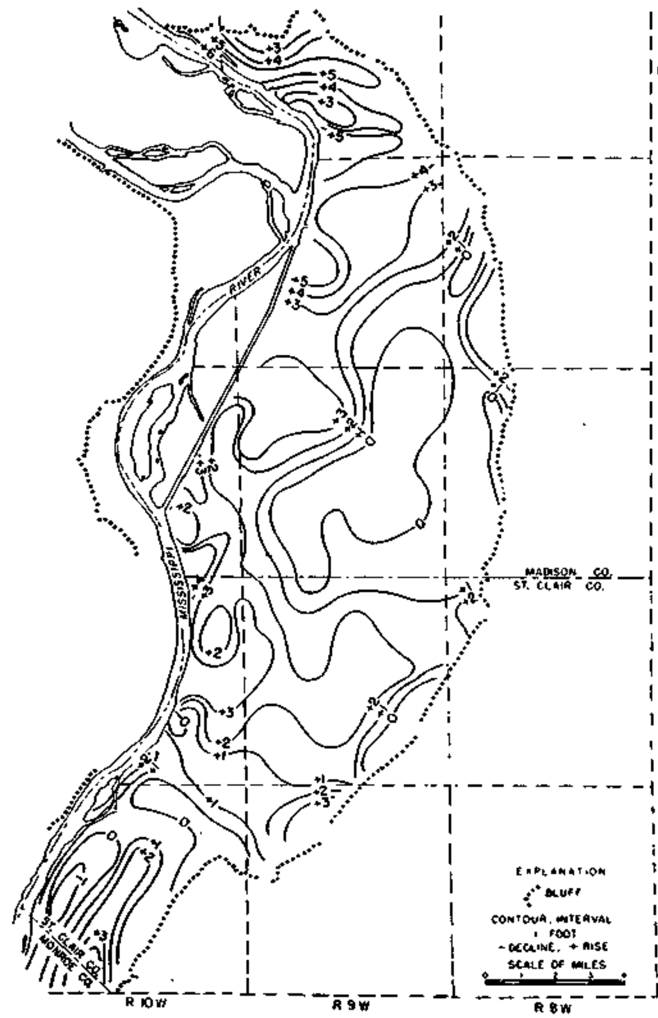


Figure 42. Estimated change in water levels, June 1961 to June 1962

streams. Water levels generally begin to recover in the early winter when conditions are favorable for the infiltration of rainfall to the water table. The recovery of water levels is especially pronounced during the spring months when the ground-water reservoir receives most of its annual recharge. Water levels are frequently highest in May and lowest in December, depending primarily upon climatic conditions, pumping rates, and the stage of the Mississippi River. Water levels in wells remote from major pumping centers have a seasonal fluctuation ranging from 1 to 13 feet and averaging about 4 feet.

Water levels in the East St. Louis area declined appreciably during the drought, 1952-1956. The records of the U.S. Weather Bureau at Edwardsville indicate that rainfall averaged about 34.3 inches per year from 1952 through 1956, or about 6.5 inches per year below normal. The hydrograph of water levels in well MAD 3N8W-31.2a and the graph of annual precipitation at Edwardsville for 1941 to 1962 in figure 45 illustrate the pronounced effect of the prolonged drought on water levels.

Examples of hydrographs of water in wells within major pumping centers are shown in figures 46-49. Comparisons of pumpage and water-level graphs indicate that in general water levels within pumpage centers fluctuate in response to changes in precipitation, river stage, and pumpage. The effects of the drought during 1952-1956 are apparent; the effects of changes in river stage are masked almost completely by the effects of the drought and pumpage changes. However, careful study of river stages and water-level data indicate that water levels in major pumpage centers do fluctuate several feet in response to large changes in river stage. If the effects

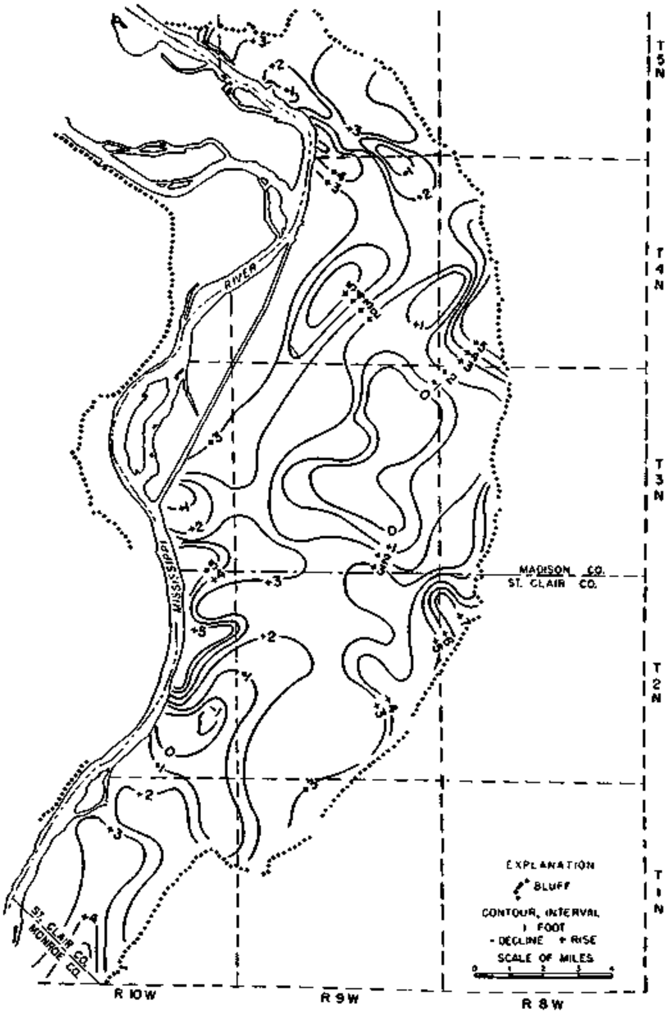


Figure 43. Estimated change in water levels, November 1961 to June 1962

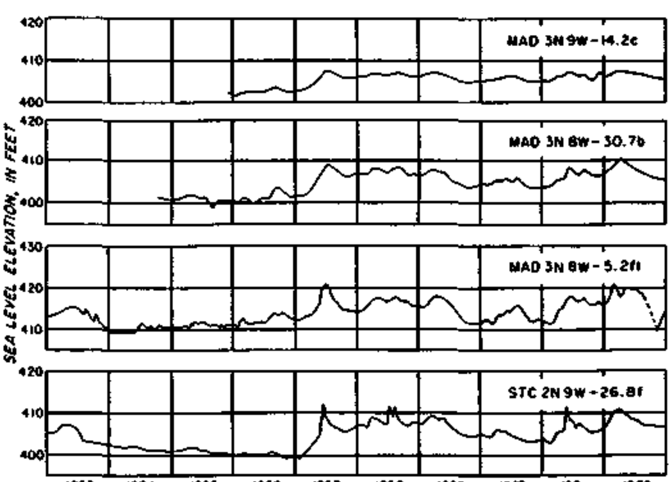


Figure 44. Water levels in wells remote from major pumping centers, 1953-1962

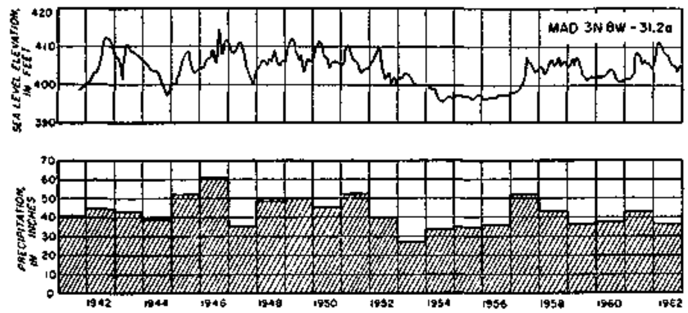


Figure 45. Water levels in well MAD 3N8W-31.2a and annual precipitation at Edwardsville, 1941-1962

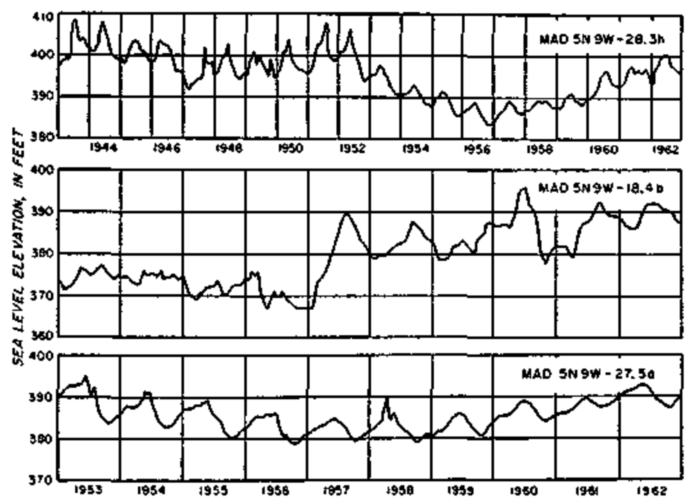


Figure 46. Water levels in Alton and Wood River areas

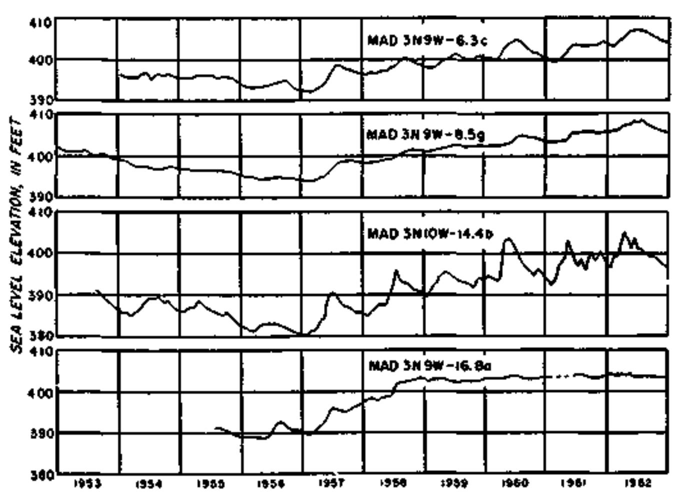


Figure 47. Water levels in Granite City area, 1951-1962

of the drought and changes in river stage are taken into consideration, water-level declines are directly proportional to pumping rates. The water levels vary from place to place within pumpage centers and from time to time mostly because of the shifting of pumpage from well to well, shifting of pumpage from pumpage centers 1 mile or more from the Mississippi River to pumpage centers near the river, and variations in total well field pumpage. At no location is there any apparent continuous decline that cannot be explained by pumpage increases. Thus, within a relatively short time after each increase in pumpage, recharge directly from precipitation and by induced infiltration of water in streams increased in proportion to pumpage as hydraulic gradients became greater and areas of diversion expanded.

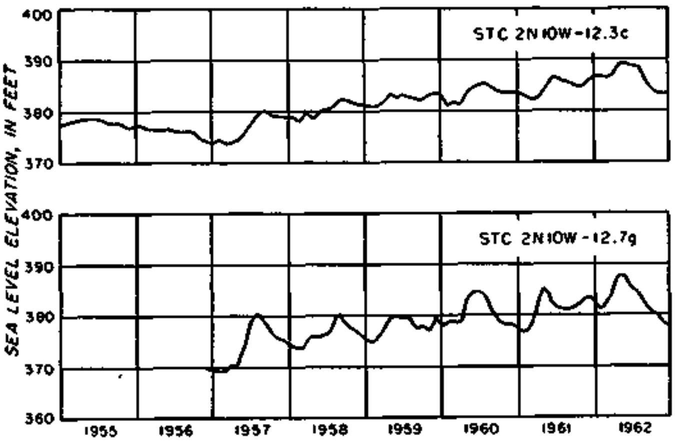


Figure 48. Water levels in wells in National City area, 1955-1962

Annual fluctuations of water levels in wells within major pumping centers are generally less than 15 feet. The average rate of decline during 1952-1956 was about 2 feet per year. The average rate of rise in the Granite City area during the period 1957-1962 was about 2 feet per year. The average rate of decline in the Monsanto area during 1930-1962 was about 1.3 feet per year.

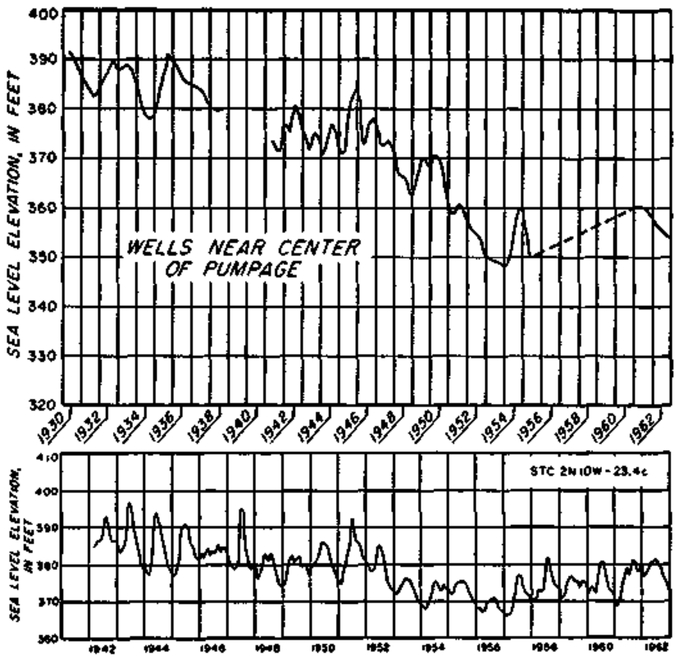


Figure 49. Water levels in wells in Monsanto area

PIEZOMETRIC SURFACE

In order to delineate areas of diversion and to determine directions of ground-water movement in the East St. Louis area, piezometric surface maps were made.

Figure 51 depicts the surface drainage system in 1900 and the estimated piezometric surface prior to heavy industrial development. The piezometric surface sloped from an estimated elevation of about 420 feet near the bluffs to about 400 feet near the Mississippi River. The average slope of the piezometric surface was about 3 feet per mile; however, the slope ranged from 6 feet per mile in the Alton area to 1 foot per mile in the Dupo area. The slope of the piezometric surface was greatest near the bluffs. The general direction of ground-water movement was west and south toward the Mississippi River and other streams and lakes. The establishment of industrial centers and the subsequent use of large quantities of ground water by industries and municipalities has lowered water levels appreciably in the areas of heavy pumping.

From 1952 through 1956 water levels declined appreciably in the East St. Louis area as the result of drought conditions, low Mississippi River stages, and record high ground-water withdrawals. Figure 52 shows the piezometric surface in December 1956, when water levels were at record low stages at many places.

The illustration shows clearly the cones of depression in the piezometric surface which have developed as the result of heavy pumping. It will be noted that a considerable lowering has taken place in the piezometric surface since 1900. In 1956 the deepest cone of depression was in the Granite City area. Other pronounced cones were centered in major pumping centers.

Figure 53 shows the piezometric surface in June 1961 after pumpage was reduced in the Granite City area. The piezometric surface map for December 1956 is similar in many respects to the piezometric surface map for June 1961. Significant differences are that the cone of depression in the Granite City area was much deeper

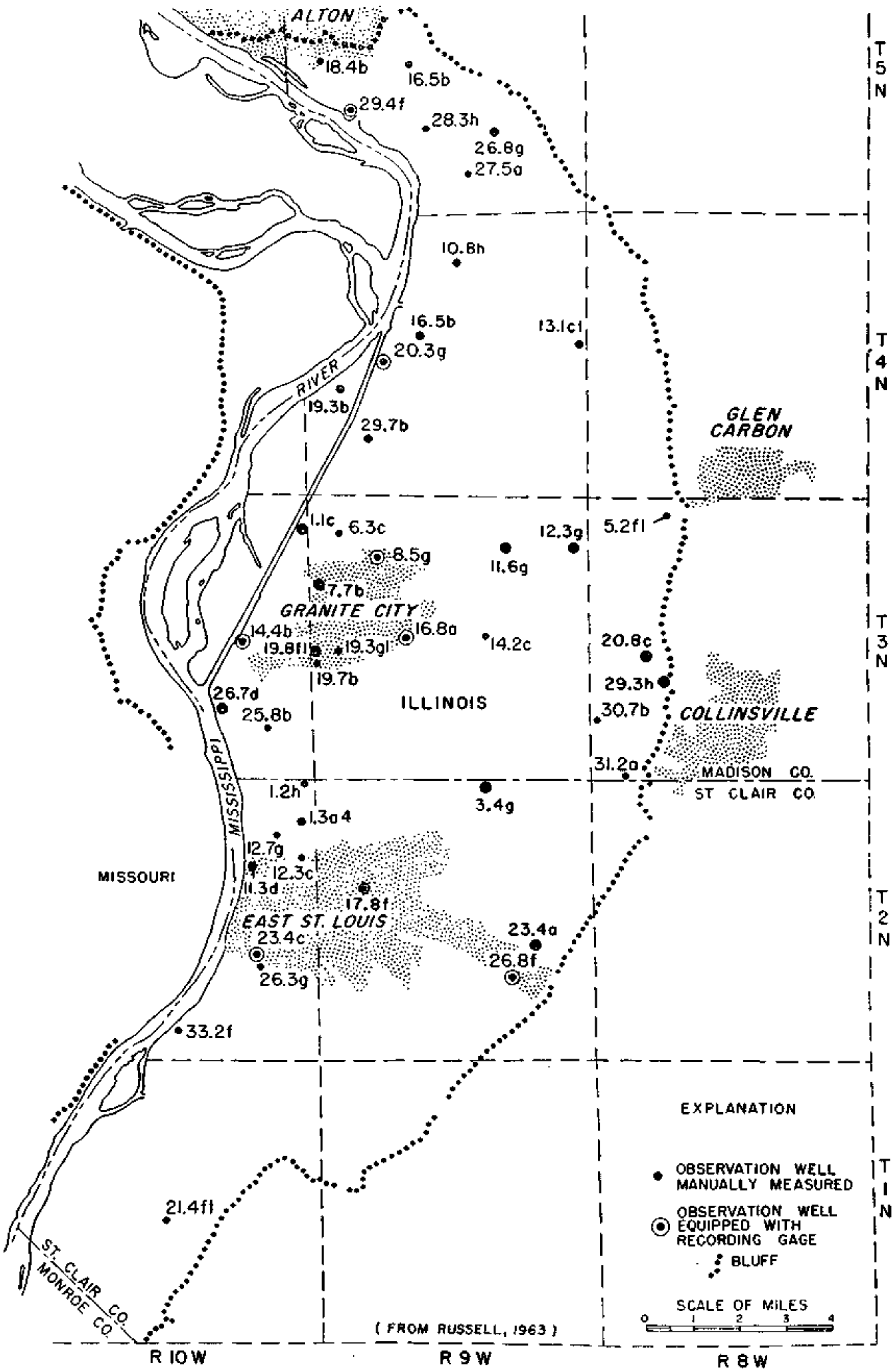


Figure 50. Locations of observation wells

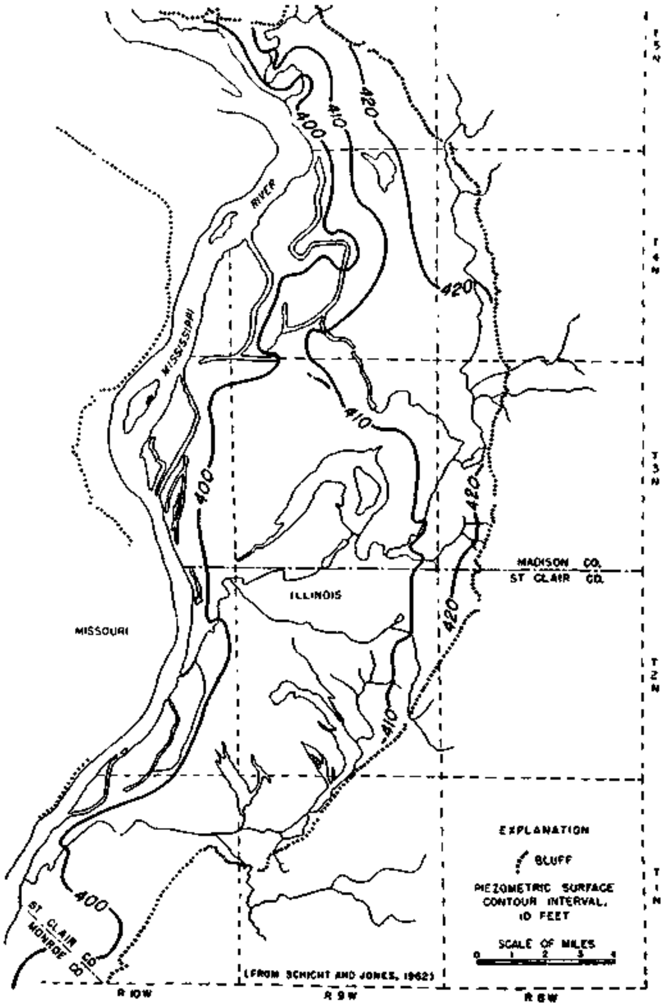


Figure 51. Drainage system and estimated elevation of piezometric surface about 1900

in 1956 than in 1961, and ground-water levels were lower in the vicinity of streams and lakes in 1956 than they were in 1961.

During June 1962, when water levels were near peak stages, a mass measurement of ground-water levels was made, and data collected are given in tables 23, 24, and 25. The piezometric surface map for June 1962 is shown in figure 54. Features of the piezometric surface maps for June 1961 and June 1962 are generally the same. The deepest cone of depression in June 1962 was centered in the Monsanto area where the lowest water levels were at an elevation of about 350 feet. A smaller cone of depression occurred near the Mississippi River about 1.5 miles west of the large Monsanto cone of depression in the vicinity of a small pumping center. The water levels in the center of this cone of depression were at an elevation of about 355 feet. The elevations of the lowest water levels in other important cones of depression were: 385 feet in the Wood River area, 390 feet in the Alton area, 395 feet in the Granite City area, and 390 feet in the National City area.

The general pattern of flow of water in 1962 was slow movement from all directions toward the cones of depressions or the Mississippi River and other streams. The lowering of water levels in the Alton, Wood River, National City, and Monsanto areas that has accompanied withdrawals of ground water in these areas has established hydraulic gradients from the Mississippi River towards pumping centers. Ground-water levels were below the surface of the river at places and appreciable quantities of water were diverted from the river into the aquifer by the process of induced infiltration. The piezometric surface was above the river at many places. For example, southwest of the Granite City cone of depression water levels adjacent to the river were higher than the normal river stage and there was discharge of ground water into the river.

The average slope of the piezometric surface in areas remote from pumping centers was 5 feet per mile. Gradients were steeper in the immediate vicinity of major

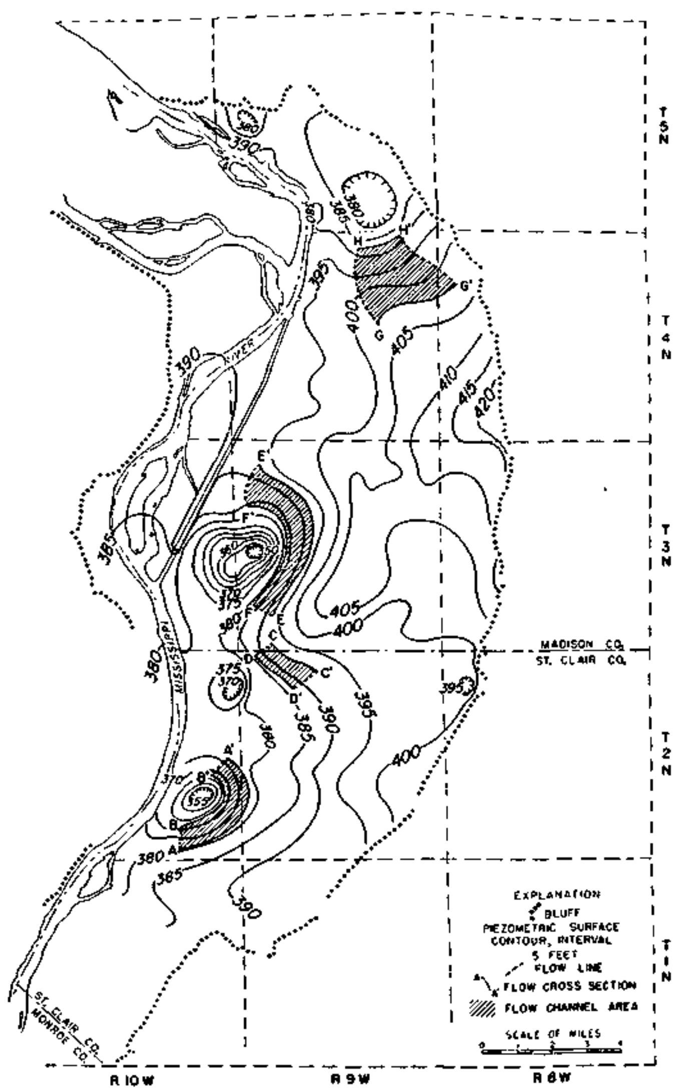


Figure 52. Approximate elevation of piezometric surface,, December 1956

Table 23. Water-Level Data for Wells

Well number	Elevation of measuring point (ft)	Water levels, June 1962: Depth to water (ft)	Water levels, June 1962: Mean sea level elevation (ft)	Water level changes (ft): From June 1961 to June 1962	Water level changes (ft): From November 1961 to June 1962
MAD—					
5N10W-					
13.2a	413.4	7.52	405.88	+5.28	+3.11
13.5c	412.8	6.44	406.36	+5.88	+1.56
13.6cl	416.1	9.42	406.68	+5.40	+1.25
13.7e	415.2	6.32	408.88	+5.60	+0.14
14.1e	411.9	1.55	410.35	+6.37	+0.54
24.1h	414.7	9.31	405.39	+5.19	+2.96
5N9W-					
16.5b	443.03	44.25	398.78	+4.42	+2.33
18.3c	436.7	49.11	387.59	+2.91	+3.19
18.4b	438.1	47.10	391	+3.00	+2.00
19.3c	415.7	7.56	408.14	+5.20	+2.54
19.4h	430	35.45	394.55	+3.20	+2.09
19.6e	415.8	7.97	407.83	+5.79	+2.67
19.7f	413.3	6.44	406.86	+5.77	+3.03
19.8g	414.7	8.02	406.68	+5.80	+3.20
20.5a	413.4	5.68	407.72	+3.18	+1.05
22.2c	440.71	47.57	393.14	+5.53	+3.99
26.8g	441.42	57.17	384.25	+4.53	+2.86
27.5a	428.52	36.25	392.27	+2.96	+3.61
28.3h	432.60	32.25	400.35	+3.20	+3.77
28.4c	413.30	11.22	402.08	+2.85	+1.09
28.8e	418.2	10.29	407.91	+4.75	+1.29
29.1e	413.4	11.15	402.25	+4.40	+1.07
29.4f	416.07	8.67	407.40	+2.68	+0.33
29.4g	414.4	7.30	407.10	+3.61	+0.83
29.5g	415.4	8.26	407.14	+3.41	+0.54
33.5el	418.44	16.00	402.44	+11.00	+5.00
33.5e2	417.89	14.00	403.89	+10.00	+3.00
34.7c	431	37.20	393.80		+3.00
35.5f	445.55	60.55	385	+5.80	+3.45
35.5h	446.53	64.53	382	+3.50	+3.47
35.6b	445.69	50.69	395	+4.81	+0.52
4N8W-					
6.8h	441.18	31.45	409.76	+3.42	+2.28
7.4b	428	18.06	409.94	+2.03	+3.02
19.4e	429	16.86	412.14	—0.42	+0.91
20.3c	452.5	30.32	422.18	+9.12	+11.48
30.1f	425	8.07	416.93	+2.38	+6.88
4N9W-					
9.2b	434.61	25.05	409.56	+3.13	+2.33
10.8h	432.57	24.8	407.77	+4.00	+3.00
12.4h	427	19.65	407.35	+2.79	
13.1c1	439.15	26.00	413.15	+2.00	
14.8h	422.89	16.50	406.39		+3.34
16.5b	417.78	7.45	410.33		
19.3b	422	18.49	403.51	+2.30	+0.46
20.3g	414.39	5.40	408.99	+5.18	+1.25
25.2d	421	8.73	412.27	+0.60	+0.32
27.8h	409	1.00	408.00	+3.66	+5.46
29.7b	421.06	13.17	407.89		+1.83
29.8d	413.42	5.58	407.84	+2.27	+2.05
30.1b	416.70	8.88	407.82	+2.53	+1.90
31.2h	416.95	9.28	407.67	+2.72	+1.86
31.3g	415.57	7.98	407.59	+2.77	+1.69
31.6a	408.02	1.09	406.93	+2.76	+1.61
34.1h	423	14.84	408.16	+0.76	+1.87
34.5a	421	10.36	410.64		+1.87

Well number	Elevation of measuring point (ft)	Water levels, June 1962: Depth to water (ft)	Water levels, June 1962: Mean sea level elevation (ft)	Water level changes (ft): From June 1961 to June 1962	Water level changes (ft): From November 1961 to June 1962
4N10W-					
36.5f	415	2.75	412.25	—0.77	+1.89
3N8W-					
5.2fl	439.65	20.00	419.65	+2.00	+3.00
6.1e	425	6.75	418.25	+0.43	+2.01
8.4h	430	10.78	419.22	—0.24	+1.82
8.8a	422	9.35	412.65	+0.68	+1.97
17.4d	416.06	4.02	412.04	—0.02	+0.76
20.5c	430	19.46	411.54		+4.10
20.8c	422	10.01	411.99	+0.69	+2.28
30.7b	421.28	12.55	408.73	+0.52	+2.16
31.2a	428.22	20.04	408.18	+0.11	+3.44
3N9W-					
3.1a	415	7.05	407.95	—0.42	—0.19
5.8b	424.45	16.78	407.67	+3.91	+3.20
6.3c	426.66	19.39	407.27	+3.54	+2.96
7.7b	425.08	19.15	405.93	+3.57	+7.95
8.5g	420.84	12.40	408.44	+3.27	+2.91
9.4e	421	12.80	408.20	+3.67	+4.34
10.4b	415	5.35	409.65	+0.26	+1.31
12.3g	420.5	4.76	415.74	—0.18	—0.55
14.2c	425.50	17.36	408.14	—0.09	—0.36
16.1d	422	14.82	407.18	—0.63	+1.23
16.8a	415.88	10.87	405.01	—0.11	+0.89
18.1f	412.90	8.88	404.02	+2.42	+2.42
19.3gl	417.74	15.78	401.96		
19.8fl	424.14	26.68	397.46		+6.74
20.2h	414.67	9.54	405.13	+0.18	+0.82
20.7e	418.73	14.87	403.86	—0.10	+1.57
20.8dl	416.68	13.57	403.11	—0.13	+2.10
20.8d2	414.71	11.89	402.82	—0.16	+1.86
20.8el	416.33	13.03	403.30	—0.12	+1.82
21.2d	408	4.85	403.15	—0.91	—0.69
23.5g	419	16.65	402.35	—0.45	+0.05
24.4g	425.90	16.55	409.35	+0.38	+1.15
28.4e	417.5	6.81	410.69	+0.27	+1.34
32.3b	410	12.15	397.85	—0.21	+3.21
32.6g	418	17.58	400.42		
35.2d	411.21	6.34	404.87	+0.31	+1.59
35.5g	415.5	8.65	406.85	+0.33	+0.95
36.2f	421.12	13.07	408.05	+0.59	+2.15
3N10W-					
1.1c	407.11	0.00	407.11	+3.22	+2.05
12.4f	406.98	0.73	406.25	+1.86	+1.25
12.6.C	407.51	1.34	406.17	+1.80	+1.34
13.8g	409.43	5.58	403.85	+2.64	+2.72
14.1f	406.78	4.11	402.67	+1.99	+1.84
14.2d	411.36	7.63	403.73	+3.05	+2.56
14.3c	413.53	9.92	403.61	+2.93	+2.44
14.4b	413.69	7.59	406.10	+5.40	+5.09
22.1a	412.2	10.82	401.38	+2.13	+0.94
22.1c	412.9	12.39	400.51	+2.15	+1.45
23.6c	413.5	13.40	400.10	+1.75	+0.68
23.7c	412.4	12.35	400.05	+1.02	+1.35
24.1cl	422.34	24.47	397.87	+1.63	+2.23
24.1c2	418.59	20.97	397.62	+1.58	+2.19
24.6c	420	29.07	390.93	+2.08	+2.18
25.8b	414.96	14.94	400.02	+1.84	+2.57
26.6b	411.3	10.73	400.57	+0.45	+1.80

Table 23 (Continued)

WPII number	Elevation of meas-uring Point (ft)	Water levels, June 1962: Depth to watpr (fQ	Water levels, June 1962: Mean sea level elevation (ft)	Water level changes (ft): From June 1961 to June 1962	Water level changes (ft): From No-vember 1961 to June 1962
3N10W-(Continued)					
26.7d	411.2	10.21	400.98	+1.24	+1.60
26.8e	411.1	10.16	400.94	+1.53	+1.81
26.8h	411.8	10.80	401.00	+1.57	+1.76
35.6f	401.8	0.97	400.80	+4.21	+5.22
35.6h	404.6	4.19	399.60	+1.02	
36.5h	414.25	13.29	400.96	+3.16	+3.81
STC__					
2N8W-					
6.1d	425	17.20	407.80	+2.02	+4.28
6.8d	429.27	15.00	414.27	+1.00	+7.00
7.2h2	430	22.63	407.37		+5.72
2N9W-					
2.4e	418.5	6.85	411.65	+0.95	+3.05
3.4g	422	15.44	406.56	+0.56	+3.25
3.8a	424	23.01	400.99	+1.98	+2.67
7.5e	420	33.60	386.40	+2.98	+2.56
7.6e	420	34.03	385.97	+5.37	
11.7h	419	11.85	407.15	+1.71	+3.53
12.5d	420	7.88	412.12	+0.76	+4.66
13.6c	421.70	12.00	409.70	+1.00	+4.33
14.5c	425	18.38	406.62	+3.17	+3.39
15.3b	413	8.16	404.84	+3.21	+4.74
15.7a	420	18.32	401.68	+2.13	+2.97
17.2d	415	17.60	397.40	+1.76	+0.88
17.8f	417.21	24.12	393.09	+2.43	+1.37
18.3a	416.5	22.60	393.90	+2.89	+1.41
19.8e	418.78	31.24	387.54		+0.81
21.7h	410	15.18	394.82	+2.38	+2.14
23.4a	423.86	10.98	412.88	—1.98	+4.87
24.6e	428	16.42	411.58	—0.89	+4.14
26.7f	424.18	15.24	408.94	—0.07	+2.86
26.8f	421.39	12.78	408.61	—0.01	+2.84
27.8g	415	9.44	405.56		+2.36
28.4g	409	1.55	407.45	+2.15	
30.6d	415	25.53	389.47	+0.88	+1.94
32.2c	408	12.28	395.72	+0.73	+2.58
34.4h	417	12.06	404.94	—0.02	+2.18
1.2h	412	20.15	391.85	+2.42	+3.38
1.3al	418.4	31.0	387.40		
12.3c	418.54	29.62	388.92	+2.53	+6.92
12.7g	410	23.91	386.09	+1.06	+2.41
23.4c	399.72	19.94	379.78	+1.25	+1.43
23.6f	415.7	23.49	392.21	+3.30	+5.37

Well number	Elevation of meas-uring point (ft)	Water levels, June 1962: Depth to water (ft)	Water levels, June 1962: Mean sea level elevation (ft)	Water level changes (ft): From June 1961 to June 1962	Water level changes (ft): From No-vember 1961 to June 1962
2N9W-(Continued)					
23.6g	397.5	4.04	393.46	+3.63	+1.37
23.7a	406.5	24.59	381.91	+0.89	+0.72
23.7b	408.2	20.63	387.57	+0.84	+1.97
26.1gl	411.37	72.50	338.87		
26.1g2	411.24	65.50	345.74	+2.50	—1.90
26.2e	413.70	61.67	352.03		—0.83
26.3g	411.80	55.33	356.47		
26.5h	408.76	34.30	374.46	+1.22	+0.15
27.2hl	415.65	62.25	353.40	—9.85	
33.2f	409.35	13.11	396.24	+2.94	+2.34
34.7c	399.1	5.25	393.85	+0.14	+2.52
34.8b	398.0	3.58	394.42	—0.08	+2.60
1N9W-					
4.5e	411	8.49	402.51	+0.84	+3.42
6.2a	416.	18.43	397.57	—0.39	+2.20
1N10W-					
4.1g	399.0	3.99	395.01	—0.49	+2.15
4.2e	396.4	1.04	395.36	—0.34	+1.76
4.3b	398.6	2.95	395.65	—0.17	+1.08
4.3c	397.7	2.27	395.43	+0.50	+1.10
4.7b	409.4	12.85	396.55	+0.95	+2.47
8.2h	407.8	11.11	396.69	+0.72	+2.76
8.5c	405.1	8.27	396.83	—0.27	+3.64
8.7a	406.3	9.89	396.41	—0.23	+2.00
9.1f	403.63	5.65	397.98	+0.84	+1.97
9.2h	404.55	6.94	397.61	+0.52	+1.68
9.4h	409.9	13.93	395.97	+0.78	+1.83
10.1c	403.29	5.14	398.15	+0.94	+1.94
10.4c	402.24	4.23	398.01	+0.82	+1.89
12.5b	401.74	3.09	398.65	—0.23	+0.95
13.3h	402.25	3.43	398.82	—0.32	+0.68
16.2g	411.5	10.96	400.54	+2.66	+3.26
17.1e	400	3.75	396.25	—0.71	+3.67
19.6f	406.4	10.21	396.19	—1.03	+2.52
21.1a	410	13.63	396.37	+0.68	—3.38
21.4f	412.01	13.85	398.16		+2.92
28.6a	405	9.54	395.46	+0.38	+2.93
30.6h	405.3	9.30	396.00	—0.99	+2.54
32.3e	414	18.81	395.91	+4.19	+5.10
MON—					
1N10W-					
30.8b	408.1	12.01	396.01	—0.85	+2.34
31.4d	407	10.55	396.45	+1.14	+3.30

pumping centers and exceeded 30 feet per mile within the Monsanto cone of depression. Gradients averaged about 10 feet per mile within the Alton, Granite City, National City, and Wood River cones of depression.

Along Canteen Creek and Cahokia Canal east of Horseshoe Lake, Long Lake, and Grand Marais State Park Lake, the piezometric surface was higher than the surface-water elevation and ground water was discharged into these streams and lakes. Below the confluence of Canteen Creek and Cahokia Canal south of Horseshoe Lake the piezometric surface was lower than surface-water elevations of Cahokia Canal at places where water levels have declined as the result of heavy pumping. Surface water in the Cahokia Diversion Channel south of the Wood River is kept above the piezometric surface at an elevation of 413 feet by a low water dam near the outlet of the channel. Surface-water levels are also controlled in Chain of Rocks Canal by Lock No. 27 near Granite City and were higher than the piezometric surface adjacent to the canal. The piezometric surface in the vicinity of Wood River near Alton and Prairie Du Pont Creek south of Monsanto was slightly higher than the surface-water elevations of the streams. At the lower end of Horseshoe Lake north of National City, ground-water levels were lower than the surface-water elevation of the lake.

South of Prairie Du Pont Creek ground water normally flows toward the Mississippi River. Ground water flows from the vicinity of Long Lake northwest towards the Mississippi River between the northern end of Chain of Rocks Canal and the outlet of the Cahokia Diversion Channel. Ground water flows toward the Mississippi River along the western half of Chouteau Island.

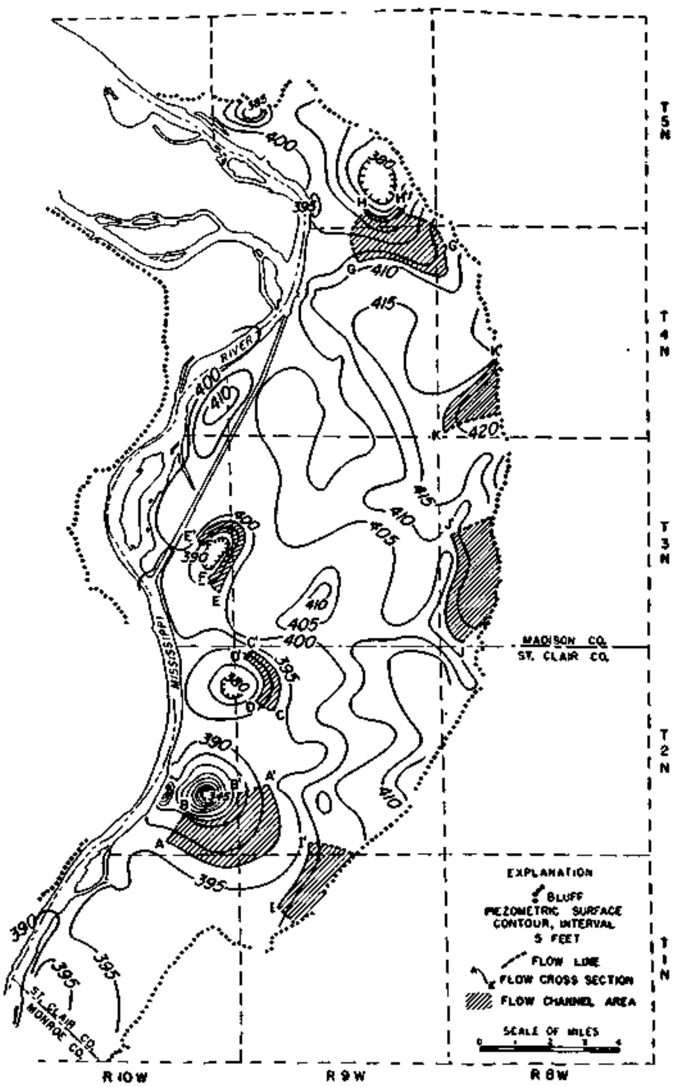


Figure 53. Approximate elevation of piezometric surface, June 1961

Table 24. Lake and Stream Elevations

Gage number	Location of gage	Elevation of measuring point (ft above msl)	Water-surface elevation June 6, 1962 (ft above msl)
2	Highway bridge 2, NW cor, sec 14, T4N, R9W	440.42	414.03
3	Highway bridge 3, NE cor, sec 14, T4N, R9W	441.38	414.09
4	Highway bridge 4, SE cor, sec 12, T4N, R9W	442.95	414.22
1	State Rte 3 bridge, SW cor, sec 5, T2N, R9W	409.80	396.43
2	Sand Prairie Road bridge, Canteen Creek, near center sec 35, T3N, R9W	418.04	400.89
3	Sand Prairie Road bridge, NW cor, sec 35, T3N, R9W	418.55	400.33
4	Hadley bridge, NW cor, sec 19, T3N, R8W	416.40	404.19
5	Black Lane bridge, Canteen Creek, near center sec 36, T3N, R9W	420.80	402.10
	Horseshoe Lake Control Works, NW cor, sec 34, T3N, R9W	403.71	403.64
	Chain of Rocks Canal (upper), SW cor, sec 14, T3N, R10W	(Surface water elevations reported)	407.90
	Chain of Rocks Canal (lower), NW cor, sec 23, T3N, R10W		401.08

Table 25. Mississippi River Stages, June 1962

Gage description	Mississippi River mile number	Water-surface elevation June 8, 1962 (ft above msl)
Lock and Dam No. 26 Alton, Ill. (lower)	202.7	410.6
Hartford, Ill.	196.8	409.4
Chain of Rocks, Mo., pool	190.4	405.5
Tailwater	190.3	404.5
Bissell Point, Mo.	183.3	401.4
St Louis, Mo.	179.6	399.8
Engineer Depot, Mo.	176.8	398.4

DIRECT RECHARGE TO AQUIFER

Only a part of the annual precipitation reaches the water table. A large part of the precipitation runs overland to streams or is discharged by the process of evapotranspiration before it reaches the aquifer. The amount of precipitation that reaches the zone of saturation depends upon several factors. Among these are the character of the soil and other materials above the water table; the topography; vegetal cover; land use; soil moisture; the depth to the water table; the intensity duration, and seasonal distribution of rainfall; the occurrence of precipitation as rain or snow; and the air temperature.

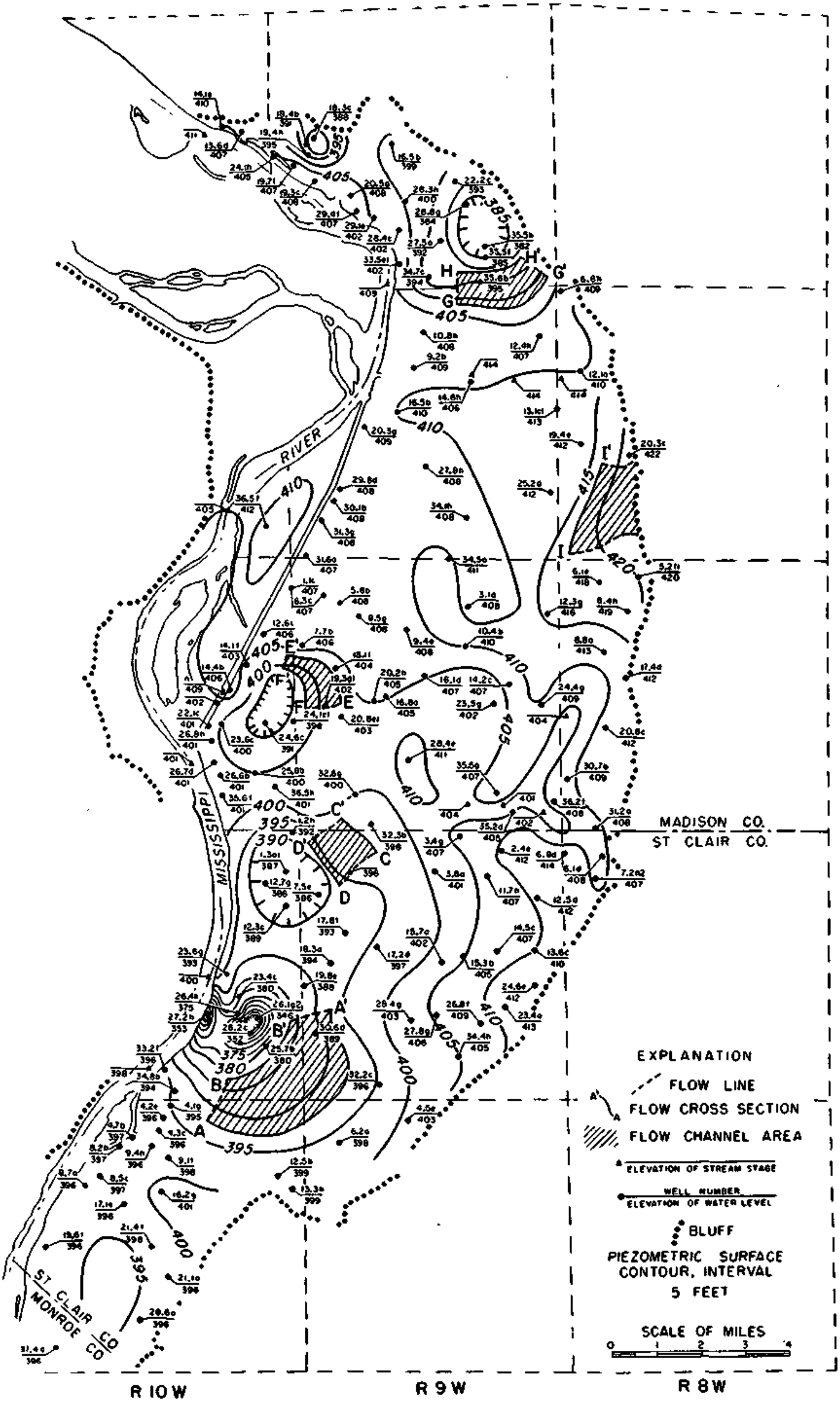


Figure 54. Approximate elevation of piezometric surface, June 1962

Generally ground-water recharge in the East St. Louis area is greatest in spring and early summer months of heavy rainfall and least in the late summer, fall, and winter months. Most recharge occurs during spring months when evapotranspiration is small and soil moisture is maintained at or above field capacity by frequent rains. During summer and fall months evapotranspiration and soil moisture requirements have first priority on precipitation and are so great that little precipitation percolates to the water table except during periods of excessive rainfall.

Recharge directly from precipitation was estimated by flow-net analyses of the piezometric surface in the vicinity of the Wood River, Granite City, National City, and Monsanto area pumping centers. The quantity of water percolating through a given cross section of an aquifer is proportional to the hydraulic gradient (slope of the piezometric surface) and the coefficient of transmissibility, and it can be computed by using the following modified form of the Darcy equation (see Ferris, 1959).

$$Q = TIL \qquad (10)$$

where:

Q = discharge through flow cross section, in gpd

T = coefficient of transmissibility, in gpd/ft

I = hydraulic gradient, in ft/mi

L = width of flow cross section, in mi

The rate of recharge directly from precipitation can be estimated on the basis of the difference in discharge of water through successive flow cross sections with the following equation (Walton, 1962):

$$R = [(Q_2 - Q_1) \pm \Delta h_t \, S \, A_t (2.1 \times 10^8)] / A_t \qquad (11)$$

where:

R = rate of recharge, in gpd/sq mi

$Q_2 - Q_1$ = difference in discharge of water through successive flow cross sections, in gpd

Δh_t = average rate of water-level decline or rise within area between successive flow cross sections, in fpd

A_t = surface area between successive flow cross sections, in sq mi

S = coefficient of storage of aquifer, 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 lines were drawn at right angles to the estimated piezometric surface contours for December 1956, June 1961, and June 1962 toward cones of depression in the Wood River, Granite City, National City, and Monsanto areas to delimit the flow channels in figures 52 through 54. The locations of flow channels were so chosen that recharge rates under all types of geologic, hydrologic, and land use conditions could be studied. The discharges through cross sections A—A', B—B', C—C', D—D', E—E', F—F', G—G', and H—H' were computed using equation 10 and figures 25 and 52 through 54. Differences in discharge of water through successive flow cross sections were determined. Average rates of water-level declines or rises within flow channel areas were estimated from hydrographs of observation wells. Surface areas of flow channels were obtained from figures 52 through 54. The average coefficient of storage of the coarser deposits was estimated to be 0.20 on the basis of aquifer-test data, and the average coefficient of storage of the finer grained alluvium was estimated to be 0.10 on the basis of studies by Schicht and Walton (1961). The data mentioned above were substituted in equation 11, and recharge rates for each flow channel area were computed.

Recharge rates vary from 299,000 gpd/sq mi in the National City area to 475,000 gpd/sq mi in the Wood River area. The average rate of recharge in the East St. Louis area is 371,000 gpd/sq mi. The East St. Louis area covers about 175 square miles. It is estimated that total recharge directly from precipitation to the East St. Louis area averages about 65 mgd.

The subsurface flow of water from the bluff was estimated by studying the movement of water through flow channels near the foot of the bluff. Flow lines were drawn at right angles to the bluff and the estimated piezometric surface contours for June 1961 and June 1962 to delimit the flow channels shown in figures 53 and 54. The discharge through cross sections I—I', J—J', and K—K' were computed using equation 10 and figures 25, 53, and 54. Average rates of water-level declines or rises within flow channel areas were estimated from hydrographs of observation wells. The average rates of changes in storage within flow channel areas were computed as the products of water-level changes, storage coefficients, and flow channel areas. Recharge directly from precipitation within flow channel areas was estimated as the products of the average recharge rate (371,000 gpd/sq mi) and flow channel areas. Recharge and changes in storage within flow channel areas were subtracted from the discharges through cross sections I—I', J—J', and K—K' to compute rates of subsurface flow of water from the bluff. The average rate of subsurface flow of water from the bluff is 329,000 gpd/mi. The length of the bluff forming the eastern boundary of the East St. Louis area is 39 miles. Thus, the total rate of subsurface flow of water from the bluffs is about 12.8 mgd.

RECHARGE FROM INDUCED INFILTRATION

The lowering of water levels in the Alton, Wood River, National City, and Monsanto areas that has accompanied withdrawals of ground water in these areas has established hydraulic gradients from the Mississippi River towards these pumping centers. In addition, lowering of water levels in the Granite City area has established a hydraulic gradient from the Chain of Rocks Canal towards the Granite City pumping center. Thus, ground-water levels are below the surface of the river and canal at places, and appreciable quantities of water percolate through the beds of the river and canal into the aquifer by the process of induced infiltration.

The volume of water percolating through the beds of the river and canal into the aquifer during 1961 was estimated by subtracting the volume of water recharged to the aquifer within areas of diversion directly from precipitation and subsurface flow from the bluff from the total volume of water pumped. In 1961 cones of depression were relatively stable and changes in storage within the aquifer during the year were very small. As shown in table 26 about 48.2 mgd or 50.0 percent of the total pumpage (96.8 mgd) was derived from induced infiltration of surface water in the Mississippi River. The piezometric surface map in figure 54 was used to delimit areas of diversion and lengths of bluff within areas of diversion. Recharge directly from precipitation was estimated as the products of areas of diversion and the average recharge rate (371,000 gpd/sq mi). Subsurface flow from the bluff was estimated as the products of lengths of bluff within areas of diversion and the average rate of subsurface flow (329,000 gpd/mi).

Table 26. Recharge by Source During 1961

Pumping center	Total pumpage (mgd)	Length of bluff within area of diversion (mi)	Recharge from bluff (mgd)	Area of diversion (sq mi)	Recharge from precipitation (mgd)	Recharge by induced infiltration (mgd)
Alton area	12.30	3.4	1.12	2.7	1.00	10.18
Wood River area	24.30	7.9	2.60	19.5	7.24	14.46
Poag area	1.20	neg	neg	3.2	1.20	0
Granite City area	8.80	0	0	20.6	7.65	1.15
Troy area	0.40	neg	neg	1.1	0.40	0
National City area	10.80	0	0	18.7	6.94	3.86
Fairmont City area	4.40	0	0	11.8	4.40	0
Caseyville area	2.40	2.9	0.95	3.9	1.44	0
Glen Carbon area	0.30	neg	neg	0.8	0.30	0
Monsanto area	31.90	2.3	0.76	34.0	12.61	18.53
Total	96.80		5.43		43.18	48.18

The amount of induced infiltration is dependent largely upon the infiltration rate of the river bed, the river-bed area of infiltration, the position of the water table, and the hydraulic properties of the aquifer.

Infiltration Rates of River Bed

The infiltration rate of the Mississippi River bed was determined with aquifer-test data. Methods of analysis of aquifer-test data affected by stream recharge were described by Rorabaugh (1956), and Hantush (1959). In addition, Walton (1963) introduced a method for determining the infiltration rate of a stream bed by aquifer-test analysis.

If the hydraulic properties of the aquifer and the distance a are known, the percentage of pumped water being diverted from a stream can be computed with the following equation derived by Theis (1941):

$$P_r = 2/\pi \int_0^{\pi/2} \exp(-f \sec^2 u)\, du \qquad (12)$$

where:

$u = \tan^{-1}(r_r/a)$
$f = 1.87a^2S/Tt$
P_r = percentage of pumped water being diverted from the stream
T = coefficient of transmissibility, in gpd/ft
S = coefficient of storage, fraction
a = distance from pumped well to recharge boundary, in ft
t = time after pumping started, in days
r_r = distance along recharge boundary measured from the perpendicular joining the real and image wells, in ft

Figure 55 gives values of P_r for various values of / and shows, therefore, the percentage of pumped water being diverted from the stream. The amount of recharge by induced infiltration is then given by the following equation:

$$Q_r = QP_r/100 \qquad (13)$$

where:

Q_r = amount of induced infiltration, in gpm
Q = discharge of pumped well, in gpm

Values of drawdown at several points within the stream bed equidistant upstream and downstream from the pumped well and between the line of recharge and the river's edge are computed, taking into consideration the effects of the image well associated with the line of recharge and the pumped well, with the following equations:

$$s = s_p - s_i \qquad (14)$$

$$s_p = 114.6QW(u_p)/T \qquad (15)$$

$$s_i = 114.6QW(u_i)/T \quad (16)$$

$$u_p = 2693r_p^2S/Tt \quad (17)$$

$$u_i = 2693r_i^2S/Tt \quad (18)$$

where:

s = drawdown at observation point, in ft
s_p = drawdown due to pumped well, in ft
s_i = buildup due to image well, in ft
Q = discharge of pumped well, in gpm
T = coefficient of transmissibility, in gpd/ft
S = coefficient of storage, fraction
r_p = distance from observation point to pumped well, in ft
r_i = distance from observation point to image well, in ft
t = time after pumping started, in min

The reach of the streambed, L_r, within the area of influence of pumping is determined by noting the location of the points upstream and downstream where drawdown is negligible (say $\leqq$ 0.01). The area of induced infiltration, A_r, is then the product of L_r and the average distance between the river's edge and the recharge boundary.

The infiltration rate of the stream bed per unit area can be computed with the following equation:

$$I_a = 6.3\times10^7Q_r/A_r \quad (19)$$

where:

I_a = average infiltration rate of stream bed, in gallons per day per acre (gpd/acre)
Q_r = amount of induced infiltration, in gpm
A_r = stream bed area of infiltration, in sq ft

Rough approximations of the average head loss, s_r, due to the vertical percolation of water through the stream bed can be determined by averaging drawdowns computed at many points within the area of infiltration. Values of drawdown within the stream-bed area of infiltration are computed, taking into consideration the pumped well and the image well associated with induced infiltration, with equations 14 through 18.

The average infiltration rate of the stream bed per unit area per foot of head loss can be estimated by use of the following equation:

$$I_h = I_a/s_r \quad (20)$$

where:

I_h = average infiltration rate of stream bed, in gallons per day per acre of stream bed per foot of head loss (gpd/acre/ft)
I_a = average infiltration rate of stream bed, in gpd/acre
s_r = average head loss within the stream bed area of infiltration, in ft

The infiltration rate of the Mississippi River bed at three sites was determined from aquifer-test data. The sites are just south of the confluence of Wood River and

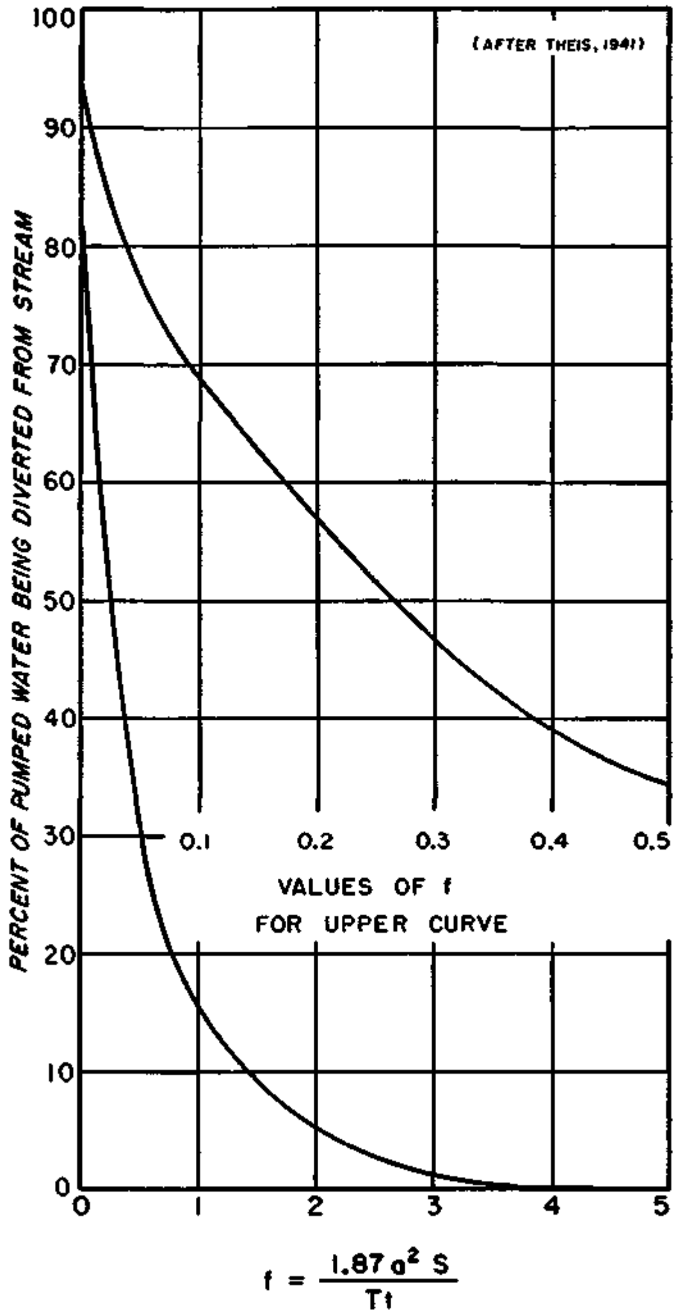


Figure 55. Graph showing the relationship between percent of pumped water being diverted from a stream and the factor 'f'

the Mississippi River, west of Wood River, and west of Monsanto. A summary of the results of aquifer tests and computed infiltration rates are given in table 27. The infiltration rate near the confluence of Wood River and the Mississippi River at a river temperature of 33F was estimated to be 305,000 gpd/acre/ft; the infiltration rate west of the city of Wood River was estimated to be 36,300 gpd/acre/ft; and the infiltration rate west of Monsanto at a river temperature of 83F was estimated to be 91,200 gpd/acre/ft.

Infiltration rates per foot of head loss vary with the temperature of the river water. Average monthly infiltra-

Table 27. Results of Aquifer Tests Affected by Induced Infiltration

Owner	Location	Date of test	Duration of test *(days)*	Pumping rate *(gpm)*	Hydraulic properties T *(gpd/ft)*	P *(gpd/sq ft)*	S *(fraction)*	I_a *(gpd/acre)*	s_r *(ft)*	I_h *(gpd/acre/ft)*	Kiver temperature *(°F)*
Olin Mathieson Chemical Corp.	Madison Cty. T5N, R9W sec 19	May 29- Jun 1, 1956; Feb 13-17, 1959	3 4	760 7000	100,000	1100	0.1	418,000	1.37	305,000	33
Shell Oil Co.	Madison Cty. T5N, R9W sec 33	Mar 3-6, 1952	3	510	190,000	1900	0.002	9,800	0.27	36,300	38
Monsanto Chemical Corp.	St. Clair Cty. T2N, R10W sec 27	Aug 4-8, 1952	4	1100	210,000	2800	0.08	15,500	0.17	91,200	83

tion rates (tables 28 and 29) were computed on the basis of average monthly river temperatures, figure 56, and the following equation:

$$I_t = I_h(\mu_a/\mu_t) \tag{21}$$

where:

I_t = average infiltration rate of river bed for a particular surface water temperature, in gpd/acre/ft

I_h = average infiltration rate of river bed determined from aquifer-test results, in gpd/acre/ft

μ_a = coefficient of viscosity at temperature of surface water during aquifer test, in centimeter-gram-seconds (cgs) units

μ_t = coefficient of viscosity at a particular temperature of surface water, in cgs units

Table 28. Average Monthly Infiltration Rates of Mississippi River Bed near Alton and Wood River

Month	Average river temperature at Alton 1940-1949 *(°F)*	Infiltration rate of river bed *(gpd/acre / ft)* West of Wood River	Near confluence of Wood and Mississippi Rivers
January	34	33,800	308,000
February	34	33,800	308,000
March	41	38,500	350,000
April	54	47,600	436,000
May	64	54,600	497,000
June	74	63,100	574,000
July	81	69,200	636,000
August	82	70,000	643,000
September	75	63,700	571,000
October	63	54,600	493,000
November	50	44,600	406,000
December	38	36,300	330,000

River-Bed Areas of Infiltration to Well Fields

Four well fields in the East St. Louis area are located close to the Mississippi River and derive most of their recharge from the induced infiltration of surface water. The well fields are south of Alton in the Duck Lake area, near the confluence of the Wood River and the Mississippi River, west of Wood River, and west of Monsanto as shown in figure 57.

One well field consisting of a collector well and two artificial pack wells is owned by the Shell Oil Refinery and is located about 100 feet east of the Mississippi River west of Wood River in sec 33, T5N, R9W. The design capacity of the well field is 5000 gpm or 7.2 mgd.

The position of the recharge boundary and the area of infiltration for the design capacity were determined by the process of trial and error. Several positions of the recharge boundary were assumed, and drawdown

Table 29. Average Monthly Infiltration Rates of Mississippi River Bed near Monsanto

Month	Average river temperature at East St. Louis 1940-1949 *(°F)*	Infiltration rate of river bed *(gpd/acre /ft)*
January	38	47,600
February	38	47,600
March	43	49,500
April	55	62,200
May	66	71,500
June	76	83,100
July	82	90,100
August	83	91,200
September	77	84,000
October	65	72,000
November	53	61,400
December	41	49,300

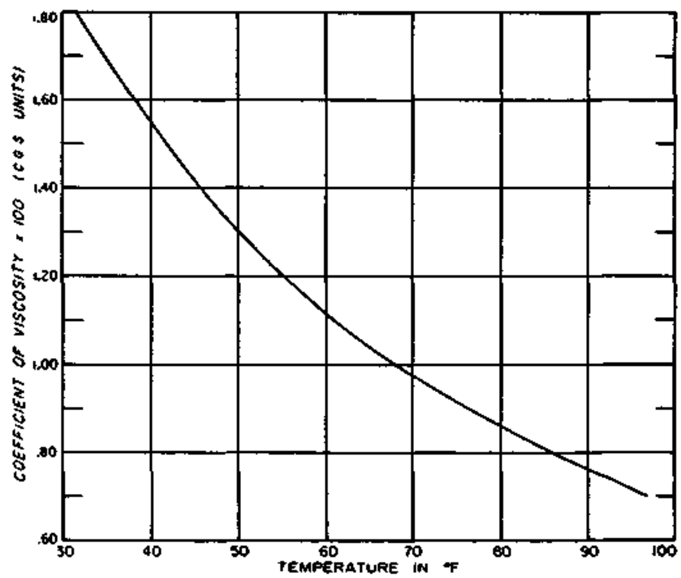


Figure 56. Graph showing relationship between coefficient of viscosity and temperature

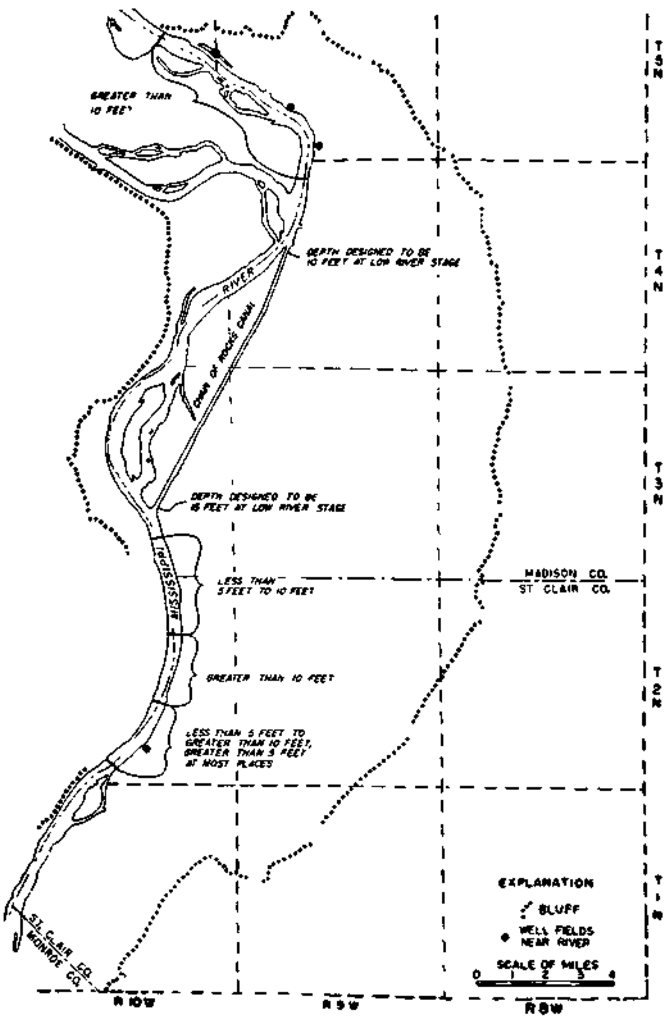


Figure 57. Estimated depths of Mississippi River and locations of well fields near river

beneath the river bed and the river-bed areas of infiltration were computed with equations 14 through 18. Values of R_i were then computed with equation 22 keeping in mind that s_r is either the average head loss within the river-bed area of infiltration or the average depth of water in the river, depending upon the drawdown beneath the river bed.

$$R_i = I_i \, s_r \, A_r \tag{22}$$

where:

R_i = potential recharge by induced infiltration, in gpd
I_i = average infiltration rate of river bed for a particular surface water temperature, in gpd/acre/ft
s_r = average head loss within river bed area of infiltration or average depth of water in river for a particular river stage, depending upon the position of the water table, in ft
A_r = river bed area of infiltration, in acres

The position of the recharge boundary and the river-bed area of infiltration which resulted in R_i balancing the design capacity were judged to be correct. The recharge boundary for the design capacity is located at a distance of 900 feet from the well field and the river-bed area of infiltration is 175 acres, as shown in figure 58.

The results of an aquifer test, made at a low pumping rate at the site of the well field, indicated a distance of 500 feet from the well field to the recharge boundary. Thus, the aquifer test at a low pumping rate indicated a certain position of the recharge boundary and a river-bed area of infiltration which were not valid for a higher pumping rate. At higher pumping rates water is withdrawn at a rate in excess of the ability of the river-bed to transmit it, and as a result the water table declines below portions of the river-bed. In such a case the recharge boundary moves away from the pumped wells as maximum infiltration occurs in the reach of the river in the immediate vicinity of the well field, the cone of depression spreads upstream and downstream, and the river-bed area of infiltration increases. Drawdowns in wells at higher pumping rates based on the position of the recharge boundary as determined from the aquifer-test data are much less than drawdowns based on the position of the recharge boundary as determined by trial and error with equation 22. Thus, the position of the recharge boundary determined from aquifer-test data cannot always be used to compute the potential yield of well fields that depend primarily upon induced infiltration of surface water as a source of recharge.

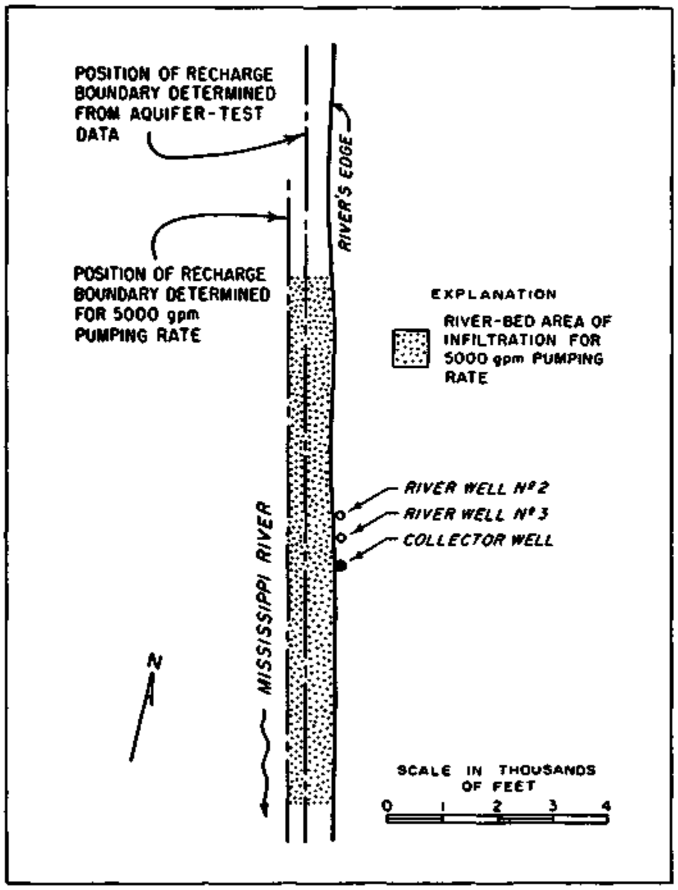


Figure 58. River-bed area of infiltration for Shell Oil Refinery well field

Potential recharge by the induced infiltration of surface water can be estimated on the basis of the infiltration rates in table 30, river depth records, water-level data, and river temperature data. Infiltration is directly proportional to the drawdown immediately below the stream bed and is at a maximum when the water table is immediately below the river bed. Under maximum infiltration conditions the average head loss within the river-bed area of infiltration is the average depth of water in the river for a particular river stage. Provided the water table remains below the river bed, the least amounts of induced infiltration occur during extended dry periods when streamflow and the temperature of the surface water are low. Profiles of the river channel can be used to determine the average depth of water in the river. Potential recharge by induced infiltration can be determined by substituting data in equation 22.

Table 30. Infiltration Rates of Stream Beds Determined from Aquifer-Test Data in Illinois, Indiana, and Ohio

Location of aquifer-test rite	Infiltration rate (gpd/acre/ft)	Surface water temperature (°F)	Infiltration rate at 40 F (gpd/acre/ft)
Along Mad River about 4 miles northwest of Springfield, Ohio*	1,000,000	39	1,010,000
Along Miami River 14 miles northwest of Cincinnati, Ohio*	168,000	82	91,100
Along White River immediately upstream from the confluence of White River and Killbuck Creek at Anderson, Indiana*	216,000	54	275,000
Along Sandy Creek 12 miles south of Canton, Ohio*	720,000	69	414,000
Along White River 1 mile west of Anderson, Indiana, ½ mile below sewage treatment plant*	39,800	35	43,600
Along Mississippi River near confluence of Wood River and Mississippi River above confluence of Mississippi and Missouri Rivers	305,000	33	344,000
Along Mississippi River west of the city of Wood River above confluence of Mississippi and Missouri Rivers	36,300	38	37,500
Along Mississippi River west of Monsanto below confluence of Mississippi and Missouri Rivers	91,200	83	48,300

**After Walton (1963)*

The average depth of water in the Mississippi River between the Illinois shore and a line 500 feet offshore was estimated from Mississippi River soundings made by the U.S. Corps of Engineers and low river stages during 1956 and 1957. The average depth of water exceeds 10 feet in places where the navigation channel is near the Illinois side, in the vicinity of Alton and Wood River, and along a small reach of the river near East St. Louis. The depth of water in the Chain of Rocks Canal is designed to be 10 feet or greater at low river stages. Estimated average depths of water in the river at low river stages are shown in figure 57.

A summary of the infiltration rates computed with aquifer-test data for the East St. Louis area is given in table 30. Infiltration rates of stream beds in Ohio and Indiana (Walton, 1963) are also listed. Infiltration rates in table 30 were adjusted to a river temperature of 40F. A comparison of the adjusted infiltration rates with infiltration rate data for slow and rapid sand filters (Fair and Geyer, 1954) indicates that all stream bed infiltration rates fall into the clogged slow sand filter category.

The least permeable reach of river bed in the East St. Louis area is west of Wood River above the confluence of the Mississippi and Missouri Rivers. The infiltration rate along this reach and the infiltration rate of the reach of river bed west of Monsanto below the confluence of the Mississippi and Missouri Rivers are low and in the same range as the infiltration rate for the White River west of Anderson, Indiana, below a sewage treatment plant. Walton (1963) states that the infiltration rate of the White River site is probably low largely because of the clogging effects of sewage.

The highest infiltration rate in the East St. Louis area was computed for the reach of river bed near the confluence of the Wood and Mississippi Rivers above the confluence of the Mississippi and Missouri Rivers. The Missouri River generally carries a greater sediment load than the Mississippi River; thus it would be expected that the average infiltration rate above the Missouri River would be greater than the average infiltration rate below it.

The infiltration rate of the Mississippi River bed west of the city of Wood River ranges from 33,800 gpd/acre/ft at an average river temperature of 34F in January and February to 70,000 gpd/acre/ft in August when the average river temperature is 82F. The infiltration rate of the river bed near the confluence of the Wood and the Mississippi Rivers ranges from 308,000 gpd/acre/ft in January and February to 643,000 gpd/acre/ft in August. West of Monsanto the infiltration rate of the river bed varies from 47,600 gpd/acre/ft at an average river temperature of 38F in January and February to 91,200 gpd/acre/ft at an average river temperature of 83F in August.

An electric analog computer (see Walton and Prickett, 1963) for the East St. Louis area was constructed so that the consequences of further development of the aquifer could be forecast, the practical sustained yield of existing pumping centers could be evaluated, and the potential yield of the aquifer with a selected scheme of development could be appraised. The electric analog computer consists of an analog model and excitation-response apparatus, i.e., waveform generator, pulse generator, and oscilloscope.

The analog model is a regular array of resistors and capacitors and is a scaled down version of the aquifer. Resistors are inversely proportional to the coefficients of transmissibility of the aquifer, and capacitors store electrostatic energy in a manner analogous to the storage of water in the aquifer. Hydrogeologic maps and data presented earlier in this report describing the following factors were used in constructing the analog model: 1) coefficient of transmissibility of the aquifer, 2) coefficient of storage of the aquifer, 3) areal extent of the aquifer, 4) saturated thickness of the aquifer, and 5) location, extent, and nature of aquifer boundaries. All nonhomogeneous and irregular hydrogeologic conditions were incorporated in the analog model.

Questions pertaining to the utilization of groundwater resources of the East St. Louis area require that pumping be related to water-level change with reference to time and space. Changes in water levels due to the withdrawal of water from the aquifer must be determined. Excitation-response apparatus force electric energy in the proper time phase into the analog model and measure energy levels within the energy-dissipative resistor-capacitor network. Oscilloscope traces, i.e., time-voltage graphs, are analogous to time-drawdown graphs that would result after a step function-type change in withdrawal of water. A catalog of time-voltage graphs provides data for construction of a series of water-level change maps. Thus, the electric analog computer provides a means of relating cause and effect relationships for the aquifer. A schematic diagram of the electric analog computer is shown in figure 59.

Analog Model

The analog model for simulating the aquifer in the East St. Louis area was patterned after analog models developed by H. E. Skibitzke, mathematician, U.S. Geological Survey, Phoenix, Arizona. The analog model consists of a regular array of 2800 resistors and 1350 capacitors. The analog model was constructed with a piece of 1/8-inch pegboard perforated with holes on a 1-inch square pattern approximately 2 x 5 feet corresponding to the dimensions of, the topographic map of the area (7.5 minute quadrangle maps). Aluminum angles (l x 1 inch) were attached along the four edges of the pegboard with metal screws to enable setting the model on a table or against a wall without disturbing capacitors of the analog model installed on the underneath side of the pegboard. Coefficient of transmissibility contours were transferred from figure 25 to topographic maps of the area which were in turn pasted on the pegboard. No. 3 brass laquered shoe eyelets were inserted in the holes of the pegboard to provide terminals for resistors and capacitors. Four resistors and a capacitor were connected to each interior terminal; the capacitor was secured to a ground wire connection of the electrical system. Two or three resistors and a capacitor were connected to boundary terminals, depending upon the geometry of the boundary. The model is bounded on the west by a recharge boundary, the Mississippi River and the Chain of Rocks Canal; the portion of the network along the recharge boundary was terminated in a short circuit. The recharge boundary of the network was adjusted in a step fashion to approximate the actual boundary of the aquifer. The model is bounded on the north, east, and southeast, by bluffs through which there is a small amount of subsurface flow. Resistors large in magnitude which simulate small amounts of subsurface flow through the bluff were connected to terminals along the north, east, and southeast boundaries of the analog model and to the ground connection of the electrical system. The model was terminated south of Dupo. A termination strip was constructed to extend the aquifer 5 miles south of Dupo (see Karplus, 1958).

Because the aquifer is a continuous phenomena while the resistor-capacitor network consists of many discrete branches, the network is only an approximation of a true analog. However, it can be shown mathematically that if the mesh size of the network is small in comparison with the size of the aquifer, the behavior of the network describes very closely the response of the aquifer to pumping.

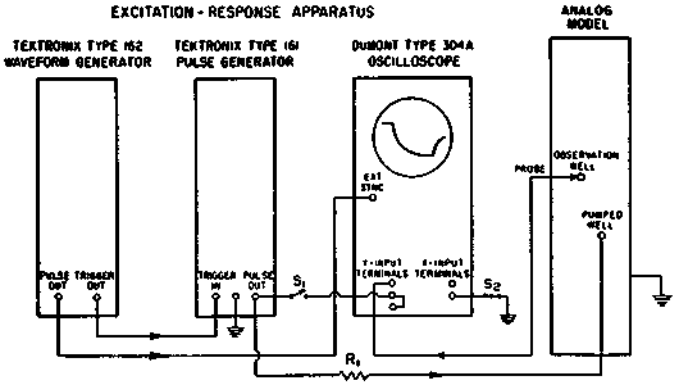


Figure 59. Schematic diagram of electric analog computer

The model was developed on the premise that ground-water flow in the East St. Louis area is two-dimensional. The finite-difference form of the partial differential equation (Jacob, 1950) governing the nonsteady state two-dimensional flow of ground-water is (see Stallman, 1956):

$$T\,(\Sigma_2^5\, h_i - 4h_1) = a^2\, S\,(\partial h/\partial t) \qquad (23)$$

where:

h_1 = head at node 1 (see figure 60A; the aquifer is subdivided into small squares of equal area, the intersections of grid lines are called nodes); h_1 (i = 2, 3, 4, and 5) = heads at nodes 2 to 5; a = width of grid interval; T = coefficient of transmissibility; and S = coefficient of storage.

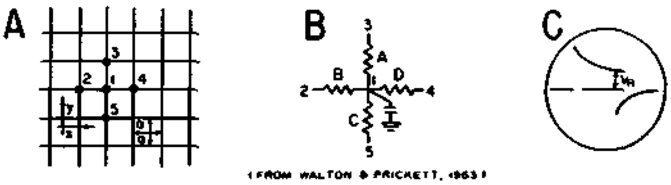


Figure 60. Finite-difference grid (A), resistor-capacitor net (B), and pumping rate oscilloscope trace (C)

Consider a resistor-capacitor network with a square pattern as shown in figure 60A and network junctions at nodes as defined in figure 60B. The junctions consist of four resistors of equal value and one capacitor connected to a common terminal; the capacitor is also connected to ground. The relation of electrical potentials in the vicinity of the junction, according to Kirchhoff's current law, can be expressed by the following equation (see Millman and Seely, 1941; and Skibitske. 1961):

$$1/R\,(\Sigma_2^5\, V_i - 4V_1) = C\,(\partial V/\partial t) \qquad (24)$$

where:

$V_{1\text{-}5}$ = electrical potential at ends of resistors; $R_{A\text{-}D}$ = resistance; and C = capacitance; V_i (i=2, 3, 4, and 5) = electrical potential at ends of resistors A–D.

Comparison of equations 23 and 24 shows that the finite-difference equation governing the nonsteady state two-dimensional flow of ground water in an infinite aquifer is of the same form as the equation governing the flow of electrical current in a resistor-capacitor network. For every term in equation 23 there is a corresponding term of the same order of differentiation in equation 24.

The analogy between electrical and aquifer systems is apparent. The hydraulic heads, *h*, are analogous to electrical potentials, *V*. The coefficient of transmissibility, *T*, is analogous to the reciprocal of the electrical resistance, *1/R*. The product of the coefficient of storage, *S*, and a^2 is analogous to the electrical capacitance, *C*.

Continuing the comparison, water moves in an aquifer just as charges move in an electrical circuit. The quantity of water is reckoned in gallons while the charge is in coulombs. The rate of flow of water past any point in the aquifer is expressed in gallons per day while the flow of electricity is in coulombs per second or amperes. The hydraulic head loss between two points in an aquifer is expressed in feet while the potential drop across a part of the electrical circuit is in volts.

Thus, there are four units which are analogous; there is necessarily a scale factor connecting each unit in one system to the analogous unit in the other system. Knowing the four scale factors the hydrologist is able to relate electrical units associated with the analog model to hydraulic units associated with an aquifer. The four scale factors, K_1, K_2, K_3, and K_4, were defined by Bermes (1960) as follows:

$$q = K_1 \Omega \qquad (25)$$

$$h = K_2 V \qquad (26)$$

$$Q = K_3 I \qquad (27)$$

$$t_d = K_4 t_s \qquad (28)$$

where:

q = gallons; Ω = coulombs; Q = gallons per day; I = amperes; h = feet; V = volts; t_d = days; t_s = seconds; K_1 = gal/coulomb; K_2 = feet/volt; K_3 = gal/day/ampere; and K_4 = days/sec.

The relation between scale factors K_1, K_3, and K_4 is expressed by the following equation (Bermes, 1960):

$$(K_3 K_4)/K_1 = 1 \qquad (29)$$

The analogy between Ohm's law and Darcy's law is established by the fact that the coefficient of transmissibility is analogous to the reciprocal of the electric resistance. Substitution of these laws in equation 27 results in the following equation which may be used to determine the values of the resistors of the interior portions of the analog model (see Bermes, 1960):

$$R = K_3/(K_2 T) \qquad (30)$$

where:

R = resistance, in ohms; and *T* = coefficient of transmissibility, in gpd/ft.

The following equation (see Bermes, 1960), which may be used to determine the values of the capacitors of the interior portions of the analog model, may be derived by taking into consideration the definitions of the coefficient of storage and capacitance and the analogy between (a^2S) and *C.*

$$C = 7.48\, a^2\, S\,(K_2/K_1) \qquad (31)$$

where:

G = capacitance, in farads; *a* = network spacing, in feet; and S = coefficient of storage, fraction.

A network spacing of 1 inch equals 2000 feet was selected to minimize the errors due to finite-difference approximation. Equations given by Karplus (1958) suggest that the selected network spacing is adequate.

By the process of trial and error, scale factors were chosen so that readily available and inexpensive resistors and capacitors and existing excitation-response apparatus could be used.

Selected analog scale factors are given below:

$K_1 = 1.826 \times 10^{15}$ gallons/coulomb

$K_2 = 1$ ft/volt

$K_3 = 1 \times 10^{10}$ gal/day/amp

$K_4 = 1.826 \times 10^{5}$ days/sec

A maximum pumping period, t_d, of 5 years was chosen, which is a sufficient period for water levels to stabilize under the influence of recharge from the Mississippi River. According to equation 28, with a K_4 = $1.826 \text{X} 10^5$ days/sec and when t_d = 5 years, the pulse duration, t_8, is equal to 10^{-2} seconds. The pulse generator has a maximum pulse duration of 10^{-2} seconds. A scale factor K_2 of 1 ft/volt was selected for ease in reading the oscilloscope graph.

A generalization of equations 23 and 24 permits accounting for variations in space of the coefficients of transmissibility and storage by varying resistors and capacitors. Fixed carbon resistors with tolerances of ± 10 percent and ceramic capacitors with tolerances of ± 10 percent were used in constructing the analog model.

Values of resistors were computed from equation 30 using data on the coefficient of transmissibility given in figure 25. Values of resistors in the internal parts of the model range in magnitude from 470,000 ohms near the bluff where T is about 20,000 gpd/ft to 33,000 ohms near Monsanto where T is about 330,000 gpd/ft. Resistors are greatest in magnitude, 2,200,000 ohms, along the valley wall where the coefficient of transmissibility is about 5000 gpd/ft

Values of the capacitors of the interior portions of the model were computed from equation 31 to be 2500 micro-micro farads. The long-term coefficient of storage substituted in equation 31 was 0.15.

Excitation-Response Apparatus

The excitation-response apparatus consists of three major parts as shown in figure 60: a waveform generator, a pulse generator, and an oscilloscope. The waveform generator which produces sawtooth pulses is connected to the trigger circuits of the pulse generator and oscilloscope, thereby controlling the repetition rate of computation and synchronizing the oscilloscope's horizontal sweep and the output of the pulse generator. The pulse generator, which produces rectangular pulses of various duration and amplitude upon command from the waveform generator, is coupled to that junction in the analog model representing the pumped well. The oscilloscope is connected to junctions of the analog model where it is desired to determine the response of the analog model to excitation. An electron beam is swept across the cathode ray tube of the oscilloscope providing a time-voltage graph which is analogous to the time-drawdown graph for an observation well. The waveform generator sends a positive pulse to the oscilloscope to start its horizontal sweep; at the same time, it sends a negative sawtooth waveform to the pulse generator. At a point along the sawtooth waveform the pulse generator is triggered to produce a negative rectangular pulse. The duration of this pulse is analogous to the pumping period, t_d, and the amplitude is analogous to the pumping rate, Q. This pulse is sensed by the oscilloscope as a function of the analog model components, boundary conditions, and node position of the junction connected to the oscilloscope. Thus, the oscilloscope trace is analogous to the water-level fluctuation that would result after a step function-type pumpage change of known duration and amplitude. To provide data independent of the pulse repetition rate, the interval between pulses is kept several times the longest time constant in the analog model. The time constant is the product of the capacitance at a point and the resistance in its discharge path.

A means of computing the pumping rate is incorporated in the circuit between the pulse generator and the analog model by the small resistor, R_i, in series, shown in figure 59. Substitution of Ohm's law in equation 27 results in the following equation which may be used to compute the pumping rate:

$$Q = (V_R / 1.44 \times 10^3 \, R_i) \, K_3 \qquad (32)$$

where:

Q = pumping rate, in gpm; V_R = voltage drop across the resistor R_i, in volts; and R_i = calibrated resistance, in ohms.

The voltage drop across the calibrated resistor is measured with the oscilloscope. Switches S_1 and S_2 are closed and opened, respectively, and the oscilloscope is connected to the pumped well junction. The waveform in figure 60C appears on the cathode ray tube; the vertical distance as shown is the desired voltage drop, V_R.

The switches S_1 and S_2 are returned to their original positions. The oscilloscope is then connected to all junctions of the analog model representing observation wells. The screen of the oscilloscope is accurately calibrated so that voltage and time may be used on the vertical and horizontal axis, respectively. The time is in seconds; the value of each horizontal division on the screen is determined by noting the duration of the rectangular pulse and the number of divisions covered by the time-voltage trace for a junction adjacent to the pumped well. The time-voltage graphs obtained from the oscilloscope can be converted into time-drawdown graphs with equations 26 and 28 which relate electrical units to hydraulic units. A catalog of time-drawdown graphs provides data for the construction of a series of water-level change contour maps. Thus, water-level changes are described everywhere in the aquifer for any desired pumping period. The pulse generator can be coupled to many junctions, and a variety of pumping conditions can be studied.

The effects of complex pumpage changes on water levels may be determined by approximating the pumpage graph by a group of step functions and analyzing the effect of each step function separately. The total water-level change, based on the superposition theorem, is obtained by summation of individual step-function water-level changes.

The pulse generator has a maximum output of 50 volts and 20 milliamperes; the pulse generator and oscilloscope have rise times less than 1 microsecond and waveform durations from less than 10 microseconds to 100 milliseconds. The performance specifications of the waveform generator, pulse generator, and oscilloscope are compatible with the following desired criteria for analog computers: low power requirements, respective calculation at variable rates, and fast computing speeds.

Accuracy and Reliability of Computer

The accuracy and reliability of the electric analog computer were assessed by a study of records of past pumpage and water levels. Water-level declines and piezometric surface maps obtained with the electric analog computer were compared with actual water-level declines and piezometric surface maps. The piezometric surface map for December 1956 (see figure 61A) was used to appraise the accuracy and reliability of the electric analog computer. The effects of the prolonged drought (1952-1956) on water levels are reflected in the piezometric surface. Hydrographs of observation wells

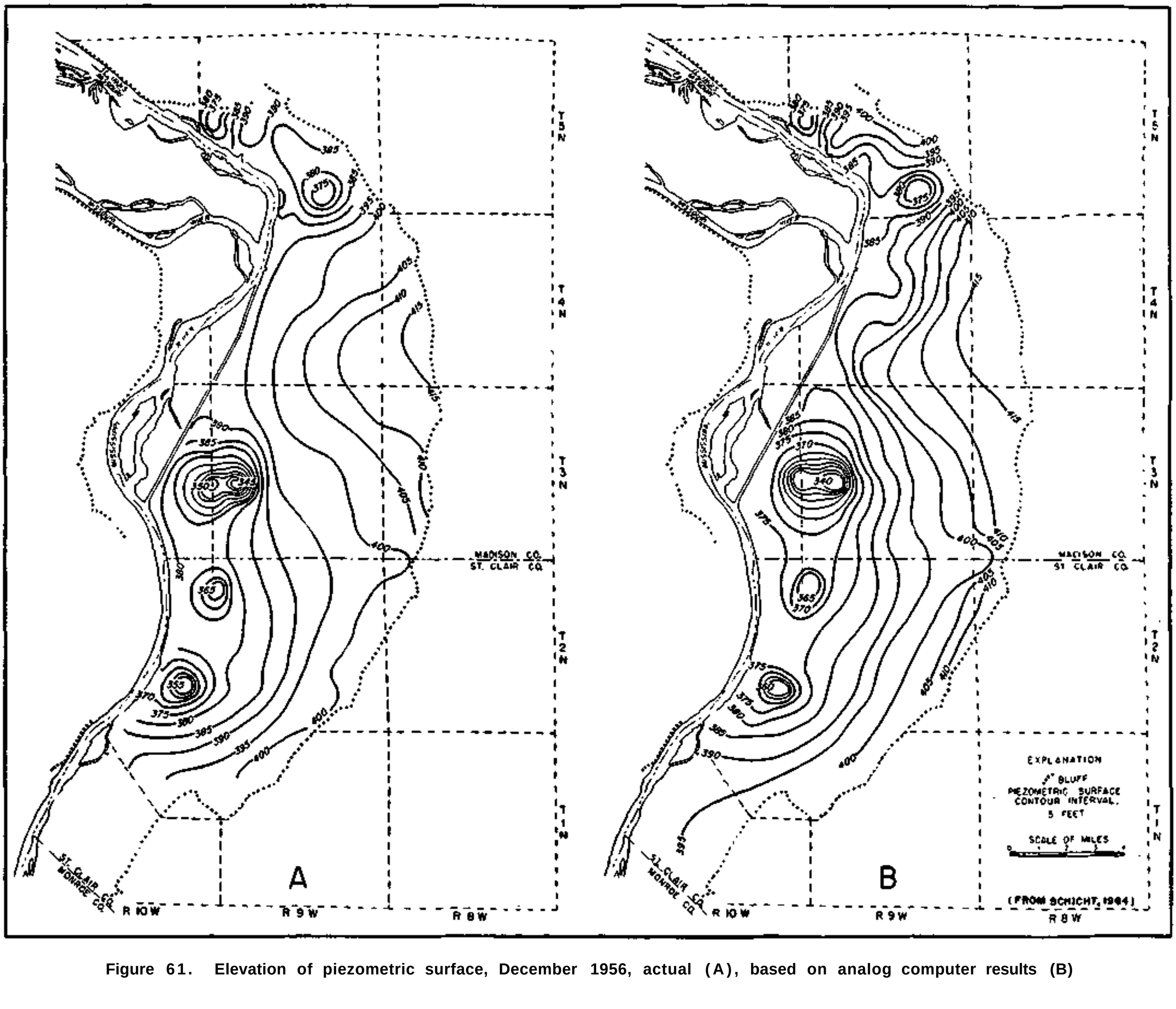


Figure 61. Elevation of piezometric surface, December 1956, actual (A), based on analog computer results (B)

indicate that stabilization of the piezometric surface during 1956 was mostly due to the effects of the Mississippi River. During much of the latter part of the drought there were long periods when little water was in the small streams and lakes in the interior portion of the East St. Louis area, and these hydrologic features had for practical purposes negligible influence on water levels.

Computations made with equation 4, taking into consideration the Mississippi River (recharge boundary) and accumulated periods of little or no recharge directly from precipitation, indicate that the piezometric surface for 1956 can be duplicated by using a time period of 5 years in estimating water-level declines.

Production wells were grouped into centers of pumping, and the average discharges during the period 1952-1956 for each pumping center were determined. The analog model was coupled to the excitation-response apparatus and the pulse generator was connected to junctions at locations of pumping centers. The output of the pulse generator was adjusted in accordance with discharge data and a maximum time period of 5 years. The oscilloscope was connected to terminals representing observation wells and water-level declines were computed. Thus, water-level declines everywhere in the aquifer were described. The total water-level decline, based on the superposition theorem, at each terminal was obtained by summation of individual effects of each pumping center. Only the effects of pumping centers were taken into account and the average stage of the Mississippi River was assumed to be the same in 1956 as it was in 1900. However, records show that the average stage of the Mississippi River was about 11 feet lower in 1956 than in 1900. The effect of the change in the average stage of the river on water levels was estimated by coupling the, pulse generator to junctions in the analog model along the river and measuring water-level changes due to the given change of the stage of the river with the oscilloscope connected to junctions in the interior portions of the analog model.

The above water-level declines due to the decline in river stage were superposed upon water-level changes due to pumpage, and a water-level change map covering the period 1900 to December 1956 was prepared. A piezometric surface map (figure 61B) was constructed by superposing the water-level change map on the piezometric surface map for 1900.

Features of the piezometric surface map prepared with data from the analog computer and the piezometric surface map prepared from actual water-level data are generally the same, as shown in figure 61. A comparison of water-level elevations for selected pumping centers, based on the analog computer and actual piezometric surface maps, are given in table 31. The average slope of

Table 31. Comparison of Analog Computer and Actual Piezometric Surface Maps for December 1956

Pumping center	Water-level elevation (ft above msl): Analog computer	Actual
Alton area	375	375
Wood River area	375	375
Granite City area	345	350
National City area	365	365
Monsanto area	360	355
Caseyville area	400	400

the piezometric surface in areas remote from pumping centers from both maps was 5 feet per mile. A comparison of gradients from analog computer and actual piezometric surface maps in the vicinity of pumping centers is given in table 32.

Table 32. Comparison of Analog Computer and Actual Hydraulic Gradients of Piezometric Surface Maps for December 1956

Pumping center	Average gradient (ft/mi): Analog computer	Actual
Alton area	15	15
Wood River area	15	15
Granite City area	20	30
National City area	10	10
Monsanto area	20	25

Differences in analog computer and actual piezometric surface maps are not significant when considered in relation to the accuracy and adequacy of geohydrologic data. The close agreement between analog computer and actual piezometric maps indicates that the analog computer may be used to predict with reasonable accuracy the effects of future ground-water development and the practical sustained yield of existing pumping centers.

PRACTICAL SUSTAINED YIELDS OF EXISTING PUMPING CENTERS

In 1962 water levels were not at critical stages in any pumping center and there were areas of the aquifer unaffected by pumping. Thus, the practical sustained yield of existing pumping centers exceeds total withdrawals in 1962. The practical sustained yield is here defined as the rate at which ground water can be continuously withdrawn from wells in existing pumping centers without lowering water levels to critical stages or exceeding recharge. Ground water withdrawn from wells less than 1 mile from the river was not considered.

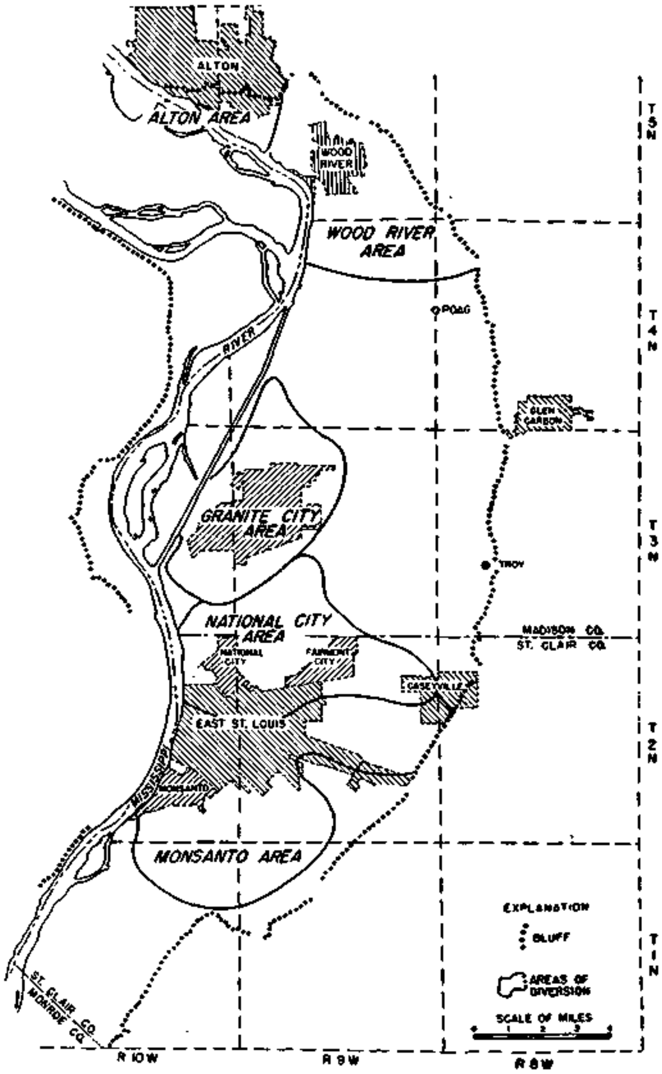


Figure 62. Areas of diversion in November 1961

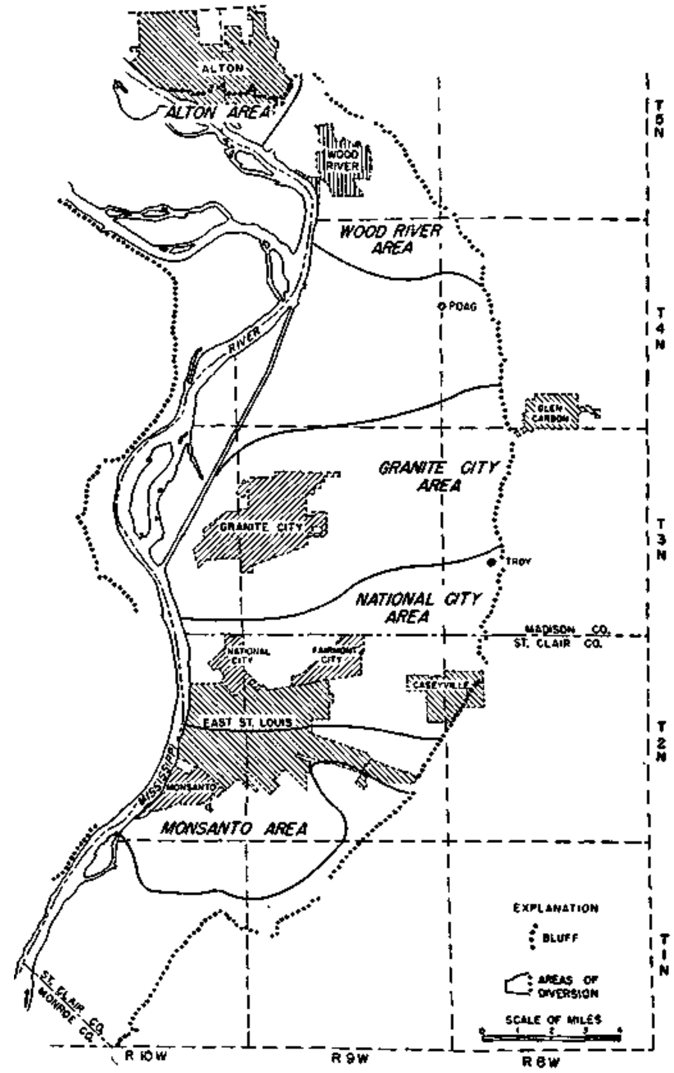


Figure 63. Areas of diversion in December 1956

Areas of diversion of pumping centers in November 1961 are shown in figure 62. The boundaries of areas of diversion delimit areas within which the general movement of ground water is toward production wells. The area (59 sq mi) north and east of Granite City and south of Wood River and a larger area south of Prairie Du Pont Creek through Dupo and south along the Mississippi River were outside areas of diversion. As shown in figure 63, the area north of Granite City outside areas of diversion was much smaller, covering about 30 sq mi, in December 1956. Pumpage in the Granite City area was 30.1 mgd in 1956 and 8.8 mgd in 1961.

Most of the coefficient of transmissibility of the valley fill deposits can be attributed to the coarse alluvial and valley-train sand and gravel encountered in the lower part of the valley fill. The thickness of the medium sand and coarser alluvial and valley-train deposits was determined from logs of wells and is shown in figure 64. The thickness of the coarse alluvial and valley-train sand and gravel exceeds 60 feet in an area south of Alton along the Mississippi River, in an area near Wood River, in places along the Chain of Rocks Canal, in a strip 1/2 mile wide and about 3 miles long through National City, in the Monsanto and Dupo areas, and in a strip about 1 mile wide and 4 miles long near Fairmont City. Thicknesses average 40 feet over a large part of the East St. Louis area. The coarser deposits diminish in thickness near the bluff, west of the Chain of Rocks Canal, and in places along the Mississippi River.

The available drawdown to the top of the medium sand and coarser deposits was estimated by comparing elevations of the top of the medium sand and coarser deposits with elevations of the piezometric surface map for June 1962 (figure 54). As shown in figure 64, available drawdown is greatest in undeveloped areas, exceeding 80 feet in the vicinity of Long Lake and in an area south of Horseshoe Lake. In a large part of the area available drawdown exceeds 60 feet. Average available drawdown within pumping centers was estimated to be 40 feet in the Alton area, 20 feet in the Wood River area, 35 feet

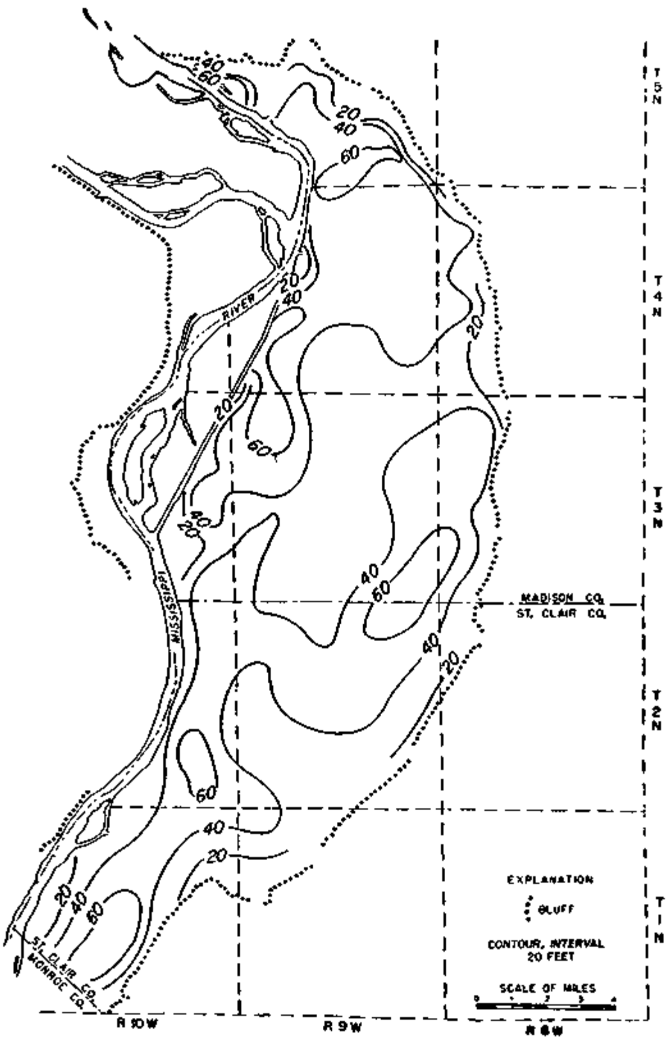


Figure 64. Thickness of medium sand and coarser deposits in lower part of valley fill

in the Granite City area, 30 feet in the National City area, and 30 feet in the Monsanto area.

When pumping water levels in individual production wells are below tops of screens, partial clogging of screen openings and the pores of the deposits in the immediate vicinity of the wells is greatly accelerated. To insure long service lives of wells, pumping water levels should be kept above tops of screens. Also, when water levels decline to stages below the top of the coarse alluvial and valley-train sand and gravel and more than one-half of the aquifer is dewatered, drawdowns due to the effects of dewatering become excessive and the yields of wells greatly decrease. Thus, critical water levels occur when pumping water levels are below tops of screens, or more than one-half of the aquifer is dewatered, or both.

Critical nonpumping water levels for existing pumping centers (table 33) were estimated on the basis of well-construction and performance data and figures 6, 64, and 65 taking into consideration the effects of dewatering. After critical water levels have been reached, individual wells in pumping centers will have yields exceeding 450 gpm.

Table 33. Critical Nonpumping Water-Level Elevations for Existing Pumping Centers

Pumping center	Average critical nonpumping water-level elevation (ft above msl)
Alton area	375
Wood River area	369
Granite City area	374
National City area	374
Monsanto area	369

The electric analog computer with a pumping period of 5 years was used to determine pumping center discharge rates that would cause water levels in all major pumping centers to decline to the critical stages in table 33. Several values of discharge were assumed and water-level declines throughout the East St. Louis area were determined. Water-level declines were superposed on the 1900 piezometric surface map together with changes in

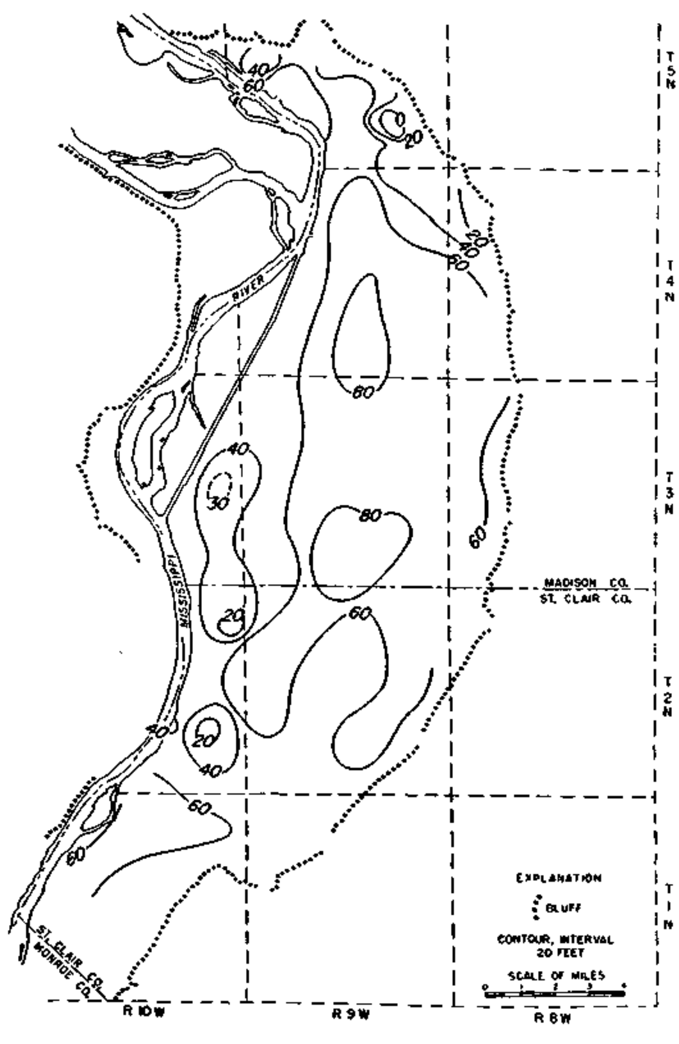


Figure 65. Estimated available drawdown to top of medium sand and coarser deposits in June 1962

water levels due to the changes in the stage of the Mississippi River, and piezometric surface maps under assumed pumping conditions were prepared. The pumping center discharge rates that resulted in a piezometric surface map with the critical water-level elevations in table 33 were assigned to the practical sustained yields of the pumping centers. The practical sustained yields of the existing pumping centers are given in table 34.

Table 34. Practical Sustained Yields of Existing Major Pumping Centers

Pumping center	1962 pumping rate *(mgd)*	Additional possible withdrawal *(mgd)*	Practical sustained yield *(mgd)*	Year after which practical sustained yield may be exceeded
Alton area	6.3	9.7	16	2000
Wood River area	14.1	5.9	20	1990
Granite City area	9.5	5.5	15	1980
National City area	11.6	6.4	18	2000
Monsanto area	22.6	0.4	23	1965
Total	64.1	27.9	92	

Estimates were made of the probable dates when practical sustained yields of existing pumping centers may be exceeded. Pumpage totals from 1890 through 1962 in the Alton, Wood River, Granite City, National City, and Monsanto areas are shown in figures 35 and 36. The past average rate of pumpage increase in each pumping center was estimated and extended to intersect the practical sustained yield of each pumping center. The assumption was made that the distribution of pumpage will remain the same as it was in 1962. It is estimated that the practical sustained yield of the Alton area pumping center (16 mgd) will be reached after the year 2000; the practical sustained yield of the Wood River area pumping center (20 mgd) will be reached about 1990; and the practical sustained yield of the Granite City area pumping center (15 mgd) will be reached about 1980.

It is estimated that the practical sustained yield of the National City area pumping center (18 mgd) will be reached about the year 2000. The rate of pumpage growth in the National City area may increase markedly, however, because of the effects of a series of drainage wells being installed to permanently dewater a cut along an interstate highway near National City. Pumpage from the drainage wells was not known at the time this report was prepared.

Pumpage in the Monsanto area during 1962 (22.6 mgd) is near the estimated practical sustained yield of 23 mgd.

No great accuracy is inferred for the estimated dates when practical sustained yields may be exceeded in table 34; they are given only to aid future water planning. A reasonable extrapolation of the pumpage graphs in figures 35 and 36 suggests that total ground-water withdrawals from wells in existing major pumping centers will exceed the practical sustained yields by about 2000.

POTENTIAL YIELD OF AQUIFER WITH A SELECTED SCHEME OF DEVELOPMENT

The electric analog computer was used to describe the effects of a selected scheme of development and to determine the potential yield of the aquifer under assumed pumping conditions. The potential yield of the aquifer is here defined as the maximum amount of water that can be continuously withdrawn from a selected system of well fields without creating critical water levels or exceeding recharge.

The distribution of pumpage with the selected scheme of development is shown in figure 66. A comparison of figures 66 and 34 shows that, with the exceptions of three new pumping centers near the river and one new pumping center in the Dupo area, the selected scheme of development is the same as the actual scheme of development in 1962.

Critical nonpumping water levels for existing and assumed pumping centers (see table 33) were estimated from figures 6, 64, and 65 taking into consideration the effects of dewatering. The electric analog computer was used to determine pumping center discharge rates that would cause water levels in all major pumping centers to decline to the critical stages in table 33. Several values of discharge in major pumping centers and anticipated discharge rates for minor pumping centers based on extrapolations of pumpage graphs for minor pumpage centers to the year 2015 were assumed and water-level declines throughout the East St. Louis area were determined. Model aquifers and mathematical models (Walton, 1962) based on available geohydrologic data and information on induced infiltration rates were used to determine the local effects of withdrawals in pumping centers near the river. Water-level declines were superposed on the piezometric surface map for 1900 together with changes in water levels due to the changes in the stage of the Mississippi River, and piezometric surface maps under assumed pumping conditions were prepared. The total pumping center discharge rate that resulted in a piezometric surface map with the critical water-level elevations in table 33 was assigned to the potential yield of the aquifer with the selected scheme of development. The potential yield, subdivided by pumping center, is given in table 35; water-level declines and approximate elevations of the piezometric surface with the selected scheme of development are shown in figures 67 and 68, respectively.

The pumpage graph in figure 32 was extrapolated into the future. Assuming that pumpage will continue to grow in the future as it has in the past, total pumpage in the East St. Louis area will exceed the potential yield with the selected scheme of development (188 mgd) after about 52 years or by 2015. A careful study of figures 25 and 66 and data on infiltration rates of the Mississippi River indicates that there are sites near the river where additional pumping centers could be developed. Thus, the potential yield of the aquifer with other possible schemes of development exceeds 188 mgd.

Recharge by Source

Flow lines were drawn at right angles to piezometric surface contours in figure 68 and areas of diversion (see figure 69) of pumping centers were delineated. Recharge directly from precipitation to each pumping center was computed as the product of areas of diversion and the average recharge rate (370,000 gpd/sq mi). Recharge from subsurface flow through the bluffs to each pumping center was computed as the product of the lengths of the bluff within areas of diversion and the average rate of subsurface flow (329,000 gpd/mi). Recharge from induced infiltration of surface water in the Mississippi River to each pumping center was determined by subtracting the sums of recharge directly from precipitation and subsurface flow from discharge rates in table 33. Recharge subdivided by source is given in table 36.

It is estimated that 36.5 percent of the total potential yield of the aquifer with the selected scheme of development will be derived from recharge directly from precipitation; about 57.3 percent will be derived from recharge by induced infiltration of surface water; and about 6.2 percent will be derived from recharge by subsurface flow through the bluffs.

Recharge amounts in 1956 and 1961, subdivided by source, are also given in table 36. The percentage of recharge from induced infiltration of surface water increases as the total withdrawal rate increases. As shown

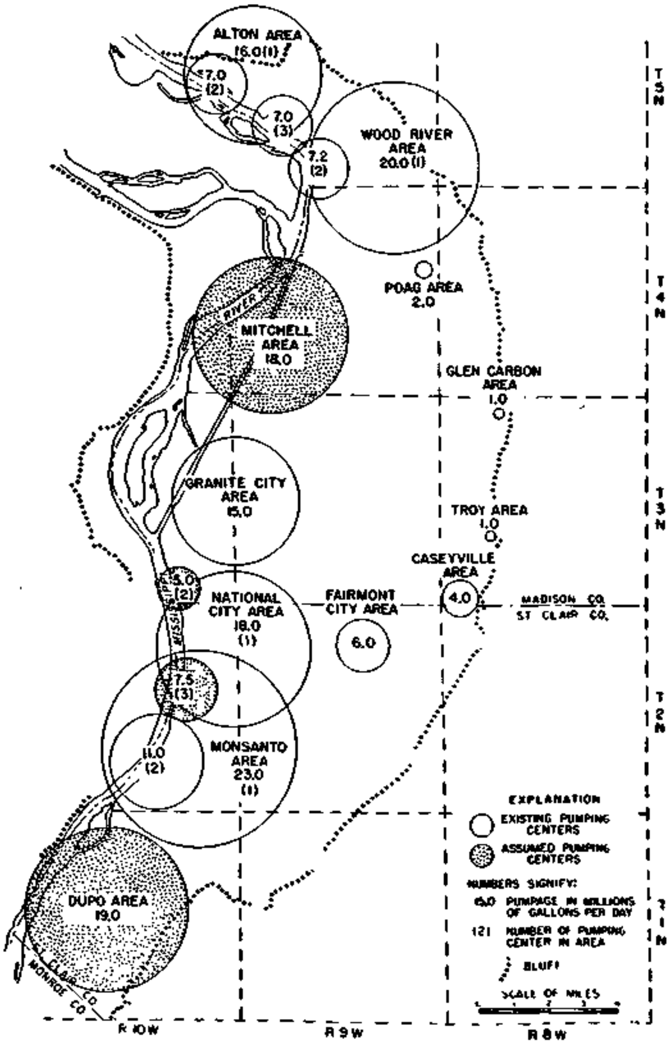


Figure 66. Distribution of pumpage with selected scheme of development

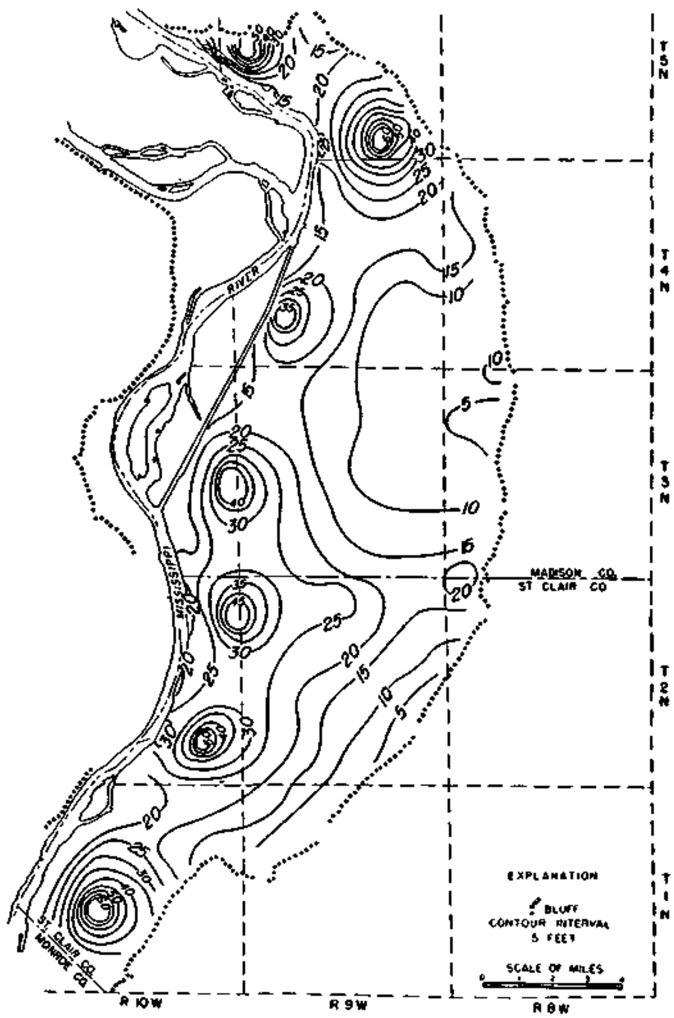


Figure 67. Water-level declines with a selected scheme of development

Table 35. Potential Yield of Aquifer with a Selected Scheme of Development

Pumping center	Pumpage with selected scheme of development (mgd)
Alton area	
1	16.0
2	7.0
Wood River area	
1	20.0
2	7.2
3	7.0
Mitchell area	18.0
Granite City area	15.0
National City area	
1	18.0
2	5.0
3	7.5
Monsanto area	
1	23.0
2	11.0
Dupo area	19.0
Poag	2.0
Glen Carbon	1.0
Troy	1.0
Caseyville	4.0
Fairmont City	6.0
Total	187.7

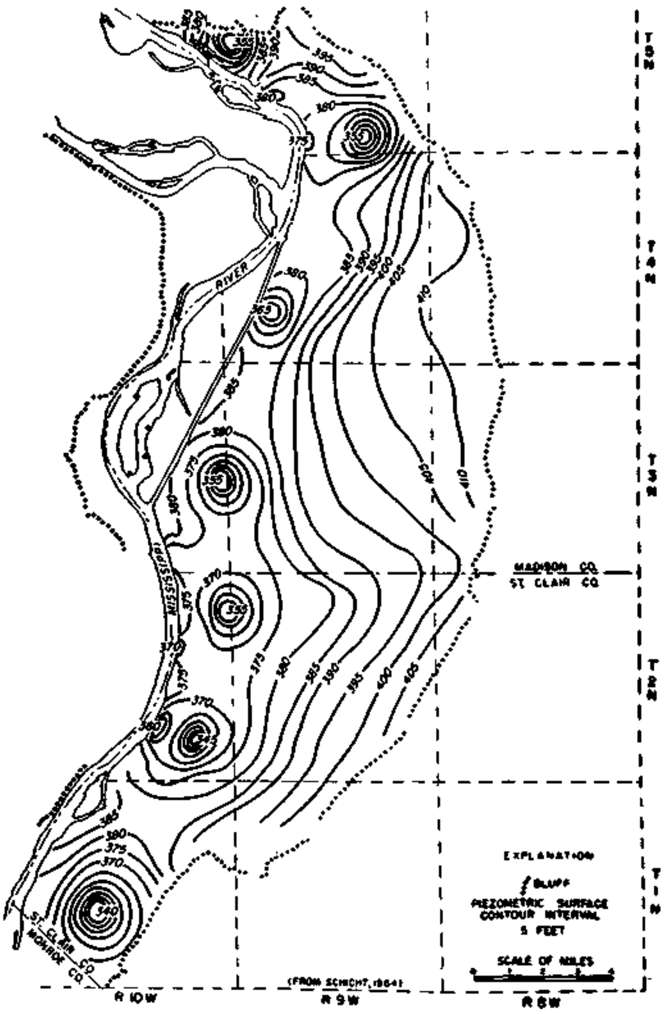


Figure 68. Approximate elevation of pieiometric surface with a selected scheme of development

in figure 69 areas of diversion with the selected scheme of development cover most of the East St. Louis area. Recharge directly from precipitation and subsurface flow through bluffs is therefore nearly at a maximum. Additional pumpage will have to be balanced with recharge mostly from induced infiltration of surface water. This can best be accomplished by developing additional well fields near the Mississippi River.

Average head losses beneath the Mississippi River bed and river-bed areas of induced infiltration, associated with pumpage in 1962 and with the selected scheme of development, were estimated based on infiltration rates and aquifer-test data. Average head losses are much less than the estimated depths of the Mississippi River given in figure 57, and river-bed areas of induced infiltration are small in comparison to the river-bed area in the East St. Louis area, indicating that recharge from the induced infiltration of surface water with the selected scheme of development is much less than the maximum possible induced infiltration.

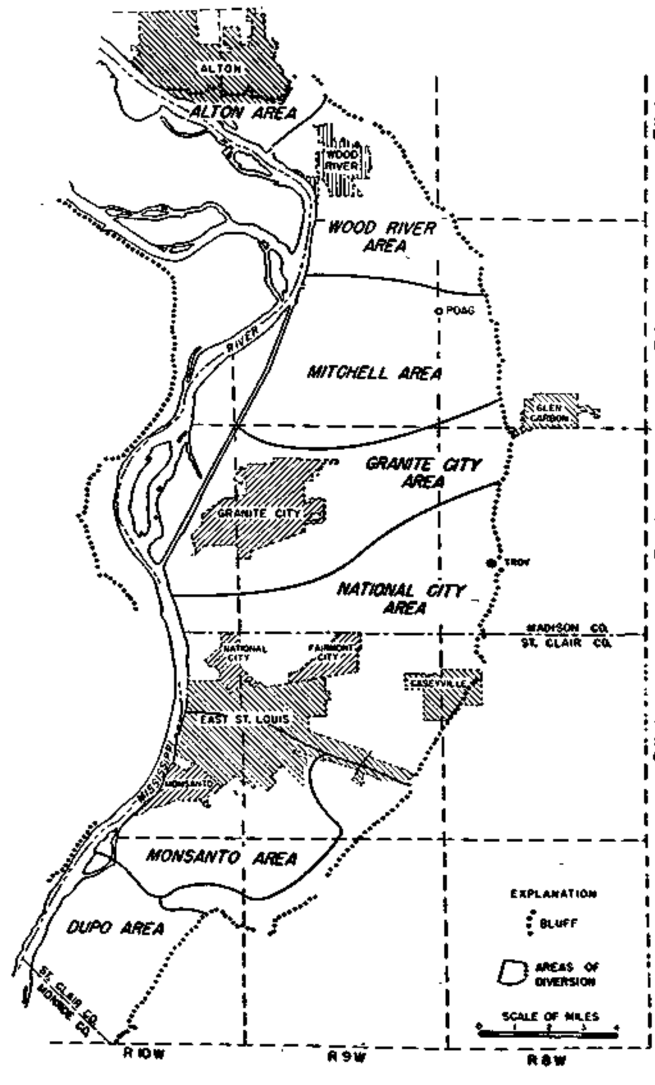


Figure 69. Areas of diversion with selected scheme of development

Table 36. Recharge with Selected Scheme of Development and in 1956 and 1961, Subdivided by Source

Pumping center	Selected scheme of development: Precipitation (mgd)	Sub-surface flow (mgd)	Induced infiltration (mgd)	Total pumpage (mgd)	1956: Precipitation (mgd)	Sub-surface flow (mgd)	Induced infiltration (mgd)	Total pumpage (mgd)	1961: Precipitation (mgd)	Sub-surface flow (mgd)	Induced infiltration (mgd)	Total pumpage (mgd)
Alton area	1.8	2.0	19.2	23.0	1.4	1.4	7.0	9.8	1.0	1.1	10.2	12.3
Wood River area	7.3	2.6	24.3	34.2	7.1	3.0	11.0	21.1	7.2	2.6	14.5	24.3
Mitchell area	9.3	1.3	7.4	18.0								
Granite City area	11.2	0.9	2.9	15.0	16.8	1.8	*	30.1	7.7		1.1	8.8
National City area	9.9	2.0	18.6	30.5	9.0	1.1	3.7	13.8	6.9		3.9	10.8
Monsanto area	9.5		24.5	34.0	10.7	0.7	18.7	30.1	12.6	0.8	18.5	31.9
Dupo area				19.0								
Poag	2.0			2.0	0.9			0.9	1.2			1.2
Glen Carbon	0.8	0.2		1.0	0.2			0.2	0.3			0.3
Troy	0.8	0.2		1.0	0.3			0.3	0.4			0.4
Caseyville	2.9	1.1		4.0	1.3	1.0		2.3	1.4	1.0		2.4
Fairmont City	6.0			6.0	2.4			2.4	4.4			4.4
Total	61.5	10.3	96.9	187.7	50.1	9.0		111.0	43.1	5.5	48.2	96.8

**Not computed; water being taken out of storage*

WATER QUALITY

The chemical character of the ground-water in the East St. Louis area is known from the analyses of water from 183 wells. The results of the analyses are given in table 37. The constituents listed in the table are given in ionic form in parts per million. The analyses of water from wells were made by the Chemistry Section of the State Water Survey. Chemical analyses of water from wells at several sites in the area are made monthly by the chemistry section. The locations of selected sites are given in figure 70. The sampling periods are listed in table 38, which provides a summary of the results of periodical chemical analyses of water from selected wells.

Ground water in the East St. Louis area varies in quality at different geographical locations. The quality of water also varies with the depth of wells, and may often be influenced by the rate of pumping and the idle period and time of pumping prior to collection of the sample. Bruin and Smith (1953) noted that relatively shallow wells of a depth less than 50 feet are in general quite highly mineralized and frequently have a high chloride content. Water samples from wells in heavily pumped areas often have high sulfate and iron contents and a high hardness.

Induced infiltration of water from the Mississippi River affects the chemical quality and temperature of water in wells at many sites. All other factors being equal, the closer the well is to the river the greater will be the effect of induced infiltration on the quality and temperature of water in the well. In most of the analyses in tables 37 and 38 the effect of induced infiltration of river water is not evident. Data in figure 71 illustrate the effect of induced infiltration of water from the river

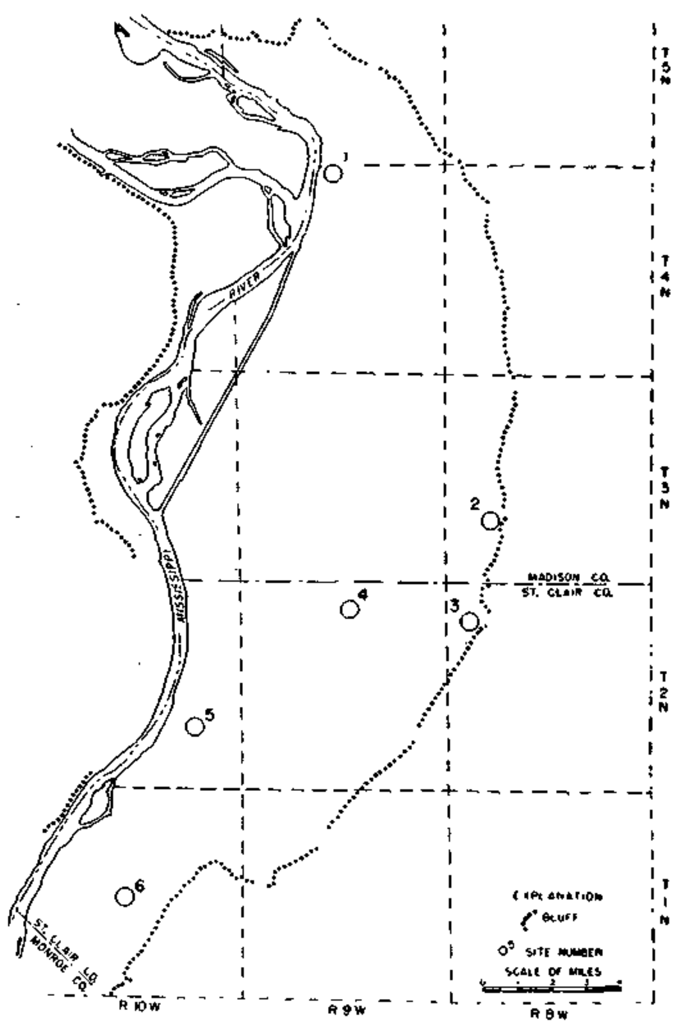


Figure 70. Sites where ground-water samples are periodically collected

Table 37. Chemical Analyses of Water from Wells

(Chemical constituents in parts per million)

Well number	Depth (ft)	Date collected	Silica (SiO_2)	Iron (Fe)	Manganese (Mn)	Calcium (Ca)	Magnesium (Mg)	Sodium + potassium (Na + K)	Boron (B)	Alkalinity (as $CaCO_3$)	Sulfate (SO_4)	Chloride (Cl)	Fluoride (F)	Nitrate (NO_3)	Hardness (as $CaCO_3$)	Total dissolved minerals	pH	Temperature (°F)
MAD—																		
5N9W-16.		12/16/48	33.6	0.2	0.3	108.7	30.5	23.9		224	171.1	27.0		11.5	398	538		
5N9W-18.4b	89	11/30/58	23.0	8.2	0.0	156.5	43.0			434	137.5	31.0		0.9	434	593		
5N9W-18.5c	93	3/31/54	12.0	2.0		123.0	61.4			314	191.3	27.0		3.7	314	560		
5N9W-22.		4/ 1/42	22.0	0.5	0.4	76.6	26.3	16.8		254	65.2	8.0		3.4	299	384	7.2	55
5N9W-		12/ 3/48	28.7	0.2	0.3	101.3	37.6	3.2		300	99.6	5.0	0.2	6.1	408	473		56
5N9W-		6/ 7/58	31.3	0.4		104	36.9	16	0.1	308	117.6	11	0.1	1.2	413	417		56
5N9W-	92	8/12/59		0.6	0.5					324		17	0.1	1.5	450	524		55
5N9W-26.7e	112	4/20/50		0.4	0.3	66.0	22.2	0.7		192	59.2	3.0	0.3	0.1	256	329		59
5N9W-26.8e	114	4/20/50	30.9	0.7	0.3	90.0	26.3	0.7		244	83.3	3.0	0.3	0.3	333	400		57
5N9W-26.8g	110	3/22/57	21.3	0.3	0.6	110.1	30.8	14.0	0.1	276	140.5	8.0	0.1	0.8	402	516		58
5N9W-26.8g	116	12/10/48	27.4	0.4	0.2	64.2	20.6	2.8		176	65.2	5.0	0.3	0.6	246	304		57
5N9W-27.1b1	122	12/ 4/48	31.3	4.8	0.5	87.0	25.0	7.6		200	105.9	19.0	0.6	0.2	321	410		57
5N9W-28.	86	11/ 1/60		5.8						284	50.4	15			335	406		55
5N9W-33.4d	100	12/16/58	11.1	1.3	0.3	52.5	18.2	14	0.0	160	45.3	20	0.1	0.9	206	270		
5N9W-33.5f	80	6/15/60		0.2	0.3					164	54.1	18		1.5	221	261		58
5N9W-35.4g	113	3/29/46		18.4						216	264.7	90.0			546	711		58
5N9W-35.4h	135	3/29/46		11.9						292	209.4	6.0			485	527		58
5N9W-35.3h	126	3/29/46		12.5						284	176.9	11.0			450	502		58
4N8W- 6.4a	71	8/30/56		6.2					0.3	556		11			448	558		58
4N8W-19.2g	1300	2/14/60		10						324	2137.8	4050			920	10373		
4N8W-19.7h	37	10/22/59	40.8	4.2	Tr	40.9	13.5	7	0	108	36.6	6	0.1	22.7	158	258		59
4N8W-19.8e	95	12/14/60		1.0						256		2	0.2	0.4	316	389		56
4N8W-29.4a	41	9/ 5/57		1.2	1.3					280		6	0.2	1.3	356	393		55
4N8W-29.4h	115	9/16/54		0.6					0	344		6			468	506		56
4N9W- 1.4d	100	6/31/61		1.6					0	236		3			289	334		
4N9W- 9.2b	85	11/10/52		2.0						308	113.1	8			420	472		
4N9W-12.5g	40	8/ 3/60		0.1						168		2		10	220	243		58
4N9W-13.	111	2/40	24.0	2.0		52.2	8.0	19.1		146	44.0	3.0		11.2		163	7.1	
4N9W-	112	11/17/48	27.1	2.8	0.3	59.9	15.0			144	51.8	5.0	0.5	8.1	211	264		57
4N9W-	114	11/ 6/61		1.6	0.3	60.0	15.6			152		4.0	0.2	9.8	214	271	7.7	58
4N9W-	117	10/26/61		2.6	0.3	65.9	17.8			180	58.4	5	0.2	9.0	238	300	7.6	59
4N9W-16.3b	35	10/21/43		0.1						334		49			914	1169		
4N9W-16.3al	27	10/21/43		Tr						334		56			895	1179		
4N9W-16.3c2	85	9/ 7/53		5.9						224	15.1	4			224	251		
4N9W-16.5bl	22	11/28/49		4.5						380	52.5	13		0.5	438	473		
4N9W-16.5c	72	1/27/53		5.9						280	43.4	3			316	339		
4N9W-19.3bl	26	1/ /44		0.2						446		28			628	1000		
4N9W-19.3b2	60	6/ 8/61		0.1						472	186.2	27		51.8	628	865		55
4N9W-20.3h	69	8/21/52		4.6						272	74.5	5			344	358		56
4N9W-20.4e		6/12/52		10.7						276	41.8	3			315	332		56
4N9W-20.4f		6/14/52		11.5						316	53.1	3			348	377		56
4N9W-20.4g		6/11/52		9.2						320	63.8	4			391	396		56
4N9W-21.5h	106	10/15/52	33.0	3.4	0.3	62.4	17.1	10.4		244	1.6	2	0.2	0.4	226	268		
4N9W-27.1f	110	10/14/43		3.5						242		8			291	345		60
4N9W-29.7c	106	4/27/54		9.4					0.1	332		3			370	380		56
4N9W-29.7e	63	9/18/52		12.1						328	79.0	6			418	418		58
4N9W-29.7g	67	9/16/52		11.0						356	79.4	3			420	435		57
4N9W-29.8e	63	9/19/52		13.0						340	66.0	5			412	412		56
4N9W-30.1a	69	9/25/52		13.0						352	72.7	6			428	430		57
4N9W-30.1b	69	9/25/52		10.3						360	87.4	6			448	458		57
4N9W-30.1c	66	9/24/52		10.1						340	80.8	9			428	433		56
4N9W-30.2a	69	8/28/52		9.5						376	56.2	4			424	424		57
4N9W-31.2g	69	8/26/52		10.8						364	76.7	5			433	440		57
4N9W-31.2h	69	8/27/52		13.0						392	90.7	5			466	492		56
4N9W-31.3f	71	8/23/52		10.9						404	71.6	5			458	486		57
4N9W-31.3g	68	8/25/52		10.8						396	98.1	8			487	515		56
4N9W-31.5b	63	7/28/52		7.6						336	43.2	5			378	392		57
4N9W-31.6a	60	8/ 5/52		10.3						348	64.4	6			416	427		57
4N9W-31.6b	58	7/25/52		9.1						352	50.6	5			395	402		57
4N9W-33.1g	110	11/21/53		5.1						296	2.5	6			268	304		
3N8W- 5.2fl	66	11/ 3/51	21.6	0.9	0.4	112.9	47.0	12.4		244	223.6	13	0.4	9.9	476	590		57
3N8W- 5.2f2	63	4/28/58	20.3	0.4	0.2	119.2	52.7	23	0.2	276	251.8	17	0.3	1.9	515	666		57
3N8W- 8.4hl	41	9/ 9/58		0.1						324		24			540	594		56
3N8W-20.5c1	100	6/30/59		0.2						320	115.8	6			424	478		57
3N8W-20.5c2	45	9/21/55		Tr					0	292		4			388	392		57
3N8W-20.8cl	100	10/ 7/43		0.1						316		7			336	365		62
3N8W-20.8c2	95	6/30/59		0.8						276	71.0	6			339	424		58
3N8W-29.3h	115	7/26/57	25.0	0.1	0.1	102.5	44.4	9	0.3	292	148.9	7	0.1	1.3	438	517		56
3N8W-30.7bl	40	10/13/54		4.2						324	49	3			340	369		
3N8W-30.7b2	104	10/13/54		1.8					0.6	296		4	0.3		332	364		
3N8W-31.2a1	99	4/ 3/52	27.0	1.4	0.9	107.6	43.8	8.1		280	170.5	6	0.4	0.6	449	552		56
3N8W-31.2a2	102	8/17/55	35.0	0.8	0.4	87.4	35.9	4	0.1	316	47.3	7	0.3	0.7	366	412		57
SN8W-31.2a3	103	8/12/58		1.6	0.5					280		18	0.1	1.0	450	465		56
3N9W- 3.	110	2/18/44		9.1						322		2.0			349	377		
3N9W- 5.8b	110	4/27/54		6.6					Tr	264		6			336	347		56

Table 37 (Continued)

Well number	Depth (ft)	Date collected	Silica (SiO_2)	Iron (Fe)	Manganese (Mn)	Calcium (Ca)	Magnesium (Mg)	Sodium + potassium (Na + K)	Boron (B)	Alkalinity (as $CaCO_3$)	Sulfate (SO_4)	Chloride (Cl)	Fluoride (F)	Nitrate (NO_3)	Hardness (as $CaCO_3$)	Total dissolved minerals	pH	Temperature (°F)
MAD—(Continued)																		
3N9W- 6.3c	110	5/13/54		7.2					Tr	276		4			352	364		57
3N9W- 6.4al	32	9/25/54		0						312	212.4	12			525	587		56
3N9W- 6.4a2	56	1/29/57		7.8					0	300		4			424	468	-	58
3N9W- 6.8f	56	8/18/52		4.1						252	60.5	3			248	323		56
3N9W- 6.8g	55	8/14/52		3.0						244	58.2	4			294	317		58
3N9W- 6.8h	59	8/12/52		3.5						320	77.8	5			395	408		57
3N9W- 8.5g	80	9/23/51	41.8	8.1	0.2	114.2	28.8	2.8		240	143.3	15	0.5	0.5	404	486		57
3N9W-10.4g	27	1/ /44		2.7						316		13			444	518		
3N9W-10.4h	23	9/ 7/55		1.8						240		4			300	439		58
3N9W-14.2g	100	11/ 2/56		4.5					0.2	232		4			256	287		57
3N9W-17.2a	106	4/21/54		5.7					0.1	296		8			384	380		56
3N9W-18.1d	90			9.4						256		5	0.1	0.7	312	338		60
3N9W-18.1f	45	2/24/44		5.2						258		4			326	356		60
3N9W-19.8h	109			7.0						288	94.6	40			372	459		
3N9W-24.4g	104	9/ 7/54		11					0.3	320	38.3	5			336	356		57
3N9W-30.7e	101	6/ 9/58		8.7						308	138.6	29	0.3		420	520		57
3N9W-32.6g	30	9/22/60		1.5						336	162.1	22		0.7	510	613		54
3N9W-35.2d	100	11/ 1/55		1.6					0.1	344		10			400	422		57
3N9W-35.4a	28	3/27/34	14	0.0	0.35	197.8	43.3			326	146.0	111.0		88.6	320	673		
3N9W-35.4d	55	3/27/34	14	1.3	0.8	92.4	21.6	12.7		286	46.7	0		0.9		320		
3N9W-35.7d	65	6/ 3/53		8.2				6.7		180		50			124	295		57
3N9W-35.8a	100	11/ 6/45		7.6						304		5			335	368		58
3N10W- 1.1c	53	7/21/52		4.0						320	65.2	6			365	392		56
3N10W- l.ldl	52	7/22/52		6.4						304	61.7	6			357	384		57
3N10W-l.ld2	72	11/21/53		10.1						376	138.2	25			528	580		56
3N10W- 1.2b	52	7/16/52		3.3						292	60.3	5			340	363		56
3N10W- 1.2c	53	7/18/52		2.2						328	76.5	4			386	405		58
3N10W- 1.3a	58	7/12/52		5.2						280	66.4	5			349	371		58
3N10W- 1.3b	58	7/15/52		3.3						312	51.6	4			361	375		57
3N10W-12.3d	65	3/ 2/61		7.0						404	193.4	12	0.3	0.8	590	699		57
3N10W-12.4g	57	7/ 2/52		4.8						196	74.7	8			265	297		56
3N10W-12.5f		6/26/52		5.5						204	82.5	8			276	318		58
3N10W-12.5e	57	6/28/52		7.2						200	103.7	8			312	335		57
3N10W-12.6d		6/20/52		8.4						232	93.2	7			320	358		57
3N10W-14.2d		9/ 2/52		9.6						268	76.5	9			336	364		57
3N10W-14.4a		9/ 6/52		8.4						204	78.0	10			286	319		60
3N10W-14.4b		9/23/52		8.7						216	68.7	11			300	316		55
3N10W-23.5e		7/15/52		9.8						276	102.0	9			365	400		56
3N10W-23.5h		9/ 9/52		8.4						224	72.4	8			286	320		53
3N10W-23.6e		9/13/52		10.8						324	129.2	13			441	500		60
3N10W-24.3h	84	2/23/44		8.8						334		38			712	825		56
3N10W-25.	119	2/24/44		16.6						362		37			720	891		60
3N10W-25.8g	95	6/26/52	45.0	22.6	3.0	162.0	39.3	35.9		344	257.9	24	0.3	0.1	567	757		
3N10W-36.5g	100	8/23/53		8.9						292		31			628	674		60
STC—																		
2N8W- 6.5h	106	4/29/55		0.6					Tr	296		7			388	408		56
2N8W- 6.6a	105	9/ 2/54		1.6					Tr	312		15			656	675		58
2N8W-7.2hl	91	4/12/35	12.0	0.6	0.6	156.8	77.7	71.5		461	316.1	52		1.3	708	1001		
2N8W- 7.3h	80	6/25/54		0.1					0.6	372	350.5	42			756	877		57
1N10W- 3.5e	31	11/ 2/43		0.2						386		71			960	1122		
1N10W- 9.1g	26	11/ 8/43		0.1						380		68			941	1123		
1N10W-12.7d	27	11/ 5/43		0.4						280		14			461	527		
1N10W-19.2h	34	11/16/43		0.1						412		141			1152	1566		
1N10W-21.4f	102	6/ 4/43		14.0						354		27			518	625		
1N10W-28.3h	51	6/21/43		1.4						120		16			322	386		
lN10W-33.7b	33	11/15/43		0.5						86		48			157	246		
2N9W- 1.3f	90	8/15/62		5.1	0.8					328		70	0.2	0.3	520	644		
2N9W- 1.4a	115	3/ /54		1.9					Tr	296		7			388	408		56
2N9W- 2.4f	36	11/ 6/45		3.6						280		10.0			338	382		
2N9W- 2.8e		9/27/45	32.6	7.5	0.5	94.4	29.6	1.6		324	29.6	5.0	0.3	0.2	358	406		
2N9W- 3.8a	115	4/ 8/43		6.2						382		5.0			408	450		
2N9W- 3.8a	117	4/ 8/43		6.6						290		12.0			617	742		
2N9W- 4.3a	112	4/ 1/43		7.6						312		52.0			938	1035		
2N9W- 4.3b	105	3/30/43		4.2						270		4.5			919	1176		
2N9W- 4.3c	122	3/30/43		6.2						314		13.0			401	477		
2N9W- 4.4a	100	4/ 1/43		12.4						270		42.5			1273	1882		
2N9W- 7.6el	106	3/10/58		5.8	0.4	190	52			326		108			690	1014		58
2N9W- 7.6e2	100	3/10/58		2.2	0.4					336		110			700	1021		58
2N9W- 9.1h		2/2/48		14.8						324	32.1	5			353	369		
2N9W- 9.7a	98	5/14/54		7.3						352	97.9	15			452	492		58
2N9W-10.6h1	122	8/ /42		4.7						356		4.0			343	387		
2N9W-10.6h2	124	8/ /42		9.4						380		3.0			350	397		
2N9W-17.2d	108	4/ /54		6.8						308		7			360	387		
2N9W-17.7g	114	8/24/48		10.1						308	111.3				425	463		
2N9W-18.5h	116	3/25/43		7.1						336		39			612	725		60
2N9W-19.	104	3/19/43		11.0						364		36			608	778		54
2N9W-26.7f	81	9/ 7/51	36.8	2.1	0.3	84.3	31	11.5		228	118.1	11	0.4	0.8	342	437		57
2N9W-26.8f	81	8/31/51	36.0	2.6	0.2	69.4	26	11.5		188	97.1	13	0.4	0.6	283	358		57

Table 37 (Continued)

Well number	Depth (ft)	Date collected	Silica (SiO_2)	Iron (Fe)	Manganese (Mn)	Calcium (Ca)	Magnesium (Mg)	Sodium + potassium (Na + K)	Boron (B)	Alkalinity (as $CaCO_3$)	Sulfate (SO_4)	Chloride CC1)	Fluoride (F)	Nitrate (NO_3)	Hardness (as $CaCO_3$)	Total dissolved minerals	pH	Temperature (°F)
STC—(Continued)																		
2N9W-28.6e	109	9/20/37	8.0	7.5		107.3	23	6.7		358		2.0		1.3	363	371		58
2N9W-29.6e	113	3/19/43		7.1						340		9.0			366	426		
2N9W-30.5h	100	3/17/43		25.0						420		33			590	850		
2N9W-30.6d	no	8/ 7/44		9.1						312		19			406	555		
2N10W- 1.3a4	110	5/16/61		11	0.8					304	149.3	16		1.9	444	569		57
2N10W- 1.3a5	110	5/16/61		14	1.0					296	146.5	13		0.7	436	551		57
2N10W-12.3c	106	9/ /54		12						396	537.7	170			884	1424		60
2N10W-12.3g2	108	3/30/43		5.6						418		640			1050	2258		
2N10W-12.3g3	108	do		3.8						404		225			803	1213		
2N10W-12.3g4	108	do		7.1						400		530			844	1810		
2N10W-12.6f	106	1/29/59		12						362	209.6	59			561	767		
2N10W-12.6h	100	1/29/59		15						436	209.6	92			625	913		
2N10W-13.6a	110	11/16/43		4.7						286		5			357	386		
2N10W-13.5d	108	3/17/43		12.8						370		53			682	840		
2N10W-13.7gl		9/ /44		12.2						352		48			686	882		
2N10W-13.7h2	38	9/ /44		0.9						368		33			616	803		
2N10W-24.1e		4/24/36	16.0	0.6	0.5	154.8	40.8	9.0		290	226.4	34		1.2	554	720		
2N10W-25.7b	100	4/ 1/43		1.7						396		32			322	469		54
2N10W-26.1e1	95	8/18/43		6.1						404		9			377	443		56
2N10W-26.1e2	105	12/12/47	37.0	12.8	0.3	130.2	40.6	15.6		360	137.0	18	0.3	0.1	493	603		
2N10W-26.2e	107	4/16/43		21.6						462		61			777	1108		60
2N10W-26.3dl	95	6/24/43		8.0						374		29			533	668		60
2N10W-26.3d2	no	6/24/43		11.3						328		23			518	676		60
2N10W-26.3g	105	12/12/47		22.9						440	486.7	41			750	1256		59
2N10W-26.3h2	105	6/10/43		1.1						386		39			770	864		
2N10W-26.3h3	105	6/10/43		12.4						402		39			770	890		
2N10W-26.4e	109	12/12/47	45.2	15.2	0.5	141.7	37	30.6		356	163.7	34	0.4	Tr	508	662		58
2N10W-26.7b	112			15.6						416	161.9	30			567	677		57
2N10W-26.7b	110	5/17/43		6.6						340		32			561	634		55
2N10W-33.2f	100	2/18/44		15.4						444		50			620	740		59
2N10W-34.	73	6/23/43		12.0						354		43			466	638		57

on water in a collector well owned by the Shell Oil Company located west of Wood River immediately adjacent to the river. The average monthly range in temperature of water in the collector well varies from about 50F during the late winter and early spring months to about 70F during the late summer and early fall months. Temperatures of the river water vary from a low of about 34F during January and February to a high of about 84F during July and August, The highs and lows of the temperature of the water from the collector well lag behind corresponding highs and lows of the temperature of the river water by 1 to 2 months, as shown in figure 71. During the period November 1953 to March 1958 the average monthly total hardness of water from the collector well varied from a low of 180 to a high of 253 ppm. During the same period the average total hardness of the river varied from a low of 150 to a high of 228 ppm. In general the water from the collector well is less hard than water in wells away from the river.

The hardness of waters in the East St. Louis area, as indicated in table 37 ranges from 124 to 1273 ppm and averages 459 ppm. In general, water in excess of 500 ppm hardness is found in wells less than 50 feet in depth. The iron content ranges from 0 to 25.0 ppm and averages 6.2 ppm. The chloride content ranges from 0 to 640 ppm and averages 27 ppm. Fluoride content ranges from 0.1 to 0.6 ppm.

The temperature of water from 121 wells in the sand and gravel aquifer ranges from 53 to 62F and averages 57.3F. A seasonal variation in temperatures of water in wells is not readily apparent.

Chemical analyses and temperatures of water from the Mississippi River at Alton and Thebes, Illinois, are given in tables 39 and 40 respectively.

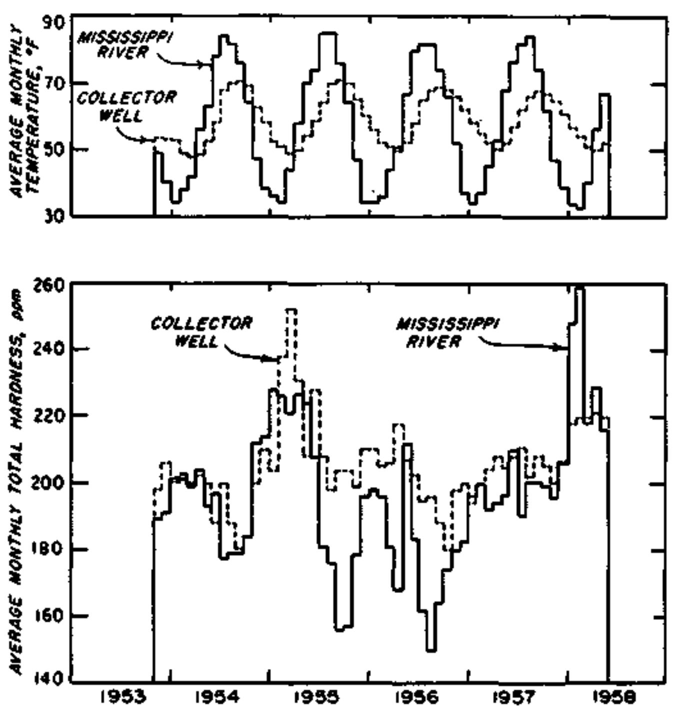


Figure 71. Chemical and temperature data for collector well and Mississippi River, 1953-1958

Table 38. Summary of Results of Periodical Chemical Analyses for Selected Wells

(Chemical constituents in parts per million)

Owner	Site number	Iron (Fe)	Chloride (Cl)	Sulfate (SO_4)	Alkalinity (as $CaCO_3$)	Hardness (as $CaCO_3$)	Total dissolved minerals	Temperature (°F)	Well number and period of record
Western Fibre Co.	3	0.9-3.6	165-260	388.8-496.4	572-872	956-1140	1400-1894	59-61	Well 3 6/25/56 to 1962
	3	1.6-6.8	150-220	371.3-485.5	408-776	820-1100	1202-1761	58-64	Well 1 10/31/49-5/28/56
Hartford (V)	1	2.3-12.2	5-30	43.4-129.8	204-384	247-482	336-580	55.5 61	Well 2 7/5/47 to 1962
Virginia Carolina Chemical Co.	4	2.3-50	2-22	1.4-1130.1	272-456	331-1560	353-2075	55-60	7/10/44 to 1962
Missouri and Pacific R. R.	6	5.4-15.0	9-28	52.2-235.5	364-428	420-532	473-731	57-62	South well 10/28/59-9/3/58
	6	0.4-20.8	2-93	55.6-374.2	76-504	129-733	240-963	57-62.5	North well 3/10/44-10/7/54
Hartford (V)	1	0.7-16.4	4-21	39.3-129.4	146-400	227-501	346-590	55.5 57	Well 1 3/31/50 to 1962
Troy (V)	2	0.1-0.6	3-11	103.1-177.1	264-310	396-464	438-563	56-57.5	Well 1 1/25/54 to 1962
American Zinc Co.	5	9.6-50.4	20-119	254.5-744.2	280-392	521-1080	624-1596		Well 2 11/22/44-11/15/48
	5	15 -23	43-118	260.4-422.7	236-320	503-638	766-1150		Well 7 1/23/61 to 1962

Table 39. Chemical Analyses of Water in Mississippi River at Alton

(Chemical constituents in parts per million)

Date	Laboratory number	Iron (Fe)	Chloride (Cl)	Sulfate (SO_4)	Alkalinity (as $CaCO_3$)	Hardness (as $CaCO_3$)	Total dissolved minerals	Temperature (°F)	Turbidity
5/ 1/51	125197	3.6	9	54.5	120	189	230	61.5	74
6/28/51	125677	5.9	9	55.1	120	189	230	75	168
9/ 7/51	126474	4.0	15	62.7	144	203	261	73	73
9/26/51	126572	8.3	12	45.9	156	210	246	72	311
12/ 6/51	127175	3.5	10	61.7	180	257	275	45	61
1/ 3/52	127416	4.5	9	59.4	172	245	276	32	115
2/ 2/52	127720	4.6	10	71.0	160	246	279	34	96
4/29/52	128690	1.8	7	56.2	136	202	231	65	43
5/ 2/52	128746	3.6	6	52.0	136	193	224	67	84
6/ 4/52	128955	3.8	11	76.9	176	256	293	75	101
2/ 4/53	131056	0.5	15	56.0	176	224	275	41	16
3/ 6/53	131345	3.7	13	57.6	132	176	232	40	124
4/ 2/53	131622	8.3	11	71.8	144	224	266	50	220
4/29/53	131853	2.4	12	79.6	152	240	279	60	60
5/28/53	132119	4.5	14	84.1	164	248	300	75	95
7/ 2/53	132404	3.1	8	52.0	144	204	226	86	70
7/30/53	132600	1.6	36	54.6	136	224	287	88	43
10/ 1/53	133068	1.2	15	61.7	152	196	262	76	38
11/ 5/53	133314	1.4	15	53.5	164	196	249	57	33
12/31/53	133676	0.6	15	38.1	156	176	225	33	32
2/25/54	134103	1.2	17	60.3	156	208	262	46.5	43
3/31/54	134363	7.1	15	61.8	120	196	268	48	234
4/30/54	134724	3.2	16	95.0	152	256	306	68	166
6/ 2/54	134966	4.6	20	104.8	160	256	322	70.5	97
6/30/54	135189	7.6	10	44.8	128	172	206	82	293
7/28/54	135447	2.3	16	44.6	132	180	209	86	49
9/ 1/54	135693	4.3	11	43.6	152	176	230	79	155
10/ 7/54	135923	4.0	14	44.8	148	188	233	72	107
10/29/54	136135	6.5	16	83.1	144	228	284	63	143
12/ 2/54	136391	0.9	16	68.3	176	220	277	40	36
1/ 6/55	136663	3.2	19	69.1	176	228	293	39	76
3/16/55	137223	11.6	14	86.4	176	260	307	49	149
3/30/55	137321	2.2	14	64.0	160	212	257	44	45

Table 39 (Continued)

Date	Laboratory number	Iron (Fe)	Chloride (Cl)	Sulfate (SO_4)	Alkalinity (as $CaCO_3$)	Hardness (as $CaCO_3$)	Total dissolved minerals	Temperature (°F)	Turbidity
5/11/55	137675	2.0	12	78.8	180	252	301	67	47
6/ 2/55	137812	8.2	8	53.9	120	160	212	73	298
6/29/55	138071	2.3	12	65.8	160	220	276	80	38
8/ 3/55	138394	1.1	15	37.8	144	180	211	92	28
9/ 8/55	138599	1.8	8	38.1	128	172	184	79	64
10/ 4/55	138785	1.4	20	49.2	144	176	238	70	93
11/ 4/55	139004	0.5	19	57.0	140	188	272	51	20
12/ 7/55	139282	0.3	17	46.3	156	184	245	37	62
12/28/55	139420	0.6	18	47.1	172	204	264	34	20
2/10/56	139760	0.5	18	45.7	168	208	273	33	13
2/29/56	139983	1.5	23	56.7	160	228	264	42	32
3/29/56	140209	0.6	16	61.5	152	228	289	50	28
5/ 1/56	140483	7.3	17	55.5	120	172	232	53	234
6/ 1/56	140716	6.9	15	64.6	136	196	262	73	181
6/30/56	140928	2.1	13	44.2	140	168	222	84	55
8/ 3/56	141156	1.1	18	53.5	140	176	255	84	24
8/27/56	141369	3.6	14	42.4	136	164	216	80	56
10/ 1/56	141601	3.7	18	43.2	140	184	222	71	69
10/29/56	141796	1.3	19	52.7	148	180	255	62	34
11/28/56	142026	0.9	18	52.2	144	204	249	41	19
12/28/56	142239	1.3	22	53.9	160	204	289	37.5	30
1/29/57	142499	2.9	17	49.2	140	188	227	34	66
3/ 4/57	142812	1.5	21	75.5	148	212	288	42	36
3/27/57	142974	3.0	15	65.2	140	192	251	44	88
4/30/57	143298	10.	10	65.6	128	184	237	67	650
5/27/57	143484	5.2	12	75.1	144	216	295	70	129
7/ 8/57	143871	3.9	12	57.6	124	194	262	81.5	99
9/11/57	144452	3.2	18	61.1	144	200	248	76	72
10/10/57	144725	3.1	19	64.8	152	204	288	65	83
11/ 6/57	145010	2.7	17	56.8	134	202	260	52	72
11/29/57	145239	3.5	19	73.8	168	230	317	42	35
1/ 6/58	145426	2.8	15	80.8	162	232	303	35	59
1/30/58	145697	2.7	16	82.3	180	260	342	35	35
2/25/58	145869	1.0	22	81.3	186	276	346	34	22
6/10/58	147827	6.3	15	69.7	152	208	287	78	107
11/24/58	148305	2.6	19	55.31	148	188	275	50	51

Table 40. Chemical Analyses of Water in Mississippi River at Thebes, Illinois*

(Chemical constituents in parts per million)

Date	Dis-charge *(ft/sec)*	Labora-tory number	Tem-pera-ture *(°F)*	Tur-bidity	Iron (Fe)	Manga-nese	Fluo-ride (F)	Boron (B)	Silica (SiO_2)	Chlo-ride (Cl)	Sul-fate (SO_4)	Ni-trate (NO_3)	Ammo-nium (NH_4)	Cal-cium (Ca)	Magne-sium (Mg)	Sodium (Na)	Alka-linity (as $CaCO_3$)	Total hard-ness (as $CaCO_3$)	Total dis-solved min-erals
1950																			
10/18	103,000	123421	65	685	13.0	1.1			15.3	14	75.7	4.9	T	55.4	13.4	28	152	194	317
11/ 8	86,000	123582	54	259	5.0	0.6			15.8	18	119.3	3.5	0.2	65.1	18.7	33	160	240	369
1951																			
1/9	101,000	124122	33	102	4.0	0.3			17.8	23	102.9	7.6	T	58.4	18.9	38	160	223	360
2/15	119,000	124410		149	7.7	0.5			12.6	16	61.5	5.7	0.1	40.8	14.4	19	112	162	238
3/13	303,000	124705	38	512	15.5	1.1			16.1	8	46.3	6.1	0.0	42.7	11.6	10	112	155	223
4/15	413,000	124887	46	763	30.8	2.4			14.4	10	46.3	5.8	0.2	47.6	12.1	11	128	169	235
5/ 9		125366	58	743	28.4	1.8			23.8	9	42.6	7.8	T	46.1	10.7	13	124	160	225
6/13	410,000	125601	70	846	51.0	2.9			15.7	10	61.9	6.1	0.0	61.3	7.8	28	156	186	284
7/24	802,000	126000	81	380	13.6	0.5			16.8	7	31.5	1.0	T	41.6	6.9	13	116	133	201
9/12	344,000	126468	74	685	23.3	0.6			13.9	10	45.5	4.4	T	46.1	4.8	20	120	141	237
10/10	228,000	126667	65	306	10.6	0.9			13.5	16	73.4	5.1	T	57.4	11.4	26	156	263	299
11/15	305,000	127030	45	220	10.5	1.3			16.3	11	71.6	4.6	0.1	54.9	16.6	13	140	206	283
1952																			
1/10	156,000	127450	34	59	2.1	0.3	0.3		14.2	18	71.4	7.8	0.3	59.0	21.0	11	152	234	319
3/20	480,000	128250	47	685	25.0	1.9	0.3		12.4	8	53.9	9.3	0.1	44.6	12.3	13	116	162	213
3/12	308,000	128251	47	167	6.1	0.6	0.3		13.2	11	67.9	6.8	T	49.6	14.7	22	140	185	274
4/16	547,000	128480		500	20.0	1.8	0.2		11.0	8	76.5	6.6	0.1	50.1	14.5	"16	124	185	259
6/11	198,000	129028	81	600	19.9	1.6	0.3		15.2	12	77.1	5.7	T	67.0	15.8	25	184	233	430
6/25	257,000	129099	82	372	11.1	0.9	0.3		32.0	9	79.0	6.0	T	57.2	16.5	24	164	211	318
7/30	162,000	129593		296	8.6	0.5	0.2		15.4	10	59.7	5.2	T	52.7	15.5	13	144	186	271
8/ 6	149,000	129660	81	136	5.2	0.3	0.3		18.0	12	58.4	4.3	0.0	51.2	16.3	20	156	195	278
9/10	125,000	130229	76	661	19.3	1.3			13.3	11	60.9	4.0	T	45.7	12.0	12	108	164	242
10/30	74,000	130337		136	6.8	0.3	0.3		10.9	19	109.6	2.5	T	57.9	19.2	36	160	224	351
12/ 3	84,300	130660		186	5.8	0.3			10.6	21	94.6	3.0	T	57.2	18.1	37	168	218	347
1953																			
1/19	69,500	131031	41	28	2.1	T			11.0	20	88.0	4.9	0.1	63.7	22.9	24	184	254	342
3/4	179,000	131346		466	14.4	0.8	0.3		10.3	14	61.7	6.2	0.1	41.5	15.0	18	116	166	242
4/8	337,000	131690		694	26.1	1.3	0.3		12.1	8	65.6	7.3	T	48.0	13.7	12	116	176	242
5/23	242,000	131976	46	431	15.7	0.9	0.3		9.9	10	90.5	3.9	0.1	58.5	16.1	23	152	213	310
6/ 9	170,000	132217	81	343	9.3	0.7	0.4		8.8	16	91.8	1.1	T	58.6	18.5	30	168	222	339
7/2	227,000	132407	85	685	52.8	2.8	0.5		14.1	10	134.3	5.4	0.0	65.6	19.0	44	180	242	414
8/ 5	128,000	132642	82	95	3.5	0.3	0.3		13.3	13	64.6	2.0	T	46.5	17.6	16	136	189	281
9/ 3	118,000	132828	80	76	1.8	0.3	0.3		9.9	15	72.7	2.8	T	48.0	16.6	29	152	188	283
10/14	67,500	133175	66	84	3.6	0.2	0.2		6.4	18	118.9	2.0	0.1	58.1	19.3	43	168	225	383
11/10	70,000	133391	72	91	3.7	0.3	0.3		7.0	17	111.1	1.7	T	57.5	19.1	40	168	223	384
12/ 9	71,100	133607	46	14	2.3	T	0.3		7.1	21	74.9	3.5	0.1	55.5	21.4	29	180	227	333
1954																			
2/10	52,800	133982	48	27	2.2	0.1	0.3		11.4	26	91.9	6.2	T	64.2	20.7	44	204	246	403
3/10	79,000	134192	48	167	6.4	0.5	0.5		9.9	20	97.0	3.8	0.4	58.0	17.8	37	168	219	365
4/21	139,000	134673	49	455	11.0	0.5	0.4		8.8	14	86.4	b.9	T	54.8	21.0	18	148	224	325
5/ 4	211,000	134888	52	1240	11.0	1.4	0.2		7.6	12	85.5	6.9	T	57.0	17.8	20	148	216	317
6/16	194,000	135070	72	1200	25.0	1.3		T	10.1	12	71.5	8.4	T	55	3.1	48	156	150	308
7/7	239,000	135263	82	750	T	0.9	0.4	0.2	12.6	6	45.7	8.2	T	47.2	19.0	0	132	196	238
8/10	114,000	135635	78	102	T	0.3	0.4	0.3	9.2	15	112.3	2.8	T	50.9	17.1	38	140	198	348
9/22	120,000	135489	66	263	1.4	0.1	0.7	0.0	6.5	12	71.2	3.1	T	48.6	15.4	20	136	185	269
10/ 4	113,000	136203	48	119	T	0.3	0.3	0.0	10.9	11	75.9	4.2	T	52.3	17.9	10	128	204	274
12/ 7	92,000	136518	41	13	0.1	0.3	0.3	0.0	20.0	16	71.6	1.8	T	55.8	19.2	12	148	219	284
1955																			
2/ 8	69,900	136898	42	44	2.8	0.1	0.3	0.0	11.6	17	80.9	4.2	0.0	66.5	23.1	12	176	262	323
5/10	137,000	137641	70	145	5.0	0.4	0.1	0.0	8.6	9	90.3	5.0	T	58.3	15.1	29	160	208	312
6/15	177,000	137915	62	500	8.6	1.2	0.2	0.0	7.0	14	80.8	3.1	T	46.4	18.5	24	140	192	285
7/ 7	129,000	138105	71	950	18.0	1.6	0.1	0.0	9.6	15	64.0	3.4	0.2	50.9	15.3	16	136	191	262
8/ 4	82,900	138357	81	112	7.0	0.4	0.3		14.4	16	111.5	5.1	T	53.8	18.6	35	144	211	354
9/ 2	109,000	138606	67	60	2.0	0.2	0.2	0.1	7.4	17	101.2	3.8	T	51.4	19.2	33	148	208	322
10/ 5	103,000	138779	62	201	1.1	T	0.2	0.1	7.2	19	95.2	4.1	T	46.9	16.3	37	136	185	318
11/ 2	82,100	139003	52	84	3.4	0.4	0.2	0.0	7.2	19	105.5	4.5	T	58.8	16.8	32	144	216	335
12/ 1	52,900	139211	44	45	1.9	0.1	0.2	0.4	7.4	22	79.4	5.4	T	54.8	18.3	34	168	212	331
1956																			
2/15	69,700	139892		600	19.0	1.9	0.2	0.2	9.5	20	66.4	4.2	0.8	43.0	17.1	34	152	178	291
2/29	88,200	139960	41	242	7.9	0.7	0.3	0.0	9.5	24	80.0	7.7	T	53.4	17.8	26	140	207	323
4/14	113,000	140301	49	86	3.8	0.5	0.3	0.1	5.4	15	75.3	4.7	0.1	48.0	18.3	26	148	195	276
5/ 1	188,000	140545	52	80	2.8	0.5	0.2	0.0	11.1	13	66.4	4.4	0.1	46.5	15.1	15	120	179	302
6/ 1	144,000	140739	69	119	6.3	0.5	0.2	0.0	6.7	16	90.1	5.2	0.1	51.5	17.7	25	136	202	296
7/6	146,000	140981	71	198	8.2	0.2	0.1	0.0	7.5	14	89.1	3.6	0.1	47.3	16.9	27	132	188	302

**From Larson and Larson (1957)*

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