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PART 2

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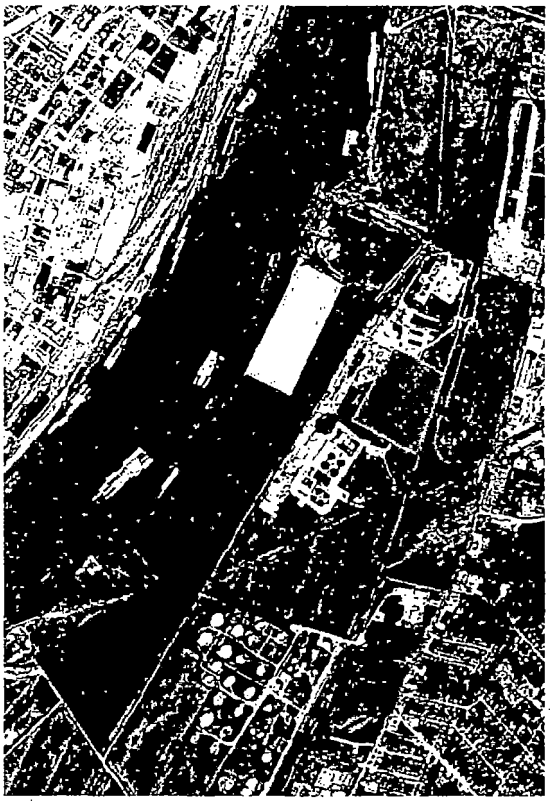
SAUGET AREA 2,
SAUGET, ILLINOIS

REMEDIAL INVESTIGATION/ FEASIBILITY STUDY REPORT VOLUME 5

Appendices H through N

Prepared for

Sauget Area 2 Sites Group
c/o Gary Uphoff
Environmental Management Services
5934 Nicklaus Drive
Fort Collins, CO 80528



January 30, 2004

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St. Louis, MO 63110
(314) 429-0100
Project #21560888.07001

SAUGET AREA 2,
SAUGET, ILLINOIS

REMEDIAL INVESTIGATION/ FEASIBILITY STUDY REPORT VOLUME 5

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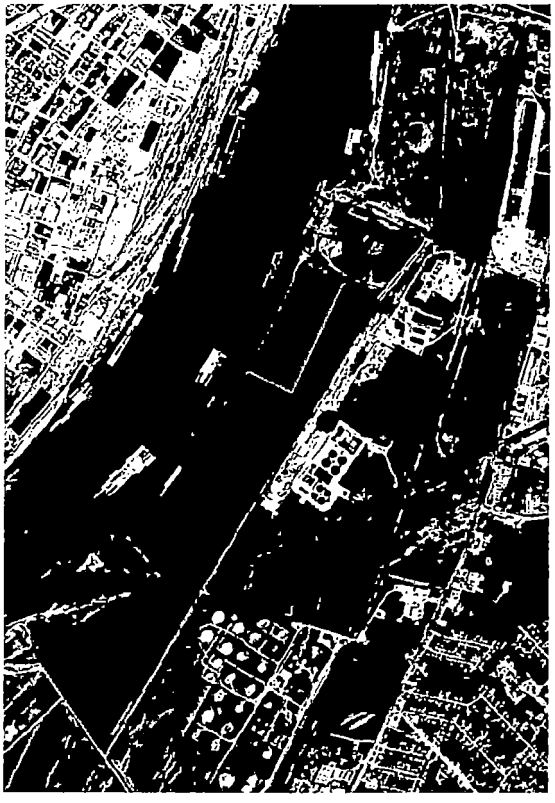

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St. Louis, MO 63110
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APPENDIX L

Attachments for the Interim Groundwater Remedy Design Basis

DRAFT
GSI Job No. G-2561
March 31, 2002

GROUNDWATER
SERVICES, INC.

Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

TABLES

Table 1: MODFLOW Calibration Results

Table 2: River Stage Estimate for MODFLOW Calibration

GROUNDWATER SERVICES, INC.

Table 1
MODFLOW CALIBRATION RESULTS
Interim Groundwater Remedy Design Basis
Sauget Area 2, Sauget, Illinois

Well ID	Roux Elevation to Measuring Point (ft MSL)[1]	OBSERVED Depth to Water (Oct. 25, 2001) (ft)	OBSERVED Water Elevation (Oct. 25, 2001) (ft MSL)[2]	SIMULATED Water Elevation (ft MSL)[3]	Residual Error (SIMULATED-OBSERVED) (ft)	Squared Residual Errors (ft)
Layer 1						
B-22A	428.16	29.16	399.00	395.2	-3.80	14.44
B-24A	422.49	23.39	399.10	394.8	-4.30	18.49
B-25A	428.47	30.02	398.45	396.4	-2.05	4.20
B-26A	423.71	27.87	395.84	393.7	-2.14	4.58
B-28A	423.04	26.18	396.86	392.5	-4.36	19.01
B-29A	429.03	32.17	396.86	396.4	-0.46	0.21
				MEAN OF RESIDUAL ERRORS:	-2.85	
				ROOT MEAN SQUARE:		3.19

Well ID	Roux Elevation to Measuring Point (ft MSL)[1]	OBSERVED Depth to Water (Oct. 25, 2001) (ft)	OBSERVED Water Elevation (Oct. 25, 2001) (ft MSL)[2]	SIMULATED Water Elevation (ft MSL)[3]	Residual Error (SIMULATED-OBSERVED) (ft)	Squared Residual Errors (ft)
Layers 2 and 3						
B-21B	428.37	38.39	389.98	391.4	1.42	2.02
B-24C	422.52	32.80	389.72	390.7	0.98	0.96
B-25B	427.35	37.21	390.14	391.8	1.66	2.76
B-26B	423.62	33.58	390.04	390.7	0.66	0.44
B-28B	423.08	33.09	389.99	390.6	0.61	0.37
B-29B	429.06	38.83	390.23	391.5	1.27	1.61
GM-27B	426.04	36.09	389.95	390.9	0.95	0.90
GM-27C	426.76	36.63	390.13	390.9	0.77	0.59
				MEAN OF RESIDUAL ERRORS:	1.04	
				ROOT MEAN SQUARE:		1.10

NOTES:
1. Obtained from Table 2 of "Summary of Ground-Water Quality Conditions", Roux Associates, Inc., Vol. II of II, December 1997.
2. Calculated by GSI using elevations obtained from Table 2 of "Summary of Ground-Water Quality Conditions", Roux Associates, Inc., Vol. II of II, December 1997.
3. Groundwater elevations obtained from MODFLOW using a river elevation of 389.5 ft.

ft = feet
MSL = mean sea level

GROUNDWATER SERVICES, INC.

Table 2
RIVER STAGE ESTIMATE FOR MODFLOW CALIBRATION
Interim Groundwater Remedy Design Basis
Sauget Area 2, Sauget, Illinois

Date and Time	Gage Level (ft)	River Stage (ft AMSL)[1]
Oct 24 2002 12:00	8.50	388.44
Oct 24 2002 13:00	8.55	388.49
Oct 24 2002 14:00	8.67	388.61
Oct 24 2002 15:00	8.68	388.62
Oct 24 2002 16:00	8.80	388.74
Oct 24 2002 17:00	8.93	388.87
Oct 24 2002 18:00	9.07	389.01
Oct 24 2002 19:00	9.21	389.15
Oct 24 2002 20:00	9.37	389.31
Oct 24 2002 21:00	9.44	389.38
Oct 24 2002 22:00	9.58	389.52
Oct 24 2002 23:00	9.57	389.51
Oct 24 2002 24:00	9.66	389.60
Oct 25 2002 01:00	9.76	389.70
Oct 25 2002 02:00	9.82	389.76
Oct 25 2002 03:00	9.92	389.86
Oct 25 2002 04:00	9.99	389.93
Oct 25 2002 05:00	10.09	390.03
Oct 25 2002 06:00	10.11	390.05
Oct 25 2002 07:00	10.18	390.12
Oct 25 2002 08:00	10.17	390.11
Oct 25 2002 09:00	10.27	390.21
Oct 25 2002 10:00	10.35	390.29
Oct 25 2002 11:00	10.39	390.33
Oct 25 2002 12:00	10.47	390.41
	MEAN OF STAGE:	389.52

NOTES:
1. Gage Zero = 379.94 ft: obtained from http://mvs-wc.mvs.usace.army.mil/MISS/MISL.html; gage number 0179A Mississippi River at St. Louis, MO.
2. ft = feet
3. AMSL = American Mean Sea Level

DRAFT
GSI Job No. G-2561
March 31, 2002

GROUNDWATER
SERVICES, INC.

Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

FIGURES

Figure 1: Generalized Hydrogeologic Cross Section

Figure 2: MODFLOW Model Configuration

Figure 3: Hydraulic Conductivity Arrays

Figure 4: Simulated Potentiometric Surface Maps

Figure 5: Well Locations

GSI Job No.G-2561
Issued: 11/29/01
Revised: 3/31/02

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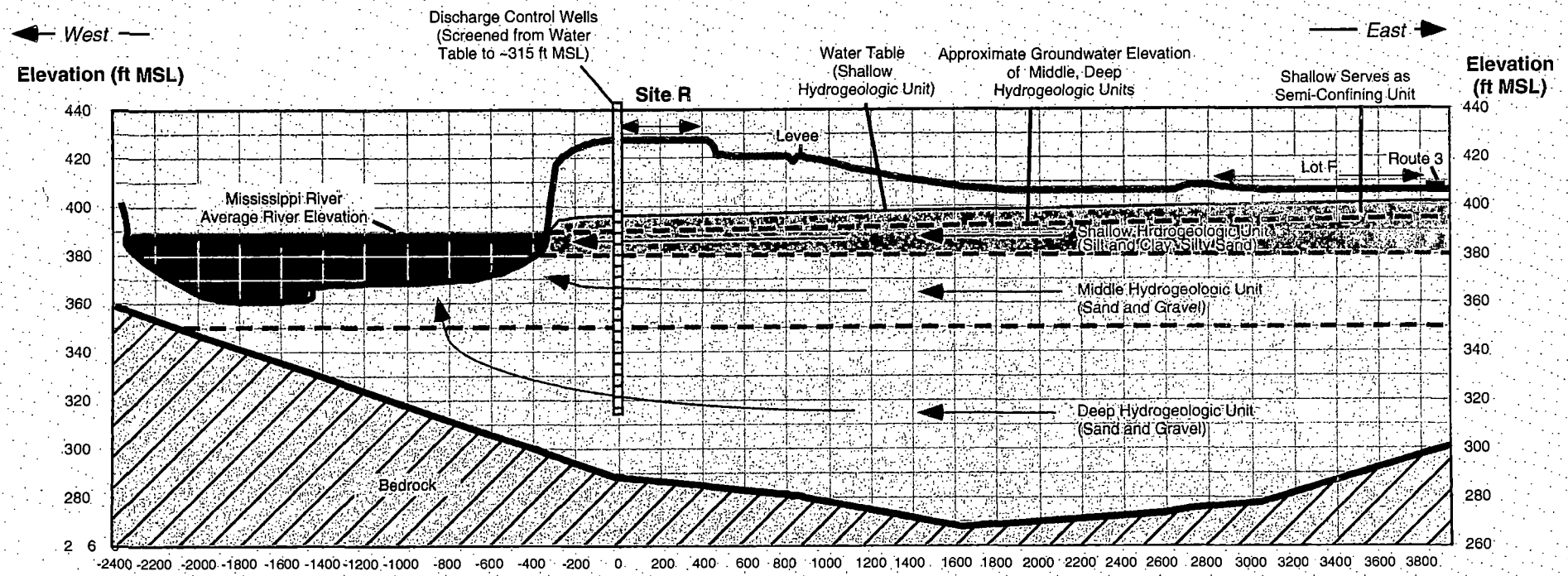
Figure 1
GENERALIZED HYDROGEOLOGIC CROSS SECTION

Interim Groundwater Remedy Design Basis
Sauget Area 2, Sauget, Illinois, Solutia Inc.



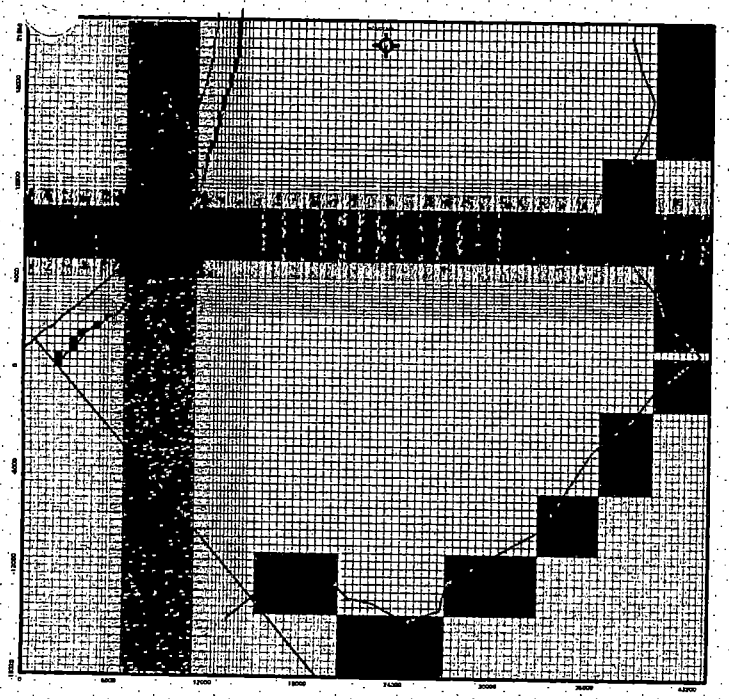
VERTICAL EXAGGERATION: APPROXIMATELY 12:1
VERTICAL SCALE: 1" = 66 FT
HORIZONTAL SCALE 1" = 752 FT

LEGEND

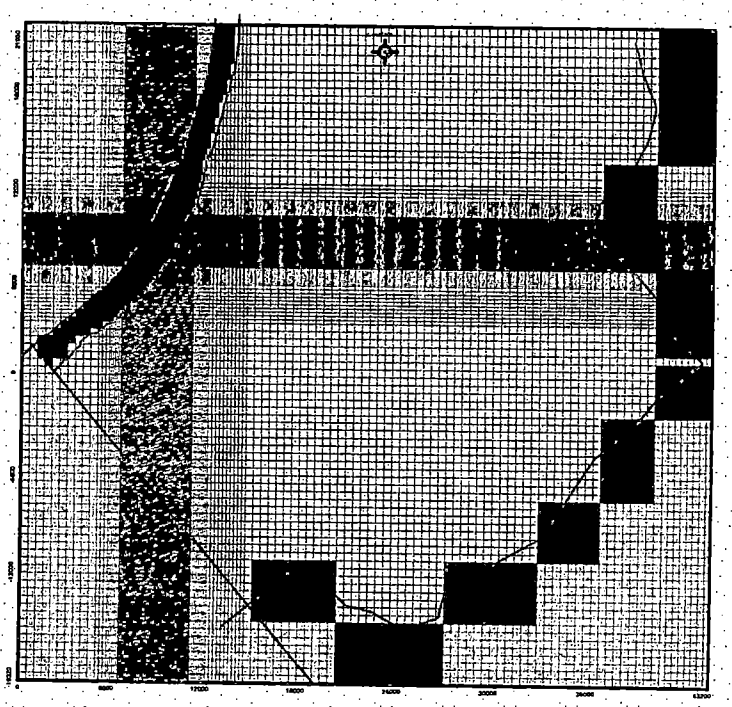
- *Approximate Direction of Groundwater Flow With No Pumping*
- *Mississippi River*
- *Unsaturated Zone Hydrogeologic Unit*
- *Shallow Hydrogeologic Unit in MODFLOW model*
- *Middle and Deep Hydrogeologic Units in MODFLOW model*
- *Bedrock*

Sources:

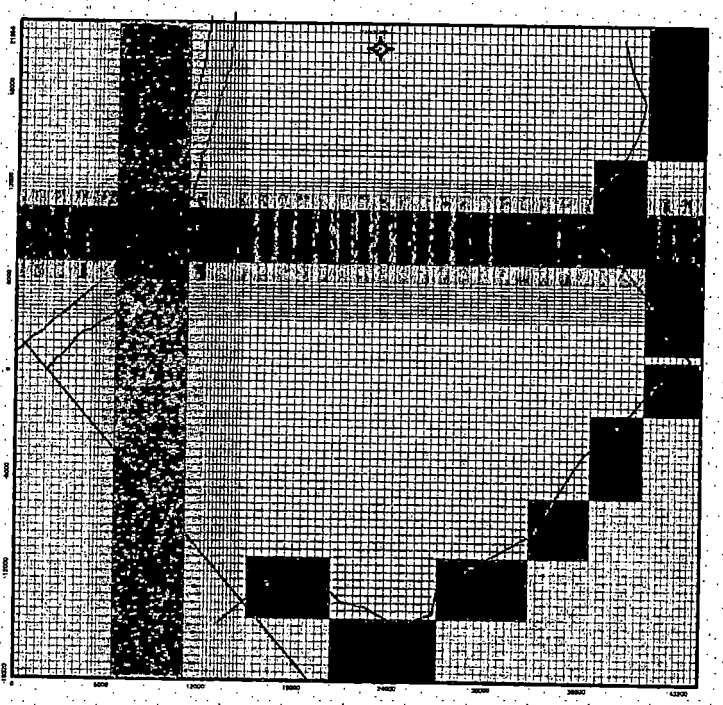
- Bedrock Elevation: Bergstrom and Walker, 1956
- Layer Elevations: URS Geologic Cross Section, 9/01, Geraghty and Miller, 1986, Groundwater Services, Inc., 2001
- Ground Surface Elevations: Geraghty and Miller, 1986 and URS Geologic Cross Section, 9/01
- Groundwater Flow Direction: Groundwater Services, Inc., 2001
- River Hydrography: 1994 Corps of Engineers data.



Layer 1: Shallow Hydrogeologic Unit



Layer 2: Middle Hydrogeologic Unit



Layer 3: Deep Hydrogeologic Unit

N

SCALE (ft)

0 6000 12,000

DRAFT

LEGEND

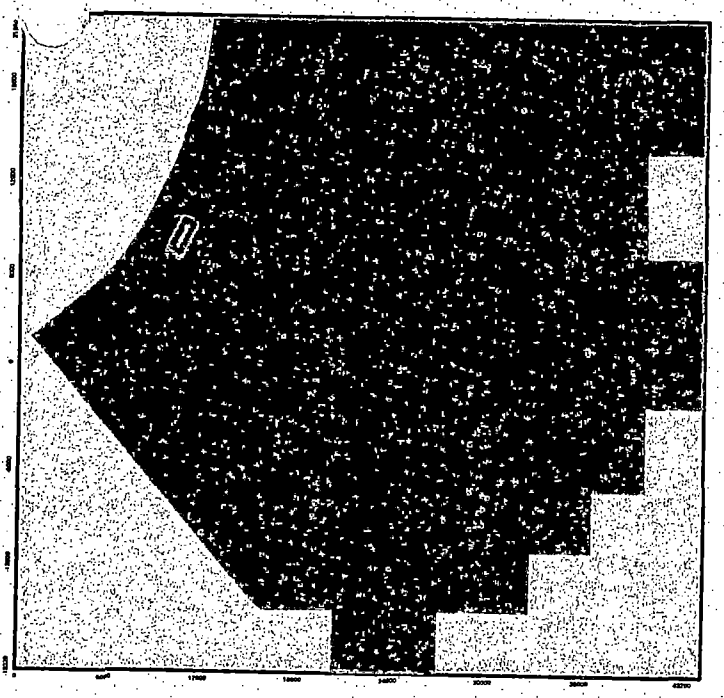
- Regional pumping center for highway dewatering
- River cells in MODFLOW model
- Approximate area of Site R
- Constant head cells
- Inactive cells

GROUNDWATER SERVICES, INC.

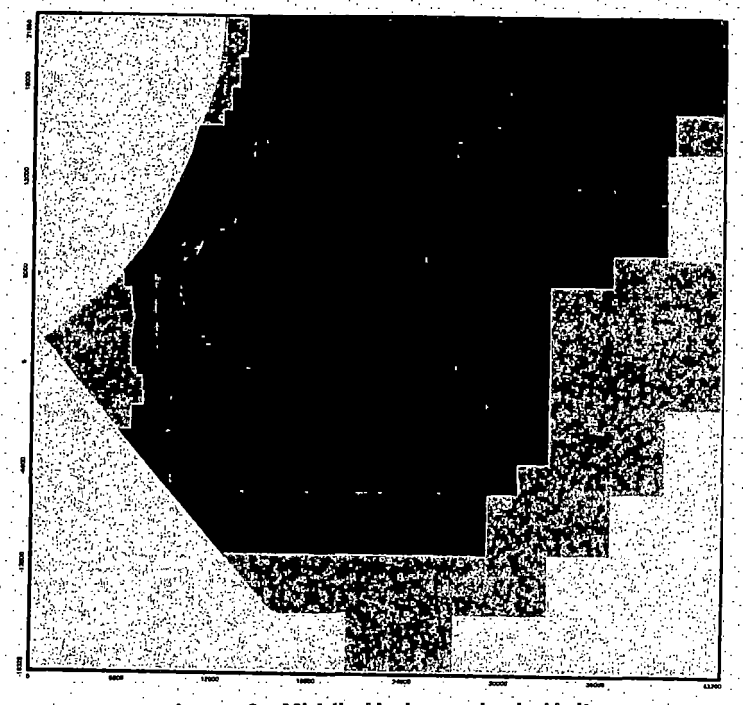
MODFLOW MODEL CONFIGURATION
INTERIM GROUNDWATER REMEDY DESIGN BASIS

Sauget Area 2, Sauget, Illinois
Solutia Inc.

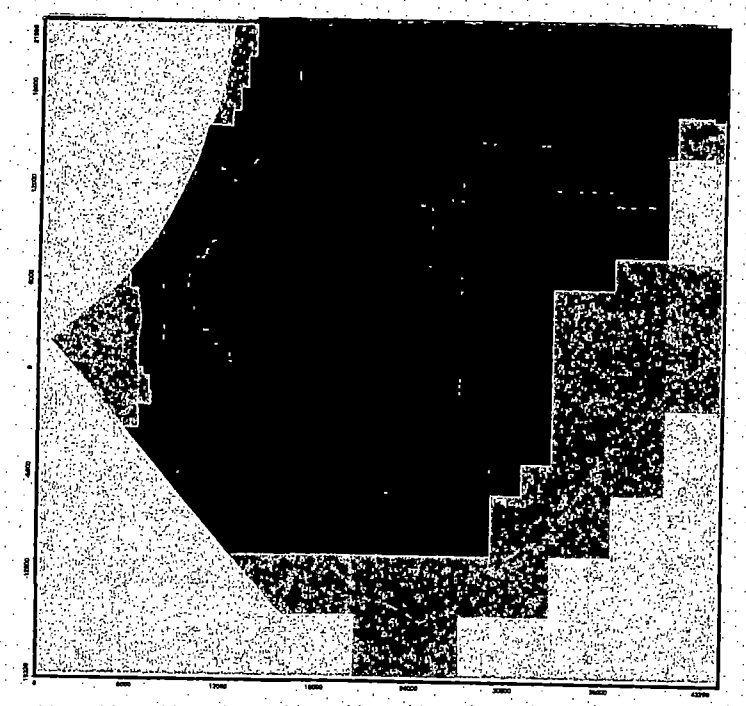
GSI Job No:	G-2561	Drawn By:	CRW
Issued:	11/30/01	Chk'd By:	SKF
Revised:	3/31/02	Appv'd By:	CJN
Scale:	As Shown		FIGURE 2



Layer 1: Shallow Hydrogeologic Unit



Layer 2: Middle Hydrogeologic Unit



Layer 3: Deep Hydrogeologic Unit

N

SCALE (ft)

0 6000 12,000

DRAFT

LEGEND

	$K_x = K_y$ (cm/sec)	K_z (cm/sec)
	5 E-4	1 E-6
	0.059	0.0295
	0.083	0.0415
	0.11	0.055
	0.15	0.075
	0.137	0.0685

Approximate area of Site R

Inactive cells

NOTE: K_x = Hydraulic conductivity in x direction
K_y = Hydraulic conductivity in y direction
K_z = Hydraulic conductivity in z direction

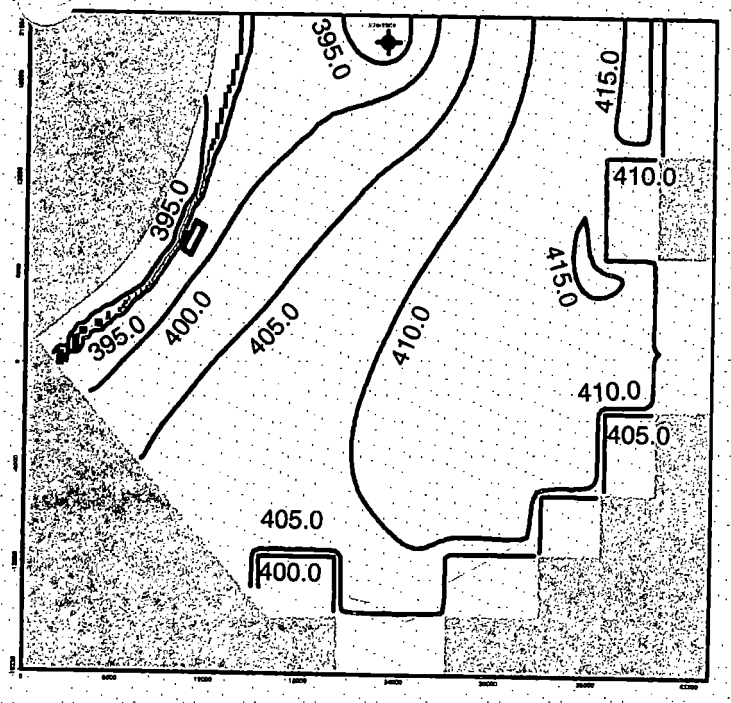
GROUNDWATER SERVICES, INC.

HYDRAULIC CONDUCTIVITY ARRAYS

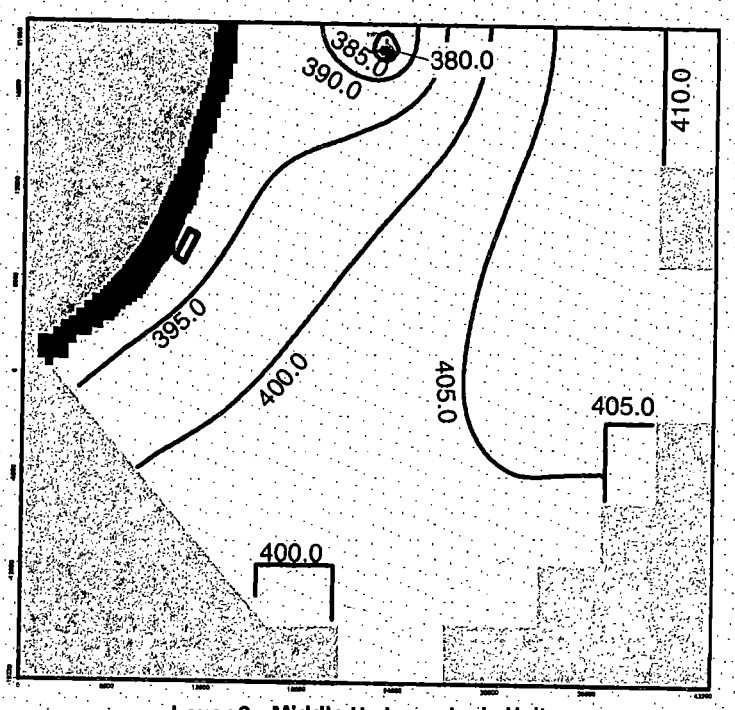
INTERIM GROUNDWATER REMEDY DESIGN BASIS

Sauget Area 2, Sauget, Illinois
Solutia Inc.

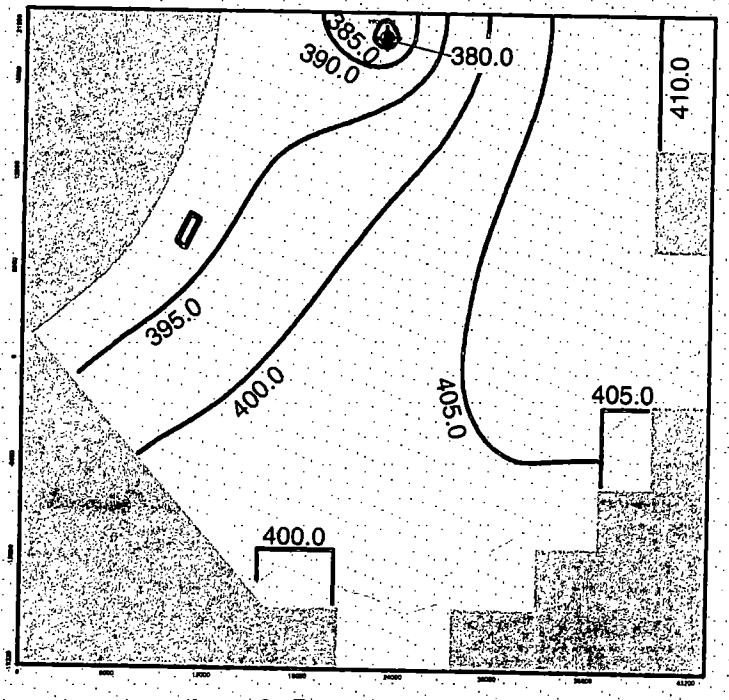
GSI Job No:	G-2561	Drawn By:	CRW
Issued:	11/30/01	Chk'd By:	SKF
Revised:	3/31/02	Appv'd By:	CJN
Scale:	As Shown		FIGURE 3



Layer 1: Shallow Hydrogeologic Unit



Layer 2: Middle Hydrogeologic Unit



Layer 3: Deep Hydrogeologic Unit

N

SCALE (ft)

0 6000 12,000

DRAFT

LEGEND

- Regional pumping center for highway dewatering
- River cells in MODFLOW model
- Inactive cells
- Approximate area of Site R
- Head equipotential contour (ft)

NOTE: 1. River elevation is 391 ft MSL.

GROUNDWATER SERVICES, INC.

SIMULATED POTENTIOMETRIC SURFACE MAPS

INTERIM GROUNDWATER REMEDY DESIGN BASIS

Sauget Area 2, Sauget, Illinois
Solutia Inc.

GSI Job No:	G-2561	Drawn By:	CRW
Issued:	11/30/01	Chk'd By:	SKF
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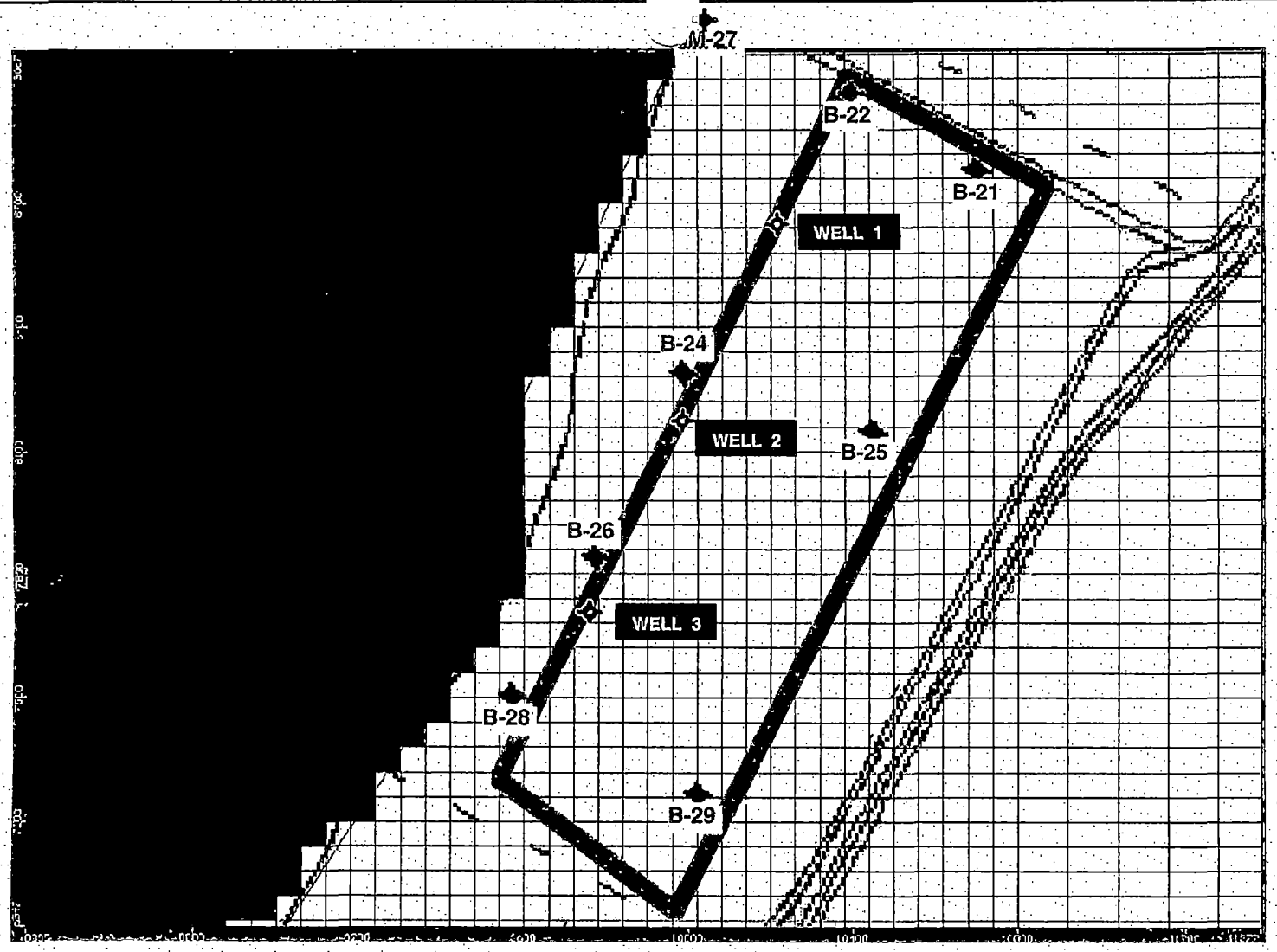
Scale:	As Shown		**FIGURE 4**



LEGEND

- Discharge control well
- Monitoring well cluster
- River cells in MODFLOW model
- Approximate area of Site R

DRAFT

NOTES:
1) Grid size in Site R = 60 ft x 60 ft
2) River elevation is 291 ft MSL

SCALE (ft)
0 200 400

GROUNDWATER SERVICES, INC.

WELL LOCATIONS
INTERIM GROUNDWATER REMEDY
Sauget Area 2, Sauget, Illinois
Solutia Inc.

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Revised:	3/31/02	Appv'd By:	CJN
Scale:	As Shown		FIGURE 5

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GSI Job No. G-2561
March 31, 2002

GROUNDWATER
SERVICES, INC.

Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

ATTACHMENTS

Attachment 1: Bedrock Elevation Map, Bergstrom and Walker, 1956

Attachment 2: MODFLOW Groundwater Flow Model, McDonald and Harbaugh, 1988

Attachment 3: Model Grid Development, Zheng and Bennett, 1995

Attachment 4: Model Grid Development, Spitz and Moreno, 1996

Attachment 5: Geologic Cross-Sections, URS, 2001

Attachment 6: Geologic Cross-Sections, Geraghty and Miller, date unknown

Attachment 7: Source Evaluation Study, Sauget Area 1, Groundwater Services, Inc., 2000

Attachment 8: Hydraulic Conductivity Data, Schicht, 1965

Attachment 9: Corps of Engineers Bathymetric Cross Section

Attachment 10: Monthly River Stage

Attachment 11: Regional Groundwater Flow Model, Clark, 1997

Attachment 12: Highway Dewatering Projects, Ritchey and Schicht, 1982

Attachment 13: Development of a Three-Dimensional Groundwater Flow Model, Geraghty and Miller, 1993

Attachment 14: Groundwater Flow Conditions, Geraghty and Miller, 1994

Attachment 15: 1990 Potentiometric Surface Map, Schicht and Buck, 1995

Attachment 16: Strategies for Design of Capture and Containment Remedial Systems, Gorelick et al., 1993

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GROUNDWATER
SERVICES, INC.

Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

ATTACHMENT 1

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GROUNDWATER GEOLOGY OF THE EAST ST. LOUIS AREA, ILLINOIS

BY

ROBERT E. BERGSTROM AND THEODORE R. WALKER

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GROUNDWATER GEOLOGY OF THE EAST ST. LOUIS AREA, ILLINOIS

BY

ROBERT E. BERGSTROM AND THEODORE R. WALKER

ABSTRACT

Geologic conditions favorable for large supplies of groundwater are among the factors promoting the concentrated industrial development of the Mississippi River bottomlands of the East St. Louis area, commonly known as the American Bottoms. The water-yielding deposits of the area are permeable sand and gravel in unconsolidated valley fill. The valley fill, which ranges to over 170 feet in thickness, consists partly of Recent alluvium and partly of older alluvium, some of which is glacial outwash material from the Upper Mississippi Valley. Valley-train sand and gravel occur beneath Recent alluvium in the northern part of the area and are present at the surface in terraces bordering the flood plain in the vicinity of Roxana. The lower alluvium south of the Missouri River mouth is older Missouri River sediment mixed with coarse glacial outwash material from the Upper Mississippi Valley. Although Recent cut-and-fill in this portion of the area has produced heterogeneity in the upper two-thirds of the valley fill, there is a general coarsening of material with depth. The most favorable water-yielding deposits usually occur below a depth of 60 to 90 feet, but clean sand and gravel are not present at all places on the American Bottoms. Distribution of permeable deposits and thickness of valley fill are controlled in part by the configuration of the bedrock valley floor.

Recharge of groundwater in the valley fill is by seepage from rainfall and floods and, in certain areas, by percolation from the Mississippi River and its tributaries. Geologic conditions appear favorable locally for greater groundwater exploitation, especially in some areas close to the river where permeable deposits are present and where river recharge might be induced by pumpage.

INTRODUCTION

LOCATION

The East St. Louis area in southwestern Illinois includes the portions of Madison, St. Clair, and Monroe counties that lie within the valley bottom of the Mississippi River between Alton and Dupo, Ill. (fig. 1). The area is known locally as the *American Bottoms*. It includes about 175 square miles, is approximately 30 miles long, and has a maximum width of 11 miles. The principal cities are East St. Louis, Granite City, Wood River, and Alton.

The area has been mapped by the United States Geological Survey, and topographic maps of the following 7½-minute quadrangles are available: Alton, Bethalto, Columbia Bottom, Wood River, Granite City, Monks Mound, Cahokia, and French Village.

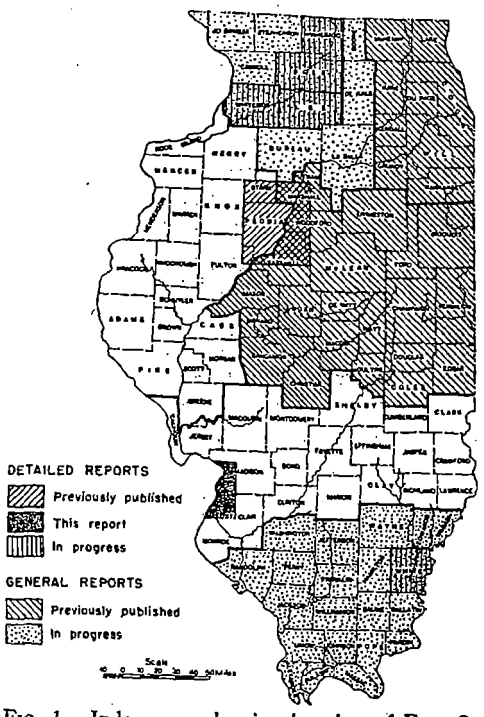


FIG. 1.—Index map showing location of East St. Louis area and major groundwater reports published since 1950 or in progress.

PURPOSE OF REPORT

The East St. Louis area is one of the most highly industrialized areas in Illinois, and the demand for groundwater supplies

has been great. The total municipal and industrial pumpage of groundwater during 1951 averaged between 100 and 110 million gallons per day (Bruin and Smith, 1953, p. 5). The expansion of existing industries and the influx of new industries indicate that even greater demands will be made on groundwater reservoirs. To develop the groundwater resources to their full potential, careful consideration must be given to the geologic conditions that control the occurrence of groundwater in the area. This report summarizes these conditions and indicates areas favorable or unfavorable for the development of additional supplies. Emphasis is placed on geologic conditions controlling development of the large supplies needed for municipal and industrial purposes. Engineering aspects of the problem, involving detailed hydrologic and production data, have been under investigation for a number of years by the Illinois State Water Survey (Bruin and Smith, 1953).

ACKNOWLEDGMENTS

The authors are indebted to many organizations and persons for assistance in the accumulation of basic data for this investigation. The U. S. Army Corps of Engineers, St. Louis district, supplied logs, samples, maps, and cross sections of test borings, river gauge records, and sounding results. Engineers of the Illinois State Water Survey furnished production figures, water levels, water-quality information, and well locations. E. G. Jones, field engineer for the State Water Survey at Alton, was especially helpful in the collection of information. Well logs and samples were obtained from water well drillers and industries in the East St. Louis area. Engineers of the St. Louis Municipal Waterworks supplied test-boring data. Seismic studies in the area were made by Robert C. Johnson and Robert C. Parks of the Illinois Geological Survey. Carl A. Bays, Consulting Geologist, Urbana, Ill., furnished data on bedrock depths in the Missouri Bottoms west of Alton.

We were assisted by many members of the Geological Survey staff, particularly those of the Division of Groundwater Geology and Geophysical Exploration. Many helpful suggestions and criticisms were made by M. M. Leighton, G. B. Maxey, J. C. Frye, F. C. Foley, H. B. Willman, Leland Horberg, and G. E. Ekblaw.

Previous Investigations

Early references to the geology of the East St. Louis area are contained in the reports on Madison and St. Clair counties of the first Geological Survey of Illinois, directed by A. H. Worthen (cited below). Subsequent reports on stratigraphy, physiography, and mineral resources were published by Fenneman, and a report on the groundwater resources was published by Bowman and Reeds. The Fenneman and Bowman reports, listed below, have been the primary sources of general geologic information on the area. Other geologic work in the vicinity of East St. Louis has been in connection with larger areal studies or on individual geologic problems. The following publications are concerned with geology of the area, with or without special reference to groundwater:

Bell, A. H., 1929, The Dupo oil field: Illinois Geol. Survey Ill. Pet. 17.

Bowman, Isaiah, and Reeds, C. A., 1907, Water resources of the East St. Louis district: Illinois Geol. Survey Bull. 5.

Drushel, J. A., 1908, Glacial drift under the St. Louis loess: Jour. Geol. v. 16, p. 493-498.

Ekblaw, G. E., and Workman, L. E., 1933, Subsurface geology in the East St. Louis region (abst.): Illinois Acad. Sci. Trans., v. 26, no. 3, p. 101.

Englemann, George, 1947, Carboniferous rocks of St. Louis and vicinity: Am. Jour. Sci., 2nd ser., v. 3, p. 119-120.

Fenneman, N. M., 1907, Stratigraphic work in the vicinity of East St. Louis: Illinois Geol. Survey Bull. 4, p. 213-217.

————, 1909, Physiography of the St. Louis area: Illinois Geol. Survey Bull. 12.

————, 1911, Geology and mineral resources of the St. Louis quadrangle,

Mo.-Ill.: U. S. Geol. Survey Bull. 438.

Flint, R. F., 1941, Ozark segment of Mississippi River: Jour. Geol., v. 49, p. 626-640.

Horberg, Leland, 1950, Bedrock topography of Illinois: Illinois Geol. Survey Bull. 73.

Leverett, Frank, 1870, The Illinois glacial lobe: U. S. Geol. Survey Mon. 38, p. 64.

————, 1895, The preglacial valleys of the Mississippi and its tributaries: Jour. Geol., v. 3, p. 740-763.

————, 1921, Outline of the Pleiscene history of Mississippi Valley: Jour. Geol., v. 29, p. 615-626.

Robertson, P., 1937, Drift exposures in St. Louis and St. Louis County (abst.): Missouri Acad. Sci. Proc., v. 3, no. 4, p. 129.

————, 1938, Some problems of the middle Mississippi River during Pleistocene time: St. Louis Acad. Sci. Trans., v. 29, no. 6, p. 169-240.

————, 1940, Some Pleistocene terraces of the Mississipi River (abst.): Geol. Soc. Am. Bull., v. 51, no. 12, pt. 2, p. 2041.

Rubey, W. W., 1952, Geology and mineral resources of the Hardin and Brussels quadrangles (in Illinois): U. S. Geol. Survey Prof. Paper 218.

Wanless, H. R., 1933, Pennsylvanian rocks of Madison and St. Clair counties, Illinois: Illinois Acad. Sci. Trans., v. 26, no. 3, p. 105.

Worthen, A. H., 1866, Madison County: Geol. Survey of Illinois, v. 1, p. 249-263.

————, 1866, St. Clair County: Geol. Survey of Illinois, v. 1, p. 231-248.

Of the following publications, pertaining more specifically to engineering phases of groundwater work, the report by Bruin and Smith contains the most recent and complete information on the hydrology and water quality in the American Bottoms:

Brittain, D., 1875, On the well at the Insane Asylum, St. Louis Co., Missouri: Am. Jour. Sci., 3rd ser., v. 9, p. 61-62.

Broadhead, G. C., 1878, On the well at the Insane Asylum, St. Louis Co.: St. Louis Acad. Sci. Trans., v. 3, p. 216.

Bruin, Jack, and Smith, H. F., 1953, Preliminary investigation of groundwater resources in the American Bottoms: Illinois Water Survey Rept. Inv. 17.

Gleason, C. D., 1935, Underground waters in St. Louis County and City of St. Louis, Missouri: Missouri Div. of Geol. Survey and Water Resources, Bienn. Rept. of State Geologist, 1933-34, app. 5.

Prout, H. A., 1853, Belcher's artesian well in St. Louis: Am. Jour. Sci., 2nd ser., v. 15, p. 460-463.

St. Louis Chamber of Commerce, 1950, The industrial water resources of the St. Louis area.

Searcy, J. K., Baker, R. C., and Durum, W. H., 1952, Water resources of the St. Louis area, Missouri: U. S. Geol. Survey Circ. 216.

Shepard, E. M., 1907, Underground waters of Missouri—their geology and utilization: U. S. Geol. Survey Water Supply Paper 195.

Suter, Max, 1942, Groundwater studies in the East St. Louis district: Illinois Engineer, v. 18, no. 2.

EXTENT AND RELIABILITY OF SUBSURFACE DATA

This report is based on a study of about 700 logs of wells and borings, supplemented by studies of available samples. Most of the logs are of water wells or of test borings made prior to the construction of water wells. Many are logs of borings made by the U. S. Corps of Engineers in connection with levee construction. A few are logs of oil wells or oil test holes. Most of the borings do not extend through the unconsolidated sediments lying above bedrock. Many of the wells were drilled to what the drillers assumed to be bedrock, but it is likely that many of these borings end at large boulders several feet above the bed-

rock, for nearby wells record greater depths to bedrock. The Corps of Engineers recognizes the possibility that many of their borings end with the bit resting on a boulder lying above bedrock; they label such depths "bit refusal" rather than "bedrock." The term "bit refusal" is preferred to an unqualified designation as bedrock in those cases where the drilling does not actually continue into bedrock for at least a few feet.

In mapping the surface of the bedrock, we have considered as reliable only those wells that have penetrated the underlying rock. The only wells that satisfy this requirement are the oil wells and oil test holes, and these are few. The reliability of the remainder of the logs is open to some question, so a subjective factor was involved in construction of the bedrock surface contour map.

Logs of oil wells and oil test holes are of little value in giving information on the lithology of the unconsolidated material in the American Bottoms because they lack detail in the upper sections. For information on the lithology of the valley fill, reliance must be placed upon logs of shallow borings. Logs obtained from the Corps of Engineers are considered to be the most reliable. The borings from which these logs were made were supervised by field engineers experienced in collecting and recording such data, and the sampling intervals were closely spaced. In addition, many of these logs have been compiled after mechanical analyses were made of the samples. Logs obtained from water-well drillers are less reliable, as many lack detail. Where drillers attempted to classify the sediments into grain sizes, a large personal factor was involved. For example, the sediment in many samples is described as "building sand" or "quicksand"; in such cases much has been left for us to interpret.

Some information also has been obtained from excavations made for the construction of piers and abutments for bridges across the Mississippi River. These give reliable information on bedrock elevations but at best furnish only very generalized information on the nature of the unconsolidated sediments. To supplement the data available on depth to bedrock, a refraction seismograph study was made at locations where well information was lacking.

An attempt was made to obtain additional information on the stratigraphy of the unconsolidated sediments by the electrical earth resistivity method. Twenty-five resistivity stations were set up adjacent to wells or borings for which detailed logs were available and which thus could serve as controls. The results of this work are inconclusive. We decided that unknown factors were influencing the resistivity readings, and this phase of the investigation was halted.

TOPOGRAPHY AND DRAINAGE

The Missouri and Mississippi rivers come together in the northern part of the area, about 5 miles downstream from Alton. Upward from this junction within the area of study and for several miles upstream, these two rivers flow southeast in the same valley, bordered on each side by bluffs of Mississippian limestone (tables 1 and 2). Below this junction the Mississippi River flows south across the area. Through the middle of the area the river valley crosses the western edge of a lowland cut in easily eroded Pennsylvanian ("Coal Measures") rocks and attains its maximum width (approximately 11 miles). In the southwestern part of the area the river crosses the more resistant Mississippian limestone and its valley narrows to about 3½ miles in width. At present, only in the area above Alton is the Mississippi River eroding the valley walls on the Illinois side. It is cutting along the western bluffs throughout the remainder of the area.

Along the river channel, the flood plain ranges in average elevation from 415 feet in the vicinity of Alton to 405 feet in the vicinity of Dupo. In this distance of 30 miles, the river falls 16 feet, a gradient of about 6 inches per mile.

In relatively recent geologic time, the Mississippi River has changed its course frequently in the East St. Louis area, producing a complex variety of land forms and

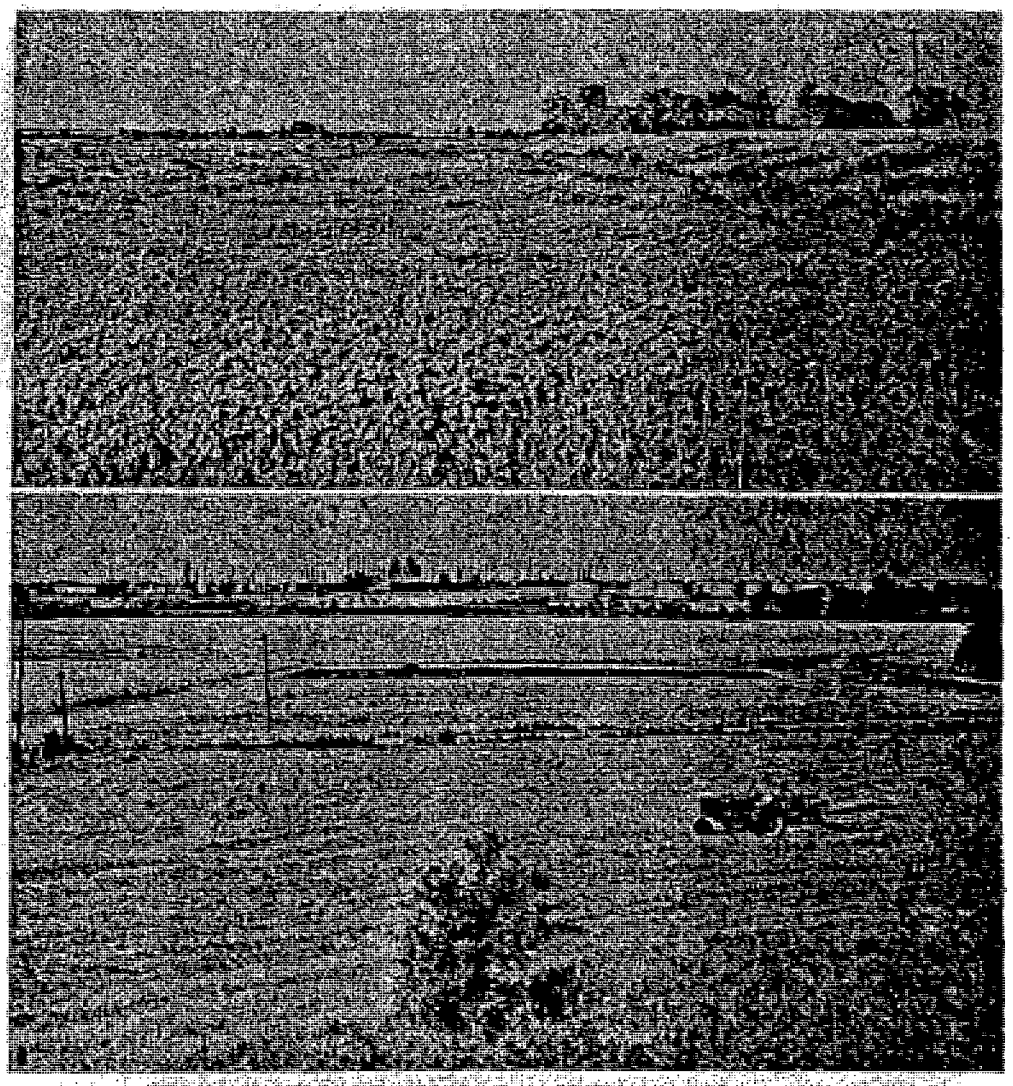


PLATE 1.—A. Horseshoe Lake bed, Madison County, Ill. View east from highway U.S. 66 near National City. B. Shell Oil Company Wood River Refinery, built on terrace above the Mississippi River flood plain. View southwest from bluffs east of Roxana, Madison County, Ill.

river deposits (Fenneman, 1909, p. 13, 29). Horseshoe Lake (pl. 1A) and other crescent-shaped lakes, swamps, and lowlands in the area mark the location of former meanders abandoned in the process of channel migration. The arcuate ridges and swales that border these meander loops on the concave side were formed as slackwater bars in former channels. East of the meander belt are discontinuous areas of poorly drained lowlands or backwater swamps which have been partially filled by silt and clay from floodwaters of the Mississippi and local tributaries.

In the northern part of the American Bottoms, deposits of sand and gravel occur in terraces that stand above the flood plain. They are eroded remnants of a valley fill of sand and gravel deposited by water from melting glaciers to the north, in the Mississippi drainage basin. These deposits formerly filled the valley to the present levels of the terraces. The low, broad ridge upon which East Alton, Wood River, Roxana,

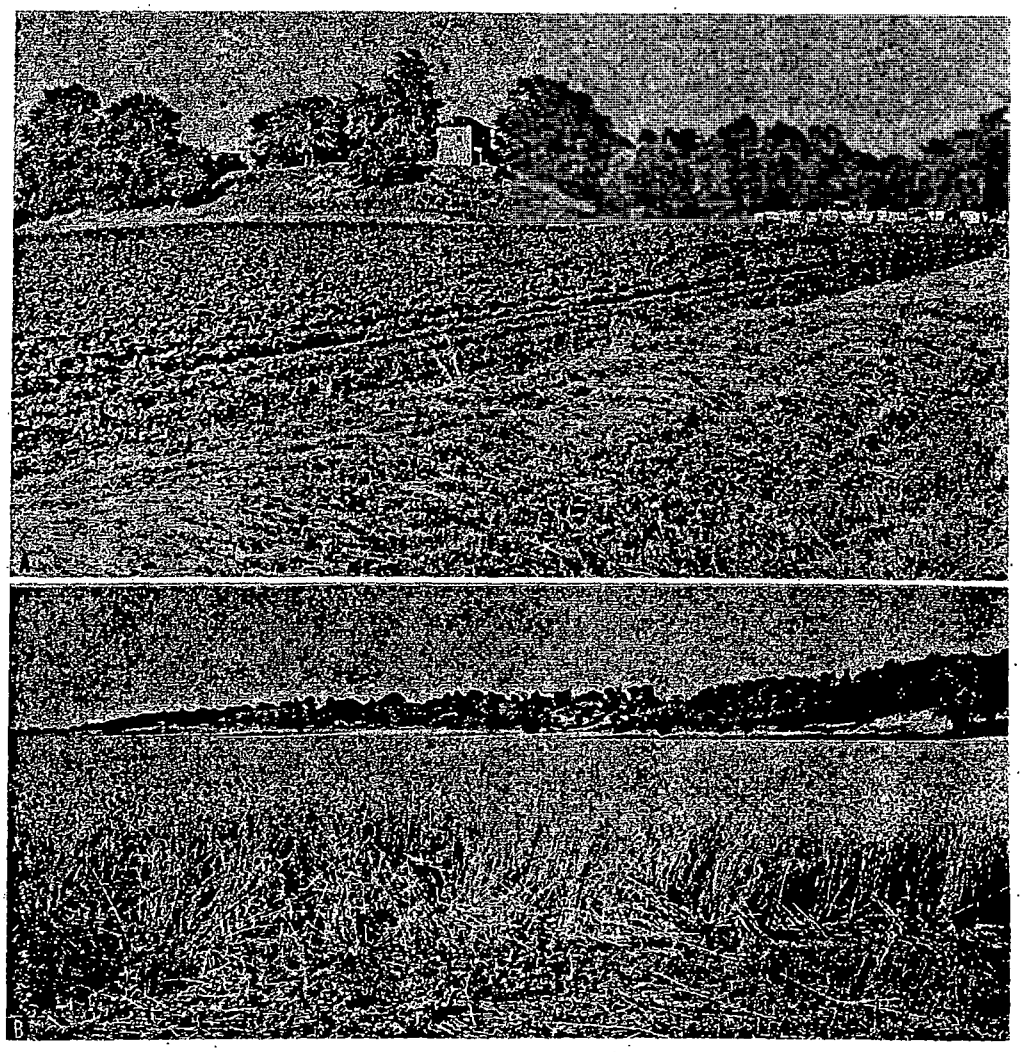


PLATE 2.—A. Mounds at Cahokia Mounds State Park, Madison Co. B. East bluffs bordering the American Bottoms. Looking northeast from 3 miles northeast of Horseshoe Lake, Madison Co.

and South Roxana are built is a terrace that stands 40 feet or more above the Mississippi River and 25 to 35 feet above the present flood plain (pl. 1B). The terrace is 440 to 450 feet above sea level. The front of the terrace has a sharp rise of 12 to 15 feet. This terrace level is also represented by low flat-topped knolls in the vicinity of Poag and just west of Indian Creek south of Roxana.

Many areas on the American Bottoms are somewhat above the flood plain but are below the level of the terrace at Wood River. North of Horseshoe Lake, the elevation of this intermediate level is 420 to 435 feet. It is more recent than the Roxana terrace but also may represent aggradation during late glacial time.

Between East St. Louis and the eastern bluff is a group of mounds occupying an area of 3 to 4 square miles (pl. 2A). The largest of these, Monks Mound, is about 85 feet high, whereas the smaller ones are only a few feet high. Although some of the mounds are symmetrical, steep, and cone-shaped, indicating an artificial origin, some of them may be remnants of an earlier higher flood plain.

The bluff that forms the eastern edge of the valley rises 150 to 200 feet above the valley bottom. Bedrock is well exposed in the bluffs on the Illinois side of the river in only two places, northwest of Alton in the northern part of the area and south of Stolle in the southern part. Most of the bluffs on the eastern side of the valley (pl. 2B) are covered by a mantle of glacial drift overlain by windblown silt called *loess*. With the exception of the two areas mentioned, the loess also blankets the face of the bedrock bluff. Between Edgemont and Caseyville, however, the loess cover is patchy and there are scattered outcrops of Pennsylvanian bedrock in the bluffs.

Many alluvial fans have been developed below the bluffs on the eastern side of the valley. These fans are composed predominantly of reworked loess which has been picked up by tributary streams in the upland and redeposited where the tributaries enter the main valley. As a result of the deposition of alluvial fans, the elevation of the valley bottom adjacent to the eastern bluff is 30 to 50 feet higher than the general elevation of the valley bottom. The alluvial fans, however, are not to be confused with the terraces of glacial sand and gravel mentioned above, for they gently slope and thin valleyward and have an entirely different lithologic composition.

The upland adjacent to the American Bottoms consists of broad, flat plains separated by relatively narrow, deep valleys. In most places the major tributary streams appear to follow preglacial bedrock valleys. The valley floors have relatively steep gradients as they join the main valley. In contrast, the Mississippi valley bottom slopes gently southward at an average rate of only about 6 inches per mile. In times of heavy rainfall the tributaries carry more water than normally can be confined within their banks in their lower courses across the Mississippi flood plain. Formerly this resulted in numerous floods along those portions of the tributaries that lie within the valley. As a corrective measure, the lower courses of the tributary streams have been straightened and levees constructed to prevent flooding of agricultural lands.

East of Dupo and south of Stolle, where easily dissolved Mississippian limestones are near the surface, the ground is pitted with hundreds of sinkholes 10 to 40 feet deep. This irregular sinkhole topography is markedly different from the flat divides and narrow valleys farther east.

OCCURRENCE AND MOVEMENT OF GROUNDWATER

General Principles

Water flowing over the ground or falling on the ground as rainfall seeps through openings between loose particles of the soil and percolates downward. Below a certain depth, all openings in the loose surface materials and underlying bedrock are filled with water.

The upper surface of the saturated zone is called the *water table*. Its position is determined by the depth at which water stands in wells, borings, and excavations. The water-table surface roughly parallels the surface topography, rising under the uplands and intersecting the ground surface along perennial streams, lakes, and swamps. Its position fluctuates from season to season and year to year. The water table is lowered during periods of prolonged drought; it rises during periods of excessive rainfall. In the East St. Louis region its position is normally at a depth of about 15 to 20 feet below the surface of the valley floor, although concentrated pumpage has lowered it considerably over much of the area.*

The water in the upper part of the saturated zone is unconfined and moves under the influence of gravity in the direction of the water-table slope. In wells that penetrate the saturated zone under these conditions, the water level indicates the level of the water table; these wells are called *water-table wells*.

Where permeable water-bearing formations (aquifers) are overlain by relatively impermeable formations and the water in the aquifers is confined under hydrostatic pressure, artesian conditions exist. Wells penetrating such aquifers are called *artesian*

* For a water-table map of this area see Illinois State Water Survey Rept. Inv. 17, p. 19.

wells. The water levels in artesian wells stand above the bottom of the confining impermeable bed and may be either above or below the level of the water table at any particular place.

Water-table and artesian systems ideally represent two fundamentally different sets of hydrologic conditions. Commonly, however, the confining layer of the artesian aquifer is only relatively impermeable and thus allows slow transmission of water from the system into adjacent aquifers. This is called a *leaky artesian* condition and it most commonly and nearly always prevails in interbedded unconsolidated deposits with different permeabilities, such as the valley fill and glacial deposits in the East St. Louis area.

Aquifers in the East St. Louis Area

VALLEY FILL

For practical purposes, the only aquifer for large-quantity production in the East St. Louis area is valley-fill material, which includes both alluvium and glacial outwash. Groundwater occurs in the valley fill, with its interbedded layers and lenses of varying permeability, primarily under water-table and leaky artesian conditions. At present, this aquifer furnishes all the groundwater pumped from wells in the valley bottom.

BEDROCK

Bedrock aquifers, although in part capable of producing large quantities of water, are now of negligible importance in the American Bottoms because of the possibility of highly mineralized water at depth, the ready availability of water from shallower valley-fill deposits, and the high cost of deep drilling. In many places on the uplands, however, the bedrock is the only groundwater source available and is tapped for domestic supplies. The shallower bedrock formations in this region are not highly productive, and the deeper ones yield highly mineralized water.

GLACIAL DRIFT

Thin deposits of glacial drift are present on the upland adjacent to the area. This material consists of glacial till overlain locally by 50 feet or more of loess. In some places thin beds of sand and gravel within the till furnish enough water for domestic supplies. These local sand and gravel beds are generally found near the base of the till. They are not persistent and their presence normally cannot be predicted prior to drilling.

GEOLOGIC HISTORY

Paleozoic Era

The present landscape of the East St. Louis area has been produced by processes acting only during relatively recent geologic time. A vast amount of earlier time is represented by the indurated sedimentary rocks that underlie the unconsolidated alluvial fill of the American Bottoms (pl. 4). There is virtually no sedimentary record in this area for the time between the formation of the youngest of these sedimentary rocks (Pennsylvanian) and the advance of Kansan ice during the Pleistocene or glacial epoch. A summary of geologic events is given in table 1.

The bedded rocks of the Paleozoic era beneath the valley fill and in the bluffs of the East St. Louis area rest on the eroded surface of much older (pre-Cambrian) rocks at a depth of over 3800 feet. The Paleozoic seas in which these rocks were deposited as sediments alternately advanced and retreated in the area. The position of the shorelines and the character of the sediments deposited were controlled to some extent by activity in the nearby Ozark area, which was uplifted from time to time, beginning early in the Paleozoic era. The sandy and shaly rocks reflect the washing of sands and muds into the shallow seas, whereas the limestones and dolomites suggest clear seas. No doubt crustal movements were gentle, and neither seas nor highlands were strongly or rapidly modified.

At the close of the Pennsylvanian period the sea withdrew and the area became land. It is likely that the area was never again submerged by the sea, though in other parts of the United States thousands of feet of marine sedimentary rocks were formed dur-

TABLE 1.—SUMMARY OF GEOLOGIC HISTORY

Geologic time division				Geologic events in East St. Louis area
Era	Period	Epoch		
Cenozoic	Quaternary	Pleistocene	Recent	Shifting of river channel; modification of flood plain; formation of alluvial fans along bluffs.
			Wisconsin	Deposition of valley trains and loess; dissection of valley-train deposits and formation of terraces.
			Sangamon	Weathering and erosion of till and valley-train deposits; reopening of valley.
			Illinoian	Advance of glacier across American Bottoms and onto bluffs at St. Louis; Mississippi River probably maintained course through or under ice.
			Yarmouth	Weathering and erosion of till and valley-train deposits; reopening of drainage through valley.
			Kansan	Advance of glacial ice; deposition of till; possible damming or restriction of Mississippi Valley.
			Aftonian	Weathering, erosion.
			Nebraskan	Advance of glacial ice, which may have reached this area; deposition of valley train.
	Tertiary			Complex series of crustal movements and erosional cycles; establishment of major drainage lines; major cutting of Mississippi bedrock valley.
Mesozoic	Cretaceous			Erosion.
	Jurassic			
	Triassic			
Paleozoic	Permian			Uplift and erosion.
	Pennsylvanian			Periodic submergences by sea with formation of coal swamps during emergent intervals.
	Mississippian			Submergence; formation of shales and thick limestone formations.
	Devonian			Deposition of lime sediments followed by emergence and erosion.
	Silurian			Deposition of limy sediments along outer margin of a great reef belt; later emergence and erosion.
	Ordovician			Continued submergence, with formation of dolomite, shale, and sandstone; intervals of emergence and erosion.
	Cambrian			Prolonged erosion; later submergence and formation of thick beds of sandstone and dolomite.
Pre-Cambrian				Long period of igneous activity, sedimentation, crustal activity, and erosion.

ing the 250 million years after the Pennsylvanian period.

Mesozoic and Tertiary History

The post-Pennsylvanian history of the East St. Louis area is mainly an account of the wearing down of the land by ancient streams during and after periods of crustal uplift. Four cycles of uplift and erosion are recorded in the bedrock surface of western Illinois (Horberg, 1953, p. 39). Each cycle of erosion was initiated by a period of uplift that gave streams more erosive power and caused them to cut into and partially destroy the existing land surface. The oldest erosion surface, because it was involved in all subsequent periods of uplift, has been largely destroyed, but remnants are preserved in the flat upland surfaces of Calhoun County, 25 miles northwest of Alton.

The crustal uplifts produced many drainage shifts. The latest movement probably established the major drainage patterns in essentially their present form, although many segments of river channels were doubtlessly inherited from early courses. Because the Mississippi River between St. Louis and Cape Girardeau cuts across resistant Mississippian rocks, which have been uplifted along the eastern side of the Ozark dome, instead of flowing across the lowland of the softer Pennsylvanian rocks farther east, it is possible that the river was established in its present channel prior to uplift of the dome. Regional structural and geomorphic relationships suggest that the Mississippi Valley is very old. Furthermore, from regional evidence it appears that it may have been cut essentially to its present depth before the advance of Pleistocene glaciers.

Pleistocene Epoch

The advance of continental glaciers into northern United States during the Pleistocene epoch profoundly modified the landscape. Areas actually overridden by the glaciers were blanketed by unsorted rock debris as the ice melted and dropped its load. Beyond the ice front, sediment-laden meltwaters escaped down valleys toward the sea, partially filling them with glacial sand and gravel deposits that became progressively finer downstream. The river flats, kept free of vegetation by frequent glacial flooding, were subject to wind erosion, and great volumes of silt were picked up and transported to the uplands bordering the valleys. The unsorted ice-laid deposits (till), the sorted water-laid material (outwash), and the wind-transported silts (loess) mantle the bedrock in the American Bottoms and adjacent area.

The history of the earlier glacial advances (Nebraskan and Kansan) in the area is obscure, but later glacial events are better documented. The presence of Illinoian till in St. Louis and along the eastern bluffs of the valley indicates that the Illinoian ice, advancing from the northeast, extended across the American Bottoms.

The "clay," "blue clay," and "blue clay and gravel" that are logged in many wells just above bedrock in the Alton–Wood River area may be pebbly glacial till which could be of Illinoian age or older. Because the Illinoian drift is thin, it is unlikely that the valley was completely filled at that time, although drainage was temporarily blocked or restricted so that ponding took place upstream in the Mississippi, Illinois, and Missouri valleys.

The Wisconsin glacial stage in the East St. Louis area was marked by the downstream spread of outwash as valley trains during ice advances in the north and by deposition of loess on the bluffs. Loess is well exposed in the uplands on the eastern side of the valley, particularly in road cuts along Highway 460 between East St. Louis and Belleville where the road first enters the uplands. The loess deposits indicate that the Mississippi valley bottom was covered with extensive valley-train deposits including glacial rock flour from Wisconsin ice sheets. The nearest approach of Wisconsin ice was during the Tazewell substage when the ice advanced into Shelby County, some 75 miles to the northeast.

During one glacial advance, the flood plain at East St. Louis was aggraded to an elevation of about 445 feet. Remnants of this surface are the terraces at Roxana and Wood River and along Cahokia Creek.

Subsequent river downcutting destroyed this surface in all but the northern portion of the American Bottoms. The Recent river scour and reworking have not been complete, however, for the lower section of the valley fill is believed to be partly glacial in origin. Wood fragments found in the lower part of the fill have been dated by the radioactive carbon method as older than 20,000 years, which dates the wood, and presumably the deposits containing the wood, as at least as old as early Wisconsin.

The large boulders commonly encountered at depths of 80 feet or more, which sometimes limit the depth of drill penetration, are probably remnants of Illinoian or older till.

In Recent time the river has scoured and reworked the upper part of the valley fill in migrating across the broad bottomlands. The channel scouring has taken place chiefly during floods when volume and velocity were high. At the same time, spreading floodwaters have deposited silt and clay along the sides of the channel and in backwater areas. In subsiding and low-water river stages, only fine-grained sediments have been transported, and silting has taken place in the channel. The channel migration, cut-and-fill, and flooding have produced complex, heterogeneous deposits which vary in depth (fig. 4). Soundings at Eads Bridge during river flood have indicated river scour as deep as 80 feet (Woodward, 1881, p. 5). This figure is thought to represent the average depth to which the valley fill has been reworked along the Recent meander belt. Below this depth the deposits are glacial outwash material and older alluvium.

The broad alluvial fans found below the bluffs are also of Recent age. They are composed of reworked loess and have been built outward across the valley fill by tributary streams and slope wash.

GEOLOGY AND WATER-BEARING PROPERTIES OF THE BEDROCK

Regional Relations

The river sediments of the American Bottoms are underlain by consolidated sedimentary rocks over 3800 feet thick, as shown by a well completed at the City Sanatorium in St. Louis in 1869 (Broadhead, 1878).

The bedrock formations, dominantly limestone and dolomite with subordinate amounts of sandstone and shale, dip gently northeastward from the Ozark highlands toward the Illinois Basin. In the area of the American Bottoms, minor folds have been superimposed upon the regional structure so that locally the beds may dip in other directions (plate 4). For example, in the southern part of the area a sharp transverse arch produces reversals of the regional dip. The axis of this fold extends from the vicinity of Waterloo in Monroe County in a northwesterly direction through Dupo on the American Bottoms and across the Mississippi at Arsenal Island into St. Louis. The steeply dipping beds on the southern limb of the arch can be seen in the bluffs south of Dupo. The arch is the controlling structure for the accumulation of oil in the Waterloo and Dupo oil fields of Illinois (Bell, 1929). The Florissant dome north of St. Louis is near the trend of the structure.

Mississippian rocks underlie the valley fill in the western part of the American Bottoms, and Pennsylvanian rocks underlie the bottom sediments in the eastern part. The approximate boundary between Pennsylvanian and Mississippian rocks is shown in plate 3. A summary of formations underlying the American Bottoms is given in table 2.

The Mississippi River now follows a channel underlain, beneath the alluvium, by Mississippian limestones. The widening of the Mississippi Valley between Wood River and Dupo is a result of the river's lateral cutting into the easily eroded shales of the Pennsylvanian and Mississippian (Chester) formations upstream from the resistant Mississippian limestones that are at the surface in the Waterloo-Dupo structure.

Lithology of the Bedrock

Most information on the bedrock formations in the American Bottoms has come

TABLE 2.—GEOLOGIC FORMATIONS OF THE EAST ST. LOUIS AREA AND THEIR GROUNDWATER POSSIBILITIES

Era	System	Series	Group	Formation	Average thickness	Material	Groundwater possibilities in East St. Louis area
Cenozoic	Quaternary	Pleistocene	Recent alluvium		0–100	Sand, gravel, silt, and clay	Permeable sands and gravels are water-yielding.
			Glacial till, outwash, and loess		10–170	Pebbly clay, sand and gravel, and silt	
Paleozoic	Pennsylvanian				100–400	Shale, sandstone, limestone, and coal	Some of the sandstones and limestones have sufficient permeability to yield water for domestic drilled wells.
	Mississippian	Chester			0–200	Sandstone, shale, and limestone	Some of the sandstones, particularly in lower part of the series (Aux Vases), have moderate permeabilities and are fair-to-good groundwater sources, if close to outcrop area or not too deeply buried.
		Iowa	Meramec	Ste. Genevieve	0–150	Sandy oolitic limestone	Yield water from joints and solution channels. Meramec limestones, particularly St. Louis, are potential water sources north of Alton, in St. Louis, and in sinkhole region south of Stolle.
				St. Louis	200–250	Limestone and dolomite, fine grained	
				Salem	50–100	Dolomite and granular fossiliferous limestone	
			Osage	Warsaw	40–140	Shale and argillaceous limestone	Keokuk-Burlington limestones are less cavernous than St. Louis limestone and therefore not as favorable as a groundwater source except along Dupo arch where limestone is close to surface.
				Keokuk-Burlington	200–270	Cherty crinoidal limestone	
				Fern Glen	45–100	Shaly limestone	
			Kinderhook	Chouteau	10–30	Slightly silty fine-grained limestone	Not water-yielding.
				Hannibal-Grassy Creek	5–50	Dark shale	

Devonian				0–30	Sandy limestone and dolomite	
Silurian	Niagaran	Bainbridge	Moccasin Springs	20–170	Shaly red limestone	Devonian-Silurian limestone may yield water from joints and solution crevices, but at depth encountered the water is highly mineralized.
			St. Clair	30–40	Crystalline pink-speckled limestone	
	Alexandrian		Sexton Creek	20–30	Cherty limestone	
			Edgewood	5–30	Silty dolomite	
Ordovician	Cincinnatian		Maquoketa	140–160	Shale and shaly limestone	Not water-yielding.
	Mohawkian		Kimmswick	75–100	Coarse-grained crinoidal limestone	Kimmswick-Joachim limestone not well jointed or cut by solution channels and not considered a likely groundwater source, even of highly mineralized water.
			Decorah	15–30	Limestone and shale	
			Plattin	100–200	Fine-grained limestone	
	Chazy		Joachim	70–120	Silty dolomite	
			St. Peter	135–155	Clean sandstone, poorly cemented	High permeability, but groundwater highly mineralized.
	Prairie du Chien			850±	Dolomite and sandstone	Most of section is dense dolomite with poor groundwater possibilities. Permeable formations contain highly mineralized water.
Cambrian	St. Croixan			1350±	Dolomite, sandstone, and shale	

Ancient granitic and other crystalline rocks referred to the Proterozoic and Archeozoic eras, called per-Cambrian rocks.

from oil test wells, most of which are drilled to the Kimmswick limestone, the producing formation in the Dupo oil field. Some wells have gone to the St. Peter sandstone; only a few have gone deeper.

A sample-study log of one of the deeper oil tests, drilled 2 miles southeast of Dupo, follows.

Lockwood-Dyroff well 1—NW corner NE¼ sec. 26, T. 1 N., R. 10 W., St. Clair Co. Drilled November 1929. Illinois Geological Survey sample-study set 723, studied by F. E. Tippie. The pre-St. Peter correlations are in part based on a study of this well by John Grohekopf and Earl McCracken, Missouri Geological Survey.

	Depth feet
Pleistocene system	
"Soil"	26
Mississippian system	
Iowa series	
Meramec group	
St. Louis limestone	
"Limestone, white, hard"	45
Limestone, slightly oolitic, finely sandy, white, extra fine	50
"Limestone, white, hard"	90
Limestone, finely sandy, light brown, sublithographic; dolomite, sandy, brown, very fine	95
"Limestone, white, hard"	150
Dolomite, cherty, silty, light gray, very fine	155
"Limestone, white, hard"	165
Limestone, slightly sandy and cherty, buff, very fine	170
Dolomite, partly sandy and argillaceous, light brown, very fine	180
"Limestone, white, brown"	210
Limestone, partly oolitic, slightly cherty, brown, very fine	215
Limestone, dolomitic, brown, extra fine	225
Limestone, dolomitic, cherty, partly oolitic, white to light brown, very fine	235
Limestone, brown, sublithographic	240
Limestone, partly oolitic, dolomitic, white to brown, very fine	265
Dolomite, slightly cherty, brown, very fine	273
Salem limestone	
Limestone, dolomitic, oolitic, slightly cherty, brown, very fine	283
Limestone, brown, lithographic	290
Limestone, dolomitic, oolitic, light brown, very fine	305
Limestone, oolitic, cherty, slightly sandy, light brown, fine	325
Limestone, dolomitic, cherty, brown, very fine	335
Limestone, slightly sandy, mottled gray and brown, medium	345
Limestone, slightly sandy, light brown, medium	385
Limestone, slightly dolomitic, gray, brown, mottled very fine	395
Limestone, slightly dolomitic and sandy, gray, brown, medium, conglomeratic	410
Dolomite, gray, very fine	415
Limestone, dolomitic, brownish, gray, very fine to fine	425
Osage group	
Warsaw formation	
Dolomite, slightly argillaceous, brown, gray, little greenish, very fine	445
Dolomite, very argillaceous, cherty, gray, very fine; shale, dolomitic, gray	460
"Shale, blue, soft"	495
Dolomite and shale as above	500
Keokuk-Burlington limestones	
Dolomite, extra cherty, light gray, very fine, glauconitic	515
Dolomite, argillaceous, extra cherty, gray, very fine, glauconitic	530
Limestone, dolomitic, slightly sandy, white, very fine, partly glauconitic; chert, white, abundant, partly glauconitic	676
Limestone, cherty, white, fine to coarse, crinoidal	681
Fern Glen formation	
Dolomite, very argillaceous, green, grading to shale	690
Limestone, cherty, light brown, reddish, sublithographic	695
Limestone, cherty, white, fine to coarse, crinoidal; shale, calcareous, green	700
Limestone, cherty, white, greenish, very fine to fine, crinoidal; shale, calcareous, green, red at base	715
"Shale, blue, soft"	730
Limestone, argillaceous, slightly cherty, white to red, very fine to coarse, crinoidal; shale, calcareous, red, green	750
"Lime, red, soft"	755
Kinderhook group	
Chouteau limestone	
"Lime, gray, hard"	770
Limestone, white, brownish, sublithographic	785
Hannibal shale	
Shale, dark gray to black, few coarse sand grains at base	798
Silurian system	
Dolomite, silty, slightly cherty, white, little pinkish, very fine	825
Dolomite, cherty, light brown, very fine	830
Limestone, dolomitic, cherty, white, very fine to medium	845
Dolomite, slightly cherty, light brown, very fine	868

	Depth feet
Ordovician system	
Maquoketa formation	
Shale, dolomitic, silty, green, gray, very weak	925
Shale, silty, dolomitic, dark brown	935
Siltstone, calcareous, light brown; dolomite, argillaceous, cherty, silty, gray, very fine	950
Shale, calcareous, brownish gray; and limestone, very argillaceous, brownish gray; little chert	1015
Kimmswick limestone	
Limestone, white to light brown, very fine, little coarse	1020
Limestone, white to light brown, fine to lithographic	1030
Limestone, white, buff, fine to coarse	1065
Limestone, cherty, white to light brown, fine to coarse	1110
Dolomite, brown, very fine	1113
Decorah formation	
Limestone, dolomitic, argillaceous, brown, very fine; little shale, gray	1130
Plattin limestone	
Limestone, slightly cherty, light brown, sublithographic	1140
Dolomite, slightly cherty, light brown, very fine	1165
Dolomite as above; limestone, partly cherty, white, brownish, sublithographic	1240
Limestone, slightly cherty, white to buff, very fine	1260
Limestone, slightly cherty, light brown to white, sublithographic	1285
Limestone, slightly cherty, light brown to white, very fine; little dolomite, dark brown, very fine to base	1325
Joachim dolomite	
Dolomite, light grayish brown, very fine; shale, dolomitic, greenish gray	1335
Dolomite, light gray to light brown, very fine	1385
Dolomite, white, buff, very fine, becoming slightly argillaceous and cherty at base	1410
"Lime, gray, soft"	1425
Shale, green, very weak; dolomite, white, light brown, very fine	1433
Dolomite, argillaceous, brown, gray, greenish, very fine	1440
Dolomite, white to brownish, very fine, finely sandy at base	1473
Shale, dolomitic, finely sandy, gray	1478
Glenwood–St. Peter sandstone	
Sandstone, white to red (iron stain), fine to coarse, incoherent, generally rounded and frosted; little shale, sandy, green at base	1632
Cotter and Everton formations	
Dolomite, cherty, white, very fine, scattered sand grains, iron stain	1645
Sandstone, white, fine to coarse, iron stain	1650
Dolomite, cherty, white, very fine	1690
Dolomite as above; little sandstone, dolomitic, medium, scattered sand grains	1705
Dolomite as above; little shale, slightly dolomitic, gray	1710
Dolomite, cherty, buff, very fine; sandstone, white, incoherent	1725
Dolomite, cherty, partly sandy, white to buff, very fine	1780
Cotter formation	
Dolomite, partly sandy, white to gray, very fine	1800
Dolomite, cherty, slightly sandy, light brown, very fine	1850
Dolomite, sandy, white to light brown, very fine; chert, banded, oolitic; little sandstone, calcareous, white, fine to coarse	1895
Upper Jefferson City formation	
Dolomite, sandy, white, brownish, very fine; chert, white; sandstone, calcareous, white, fine to coarse; shale, calcareous, gray at base	1985
Lower Jefferson City dolomite	
Dolomite, slightly sandy, very cherty, white, gray, light brown, very fine; chert, bluish, white, translucent	2155
Roubidoux formation	
Dolomite, silty, sandy, gray, buff, very fine; much chert, white, opaque, partly sandy	2240
Sandstone, white, fine to medium, subangular, incoherent; dolomite, as above; little bright green shale at base	2285
Gasconade formation	
Dolomite, white, fine to coarse, scattered sand	2307
Dolomite, very cherty, white, very fine to fine, scattered sand	2450
Dolomite, cherty, partly sandy, white to light gray, very fine to fine	2495
Gunter formation	
Dolomite, very sandy, cherty, white, very fine to fine; sandstone, dolomitic, white, fine	2530
Cambrian system	
Eminence dolomite	
Dolomite, very cherty, white, very fine to fine, scattered sand	2575
Shale, sandy, white, very weak, slightly glauconitic	2580
Dolomite, cherty, partly sandy, white to light brown, very fine to fine	2730

	Depth feet
"Lime, gray, hard"	2740
"Sand, gray"	2764
Potosi dolomite	
Dolomite, cherty, sandy, light brown, very fine to fine with some medium, pyritic	2900
"Sand, white; oil"	2904

The log of the City Sanatorium well in St. Louis (Broadhead, 1878) suggests that the Potosi dolomite, encountered in the lower part of Lockwood-Dyroff well 1, may be underlain by at least 800 feet of Cambrian beds, principally dolomite except for shale beds of the Davis formation and basal Lamotte sandstone.

In the eastern portion of the American Bottoms, wells drilled into bedrock penetrate several hundred feet of shale, sandstone, and thin limestone beds of the Pennsylvanian system and the Chester series (Mississippian) before reaching the massive Mississippian limestones that are near the surface south of Stolle and north of Alton. The sample-study log of a well 1½ miles northeast of Horseshoe Lake illustrates the nature of these upper beds.

Kesl–Kusmanoff well 1—660 feet N line, 330 feet W line, SW¼ SE¼ sec. 12, T. 3 N., R. 9 W., Madison Co. Drilled July 1947. Illinois Geological Survey sample-study set 17178, studied by M. P. Meyer and Heinz Lowenstam. Depths adjusted to electric log and drilling time. Core study from 1215 to 1227 and from 1641 to 1687 feet.

	Thickness feet	Depth feet
Pleistocene and Pennsylvanian systems		
No samples	165	165
Samples not studied	35	200
Shale, gray, carbonaceous, micaceous, weak	30	230
Sandstone, argillaceous, silty, gray, very fine to fine, friable; interbedded shale, sandy, gray, carbonaceous	6	236
Mississippian system		
Chester series		
Paint Creek formation		
Limestone, sandy (very fine), buff, very oolitic, medium to coarse, compact	12	248
Limestone, partly argillaceous, buff, fine to medium, crinoidal	3	251
Shale, calcareous, green, weak	12	263
Limestone, argillaceous at top, brown, medium to coarse, fossiliferous, crinoidal	10	273
Shale, calcareous, mottled red and green, weak	8	281
Yankeetown siltstone		
Siltstone, very cherty, calcareous, white, compact; little sandstone, cherty, calcareous, very fine at top	6	287
Renault formation		
Shale, slightly calcareous, green and gray variegated, weak	3	290
Limestone, sandy (fine), glauconitic, light gray, medium	3	293
Sandstone, calcareous, argillaceous, green, very fine, tight; shale, green, gray to purple, weak	7	300
Siltstone, greenish gray, friable; shale, silty, mottled purple and green at top	10	310
Shale, silty, green, purple, weak; shale, red at base	8	318
Sandstone, silty, light gray to green, very fine to fine, friable	13	331
Shale, red and green variegated, weak	9	340
Shale, silty and sandy, calcareous, green, weak; grading to sandstone, argillaceous, silty, very fine, green	15	355
Shale, as above; pyrite	10	365
Aux Vases sandstone		
Sandstone, slightly calcareous, silty, light gray, very fine, friable	5	370
Sandstone, calcareous, light gray, fine, friable	16	386
Iowa series		
Meramec group		
St. Louis formation		
Limestone, buff, partly sandy, fine to oolitic to lithographic	14	400
Samples not studied	295	695
Limestone, very cherty, buff, fine, oolitic	10	705
Dolomite, very cherty, buff, red speckled, extra fine	50	755
Salem limestone		
Limestone, brown, gray speckled, medium, fossiliferous	5	760
Limestone, dolomitic, grayish brown, fine to medium, fossiliferous, oolitic (*Endothyra*)	15	775

	Thickness feet	Depth feet
Limestone, dolomitic, gray, black specked, extra fine, medium	20	795
Limestone, grayish brown, medium to coarse, fossiliferous	16	811
Osage group		
Warsaw formation		
Dolomite, silty, slightly glauconitic, gray, extra fine; quartz	14	825
Limestone, dolomitic, silty, slightly glauconitic, gray, extra fine; quartz	5	830
Shale, very dolomitic, calcareous, silty, gray, brittle; quartz	27	857
Limestone, argillaceous, silty, gray, fine; quartz	31	888
Keokuk-Burlington limestone		
Limestone, glauconitic, cherty, buff, coarse	14	902
Limestone, very cherty, glauconitic, light buff, medium to coarse	23	925
Samples not studied	125	1050
Limestone, very cherty, white, medium to coarse	15	1065
Fern Glen limestone		
Limestone, dolomitic, silty, cherty, light gray to green, extra fine	30	1095
Limestone, cherty, argillaceous, silty, green, sublithographic	30	1125
Limestone, as above; grading to little shale, calcareous, mottled red and green	27	1152
Kinderhook group		
Chouteau limestone		
Limestone, white to light buff, lithographic	21	1173
Limestone, red, sublithographic	6	1179
Limestone, light green, sublithographic	5	1184
Hannibal-Grassy Creek shale		
Shale, black, weak	26	1210
Shale, silty, gray, weak	5	1215
Shale, brown, tough, sporangites; "Hardin sand" 1 inch at base, argillaceous, coarse, fine, pyritic at base	4	1219
Silurian system		
Niagaran series		
Dolomite, argillaceous, silty, light gray, pyritic	37	1256
Limestone, dolomitic, argillaceous, gray to greenish gray, fine, some red shale partings	22	1278
Shale, calcareous, greenish gray; few limestone streaks, as above	7	1285
Limestone, dolomitic, argillaceous, greenish gray, fine, with pink and red silty shale partings	20	1305
Limestone, silty, argillaceous, red, fine, scattered coarse crinoidal fragments	14	1319
Shale, calcareous, red, brittle	6	1325
Limestone, dolomitic, argillaceous, silty, red, crinoidal	27	1352
Limestone, white to buff, fine to medium, with red crinoidal; streaks siltstone, argillaceous, red	48	1400
Limestone, as above, dolomitic, less crinoidal	6	1406
Alexandrian series		
Kankakee formation		
Dolomite, slightly calcareous, buff, light brown, fine	15	1421
Limestone, slightly glauconitic, white to light gray, medium crystalline, pyritic, very cherty from 1435 to 1452 feet	31	1452
Edgewood dolomite		
Dolomite, calcareous, light brown, fine, sucrose	20	1472
Ordovician system		
Maquoketa shale		
Shale, light greenish gray, weak; streaks siltstone to sandstone, very fine, friable	28	1500
Samples not studied	95	1595
Shale, silty, green, brown speckled, weak	26	1621
Shale, silty, calcareous, green, grayish brown, weak	11	1632
Kimmswick limestone		
Limestone, buff, red speckled, medium	9	1641
Limestone, buff, medium to coarse, fossiliferous, compact, brown and gray shale partings	46	1687

Plate 4 shows representative graphic logs from several deep wells in the American Bottoms.

Groundwater in the Bedrock Formations

No groundwater supplies are being withdrawn from bedrock formations in the American Bottoms, mainly because ade-

quate water supplies of suitable quality are available in the shallower valley-fill material. Groundwater is obtained in St. Louis from wells drilled into upper Mississippian limestones, although the municipal water supply of St. Louis and the major cities of the American Bottoms is obtained from the Mississippi River.

On the eastern upland bordering the valley, water is obtained from sandstones of the Chester series, from sandstones and fractured limestones of the Pennsylvanian system, and from Mississippian limestones. Belleville formerly obtained its water supply from wells drilled 500 to 600 feet deep, into Chester sandstones, but now obtains its supply from East St. Louis.

Beneath the uplands from East Alton to Belleville, Pennsylvanian and Chester sandstones are potential sources of water. Because of their thinness and low permeability, Pennsylvanian sandstones are rarely suitable for other than domestic wells. Mississippian limestones yield groundwater from solution channels and joints. They are potential sources of groundwater mainly between Prairie du Pont Creek and the Mississippi River in the southern part of the area and north and west of Alton in the northern part of the area.

Water obtained from bedrock commonly is too highly mineralized to be acceptable for domestic or industrial use, particularly at depths greater than 370 to 420 feet below ground level on the flood plain and 515 feet below ground level on the uplands (Bowman and Reeds, 1907, p. 56). In general, mineralization increases with formation depth. Analyses of water from bedrock formations in St. Louis County, Mo., show from 4,415 to 11,010.6 ppm total dissolved solids from pre-St. Peter formations and more than 1,000 ppm from the St. Peter at depths below 800 feet (Gleason, 1935). Because the beds dip to the northeast, a given formation generally yields progressively more highly mineralized water in that direction.

The general movement of groundwater is to the northeast, in the general direction of the regional dip of the bedrock formations. Minor structures, as at Dupo, may modify the direction of this movement. The dip of permeable rocks that crop out around the Ozark highlands and the presence of interbedded relatively impermeable shales produce artesian conditions. In the St. Louis–East St. Louis area, the St. Peter sandstone yields water under artesian pressure, although the pressure is insufficient to produce a flowing well. Artesian wells of low yield also have been reported from other formations in the area.

GEOLOGY AND WATER-BEARING PROPERTIES OF THE VALLEY FILL

Bedrock Valley

As shown in the bedrock surface map (fig. 2) and cross sections (fig. 4 and plate 4), the present Mississippi River Valley occupies a deep bedrock valley that has been partially filled by aggrading processes of the river. In much of the area, the bedrock valley floor lies 100 feet or more beneath the bottom of the present valley; in at least one place its depth is over 170 feet (see fig. 3 for thickness of valley fill above the bedrock). Available data indicate that the bedrock valley has steep walls along the present bluff line but that the valley bottom slopes gently toward the middle. In the vicinity of Dupo, the valley narrows as the river crosses resistant Mississippian limestones. Between Dupo and Alton, soft Pennsylvanian sandstone and shale beds form the eastern wall of the bedrock valley. The limestone at Dupo may have resisted downcutting by the river and thus promoted upstream lateral cutting of the Pennsylvanian strata, causing widening of the valley in the middle of the area. Valley widening probably has been aided further by the coincidental location of the weaker beds outside a major bend in the river. The elevation of the bottom of the bedrock valley averages about 310 feet. The bedrock upland bordering the valley on the east ranges in elevation from about 500 feet east of Horseshoe Lake to over 600 feet east of Dupo.

Several types of data suggest that an inner channel, shown within the 280-foot

contour lines in figure 2, has been cut at least 20 feet below the average level of the bedrock valley floor.

The log of a test well at Roxana (location A-10) shows 171 feet of valley fill, with bedrock not yet reached. The elevation of bedrock here must be less than 281 feet above sea level. Although there is abundant information from wells in the vicinity of the test hole, reliability of the data concerning depth to bedrock is uncertain. It is likely, however, that bedrock elevation at this location is at least 20 feet below that found in the adjacent area. An oil well between Dupo and East Carondelet penetrated 122 feet of valley fill before reaching bedrock. The bedrock elevation here is 280 feet above sea level, approximately 20 feet lower than in nearby wells. In excavating for the east abutment of Eads Bridge, which connects East St. Louis with St. Louis, bedrock was encountered at 284 feet above sea level. This, too, is approximately 20 feet below the general elevation of the bedrock valley floor.

Another indication of the channel has resulted from seismic work in the area. At several locations in the middle of the valley, bedrock elevations were calculated to be substantially below the elevation of the adjacent bedrock valley floor. Seismic data give elevations for the middle channel that range from 235 feet near the southern border of the area to 260 feet just west of Wood River. It is believed that the indicated 235-foot elevation is too low (possibly by 25 feet) and that the channel floor in this part of the valley is closer to 260 feet above sea level. The basis for this estimate is a Corps of Engineers line of test holes across the Mississippi River four miles to the south, in Monroe County, where the elevation of the channel floor is 256.75 feet. Other seismic stations, apparently over the channel, give elevations of 273, 280, 266, and 263 feet. The linear arrangement of these low elevations and the generally good agreement between seismic results and known elevations tend to confirm the existence of a channel cut below an elevation of 280 feet as far north as Wood River. It is also possible that the channel, at least in the southern part of the area, has an elevation as low as 260 feet. Additional information must be obtained before the exact position and maximum depths of this channel can be determined. On the basis of bedrock elevations given for the Illinois and Upper Mississippi valleys by Horberg (1950), the 280-foot contour line is carried north of Wood River in the bedrock surface map (fig. 2).

Three wells more or less in a line from Monks Mound northeastward also give bedrock elevations somewhat below adjacent areas. These wells record bedrock at an elevation below 290 feet and suggest the presence of a channel—possibly a tributary of the main channel—that swings close to the bluffs north of Caseyville.

In the reach of the Mississippi River known as "Chain-of-Rocks," west of Granite City, the present channel crosses a gently sloping bedrock bench. Along this part of the channel, from approximately a mile north of Merchant's Bridge to a mile north of Chain-of-Rocks Bridge, the river flows partly on bedrock. The shallowness of the water here interferes with river shipping and has led to the construction of Chain-of-Rocks Canal, which serves as a bypass.

Bedrock in the Chain-of-Rocks area is 20 to 80 feet higher than in the remainder of the valley; as a result, the valley fill is thinner by the same amount (fig. 3). As the river is actively eroding the bedrock here, this portion of the bedrock valley is undoubtedly younger than the deeper valley to the east.

The bedrock tributary valleys shown in figure 2 coincide with the present stream valleys. There is, however, a discordance between the bedrock valley and the present Wood River channel between East Alton and Alton where the river enters the American Bottoms. Here the river follows the western side of a mile-wide valley and flows across a spur of Mississippian limestone at an elevation of about 420 feet; half a mile to the east, the bedrock valley is 100 feet deeper and contains about 110 feet of fill.

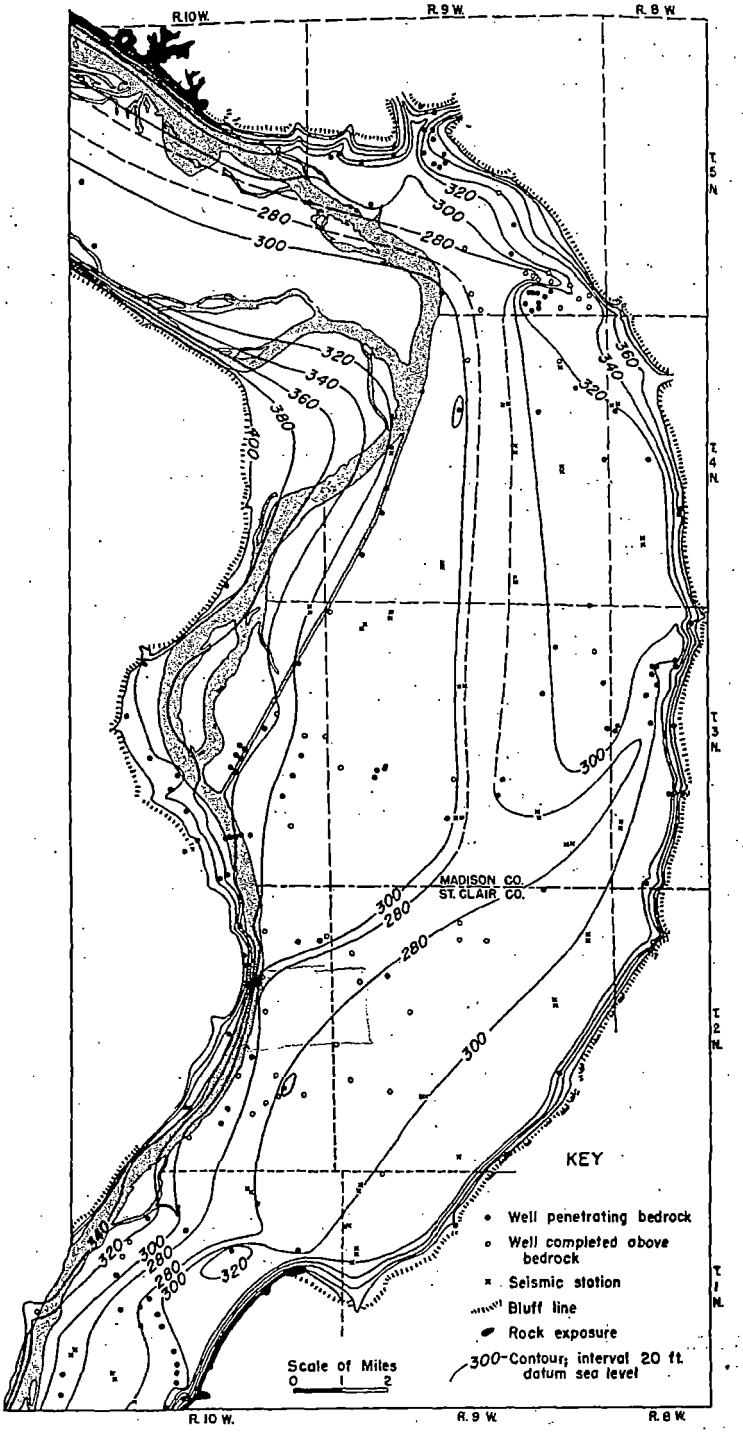


FIG. 2.—Bedrock surface map of the East St. Louis area, Ill.

1" = 5415

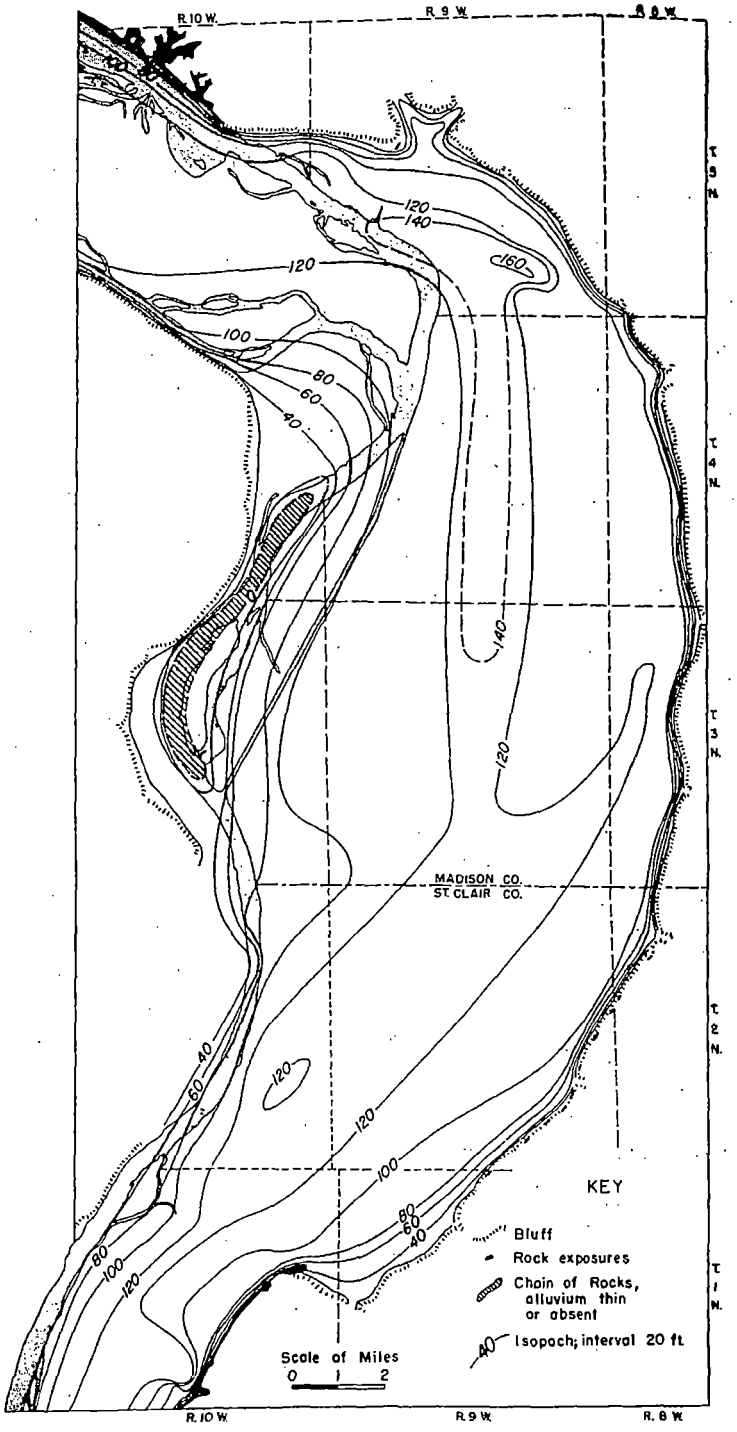


FIG. 3.—Thickness of the valley fill in the East St. Louis area, Ill.

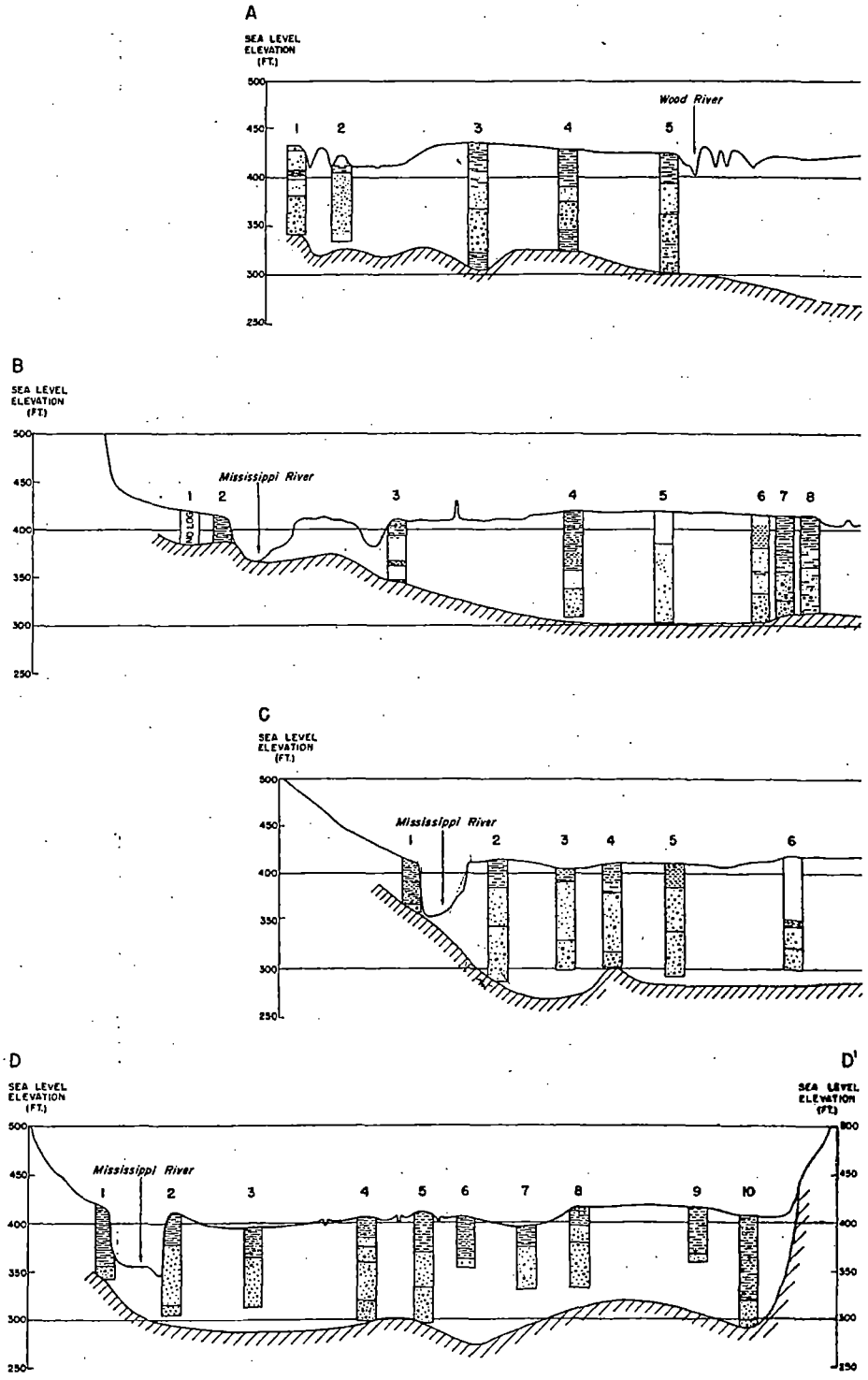


FIG. 4.—Cross sections of the valley fill in the East St. Louis area, Ill.

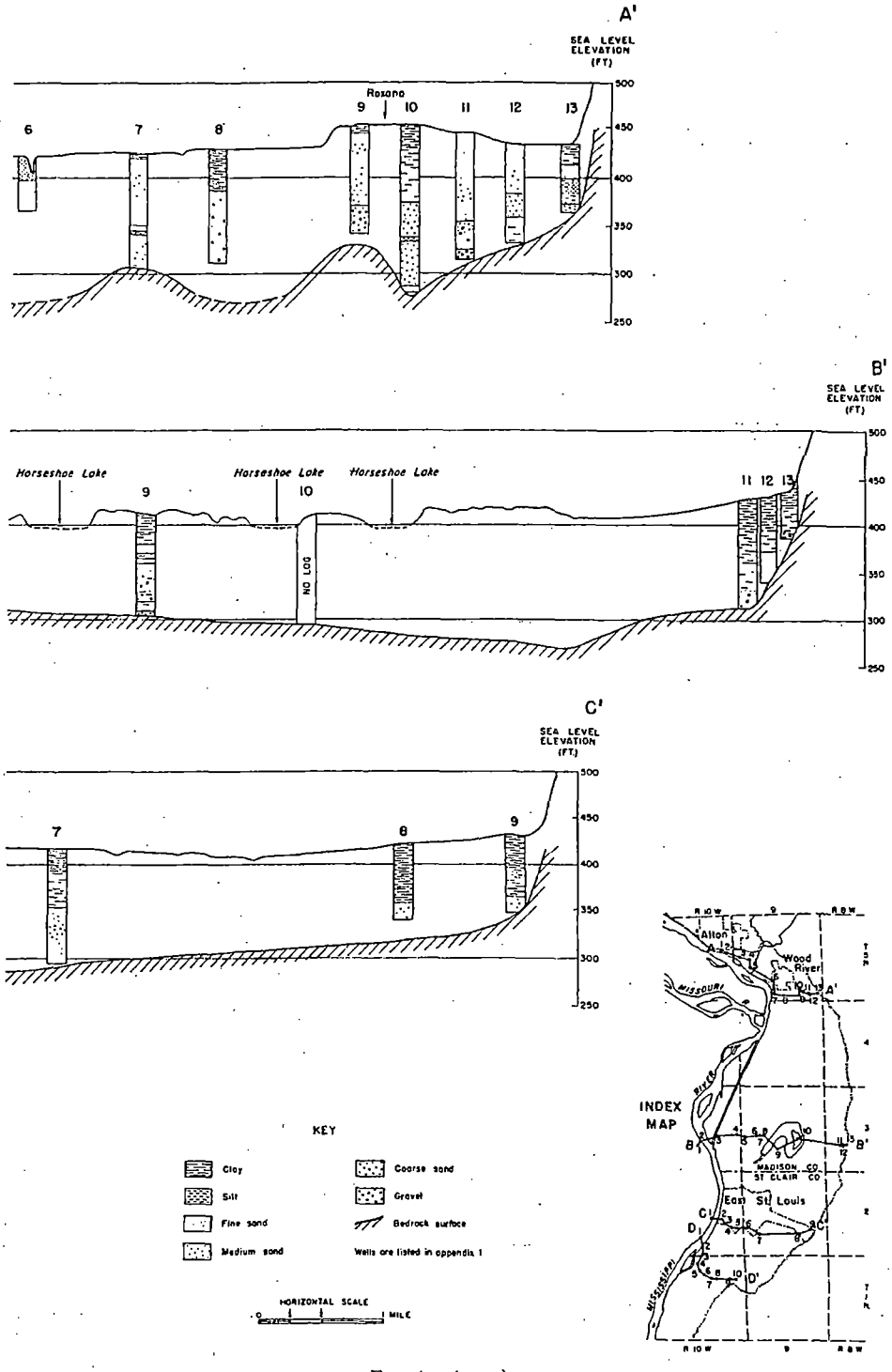

A'
SEA LEVEL ELEVATION (FT)
Roxana
6
7
8
9
10
11
12
13
500
450
400
350
300
250
B'
SEA LEVEL ELEVATION (FT)
Horseshoe Lake
Horseshoe Lake
Horseshoe Lake
9
10
11 12 13
NO LOG
C'
SEA LEVEL ELEVATION (FT)
7
8
9
KEY
Clay
Silt
Fine sand
Medium sand
Coarse sand
Gravel
Bedrock surface
Wells are listed in appendix 1
HORIZONTAL SCALE
MILE
INDEX MAP
Alton
Wood River
MISSOURI
RIVER
MISSISSIPPI
MADISON CO
ST CLAIR CO
East St Louis
R 10 W
R 8 W

FIG. 4.—(cont.)

Valley Fill

The valley fill of the American Bottoms is composed of Recent alluvium and glacial valley-train material derived from the drainage areas of the upper Mississippi and Missouri rivers. Thickness and cross sections of the valley fill are shown in figures 3 and 4.

Valley-train material is found at the surface in the valley only in terraces in the vicinity of Roxana and Wood River. This material is distinctive in composition and texture (see below). Similar material has been found at depth in a few wells near the terrace, separated from overlying Recent alluvium by a rather marked lithologic break. In most of the area, valley-train material is buried beneath the Recent alluvium.

In most of the American Bottoms, differentiation of valley-train and other alluvial deposits, on the basis of mineralogical and textural characteristics or on lithologic breaks, is not possible. South of the Missouri River mouth, the valley fill contains no apparent discontinuity; valley-train material in this area is apparently mixed with older Missouri River alluvium. These deposits, in addition, have been reworked to varying depths by Recent river scour-and-fill.

GLACIAL VALLEY-TRAIN DEPOSITS

In the Roxana-Hartford area there is a mineralogical difference between the valley-train and Recent alluvial deposits, but south of the Missouri River mouth the valley fill cannot be separated into glacial outwash and alluvial deposits. The sands of the Roxana–Wood River terrace and those in the lower portion of the valley fill at Hartford average 75 to 80 percent quartz, 8 to 15 percent potash feldspar, 5 to 10 percent plagioclase feldspar, and 2 to 6 percent other material. Over 85 percent of the quartz grains are clear and untinted, and the majority are subrounded to rounded. About 10 percent of the quartz grains are pink. Many have flecks of reddish stain in tiny pits on their surfaces. Washing the sand in dilute hydrochloric acid virtually eliminates the pink color of the quartz grains. However, owing to the large proportion of potash feldspar and pink-tinted quartz grains, dry valley-train sands commonly look pink.

The valley-train deposits underlying the terrace at Roxana are texturally quite distinctive. The bulk of the material below shallow depths consists of well-sorted medium-to-coarse sand; median diameters range from .01 inch (.25 mm) to .03 inch (.76 mm). The small amount of gravel present is of granule size (between 4 and 9 mesh).

The sample study of a well at Roxana illustrates the nature of the valley-train material underlying the terrace.

Illinois State Geological Survey test hole 3 (1954) —Roxana Water Works, SE¼ NE¼ SE¼ SE¼ sec. 27, T. 5 N., R. 9 W., Madison Co. Samples studied by R. E. Bergstrom. Est. elev. 445 feet.

	Thickness feet	*Depth feet*
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, graywacke, 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, graywacke	5	127
Pennsylvanian system		
Shale, gray and brown	9½	136½

Textural uniformity, which characterizes the deposits of the terrace, does not appear to be a general feature of the valley-train material. Wells near the terrace but on lower levels in the Hartford–Wood

River area pass through deposits that resemble the valley train mineralogically but range from medium sand to pebble gravel. These deposits occur in the lower 20 to 40 feet of the valley fill; in a few wells there is a rather sharp break in composition between them and the overlying alluvium.

The sample study from a well drilled at the Sinclair refinery at Hartford, one mile west of the Wood River terrace, illustrates the nature of the valley-train material beneath Recent alluvium and the lithologic break that separates them.

Sinclair Oil Company well 2 (1952)—150 feet N, 1750 feet E of SW corner sec. 34, T. 5 N., R. 9 W., Madison Co. Samples studied by R. E. Bergstrom. Est. elev. 431 feet.

	Thickness feet	*Depth feet*
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 ¾ 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 1½ inches in diameter, abundant chert, limestone, graywacke, rhyolite	7½	122½

At the Shell Oil Company loading dock, a mile west of the above location, the lower part of the river fill is also interpreted as glacial valley train. A sample of wood from this material was obtained from a Shell Oil Co. collector well (fig. 5). It is dated as "older than 24,000 years" by the carbon 14 method, which tends to corroborate the valley-train interpretation (Libby, 1954).

South of the Missouri River mouth, valley-train and other alluvial deposits cannot be differentiated. Wells here penetrate, from top to bottom, 10 to 30 feet of surficial silt and clay, silty sand and gravel, and cleaner sand and gravel. At many places coarse bands, generally at depths greater than 75 feet, contain substantial deposits of granule and pebble gravel. Well samples from these zones have numerous pebbles ranging up to 1½ inches in diameter. Some larger pebbles and even large boulders are reported from the lower depths. Median diameters of the water-yielding deposits below the surficial silt and clay range from .008 inch (.22 mm; fine sand) to .08 inch (2.2 mm; granule gravel) in sieved well samples. It is likely that the larger size does not represent the median diameter of the coarsest deposits in the American Bottoms.

Although logs and samples of most wells south of the Missouri River mouth show a general coarsening with depth and give little evidence of a break within the valley fill, it seems reasonable to refer some of the deeper and coarser sand and gravel to glacial origin and the upper material to Recent alluviation. The evidence for this interpretation is: 1) the presence of glacial valley-train material beneath the Wood River terrace and at lower depths at Hartford, as indicated by distinctive composition and carbon 14 dating; 2) studies of present Mississippi River erosion and sedimentation, which show scour up to 80 feet along the present channel but general transportation of mainly fine material; and 3) the presence of extensive deposits containing pebble gravel and boulders, indicative of high velocities and large volumes of water, 100 feet and more beneath the present flood plain.

The coarse deeper deposits are shown by the sample study of a well between Dupo and East Carondelet, in the southern part of the area. In this well the driller reported a thickness of 20 feet of sand, gravel, and boulders below a depth of 75 feet and, below this material, 17½ feet of sand, gravel, and broken rock.

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 feet	*Depth feet*
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

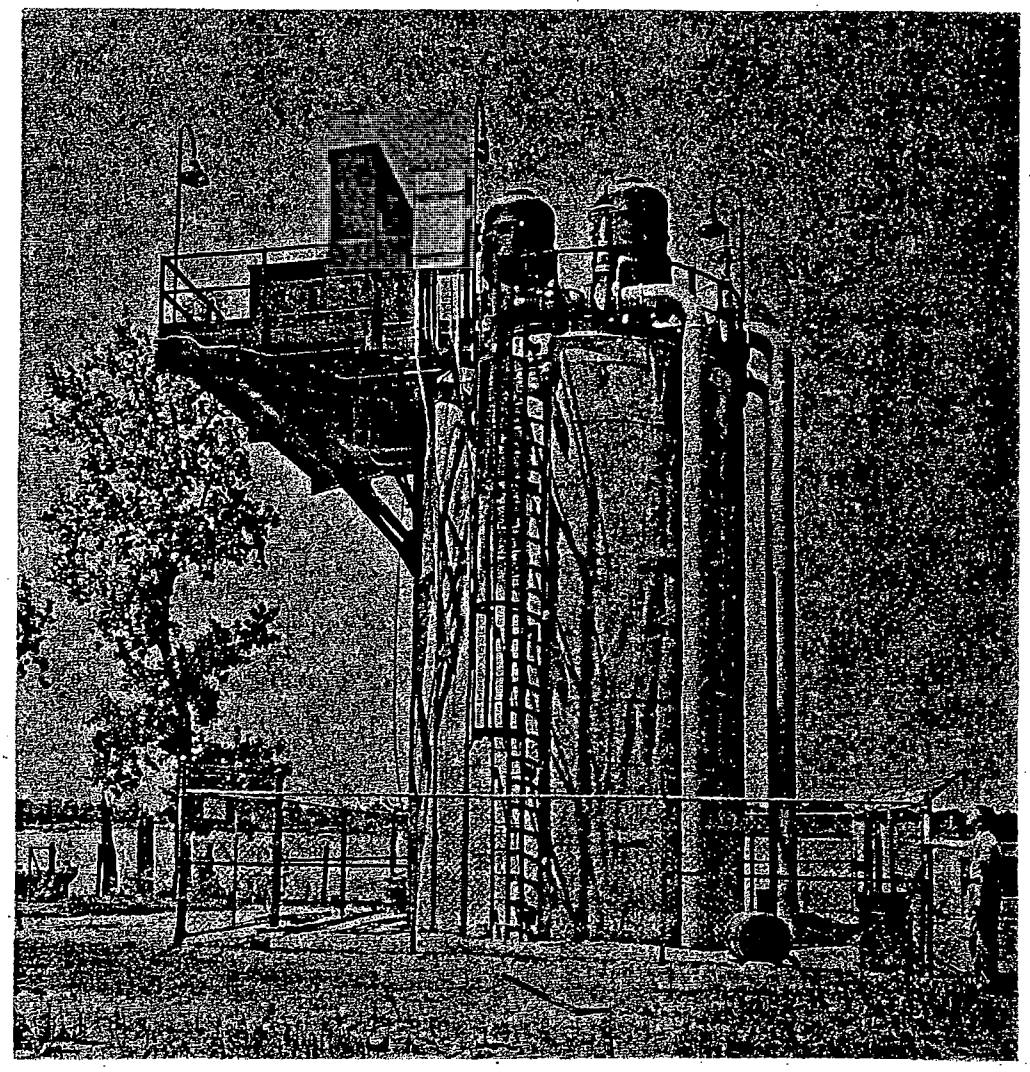


FIG. 5.—Shell Oil Co. high-capacity well at Hartford, Ill. Mississippi River in the background.

	Thickness feet	*Depth feet*
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, graywacke, 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 graywacke 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 . . .	7½	112½

The lack of a diagnostic composition in the valley-train material in the southern portion of the American Bottoms may be a result of mixing sediments from the Upper Mississippi Valley with those brought in from the Missouri River drainage basin.

OTHER ALLUVIAL DEPOSITS

Samples of Recent alluvial deposits, obtained from wells at shallow depths close to the present river channel, differ from the valley-train deposits in the Hartford–Wood River area. The sands average 65 to 75 percent quartz, 10 to 13 percent potash feldspar, 12 to 15 percent plagioclase feldspar, and 4 to 7 percent other materials. The quartz grains are dominantly clear, untinted and unstained, and subangular to subrounded. The sand samples commonly look gray, in contrast to the valley-train sands, which look pink.

The grains classified above as "other materials" are chert, limestone, jasper, shale, coal, graywacke, and heavy minerals. The alluvial deposits, like the valley-train deposits, are only slightly calcareous, averaging 3 to 4 percent soluble material by weight.

A further characteristic of the alluvium at Hartford and the upper portion of the valley fill in the area in general is the presence of abundant flakes of mica of the phlogopite and biotite varieties, scattered fragments of pearly mollusk shells, tiny rod-like calcium carbonate spicules, and abundant coal fragments.

The Recent alluvium ranges in texture from clay to granule gravel. The upper 15 to 30 feet is commonly silt and clay with some fine sand. Below this depth the deposits are highly variable, consisting of clean to dirty sand and gravel. These deposits are underlain in most of the area by coarser sands and gravels. Carbon 14 dating of wood obtained from this lower material indicates that in part at least it is older than Recent. Its exact origin is uncertain. It may be older alluvial, valley-train, or reworked valley-train material. The vertical variations in texture contrast with deposits of the Roxana–Wood River terrace.

The sample study from a well at Granite City is typical of many wells on the American Bottoms. It illustrates the occurrence of the upper silt and clay zone, interbedded sand and gravel deposits below the upper fine-grained beds, the coarser material in the lower part, and the lack of a conspicuous break in lithology.

Union Starch and Refining Company (1952)—950 feet S of 38° 42′ 30″ N, 2350 feet E of 90° 10′ W, T. 3 N, R. 10 W., Madison Co. Illinois Geological Survey sample set 23406. Studied by R. E. Bergstrom. Est. elev. 422 feet.

	Thickness feet	*Depth feet*
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, and 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
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

In figure 6, four mechanical analyses plotted as cumulative frequency curves illustrate the consistency of the valley-train deposits of the Roxana–Wood River terrace compared with deposits of other parts of the American Bottoms. The good sorting of the terrace deposits is indicated in the upper two curves by their steepness. The consistency of the textures with depth is shown by the close spacing of the curves representing different depths. The lower curves, of sam-

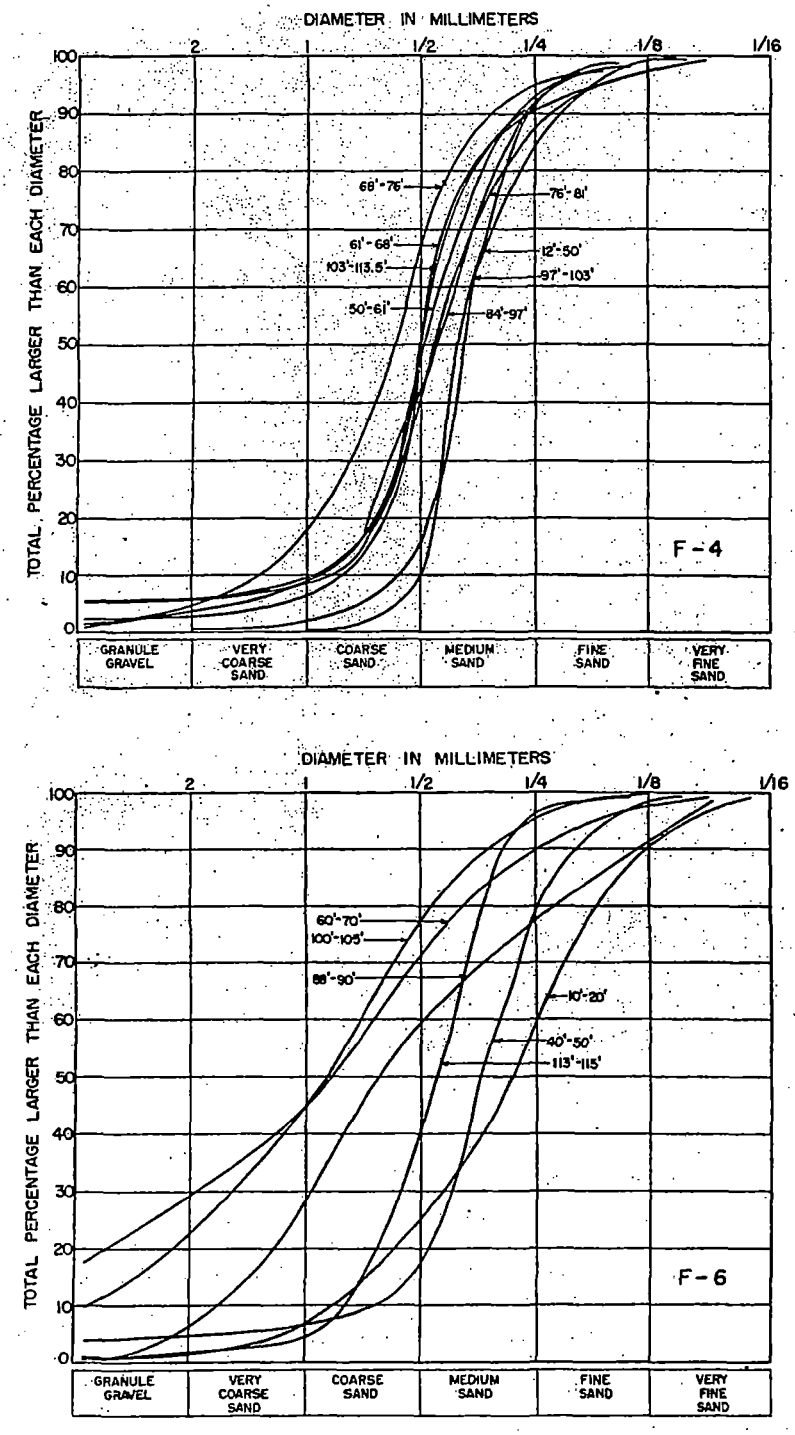
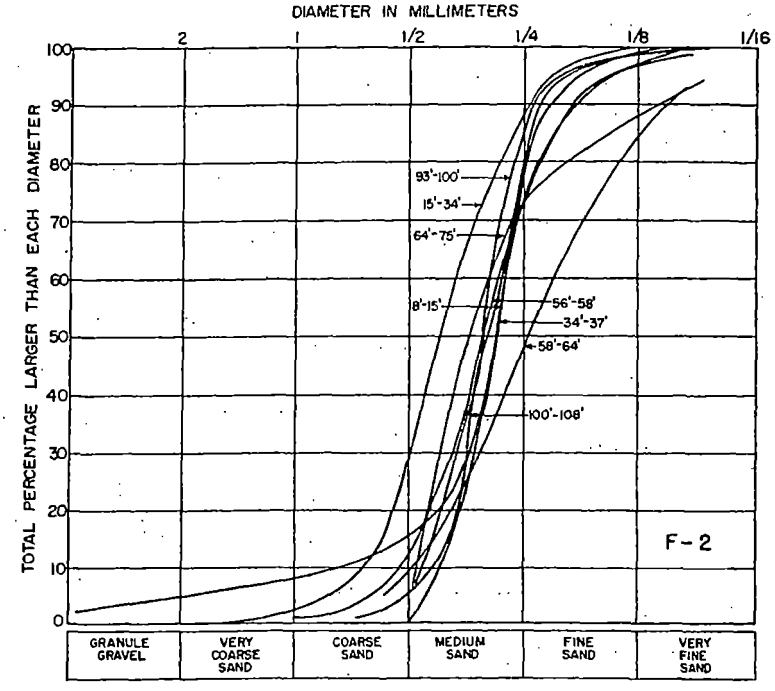


FIG. 6.—Cumulative frequency curves showing mechanical composition of well samples. Wells F-4 and F-2 (top, above and right) are located on terrace at Roxana and Wood River. Wells F-6 and F-9 are on flood plain at Granite City and Monsanto, respectively. Figures beside curves are depths of sample in well. Note good sorting (shown by steepness of curve) and similarity of textures at different depths in well (shown by close grouping of curves) of wells F-4 and F-2 on terrace.


DIAMETER IN MILLIMETERS
2
1
1/2
1/4
1/8
1/16
TOTAL PERCENTAGE LARGER THAN EACH DIAMETER
100
90
80
70
60
50
40
30
20
10
0
93'-100'
15'-34'
64'-75'
8'-15'
56'-58'
34'-37'
58'-64'
100'-108'
F-2
GRANULE GRAVEL
VERY COARSE SAND
COARSE SAND
MEDIUM SAND
FINE SAND
VERY FINE SAND

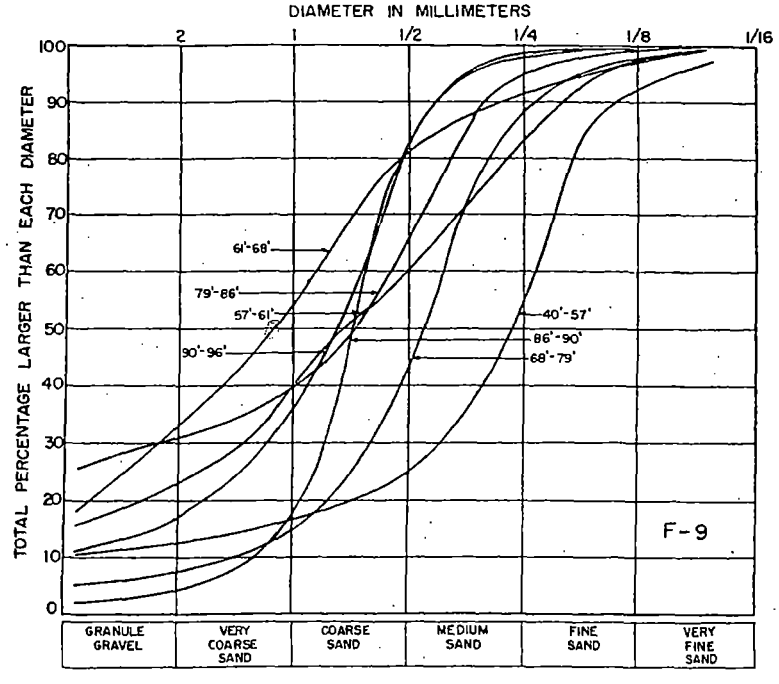

DIAMETER IN MILLIMETERS
2
1
1/2
1/4
1/8
1/16
TOTAL PERCENTAGE LARGER THAN EACH DIAMETER
100
90
80
70
60
50
40
30
20
10
0
61'-68'
79'-86'
57'-61'
90'-96'
40'-57'
86'-90'
68'-79'
F-9
GRANULE GRAVEL
VERY COARSE SAND
COARSE SAND
MEDIUM SAND
FINE SAND
VERY FINE SAND

FIG. 6.—(cont.)

ples from wells at Monsanto and Granite City, indicate poorer sorting, greater variation in texture with depth, and occurrence of fairly coarse deposits in the lower part of the valley fill.

The results of mechanical analyses of well samples (appendix 2) must be accepted with caution. The valley-fill material is highly variable throughout, so a small sample is at best characteristic only of the sediment in its immediate vicinity. In addition, these are not undisturbed samples. Some have been collected from wells drilled with cable tool rigs, some from wells drilled with rigs of the reverse rotary type, and others from wells dug with a clam-shell type digger. Most of the samples were collected by the driller or an assistant, so the conditions of collecting are not known. The evidence that these analyses present, therefore, is only suggestive.

DISTRIBUTION OF VALLEY-TRAIN AND OTHER ALLUVIAL DEPOSITS

Alluvium of Recent age probably comprises the major portion of the valley fill, although its thickness varies considerably. Beneath the terrace it is absent and valley-train material is at the surface, whereas in some areas of shallower bedrock, as in the vicinity of Chain-of-Rocks, Recent alluvium extends to bedrock.

In general, the thickness of Recent alluvium is a measure of the scouring effect of the river since the latest Pleistocene glaciation. Deep scouring occurs in the spring when there are floods and in the winter when thick ice jams cause the river to deepen its channel in order to pass beneath the ice. Soundings taken through the river ice prior to the construction of Eads Bridge indicate that at least 80 feet of channel deepening (scour) takes place (Woodward, 1881, p. 5). The effect of this scour (in combination with channel migration) has been to produce an upper blanket of Recent alluvium resting on older deposits, some of them glacial valley-train. The Recent alluvium coarsens with depth as a result of successive periods of scour and deposition, the largest particles settling out first. Coarsening is also general in the older material, below the Recent alluvium. The uppermost portion of the alluvium contains only fine-grained material; its thickness is further increased at the surface by deposition of silt and clay from floodwaters that cover the area after the channel has migrated to a new position. The cross sections (fig. 4) and cumulative frequency curves (fig. 6) illustrate the increase in grain size from the surface down.

The deposits of the Roxana–Wood River terrace and those in the area just south of Alton are exceptions to the general textural pattern of the fill. Several wells just south of Alton (wells A-3 and A-4, fig. 4) penetrate sections of "clay," "clay and silt," and "clay and gravel" at the bottom of the valley fill. The maximum thickness of the material is 25 feet. These deposits may be Illinoian or older. No samples of the lower material could be obtained for study, so the origin of the material is uncertain.

WATER-YIELDING CHARACTERISTICS

The valley-train material underlying the terrace at Roxana and Wood River is well-sorted medium-to-coarse sand throughout most of its thickness, whereas the complex alluvial deposits in other parts of the American Bottoms generally show poor sorting in the upper part and an increase in coarseness with depth. Permeabilities in these deposits are therefore greatest in the deeper parts, especially where clean coarse sand and gravel occur. The sand and gravel, 20 to 50 feet thick at many places, appear to be the most permeable of any deposits in the area, surpassing the finer material of the terrace. From the standpoint of actual well yield, however, the terrace deposits may be as favorable an aquifer as the coarser sand and gravel—despite lesser permeability—because they are considerably thicker, averaging more than 80 feet.

Evaluation of pumping tests in progress in the American Bottoms by the State Water Survey will yield quantitative data on permeabilities and transmissibilities of the deep coarse sand and gravel and the Roxana–Wood River terrace deposits.

The valley fill in some areas, however, such as north of Horseshoe Lake, is com-

posed of fine-to-medium sand and silt throughout most of its thickness and has poor groundwater possibilities. Thus the valley-fill deposits, except for those on the terrace, are characterized not only by excellent groundwater supply potentialities but by inconsistency. The terrace material, on the other hand, probably is somewhat less permeable but is a thicker and more consistent aquifer, although somewhat restricted in lateral extent.

Some drillers in the area drill to bit refusal and then set screen in the lower 10 to 40 feet of the section. However, good water-yielding beds, in Recent alluvium as well as in glacial outwash, are not everywhere restricted to the lower part of the section. In many instances shallower deposits, which might increase the yield of the completed well, are cased off. In the drilling of new wells it is recommended that, where maximum yield and specific capacity are desired, setting screen opposite the shallow permeable deposits as well as opposite the deep permeable deposits be considered.

Groundwater Recharge

The principal means of recharge of groundwater in the valley fill are seepage from rainfall and floods, and percolation from the Mississippi River and its tributaries. Rainfall is probably the most important source for the area as a whole, although where heavy pumpage is concentrated near the river the recharge from the river itself is undoubtedly great. The effectiveness of recharge from both rainfall and floodwaters is significantly influenced by the nature of the material in the upper portion of the valley fill, which throughout most of the area is 10 to 30 feet of silt and clay. This fine-grained material is usually not so impermeable as to prevent appreciable recharge. There is very little runoff because of the low relief; hence most of the rainfall either evaporates or seeps into the soil. Recharge from floodwaters is undoubtedly much less at present than it has been in the past because of the extensive flood-control program, which is continually being expanded. Where floods do occur they probably result in appreciable recharge.

The recharge from tributary streams that cross the valley flat is probably seasonal for the most part. As the gradient of the streams is very low, the normally slow-moving water can carry only the finest material. The bottoms of the channels probably are covered with a relatively thick deposit of mud, which permits only very slow movement of water into the material below. After periods of prolonged rains in the upland watershed areas, the streams rise, their velocities are greatly increased, and they probably scour their channels sufficiently to remove the impermeable mud, which temporarily permits more rapid recharge. Under natural conditions the streams would be subject to considerable periodic flooding, but man-made changes have prevented most of the floods. Courses have been straightened, channels deepened and widened, and levees constructed. As a result, the tributary streams are not now as large a source of recharge as they once were.

The Mississippi River is an important source of recharge where heavy pumpage has lowered the water table below the level of the river (Bruin and Smith, 1953). Lowering the water table causes the development of hydraulic gradient away from the river and toward the area of pumpage. During high-water stages the hydraulic gradient is increased, which in turn increases the effectiveness of recharge.

Although many areas of the river channel are normally floored with silt, which limits water infiltration, permeable sandy areas are probably present in the channel. Observations on the Mississippi indicate that even in comparatively straight reaches, the thread of the stream moves from one side of the channel to the other, producing shoals and deeps and accompanying differences in bottom deposits. Therefore even under ordinary conditions some groundwater recharge from the river is likely. During high-water stages, when the river scours its channel, recharge conditions are improved.

The only area of notably unfavorable conditions for recharge is west of Granite City where the bedrock lies at a shallow depth and the coarse deposits generally

found in the lower part of the fill are either very thin or missing (fig. 4, B-B′).

Local Groundwater Conditions in the American Bottoms

The occurrence of thick clean deposits of deep sand and gravel over wide areas in the American Bottoms has been partly responsible for the heavy industrial development of the area. Over 100 million gallons of groundwater a day is consumed by industries. Monsanto, Granite City, and Wood River–Roxana–Hartford are the major pumpage centers (Bruin and Smith, 1953). Major cones of depression have been produced by heavy pumpage in these areas.

Despite the present heavy industrial groundwater consumption, it is likely that much more groundwater could be available if industrial expansion takes place in favorable but unexploited areas, particularly near the river where recharge might be induced.

Although the variability of the valley fill and deficiency of well data in many parts of the American Bottoms make it impractical to show groundwater supply potentialities on a map, a summary of groundwater conditions in the various parts of the American Bottoms follows.

Alton–Wood River–Hartford–Roxana area.—Graphic sections showing the lithology of valley-fill material in the area are given in figure 4. They show that the bedrock surface is quite irregular. The eastern part of the section, beginning with well A-9, shows the nature of the terrace material. It is dominantly medium-to-coarse sand, with little gravel, and fairly uniform from top to bottom. Eastward the terrace surface becomes lower and the deposits are finer and contain more silt.

Clean deposits of sand and gravel are found at depths below 50 feet from Alton southeast to Hartford. Many wells in this belt have encountered clay as much as 25 feet thick overlying the bedrock, but above this material coarse sand and gravel are found. The river-front area from Alton to Hartford is geologically favorable for further groundwater development.

Area along Cahokia diversion channel and Chain-of-Rocks Canal.—The valley-fill material in this area has been investigated in connection with U. S. Army Corps of Engineers channel and levee projects (unpublished data, U. S. Army Corps of Engineers, St. Louis district). Borings penetrated thick deposits of clean sand and gravel, except near the southern end of Chain-of-Rocks Canal, west of Granite City, where the bedrock is shallow and coarse deposits are thin (fig. 3).

Area north and east of Horseshoe Lake along bluffs.—The area just west of the bluffs from the vicinity of Poag south to the Madison County line is the site of the Edwardsville, Troy, and Collinsville wells. The bedrock rises sharply at the eastern margin of the flood plain, but from one-half to three-fourths of a mile west of Highway 157, which follows the base of the bluffs, the bedrock floor is reached at a depth of 100 feet or more. Deposits of clean sand and gravel 20 to 40 feet thick have been penetrated. The coarseness of these deposits decreases toward Horseshoe Lake. Some of the coarsest sand and gravel studied came from the valley fill near the bluffs.

Because of the thick, deep sections of clean sand and gravel, this area is considered geologically favorable for greater groundwater development.

Granite City–Madison area.—The lithology of the valley fill in the Granite City area is shown in figure 4, B-B′. The bedrock surface slopes eastward. Bedrock is exposed in the river channel west of Cabaret Island during low-water stages, but between Granite City and Horseshoe Lake it is about 115 feet below the surface of the flood plain. Deposits of clean sand and gravel 20 to 35 feet thick are encountered at the base of the fill at Granite City and Madison. These deposits become finer toward the east, and within half a mile of Horseshoe Lake they pass into dominantly sand and silt deposits unfavorable for industrial groundwater supplies.

Central belt.—A north-south belt 3 to 4 miles wide, extending from a point opposite the mouth of the Missouri River south to

the Madison County line, does not appear to be favorable for the development of large supplies of groundwater. The valley fill in this belt is fine-grained material, apparently of low permeability. The nature of this material is illustrated by well B-9 in figure 4.

East St. Louis.—The deepest part of the bedrock channel appears to pass under East St. Louis, not far east of the eastern pier of Eads Bridge, where the bedrock surface is 284 feet above sea level. Wells in East St. Louis and east of the city were completed in clean sand and gravel of high permeability 20 feet or more thick. To the north, well logs at the National City stock yards record mainly medium-to-coarse sand, with little gravel.

Monsanto–Cahokia–Prairie du Pont–Dupo area.—The southern part of the area, south of East St. Louis, is highly favorable for industrial supplies of groundwater. Monsanto and Cahokia are already heavily developed, but the area to the south, with the same possibilities, has not been exploited. Coarse, permeable sand and gravel deposits are present throughout the area, as indicated by industrial wells and Corps of Engineers levee borings. C-C′ and D-D′ of figure 4 illustrate the lithology of the valley-fill materials and the nature of the bedrock surface. The presence of coarse deposits close to the river in this area favors recharge from the river, if water levels on the flood plain are sufficiently lowered by pumpage.

CONCLUSIONS

Certain generalizations on present and future development of groundwater supplies in the American Bottoms can be made from the preceding discussion.

1. Coarse alluvial and valley-train sands and gravels, generally concentrated near the base of the valley fill, have high permeabilities and are the most favorable deposits for yielding industrial supplies of groundwater.

2. The medium-to-coarse sands that underlie the terrace at Wood River and Roxana are excellent deposits for yielding industrial supplies of groundwater, although they are somewhat restricted in lateral extent and may have slightly lower permeabilities than the coarser deposits in other parts of the American Bottoms.

3. Because the terrace deposits are consistently finer in texture than are the deeper sand and gravel deposits elsewhere in the area, wells situated on the terrace in the Roxana–Wood River area would require finer gravel packs and screens for maximum efficiency than wells constructed in the lower coarse sand and gravel at East St. Louis, Granite City, Monsanto, and Cahokia. Median diameters of the terrace material range from .01 to .03 inches; median diameters of the coarse sand and gravel, .02 to .08 inches.

4. Because of the variable nature of the alluvium over much of the American Bottoms, highly permeable zones are present in some places at depths as shallow as 60 to 70 feet. The practice of setting screens only in the lower portion of wells may result in failure to take full advantage of the water-yielding capabilities of these shallower permeable zones. Therefore, where maximum yield and highest specific capacities are desired, consideration should be given to setting screens through all zones of high permeability that are of sufficient depth that the screens will not be exposed to air as a result of drawdown from heavy pumpage.

5. Greater appreciation of the variability of the valley fill during design and construction would lengthen the life and improve the efficiency of wells. Wells in the American Bottoms have been found to have a much shorter life expectancy than those in the State as a whole. The principal causes of well failures in the area are screen-clogging and the filling of wells with sand. Screen-clogging is partly chemical and partly mechanical (Bruin and Smith, 1953). Sand-clogging will be reduced if careful consideration is given to the texture ranges throughout the screened intervals. The texture of the alluvium may vary greatly within a few feet vertically, making it impossible to select a screen with one slot size optimum for the entire screened interval. Therefore, consideration should be given to the use of

composite screens made up of sections of different slot sizes. The life of many wells also would be increased by the use of carefully constructed gravel-packed wells. Clogging of screens and filling of wells with sand will be at a minimum if the gravel pack surrounding the screen has the proper textural relationship to the material in the adjacent alluvium. A uniform-grain-size gravel with a median grain size between 5 and 10 times the median grain size of the water-yielding formation has been found to give excellent results (Smith, 1954, p. 15).

6. The valley fill appears to be unfavorable for yielding industrial supplies of groundwater in portions of the American Bottoms where glacial alluviation and Recent river cut-and-fill have produced silt and fine sand extending almost to bedrock. Such conditions are believed to be present in a wide belt extending from opposite the mouth of the Missouri River to the area south of Horseshoe Lake.

7. Owing to the shallow permeable deposits along the present Mississippi channel, conditions are probably favorable for recharge from the river in most areas along the river where the water table is sufficiently lowered by pumpage. Induced recharge from the river becomes especially important in the face of increased demand for groundwater because flood-control measures have restricted the normal spreading of floodwater over the American Bottoms, formerly an important factor in recharge.

8. Increased groundwater development appears possible in three areas where present withdrawals are small compared to the potentialities believed to exist. These areas are: 1) between the eastern bluffs and Horseshoe Lake from the Madison–St. Clair county line north to Roxana; 2) along the Mississippi River near the mouth of Wood River; and 3) the East Carondelet–Dupo area in the south, extending to an area east of Cahokia.

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APPENDIX 1

PARTIAL LIST OF WELLS IN THE EAST ST. LOUIS AREA

A-1 Corps of Engineers, boring, 1948. 400 feet E of center W line, sec. 13, T. 5 N., R. 10 W., Madison Co. Elev. 434 feet. Total depth 92 feet, bit refusal. Engineer's field log.

A-2 Owens Illinois Glass Co. well 9. Thorpe Concrete Well Co., 1950. Center of NE ¼ SW¼ sec. 13, T. 5 N., R. 10 W., Madison Co. Elev. 422 feet. Total depth 88 feet. Finished in sand. Driller's log.

A-3 Alton Boxboard Co. test hole H. Layne-Western Co., 1944. 2400 feet E, 1300 feet N, SW corner sec. 18, T. 5 N., R. 9 W., Madison Co. Elev. 436 feet. Total depth 131 feet, on rock. Driller's log.

A-4 Alton Boxboard Co. test hole J. Layne-Western Co., 1944. 200 feet N, 200 feet W, SE corner sec. 18, T. 5 N., R. 9 W., Madison Co. Elev. 428 feet. Total depth 104 feet, on rock. Driller's log.

A-5 Illinois Power Co., Wood River Power Station, test boring 4, 1947. 1500 feet N, 1900 feet E, SW corner sec. 20, T. 5 N., R. 9 W., Madison Co. Elev. 425 feet. Total depth 123 feet, bit refusal. Driller's log.

A-6 Shell Oil Co., loading dock, well W-1, Ranney Well Co., 1952. 2600 feet N, 2700 feet W, SE corner sec. 33, T. 5 N., R. 9 W., Madison Co. Elev. 425 feet. Total depth 118 feet, on bedrock. Driller's log.

A-7 International Shoe Co., Layne-Western Co., 1951. 2200 feet N, 800 feet E, SW corner sec. 34, T. 5 N., R. 9 W., Madison Co. Elev. 429 feet. Total depth 117 feet, finished in clay. Driller's log.

A-8 Shell Oil Co. well 15, Thorpe Concrete Well Co., 1927. 2110 feet from W line, 278 feet from N line SW ¼ sec. 35, T. 5 N., R. 9 W., Madison Co. Elev. 454 feet. Total depth 112 feet 11 inches, finished in coarse sand and gravel. Driller's log.

A-9 Shell Oil Co. well 54, Thorpe Concrete Well Co., 1949. 1900 feet S, 1000 feet W, NE corner sec. 35, T. 5 N., R. 9 W., Madison Co. Elev. 446 feet. Total depth 131 feet, finished in gravel. Driller's log.

A-10 Shell Oil Co., Wood River, test hole 6, Layne-Western Co., 1942. 1100 feet S, 2300 feet E, NW corner sec. 35, T. 5 N., R. 9 W., Madison Co. Elev. 452 feet. Total depth 171 feet, finished in sand. Driller's log.

A-11 Shell Oil Co. test hole 10, Thorpe Concrete Well Co., 1946. 2200 feet S, 1250 feet E, NW corner sec. 36, T. 5 N., R. 9 W., Madison Co. Elev. 435 feet. Total depth 102 feet, finished in sand. Driller's log.

A-12 Shell Oil Co., Recreation Center test well, Roxana, Ill., Harold L. Watson Drilling Co., 1950. 2900 feet N, 1750 feet W, SE corner sec. 36, T. 5 N., R. 9 W., Madison Co. Elev. 270 feet. Total depth 71 feet. Driller's log.

B-2 City of St. Louis River Front Project D. H. 102, 1951. 5350 feet S of 80° 42′ 30″ N, 100 feet E of 90° 12′ 30″ W., St. Louis Co. Elev. 414 feet. Total depth 22.7 feet, bit refusal. Engineer's field log.

B-3 Corps of Engineers, Chain-of-Rocks lock site, boring H-1, 1941. 2600 feet from N line, 240 feet from W line, sec. 23, T. 3 N., R. 10 W., Madison Co. Elev. 412.4. Total depth 73.7 feet, finished in gray limestone. Engineer's field log.

B-4 Hoyt Metal Co., Granite City, Thorpe Concrete Well Co., 1936. 4200 feet S of 38° 42′ 30″ N, 2600 feet E of 90° 10′ W, T. 3 N., R. 10 W., Madison Co. Elev. 421 feet. Total depth 111 feet 6 inches, finished in boulders and sand. Driller's log.

B-5 Granite City Steel Co. well 21, Harold L. Watson Drilling Co., 1946. 4700 feet S of 38° 42′ 30″ N, 5400 feet W of 90° 07′ 30″ W, T. 3 N., R. 9 W., Madison Co. Elev. 421 feet. Total depth 116 feet, finished in sand. Driller's log.

B-6 St. Louis Gas and Coke Co. well. SW ¼ NW ¼ sec. 20, T. 3 N., R. 9 W., Madison Co. Elev. 417 feet. Total depth 114 feet, finished in sand and gravel. Driller's log.

B-7 Koppers Co. test hole 3, Layne-Western Co., 1948. 1900 feet S, 1400 feet E of NW corner sec. 20, T. 3 N., R. 9 W., Madison Co. Elev. 416 feet. Total depth 104 feet, on rock. Driller's log.

B-8 Koppers Co. test hole 4, Layne-Western Co., 1948. 1800 feet S, 2900 feet W, NE corner sec. 20, T. 3 N., R. 9 W., Madison Co. Elev. 417 feet. Total depth 103 feet, finished in sand and boulders. Driller's log.

B-9 Illinois Geol. Survey test hole 1, Charles M. Hayes, 1954. 125 feet E, 250 feet N, SW corner NW ¼ sec. 28, T. 3 N., R. 9 W., Madison Co. Elev. 413. Total depth 111 feet, finished in bedrock. Samples studied by R. E. Bergstrom. Sieve analysis.

B-10 Neidringhous-Sullivan well 2, 1932. 1600 feet from S line, 1825 feet from E line, sec. 22, T. 3 N., R. 9 W., Madison Co. Elev. 411. Total depth 1105, finished in Hannibal shale. Driller's log.

B-11 Village of Troy test hold 3, Layne-Western Co., 1953. Approx. 100 feet N, 3310 feet W of SE corner sec. 20, T. 3 N., R. 8 W., Madison Co. Elev. 430 feet. Total depth 115 feet, finished in shale. Driller's log.

B-12 Village of Troy test hole 4, Layne-Western Co., 1953. Approx. 100 feet N, 2910 feet W of SE corner sec. 20, T. 3 N., R. 8 W., Madison Co. Elev. 432 feet. Total depth 88 feet, finished in shale. Driller's log.

B-13 Village of Troy test hole 1, Layne-Western Co., 1953. Approx. 100 feet N, 1860 feet W of SE corner sec. 20, T. 3 N., R. 8 W., Madison Co. Elev. 437 feet. Total depth 48 feet, finished in shale. Driller's log.

C–1 City of St. Louis River Front Project D. H. 116, 1951. 5700 feet S of 38° 37′ 30″ N, 5300 feet E of 90° 12′ 30″ W, St. Louis Co. Elev. 412 feet. Total depth 53.5 feet, finished in sand and gravel. Engineer's field log.

C–2 Corps of Engineers test hole W–77, 1952–53. 8400 feet N of 38° 35′ N, 3600 feet W of 90° 10′ W, T. 2 N., R. 10 W., St. Clair Co. Elev. 415 feet. Total depth 127 feet, bit refusal. Engineer's field log.

C–3 American Zinc Co., Monsanto, well 6, Harold L. Watson Drilling Co., 1940. 6900 feet N of 38° 35′ N, 750 feet W of 90° 10′ W, T. 2 N., R. 10 W., St. Clair Co. Elev. 405 feet. Total depth 107 feet, finished in soapstone. Driller's log.

C–4 Monsanto Chemical Co. test hole 4, Layne-Western Co., 1948. 5100 feet N of 38° 35′ N, 250 feet W of 90° 10′ W, T. 2 N., R. 10 W., St. Clair Co. Elev. 411 feet. Total depth 110 feet, finished on rock. Driller's log.

C–5 Socony-Vacuum Oil Co. well, Layne-Western Co., 1952. 2400 feet E of 90° 10′ W, 4400 feet N of 38° 35′ N, T. 2 N., R. 10 W., St. Clair Co. Elev. 410 feet. Total depth 117½ feet, finished in gravel and sand. Sample set 22655, studied by P. M. Busch.

C–6 Key Co. well, East St. Louis, Harold L. Watson Drilling Co., 1943. 6200 feet N of 38° 35′ N, 4700 feet W of 90° 07′ 30″ W. Total depth 117 feet, finished in sand and gravel. Driller's log.

C–7 Aluminum Ore Co. well, East St. Louis, Harold L. Watson Drilling Co., 1940. 4100 feet N of 38° 35′ N, 90° 07′ 30″ W, T. 2 N., R. 9 W., St. Clair Co. Elev. 417 feet. Total depth 121 feet, finished in fine sand and mud. Driller's log.

C–8 Illinois State Water Survey well 1, Layne-Western Co., 1951. 1800 feet S, 800 feet E of NW corner sec. 26, T. 2 N., R. 9 W., St. Clair Co. Elev. 422 feet. Total depth 81 feet, finished in sand. Sample set 21485, studied by W. H. Bierschenk.

C–9 Drive-in Theater well, French Village, Harold L. Watson Drilling Co., 1941. 450 feet W of SE corner sec. 23, T. 2 N., R. 9 W., St. Clair Co. Elev. 433 feet. Total depth 82½ feet, finished at shale. Driller's log.

D–1 Anheuser-Busch Co. test hole 1, Ranney Well Co. 2600 feet N of 38° 35′ N, 800 feet E of 90° 12′ 30″ W, St. Louis Co. Elev. 417 feet. Total depth 73 feet, finished on rock. Driller's log.

D–2 Alton and Southern Railroad well 2, Fox Terminal, Harold L. Watson Drilling Co., 1950. 100 feet S of 38° 35′ N, 1100 feet E of 90° 12′ 30″ W, T. 2 N., R. 10 W., St. Clair Co. Elev. 410 feet. Total depth 104 feet, finished in sand. Driller's log.

D–3 Corps of Engineers test hole W–95, 1952–53. 3400 feet S of 38° 35′ N, 1900 feet E of 90° 12′ 30″ W, T. 2 N., R. 10 W., St. Clair Co. Elev. 396 feet. Total depth 82 feet, finished in gravelly sand. Engineer's field log.

D–4 Corps of Engineers seepage well 2, Cahokia, 1952–53. 7250 feet N of 38° 32′ 30″ N, 90° 12′ 30″ W, T. 1 N., R. 10 W., St. Clair Co. Elev. 406 feet. Total depth 108 feet, finished on bedrock. Engineer's field log.

D–5 Corps of Engineers well W24B, Prairie du Pont, 1952–53. 5000 feet N of 38° 32′ 30″ N, 600 feet E of 90° 12′ 30″ W, T. 1 N., R. 10 W., St. Clair Co. Elev. 413 feet. Total depth 117 feet, bit refusal. Engineer's field log.

D–6 Corps of Engineers test hole DH-6-S, 1952. 3600 feet S, 1200 feet W of NE corner sec. 10, T. 1 N., R. 10 W., St. Clair Co. Elev. 416 feet. Total depth 84½ feet, bit refusal. Engineer's field log.

D–7 Corps of Engineers test hole DH, 1950–54. 2300 feet N, of 38° 32′ 30″ N, 1650 feet E of 90° 10′ W, T. 1 N., R. 10 W., St. Clair Co. Elev. 408 feet. Total depth 116 feet, bit refusal.

E–1 Tarlton and Sklar–Dyroff well 1–A, 1943. 1070 feet N, 820 feet W of SE corner sec. 28, T. 1 N., R. 10 W., St. Clair Co. Elev. 403 feet. Total depth 1800 feet, finished in Gasconade dolomite. Sample study 9318, studied by D. Speziale.

E–2 Lockwood–Dyroff well 1, 1924. 150 feet S of NW corner NE ¼ sec. 26, T. 1 N., R. 10 W., St. Clair Co. Elev. 590 feet. Total depth 2904 feet, finished in Potosi dolomite. Sample study 423, studied by F. E. Tippie.

E–3 Sewell–Bayless–Sparks well 1, 1931. SW ¼ NE ¼ SW ¼ sec. 2, T. 1 N., R. 10 W., St. Clair Co. Elev. 410.5 feet. Total depth 2002 feet, finished in Jefferson City dolomite. Sample study 1001, studied by Margaret Blair.

E–4 Monk's Mound well. Center NW ¼ NW ¼ NE ¼ sec. 2, T. 2 N., R. 9 W., St. Clair Co. Elev. 437 feet. Samples studied by J. A. Udden.

E–5 Commonwealth Steel Co. well. NW ¼ SW ¼ sec. 24, T. 3 N., R. 10 W., Madison Co. Elev. 423 feet. Total depth 2085 feet, finished in Jefferson City dolomite. Sample study 226, studied by A. Thurston.

E–6 Kesl–Kusmanoff well 1, 1947. 660 feet from N line. 330 feet from W line, SW ¼ SE ¼ sec. 12, T. 3 N., R. 9 W., Madison County. Elev. 410.6 feet. Total depth 1687 feet, finished in Kimmswick limestone. Sample study 17178, studied by M. P. Meyer and Heinz Lowenstam.

E–7 Penn–Illinois–Poag well 1, 1938. 2400 feet from S line, 3630 feet from E line, sec. 12, T. 9 N., R. 9 W., Madison Co. Elev. 424.6 feet. Total depth 2093 feet, finished in St. Peter sandstone. Sample study 8582, studied by T. C. Buschbach.

E–8 Lindberg Park well, 1932. 1830 feet from N line, 2320 feet from W line, sec. 8, T. 5 N., R. 9 W., Madison Co. Elev. 446.9 feet. Total depth 1200 feet, finished in Maquoketa shale. Sample study 935, studied by L. E. Workman.

F–1 Bethalto city well 3, Thorpe Concrete Well Co., 1951. 2200 feet N, 1200 feet W, SE corner sec. 22, T. 5 N., R. 9 W., Madison Co. Elev. 437 ± feet. Total depth 95 feet, finished in coarse sand, gravelly. Driller's log and sieve analysis.

F–2 Wood River city well 1, Thorpe Concrete Well Co., 1930. 860 feet S, 300 feet E, NW corner sec. 26, T. 5 N., R. 9 W., Madison Co. Elev. 446.7 feet. Total depth 109 feet, finished in pink sand. Sample study 1056, studied by L. E. Workman.

F-3 Shell Oil Co. well 59, Thorpe Concrete Well Co., 1952. NE ¼ SE ¼ SW ¼ sec. 35, T. 5 N., R. 9 W., Madison Co. Elev. 442 feet. Total depth 110 feet, finished in fine sand and small gravel. Samples studied by R. E. Bergstrom and T. R. Walker.

F-4 Shell Oil Co. well 61, Thorpe Concrete Well Co., 1952. NW ¼ SW ¼ SE ¼ sec. 35, T. 5 N., R. 9 W., Madison Co. Elev. 442 feet. Total depth 113 feet, finished in sand and gravel. Sieve analysis.

F-5 Sinclair Oil Co. well 1, Harold L. Watson Drilling Co., 1952. 1750 feet E., 460 feet N, SW corner sec. 34, T. 5 N., R. 9 W., Madison Co. Elev. 431 feet. Total depth 126 feet, finished in medium sand and gravel. Sample study 23403, studied by R. E. Bergstrom. Sieve analysis.

F-6 Union Starch and Refining Co. well, Harold L. Watson Drilling Co., 1952. 1000 feet N, 2800 feet E, SW corner sec. 13, T. 3 N., R. 10 W., Madison Co. Elev. 422 feet. Total depth 115 feet, finished in medium sand. Sample study 23406, studied by R. E. Bergstrom. Sieve analysis.

F-7 Collinsville city well 8, Layne-Western Co., 1951. SE ¼ SE ¼ sec. 31, T. 3 N., R. 8 W., Madison Co. Elev. 424 feet. Total depth 98 feet, finished in shale. Samples studied by W. H. Bierschenk. Sieve analysis.

F-8 Hunter Packing Co. well, Harold L. Watson Drilling Co., 1948. SE ¼ NW ¼ sec. 7, T. 2 N., R. 10 W., St. Clair Co. Elev. 418 feet. Total depth 115½ feet, finished in sand and gravel. Samples studied by R. E. Bergstrom. Sieve analysis.

F-9 Monsanto Chemical Co. well Z-12, Ranney Well Co., 1952. SE ¼ SE ¼ SE ¼ sec. 22, T. 2 N., R. 10 W., St. Clair Co. Elev. 400 feet. Total depth 97 feet, stopped on rock. Sample study 23443, studied by J. W. Baxter. Sieve analysis.

F-10 Cargill Co., Fox Terminal Elevator well, Harold L. Watson Drilling Co., 1952. NE ¼ SE ¼ NE ¼ sec. 33, T. 2 N., R. 10 W., St. Clair Co. Elev. 410 feet. Total depth 110 feet, finished in medium sand. Sample study 23404, studied by F. B. Titus. Sieve analysis.

APPENDIX 2

SIEVE ANALYSES OF LOWER PART OF VALLEY FILL, AMERICAN BOTTOMS

Sample	Percent by weight retained on screen										
Well number; depth	Mesh										
	4	9	16	24	32	42	60	80	115	170	Pan (silt and clay)
F-1											
65-70	—	—	—	1.2	3.7	7.	22.5	43.7	16.2	5.	.5
75-80	—	—	1.3	1.3	2.6	2.6	22.4	55.4	10.5	2.6	1.
85-90	—	6.6	22.8	27.6	10.4	1.9	13.3	13.3	2.8	.9	—
90-93	—	4.8	22.5	30.6	22.5	4.8	4.8	8.	1.6	—	—
F-2											
76.6-93.1	—	17.	26.1	19.3	21.5	6.8	4.5	3.4	1.1	—	—
93.1-100.1	—	—	—	.9	4.8	29.8	50.9	10.5	1.9	.9	—
100.1-108.1	—	—	—	—	5.1	23.3	48.	19.4	2.5	1.2	—
F-3											
69-81	—	—	.5	1.	7.9	24.4	44.5	15.3	4.6	1.	.2
81-96	1.7	3.7	16.8	24.1	30.9	15.5	4.7	1.2	.7	.2	—
96-103	1.5	2.7	4.5	8.8	25.9	25.6	19.8	8.5	1.7	.5	—
103-110	1.2	1.2	1.9	2.6	24.3	42.4	15.1	7.5	2.4	.7	.2
F-4											
76-84	1.3	1.8	5.9	6.6	20.9	33.8	15.2	8.2	3.6	1.6	.4
84-97	25.5	14.	13.8	7.9	12.5	11.8	10.8	2.3	.8	.1	—
97-103	—	.9	2.4	2.4	9.8	49.6	26.5	6.3	.9	.4	.2
103-113	4.7	1.4	2.3	3.8	38.6	30.5	10.9	5.2	1.4	.4	.2
F-5											
70-75	2.9	9.7	20.	16.2	25.4	17.3	7.8	1.2	.2	—	—
80-85	7.9	11.	10.1	8.6	25.4	22.4	7.9	6.2	.2	—	—
90-95	—	.9	2.2	2.9	11.7	41.	31.2	7.6	1.7	.2	.2
100-105	11.7	11.2	14.4	10.8	24.2	13.9	9.1	3.1	.2	—	—
F-6											
60-70	13.4	16.5	14.6	12.8	14.7	10.5	8.2	3.7	3.5	1.5	.4
80-88	.5	1.6	4.9	3.8	6.9	8.3	15.9	23.9	16.8	12.1	4.9
95-100	10.7	20.6	30.5	11.3	11.1	6.1	4.4	2.8	1.6	.2	.4
105-110	12.4	6.8	9.0	5.6	22.0	27.7	12.4	2.3	.6	.6	.6
B-9											
60-65	8.5	12.8	12.8	9.4	15.6	15.6	12.8	7.1	2.3	.9	.9
70-75	5.9	8.9	13.1	16.5	21.2	18.6	9.7	4.2	.8	.4	.4
80-85	.4	2.2	10.9	14.5	35.9	22.7	8.2	3.2	.9	.4	.4
90-95	—	7.7	18.8	24.3	24.8	14.9	6.	2.2	.5	.2	.2
100-105	—	.9	27.7	27.2	25.3	9.5	5.7	2.4	.4	.2	.4
F-7											
75-80	28.0	24.3	26.8	7.3	6.0	3.6	1.2	.6	.6	.6	.6
85-90	15.9	11.6	31.	23.5	12.6	3.3	.8	.4	.4	—	—
95-100	1.2	2.4	17.	30.4	31.7	12.8	2.4	1.2	.6	—	—
F-8											
75-80	5.8	7.1	8.1	8.1	17.8	22.4	15.8	9.9	2.5	1.5	.5
85-90	11.9	13.5	10.4	17.1	32.8	10.4	2.3	1.	.2	—	—
95-100	3.8	5.3	18.	20.7	39.7	9.5	1.6	.4	—	—	—
F-9											
68-79	5.3	2.3	6.5	8.9	20.1	29.	16.3	5.9	3.2	1.4	.5
79-86	22.5	8.5	8.5	7.9	16.8	19.4	11.4	2.8	.8	.5	.2
86-90	1.1	3.0	13.6	28.2	35.	14.8	2.8	.2	.4	—	.4
90-96	10.2	7.0	17.2	17.2	30.6	13.1	2.9	.4	.4	—	.2
F-10											
70-75	2.1	8.3	11.9	7.6	16.2	23.4	17.4	8.8	2.6	.9	.2
80-85	1.4	1.7	2.4	2.9	11.8	12.5	24.8	28.8	6.1	4.6	2.4
90-95	—	.2	.7	.7	19.	54.3	17.2	4.9	1.9	.4	.2
95-100	—	—	.7	.4	19.8	55.2	19.5	3.4	.4	.2	—

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ATTACHMENT 2

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Techniques of Water-Resources Investigations
of the United States Geological Survey

A MODULAR THREE-DIMENSIONAL FINITE-DIFFERENCE GROUND-WATER FLOW MODEL

BOOK 6

CHAPTER A1

Chapter A1

A MODULAR THREE-DIMENSIONAL FINITE-DIFFERENCE GROUND-WATER FLOW MODEL

By Michael G. McDonald and
Arlen W. Harbaugh

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Book 6

MODELING TECHNIQUES

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A MODULAR THREE-DIMENSIONAL FINITE-DIFFERENCE GROUND-WATER FLOW MODEL

By Michael G. McDonald and Arlen W. Harbaugh

ABSTRACT

This report presents a finite-difference model and its associated modular computer program. The model simulates flow in three dimensions. The report includes detailed explanations of physical and mathematical concepts on which the model is based and an explanation of how those concepts are incorporated in the modular structure of the computer program. The modular structure consists of a Main Program and a series of highly independent subroutines called "modules." The modules are grouped into "packages." Each package deals with a specific feature of the hydrologic system which is to be simulated, such as flow from rivers or flow into drains, or with a specific method of solving linear equations which describe the flow system, such as the Strongly Implicit Procedure or Slice-Successive Overrelaxation.

The division of the program into modules permits the user to examine specific hydrologic features of the model independently. This also facilitates development of additional capabilities because new packages can be added to the program without modifying the existing packages. The input and output systems of the computer program are also designed to permit maximum flexibility.

Ground-water flow within the aquifer is simulated using a block-centered finite-difference approach. Layers can be simulated as confined, unconfined, or a combination of confined and unconfined. Flow associated with external stresses, such as wells, areal recharge, evapotranspiration, drains, and streams, can also be simulated. The finite-difference equations can be solved using either the Strongly Implicit Procedure or Slice-Successive Overrelaxation.

The program is written in FORTRAN 77 and will run without modification on most computers that have a FORTRAN 77 compiler. For each program module, this report includes a narrative description, a flow chart, a list of variables, and a module listing.

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Sauget, Illinois

ATTACHMENT 3

Applied Contaminant Transport Modeling

Theory and Practice

Chunmiao Zheng
Gordon D. Bennett

APPLIED CONTAMINANT TRANSPORT MODELING

Theory and Practice

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8

Building a Contaminant Transport Model

8.1 GETTING STARTED

One of the difficult things about developing a transport model is getting started. There are many preliminary decisions to be made; and there is always a question as to whether enough data have been assembled to begin model development. The preliminary questions and the desire to assemble further data are often reasons for delaying the actual use of simulation. In this section we consider some of the questions that have to be addressed early in any attempt to begin building a contaminant transport model.

8.1.1 Present Understanding of the Flow System

One of the first questions in a transport simulation project relates to the present level of understanding of the flow system. In some cases the hydrogeologic system may be thoroughly defined, and a flow model may already exist; if the scale of that flow model is appropriate, velocity distributions for use as input to a transport simulation can be taken directly from its output. At the other end of the spectrum, very little may be known about the flow system, and a hydrogeologic investigation, culminating in the development of a flow model, may be required as the first stage of transport model development. In most cases, however, the need for solute transport simulation arises after a certain amount of hydrogeologic investigation has been completed; thus some understanding of the flow regime exists, although very often no flow simulation, or at least no flow simulation at the scale required to support transport analysis, has been attempted. In these cases the first task in transport simulation is the development of a reasonable flow

model, at the appropriate scale, on the basis of existing hydrogeologic data and interpretations. It should be stressed that the velocity distribution as determined by the flow model is by far the most important factor in controlling solute transport under most circumstances.

8.1.2 Dimensions of Analysis

Next one must ask whether the problem at hand requires three-dimensional analysis, or can be adequately represented in two-dimensional simulation. This question relates both to the flow system and to the transport problem. Virtually all field problems are three-dimensional to some degree, but in some cases the components of flow or transport in the vertical direction are small enough to be neglected, or the problem displays symmetry through which it can be reduced to two dimensions. One can imagine situations in which three-dimensional flow simulation is needed to account for various hydrologic processes, but two-dimensional transport simulation, based on horizontal velocity components taken from one layer or one cross section of the flow model, may be adequate to characterize solute movement over the time span of interest. On the other hand, there may be situations in which two-dimensional areal flow simulation is adequate because of negligible vertical flow components, but three-dimensional transport simulation is needed to define vertical dispersion.

8.1.3 Domains of Simulation

Another issue which must be addressed for both flow and transport relates to the spatial and temporal domains of the simulation, i.e., the size of the region which must be included in the simulation, and the time span which must be covered. In terms of the region to be included, the answer is generally not the same for the flow and transport regimes. Adequate simulation of hydrogeologic influences often requires that the flow model extend over a large region, either to intersect recognized hydrologic boundaries to facilitate assignment of boundary conditions supported by field data, or to minimize the impact of boundary conditions not well constrained by field data on the local study site. Frequently, however, the transport which has occurred in the past, or that which can be anticipated over a reasonable future time, affects only a small part of the area which must be included in flow simulation. For example, the flow regime controlling transport of contaminants from a waste site may be governed by recharge areas, surface drainage features, or well fields located many miles from the site, whereas the existing or projected contaminant plume may be only a few thousand feet in length. In cases such as this the transport model domain can be restricted to the

projected plume area, while the flow model must be extensive enough to include all of the controlling features. Velocities taken from the section of the flow model corresponding to the plume area are used as input to the transport simulation.

8.2 SPATIAL DISCRETIZATION

As noted above, a flow model must exist prior to assembly of a transport model, or must be developed as the first step in the transport simulation project. Most computer codes for transport modeling, such as FTWORKS (Faust et al., 1990), MT3D (Zheng, 1990, 1992) and SUTRA (Voss, 1984) use the same model grid for both flow and transport simulation. There are a few exceptions, such as the MOC code (Konikow and Bredehoeft, 1978; Goode and Konikow, 1989) which allows a subgrid with finer spacing for transport simulation. The design of a model grid may seem, at first glance, a simple task. However, it is one of the most important steps in model development; without proper grid design, a numerical model cannot achieve reasonable representation of the conceptual model and at the same time meet the practical constraints of simulation time and computer memory.

Anderson and Woessner (1992a) discuss the steps in going from a conceptual model to a numerical model grid. Although their development is phrased largely in terms of flow modeling, most of the discussion is equally applicable to transport modeling; and their text is an excellent source of general information relating to spatial discretization and grid design in numerical modeling. The discussion in this section focuses on special considerations applying to the design of model grids which are intended for use both in flow and transport simulation. As the development will show, the spatial discretization requirements of transport simulation are usually more stringent than those of flow simulation alone, and certain spatial discretization practices that are quite acceptable in flow modeling may lead to problems in transport simulation.

8.2.1 Horizontal Nodal Spacing

Distance between adjacent nodal points, usually referred to as nodal spacing or mesh spacing, determines the resolution of a numerical model. In most modeling projects, the nodal spacing of the numerical model is dictated by the scale of the problem to be modeled and the limitations of available computer resources. For example, if it is important to examine flow and contaminant transport between a landfill and a river which are only 100 feet apart, the nodal spacing in and near this area must clearly be smaller than 100 feet. At the other extreme, simulation of solute movement in a plume

which spans several miles in the longitudinal direction and several thousand feet in the lateral direction may require very large nodal spacing to maintain a model of manageable size.

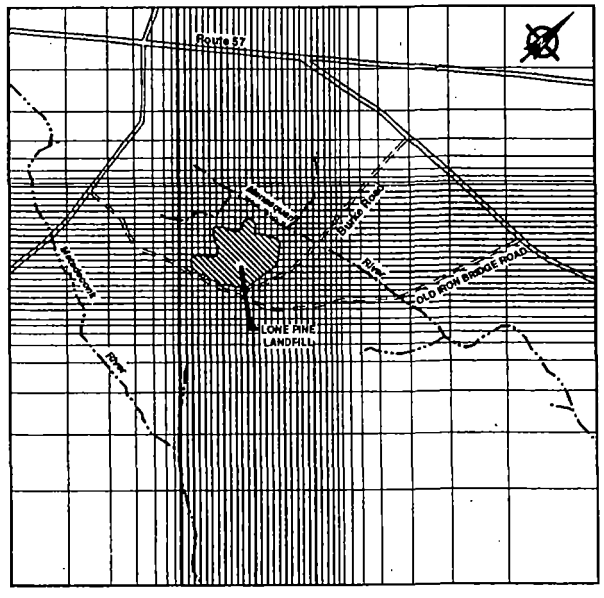
In the horizontal directions, nodal spacing may be uniform (i.e., the same in each coordinate direction and throughout a given part of the mesh) or variable. Note that meshes of uniform or variable spacing are also referred to as regular or irregular in the literature. In flow simulation, a uniform and relatively close nodal spacing is generally used within the area of greatest interest, while variable spacing may be used to extend the mesh outward to recognized hydrogeologic boundaries. In either flow or transport simulation, however, there are certain advantages to the use of uniform spacing wherever feasible.

First, the numerical truncation errors resulting from the approximation of the flow and transport equations are generally smaller for regular nodal spacings than for variable spacings. Second, in developing model input parameters, an interpolation scheme is often used to obtain nodal values on the basis of a limited number of data points. Most commercial or public domain data interpolation programs assume regular nodal spacing, and thus cannot be used easily to assign parameter values for a model grid of irregular nodal spacing. Finally, calculated hydraulic heads or solute concentrations from a regular portion of the model grid can be directly ported to a contouring program to create contour maps, without the necessity of an interpolation procedure which can smear or distort the calculated distribution.

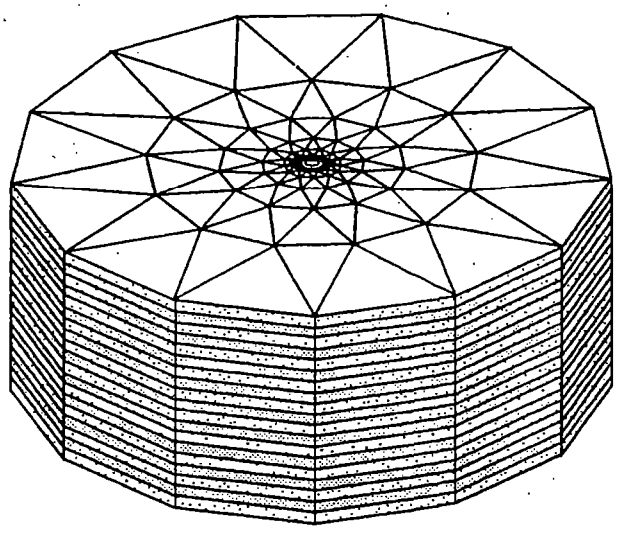
For these reasons, it is advantageous to use a regular nodal spacing throughout the entire grid. However, where the natural hydrogeologic boundaries fall at some distance from the area of interest, it is usually necessary to use an irregular spacing so that those boundaries can be incorporated in the simulation without using an impractically large number of nodal points (see Figure 8-1). In these cases, a practical rule of thumb for finite-difference simulation is to increase the spacing from one node to the next by a factor of no more than 1.5 or 2, as advocated originally by Trescott et al. (1976).

The advantages of uniform spacing noted above are equally applicable to flow and transport simulation. In transport simulation, however, if the standard finite-difference or finite-element method is used, the nodal spacing is required to meet an additional criterion in order to minimize numerical dispersion (see Chapter 6). This criterion is expressed in terms of the grid Peclet number as defined previously:

$$Pe_x = \frac{v_x \Delta x}{D_{xx}} \tag{8-1}$$



(a) A Finite-Difference Grid (modified from Zheng et al., 1991).



(b) A Finite Element Grid (after Gambolati et al., 1986).

FIGURE 8-1. Illustration of irregular finite-difference and finite-element model grids.

where Pe_x is the Peclet number for advective and dispersive components in the x direction; the Peclet numbers for the y and z directions are similarly defined. The larger the Peclet number, the more dominant is the advective transport component, and thus the more significant is the numerical dispersion problem that is associated with standard finite-difference or finite-element methods. To mitigate numerical dispersion, the nodal spacing in the x, y, and z directions must be sufficiently fine to insure a small Peclet number at the prevailing velocity components and dispersivity values. As noted in Chapter 6, numerical dispersion becomes negligible when the grid Peclet number approaches 2. Voss (1984) suggests keeping the grid Peclet number equal to or smaller than 4 when setting up nodal spacing for the SUTRA finite-element transport model.

Obviously, the Peclet number criterion for spatial discretization is stringent, and can require exceedingly small nodal spacings if flow velocity is large or dispersivity is small. This is one of the reasons that Lagrangian or Eulerian-Lagrangian approaches, such as the random walk method or the method of characteristics, are often preferable for solving advection-dominated problems (see Chapter 6). Using a Lagrangian or Eulerian-Lagrangian method, transport problems with very large Peclet numbers (infinite in the purely advective case) can be handled effectively without resorting to very small nodal spacings.

8.2.2 Vertical Discretization

Discretization in the vertical direction presents a number of mesh design problems. The finite-difference scheme is often used in the vertical direction, even where calculations in the horizontal plane are carried out using the finite-element approach, for example in the PTC code (Babu and Pinder, 1984). It is theoretically possible to impose uniform discretization in the vertical direction, i.e., to divide the model into horizontal layers of equal thickness, or at least into layers which are each of uniform, if not equal, thickness. The total number of layers is then determined simply by the required vertical resolution. If the layers differ from one another in thickness, numerical problems can be minimized by limiting the change in thickness between adjacent layers to a factor of 1.5 or 2, as in horizontal discretization.

What makes this approach difficult in practice is that most groundwater flow systems contain hydrostratigraphic units which vary significantly both in thickness and hydraulic properties, as illustrated in Figure 8-2(a). When the system is discretized using uniform horizontal layers as illustrated in Figure 8.2(b), several hydrostratigraphic units of very different characteristics may be contained in a single model cell, and estimation of composite hydraulic or transport parameters for such cells may be very difficult.

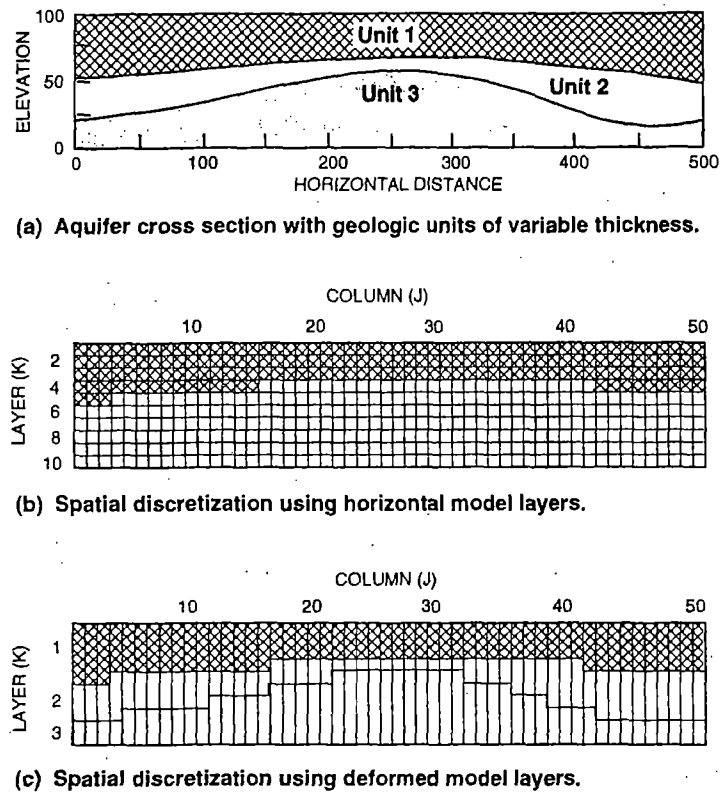


FIGURE 8-2. Illustration of different approaches for vertical discretization in a finite-difference model.

Furthermore, an impractically large number of layers may be required to maintain the continuity of aquifers or aquitards across the system; the hydrologic continuity of such units is often the most important factor in controlling the flow system and the transport process. To limit the required number of model layers while maintaining that continuity, several commonly used three-dimensional flow and transport codes (e.g., MODFLOW, MT3D, and FTWORKS) allow the use of a vertically deformed model grid, see Figure 8-2(c). In this vertical discretization approach, the thickness of an individual layer may vary from one location to another, so that the layer can represent an individual aquifer or aquitard unit.

The use of a vertically deformed grid simplifies the assignment of hydraulic parameters, and allows the continuity of aquifer or aquitard units to be maintained with fewer layers than a regular grid, as demonstrated by

Figures 8-2(b) and (c). However, vertically deformed grids introduce additional numerical errors as they depart from certain of the assumptions upon which the standard finite-difference method is based. While the magnitude of the error resulting from this departure is hard to quantify, it can be demonstrated that a vertically deformed grid causes substantially greater error in transport simulation than in flow simulation (e.g. Zheng, 1994). This is because in transport simulation, velocity must be calculated from fluxes and cross-sectional areas at model cell interfaces. In a vertically deformed grid, the cross-sectional areas at cell interfaces must be estimated from some sort of averaging, thereby suffering loss in accuracy and resolution. In addition, as noted in Chapter 5, cell-to-cell variation in layer thickness may cause false movement of fluid particles across layer boundaries in any method which incorporates a particle tracking component. For these reasons, more layers may be needed to achieve the required accuracy when both flow and transport simulations are undertaken than when flow simulation alone is involved.

In flow simulation, an aquitard or low permeability interval is sometimes represented only by the vertical conductance between the overlying and underlying transmissive layers; in effect this introduces the assumption that flow in the low permeability material is entirely vertical and that storage effects within it can be neglected. In this discussion the term "quasi-three-dimensional" is used for vertical discretization schemes which incorporate this assumption (although it should be noted that this term has also been used in a more general sense to describe any vertically deformed grid). A quasi-three-dimensional model in the sense used here (see Figure 8-3) requires much less computer storage and has much shorter run times than a model in which aquitard units are represented by individual layers or groups of layers, and can give comparable results in flow simulation provided the underlying assumptions are satisfied. However, a quasi-three-dimensional grid presents problems in transport simulation. If an aquitard separating two aquifers is not represented by an individual model layer, the calculated travel time between the aquifers is underestimated by an amount of t', where under the assumption that flow in the aquitard is vertical:

$$t' = B'/v' \tag{8-2}$$

in which B' is the thickness of the unrepresented aquitard and v' is the seepage velocity through the acquitard, given by

$$v' = q'/\theta' \tag{8-3}$$

In equation (8-3) q' is the Darcy velocity between the two aquifer units as

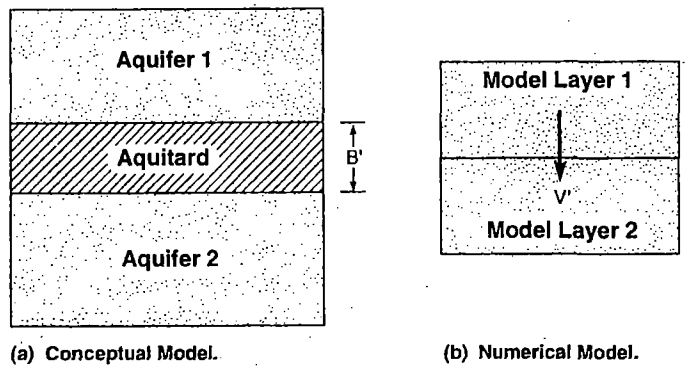


FIGURE 8-3. Illustration of the quasi-three-dimensional approach in vertical discretization.

simulated by the quasi-three-dimensional model and θ' is the porosity of the confining unit.

A further problem arises in transport simulation when a quasi-three-dimensional grid of the type shown in Figure 8-3 is used. When simulating the concentration distribution with a contaminant transport model, an aquitard acts to store or release contaminant mass. The amount of mass stored in the aquitard per unit surface (map) area, considering the dissolved phase only, is

$$M' = B'\theta'C' \tag{8-4}$$

where C' is the concentration within the aquitard. When sorption is present, the amount of mass stored in the solid phase must also be considered. When the aquitard is not represented by an individual model layer or groups of layers, concentrations in an adjacent aquifer will be overcalculated if the problem is one in which solute mass would accumulate in the aquitard. On the other hand, concentrations in the aquifer will be undercalculated if the problem is one in which solute mass would be released from the aquitard.

For the reasons outlined above, the quasi-three-dimensional approach as illustrated in Figure 8-3 should be avoided in transport modeling. If the use of this type of model cannot be avoided, due to limitations in computer resources, certain remedies may be applied to compensate at least partially for the effects of omitting the aquitard units. For example, in particle tracking calculations, travel times for particles moving between aquifer units may be increased by the amount given by equation (8-2) to obtain a corrected total travel time. In modeling the concentration distribution, simulated concentrations for aquifer units above and beneath the unrepresented aquitard may be adjusted to reflect the mass accumulating within or released from the

aquitard. However, these methods of correction cannot yield results as satisfactory as those obtained by representing the aquitard by one or more model layers.

8.3 TEMPORAL DISCRETIZATION

In transient flow simulation, the total length of time to be simulated is normally divided into a series of stress periods (often referred to as pumping periods), each of which is in turn divided into one or more time steps. Stress periods relate to the nature of external stresses; they represent time periods over which external stresses such as pumping rates, recharge rates, or river stages are held constant. Time steps, on the other hand, represent the finite time increments used in approximating the time derivative in the governing differential equation. Generally speaking, smaller time steps lead to a more accurate numerical solution; however, increasing the number of time steps increases the computer time required to complete a simulation. In practice, therefore, a compromise always has to be struck between accuracy and efficiency. Almost all commonly used flow models, whether solved through iterative or direct techniques, are based on the backward-difference approximation of the governing equation. Therefore, there are no stability criteria or constraints associated with the selection of time step size. Most flow models allow the use of a time step multiplier (normally between 1 and 1.5) so that time step sizes may be increased progressively as simulation moves on from one time step to the next within a stress period. This is desirable because the rate of change in the flow field is greatest immediately after a change in external stress, and often decreases rapidly over time.

Unlike flow simulation, which may be carried out in either the transient or steady-state modes, transport simulation is almost always transient in nature, i.e., concentration must be considered a function of time, even when the governing flow field is steady-state. It is true that a contaminant plume may eventually reach steady state if there are continuous sources and either continuous sinks or sustained reactions which reduce concentrations effectively to zero over a certain distance. However, these conditions occur infrequently in field situations, and most transport simulations must be considered transient.

In transient transport simulation, the total length of time to be simulated is also discretized into stress periods, as in flow simulation (see Figure 8-4). During each transport stress period, both the flow rate and the specified concentration of each external source remains constant. If the flow rate of an external source changes, a separate stress period is required, even if the concentration of this source remains the same, as illustrated in Figure 8-4.

A transport stress period is in turn divided into one or more time steps.

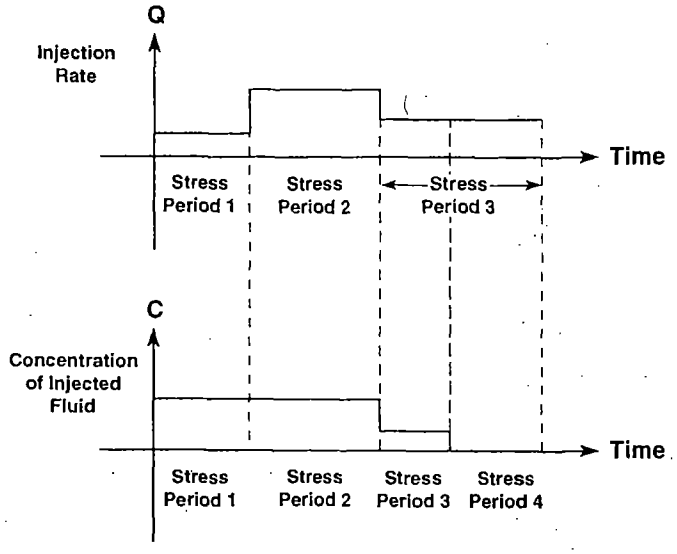


FIGURE 8-4. Relationship of stress periods used in flow and transport simulations.

Smaller time step sizes are generally needed in transport simulation than in flow simulation; thus a single time step of the flow simulation may correspond to several smaller steps in the transport simulation (referred to here as transport steps). There are a number of reasons for this finer time discretization in transport simulation. When the standard finite-difference or finite-element method is used as the solution technique, transport step sizes are generally required to meet a certain accuracy criterion expressed in terms of the Courant number, in order to minimize numerical dispersion and/or artificial oscillation (see Chapter 6). The Courant number has been defined previously as

$$C_r = \frac{v_x \Delta t}{\Delta x} \tag{8-5}$$

where C_r is the Courant number, defined for the x direction using the velocity component v_x and nodal spacing Δx. Equation (8-5) can be rewritten to obtain the criterion for the transport step size in terms of the Courant number:

$$\Delta t = C_r \frac{\Delta x}{v_x} \tag{8-6}$$

In applying equation (8-6) a Courant number equal to unity ($C_r = 1$) is the

most commonly used criterion (e.g., Voss, 1984; Kipp, 1987). In a three-dimensional model in which a sorption isotherm is simulated, equation (8-6) may be generalized as

$$\Delta t = RC_r \min\left(\frac{\Delta x}{v_x}, \frac{\Delta y}{v_y}, \frac{\Delta z}{v_z}\right) \tag{8-7}$$

where R is the retardation factor for the assumed sorption isotherm.

When a Lagrangian or mixed Eulerian-Lagrangian method is used as the solution technique in transport simulation, numerical dispersion is not a problem. However, because particle tracking is used to approximate the advection term, transport steps must be limited in size to maintain sufficient accuracy in the particle tracking solution (see Chapter 5). When the first-order Euler's method is used for the particle tracking solution, the Courant number normally should not exceed unity (e.g., Konikow and Bredehoeft, 1978; Prickett et al., 1981). When a higher-order method such as the fourth-order Runge-Kutta method is used for the particle tracking solution, a Courant number greater than 1 but generally not greater than 2 may be used (e.g., Zheng, 1990).

Transport simulation also requires smaller step sizes when an overall explicit solution scheme is used in the transport code. An overall explicit solution scheme is often used in mixed Eulerian-Lagrangian approaches, for example, in codes such as MOC or MT3D codes which solve advection by particle tracking and dispersion by an explicit finite-difference or finite-element method. The time step restriction in these cases arises because of the stability criteria associated with explicit solution schemes. For example, when dispersion is solved with an explicit block-centered finite-difference method, the maximum transport step size for a stable solution in a three-dimensional simulation is

$$\Delta t \leq \frac{0.5\,R}{D_{xx}/(\Delta x)^2 + D_{yy}/(\Delta y)^2 + D_{zz}/(\Delta z)^2} \tag{8-8}$$

where D_{xx}, D_{yy}, and D_{zz} are the principal components of the dispersion coefficient. In a two-dimensional simulation, equation (8-8) is reduced to

$$\Delta t \leq \frac{0.5\,R}{D_{xx}/(\Delta x)^2 + D_{yy}/(\Delta y)^2} \tag{8-9}$$

These equations show that the explicit formulation in three-dimensional simulation requires a smaller time step size than in two-dimensional

simulation, given the same horizontal dispersion coefficient components. The equations also show that the step size is inversely proportional to the dispersion coefficient components. Therefore, if large dispersivity values are required in a transport simulation, small transport steps may be needed to meet the stability criterion. If transport components other than dispersion are also solved explicitly, there will be additional stability requirements as discussed in Chapter 6. In summary, therefore, the transport step size should be the smallest step size computed for the entire mesh as satisfying the Courant number constraint and all stability criteria.

8.4 INITIAL CONDITIONS

Initial conditions are required for all transient flow and transport models. In flow simulation, initial conditions for a transient simulation are often taken from the results of a steady-state simulation representing the system prior to the imposition of transient stresses. In transport simulation, initial conditions are usually formulated according to the objectives of simulation.

As noted previously, the objectives of transport simulation are usually: (1) to achieve a better understanding of the transport regime; (2) to reconstruct the evolution of an existing plume from its origin to the present; or (3) to evaluate the future response of an existing plume to various proposed containment or remedial actions. For either of the first two objectives, initial concentrations are usually set to zero or to background values everywhere in the problem domain. To address the third objective, the concentration distribution of the existing plume must be used as the initial condition. The simplest way to do this would be to use the existing field data directly as the model input; however, the concentration distribution of an existing plume is rarely known in sufficient detail to allow this approach. Very often, one has only a limited number of concentration measurements, and frequently these are focused in a small area of the plume. In an ideal case, one should utilize transient simulation starting with zero (or background) initial concentrations, and adjust parameters in calibration until the simulated plume for present conditions matches measured concentrations at observation wells (see Chapter 10). The simulated plume could then be used as the initial condition for subsequent predictive runs. However, this process requires detailed information on the location and history of the contaminant sources, which is rarely available; and the time and effort involved would in many cases be prohibitive. For these reasons, the concentration distribution of an existing plume is often constructed from the measured data using an interpolation scheme, or a more sophisticated geostatistical tool such as kriging (e.g. de Marsily, 1986; Deutsch and Journel, 1992).

The diskette designed to accompany this volume includes a program, INTERP, which contains several interpolation and kriging options which may be used to obtain an initial plume distribution from scattered concentration measurements. This program also provides a logarithmic conversion option which can be used to convert the measured concentrations to a logarithmic scale before interpolation or kriging, and afterward to convert the interpolated or kriged values back to an arithmetic scale. This conversion may be desirable if the concentration data appear to follow a logarithmic distribution. To apply program INTERP, the user enters the spatial coordinates of the data points and of one nodal point, and the model grid spacing. The program returns concentration values of all nodal points, in a format specified by the user. Further information on the diskette is provided in Appendix B.

When interpolation is used to generate initial plume concentrations from a sparse set of field data, the resulting initial conditions for the predictive model carry significant uncertainty. Techniques of sensitivity and uncertainty analysis (as discussed in Chapters 10 and 11) can be used to address this problem.

8.5 BOUNDARY CONDITIONS

Generally, there are three types of boundary condition for transport models (see Chapter 4). They are: (a) specified concentration (Dirichlet condition); (b) specified concentration gradient or dispersive flux (Neumann condition); and (c) specified concentration and concentration gradient or total flux (Cauchy condition). The treatment of these boundary conditions in transport simulation has been described in Chapter 6. However, solute transport is influenced not only by the boundary conditions of the transport simulation, but also by those of the supporting flow simulation, and by the way these interact with the transport boundary conditions. In this section we discuss the combined influence of flow and transport boundary conditions in controlling the rate at which solute mass enters or leaves the model at boundary cells.

8.5.1 Use of the Specified-Concentration Condition

The Dirichlet type of transport boundary condition, or a specified (constant) concentration condition, is commonly used to represent a solute source area. For example, model cells representing zones of extensive NAPL accumulation are often treated as constant concentration cells, as dissolved concentrations in groundwater near the NAPL can be expected to remain high and essentially constant over long periods of time. As another example, concentration would normally be specified for a cell containing an injection well, if

the injection is capable of rapidly replacing the pore volume of the cell; the specified concentration in this case would be that of the injected water.

The mass fluxes from and to a specified concentration boundary node are determined internally by the simulation program. Both the advective and dispersive components of the mass exchange between the boundary nodes and the interior nodes are accounted for. Obviously, the exact amount of mass actually introduced into or removed from the transport regime is not only a function of the specific concentration, but also of factors such as the velocity and dispersivity values at the boundary nodes.

8.5.2 Use of the Specified-Mass-Flux Condition

The mass flux from and to a transport boundary can be prescribed before the transport simulation through the use of the Neumann or Cauchy type of transport boundary condition. The advective mass flux into or from the transport regime is determined by the specified flow rate and the concentration at the boundary nodes. The dispersive mass flux is determined by the specified concentration gradient and the dispersion coefficient at the boundary nodes.

In many practical model applications, the rate at which solute mass enters or leaves a boundary cell by dispersive transport is very small and thus can be neglected in calculation. As a result, the Neumann condition, where the concentration gradient must be specified across the boundary, is rarely applied. Where the Cauchy condition is applied the concentration gradient across the boundary is usually not explicitly specified, thus in effect is considered zero. In other words, the advective component is generally used to approximate the total mass flux specified across a Cauchy boundary.

8.5.3 Role of Flow Model Boundary Conditions in Solute Transport

Most of the boundary conditions commonly used in flow simulation result in flow of water into or out of the model at cells subject to the condition. Along a specified-flow boundary, water enters or leaves the model at each boundary cell at a rate Q_S, which remains fixed within each stress period but may vary from one stress period to another, and is specified for each stress period in advance of simulation. An important special case of the specified-flow boundary is the no-flow boundary, in which Q_S is zero for all stress periods. A no-flow boundary represents the only boundary condition under which flow into or out of the model cannot occur at the boundary cells.

Other commonly used conditions in flow simulation are the specified-head boundary, in which head or pressure is maintained at a fixed value in each

stress period at boundary nodes subject to the condition; and the closely related head-dependent flow boundary, in which flow into or out of the model at a boundary cell occurs in proportion to the difference between a specified head external to the model and the calculated head at the boundary node. Under both of these types of boundary condition, a flow Q_S, entering or leaving the model at the cell, is determined in the simulation at each time step. Under the head-dependent boundary condition, Q_S is calculated as

$$Q_S = COND(h_S - h_{aq}) \tag{8-10}$$

where h_S is the specified head at the external sink/source, h_{aq} is calculated head at the boundary node and $COND$ is the conductance or constant of proportionality. Under the specified-head boundary condition, Q_S is simply the algebraic sum of the flows toward and away from the cell within the model, as calculated in flow simulation.

The simplest case to consider is that of the no-flow boundary. This type of boundary condition is often used to represent a contact with material of very low permeability, but can also be used to simulate any surface on which the normal component of the hydraulic gradient is essentially zero, and can be expected to remain so throughout the simulation. In either of these cases, the velocity component normal to the boundary is zero, and advective transport into or out of the model across the no-flow boundary is clearly zero. If the boundary represents a contact with low permeability material, dispersive transport across it can also be taken as zero. In theory, dispersive transport may not be zero across a no-flow boundary which is used to represent an arbitrary surface on which the normal hydraulic gradient is negligible, since the concentration gradient may not be zero at a surface of this kind. In most cases, however, the dispersive flux of solute mass across such a surface is very small and can be neglected. Thus in effect a no-flow boundary in the flow simulation can usually be treated as a no-mass-flux boundary of the transport model.

For all other types of flow boundary, some nonzero flow, Q_S, will in general enter or leave the model at each boundary cell (where as discussed above, Q_S may be specified or determined in the course of simulation, and may vary both from cell to cell and with time). This implies that solute mass may also enter or leave the transport model through these boundary cells. If the flow model boundary coincides with a boundary of the transport model on which concentration is specified (i.e., on which either the Dirichlet or Cauchy condition is implemented) the rate at which solute mass enters or leaves the model advectively at a given boundary cell is $Q_S C_b$, where C_b is the specified concentration of the cell. Otherwise, if concentration is not specified along the boundary, two cases must be considered. If the boundary

cell functions as a sink, i.e., if Q_S is either specified as a withdrawal or is determined in calculation to be a withdrawal, the rate at which solute leaves the model advectively at the cell is $Q_S C_a$, where C_a is the calculated concentration in the groundwater at the cell. If the cell is specified as a source, or determined in calculation to function as a source, the rate at which solute mass enters the model advectively at the cell is $Q_S C_S$, where C_S is the concentration of the water entering at the cell. C_S must be specified in advance for any cell which has the potential to function as a source at any time in the simulation.

8.5.4 Comparison Between Flow and Transport Boundary Conditions

While an analogy exists between transport model boundary conditions and flow model boundary conditions, one must be careful to note the differences in the way corresponding conditions influence the respective simulations. The specified-concentration boundary condition of the transport equation is analogous to the specified-head boundary condition of the flow equation; however, it influences solute mass inflow in a very different way than the specified-head boundary influences water inflow. Similarly, the specified-flow boundary condition of the flow equation is actually a specified hydraulic gradient condition, and is thus analogous to the specified concentration gradient condition of the transport equation; but whereas the inflow or outflow of water is completely determined when the hydraulic gradient is specified, only the relatively small dispersive component of the solute mass flux is determined by specifying the concentration gradient. There is in fact no transport boundary condition, taken alone, which can specify the total rate at which solute mass enters or leaves at a boundary, except for the special case of zero concentration and zero concentration gradient noted above. To generate a transport model boundary for which the solute mass flux is exactly and fully determined, a specified-flow boundary of the flow equation must be combined with a specified-concentration and concentration gradient boundary of the transport equation.

8.5.5 A Note on Scale Difference in Flow and Transport Simulation

As stated previously, quite often a flow model is of regional scale with a large and extensive grid. A transport model, on the other hand, is of local scale under many circumstances. Using the same model grid for flow and transport simulation may lead to unnecessary transport calculations in many of the model cells outside the plume area. Some transport codes provide

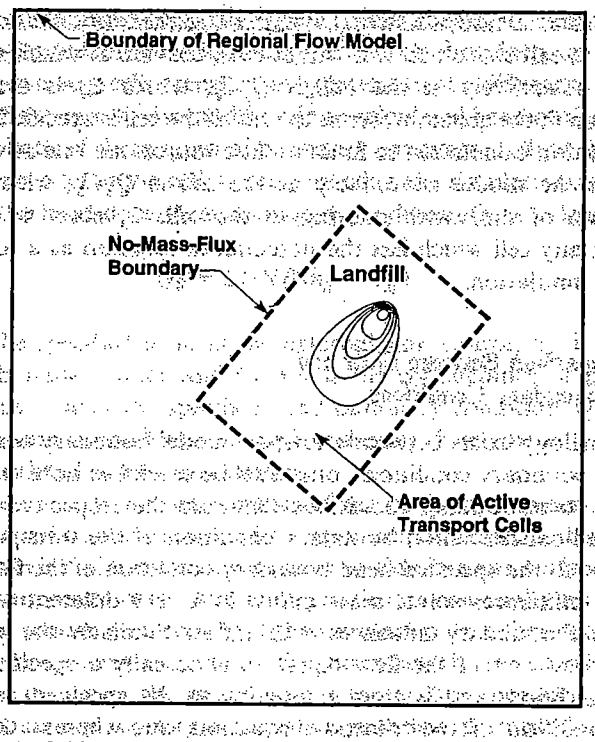


FIGURE 8-5. Definition of a local area for transport calculation within the regional flow model.

certain means to handle this situation more efficiently. For example, the MT3D code (Zheng, 1990) allows setting the no-mass-flux boundary inside the active flow domain to remove from the transport calculation any active flow cells which are not anticipated to exhibit transport (see Figure 8-5). The MOC code (Konikow and Bredehoeft, 1978; Goode and Konikow, 1989) permits the definition of a subgrid for transport simulation within a portion of the flow simulation grid. Under this option the transport grid does not coincide with the flow grid, and velocity components at cell interfaces of the transport grid have to be interpolated from the flow simulation results.

Another approach to the problem of different scales in flow and transport simulation is based on the concept of grid refinement. With this approach, termed "telescopic" by Ward et al. (1987), a regional-scale flow model with coarse nodal spacing is first developed utilizing as many natural boundary conditions as possible. A local-scale flow model is subsequently developed, with boundary conditions interpolated from the flow solution

obtained with the regional-scale flow model. (An intermediate step may also be taken, if necessary, to bridge the regional- and local-scale models.) Transport may then be simulated based on the local flow model.

8.6 SOURCES AND SINKS

8.6.1 Types of Sources and Sinks

A source or sink represents a mechanism through which water enters or leaves the system; thus it is first and foremost a feature of the flow simulation. The source/sink term in the governing transport equation represents solute mass dissolved in water which enters the flow domain through fluid sources, or leaves it through fluid sinks. The flow rates of sources and sinks are either specified or computed in the flow simulation. Sources and sinks may be divided broadly into two types, internal and external.

External sources and sinks actually represent boundary conditions, as discussed in the preceding section; examples, as implemented in simulation, would include specified-head, specified-flow, and head-dependent flow cells along a model boundary. The distinction as to whether these should be considered boundary conditions or sources and sinks is one of terminology, or of the interpretation given to the source-sink term in the governing flow and transport equations. However, whether they are described as boundary conditions or sink/source terms, boundary processes are represented in simulation as discussed in Section 8.5.

Internal sources and sinks are those located in the interior of the active flow domain. Examples include wells, buried drains, recharge, evapotranspiration, and surface water features such as rivers, lakes, or ponds. In three-dimensional or cross-sectional simulation, processes which act only at the upper surface of the groundwater regime, such as recharge, evapotranspiration, and leakage to or from surface water features, should actually be considered boundary conditions. Again, the distinction is one of terminology, not of the way these processes are represented in simulation. In this text, we follow the conventional approach of treating these processes as sink/source terms even though in certain mesh configurations they may be limited to boundary nodes.

There is in fact no essential difference in the way boundary conditions and internal sources or sinks are simulated in most computer codes. For example, in MODFLOW and MT3D, the procedures for simulating a well can be used to represent an actual injection or extraction well within the flow domain, or can be used to implement a specified-flow or specified-mass-flux boundary condition at a cell on the model perimeter. Certain other source/sink processes are similar to the head-dependent flow boundary of

equation (8-10) in the way they are simulated; in particular, outflow to drains, flow between the groundwater regime and surface water bodies, and outflow by direct evapotranspiration from the water table are usually represented in this way. In these cases the linear relationship of equation (8-10) is usually constrained to a certain range of flow values. For example, in the case of evapotranspiration or flow to a drain, the function is limited only to outflow from the aquifer, and a maximum outflow rate is normally specified; in the case of seepage to or from a stream, a maximum rate of flow from the stream into the aquifer is usually specified. However, within the specified range, flow enters or leaves the model at a rate proportional to the difference between the calculated head at the model node and an external control head.

It should also be recognized that certain processes which are sometimes thought of as solute sources are not hydraulic sources in the sense used in the equations of flow and transport. In the example of Section 8.5.1, in which NAPL entrained in an aquifer is dissolved gradually by the natural groundwater through the affected region, a source of solute exists but no water is added to the system. In cases of this kind, the source may be represented as discussed in Section 8.5.1, using one or more constant-concentration cells, provided the NAPL pool is expected to persist over the entire simulation period; or it may be treated through a dissolution reaction term. Alternatively, the mass could be introduced by specifying a fictitious injection well, where the injection rate is set sufficiently low that it does not affect the flow field in any significant way. The concentration of the injected water can then be manipulated so that the desired rate of solute mass inflow is achieved.

8.6.2 Concentrations of Sources and Sinks

The concentration of the entering water at any internal source or potential source must be specified in advance of simulation, just as for any boundary cell which has the potential to act as a source. On the other hand, the concentration of the water removed by an internal sink is usually set equal to the calculated concentration of the groundwater in the cell containing the sink. Thus a discharging well removes mass at a rate $Q_w C_a$, where Q_w is the specified well discharge rate and C_a is the calculated concentration of the groundwater in the cell containing the well; on the other hand, an injection well supplies mass at a rate $Q_w C_S$, where C_S is the concentration of the injected water, which must be specified in advance. Similarly, the rate at which solute mass is transported out of an aquifer by groundwater discharge to a surface feature is taken as the product of the calculated concentration in the groundwater and the calculated flow to the surface feature. For seepage in the reverse direction, from a surface water body into an aquifer, a

concentration must be specified for the water entering from the surface feature, unless conjunctive transport simulation of the surface and groundwater is undertaken. Evapotranspiration represents an exception to the normal rule for a sink, in that evapotranspiration can be assumed to remove only water, not solutes; thus the concentration of water removed by evapotranspiration is usually specified as zero.

The concentration of water entering from a source is often unknown and difficult to estimate. For example, if the source is seepage from a landfill above the water table, a major effort may have to be made in characterizing the complex chemical, biological, and physical processes operating within the landfill in order to estimate solute concentrations in the seepage reaching the water table. The concentrations of many types of source, moreover, may vary with time, and specification of a constant value may thus not be adequate. Figure 8-6 shows some general concentration-time functions which have been used to characterize contaminant loadings from various types of source.

It should also be noted that source concentration is often one of the targets of the model calibration process, as discussed in Chapter 10. Finally, no

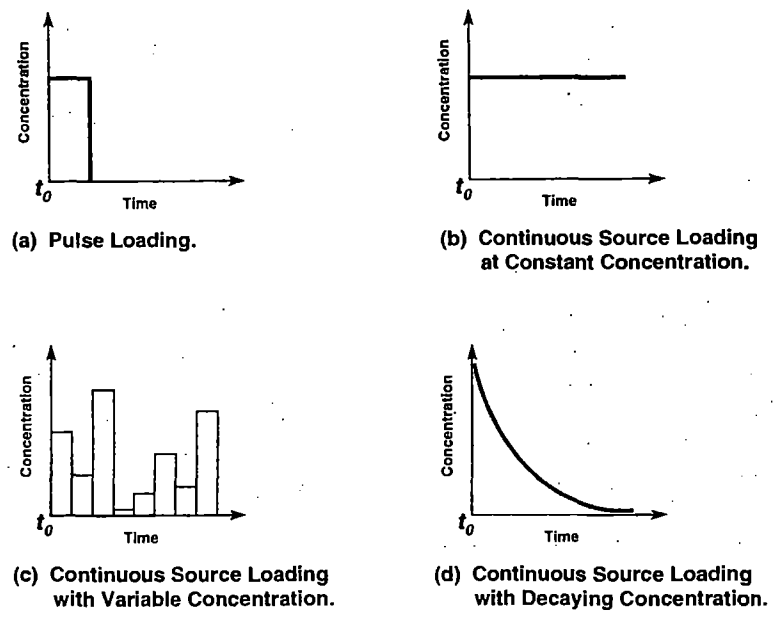


FIGURE 8-6. Examples of functions used to characterize contaminant loading from a short short-term solute input (a) or long-term leakage (b, c, and d) (after Domenico and Schwartz, 1990). Reprinted by permission of John Wiley & Sons, Inc.

matter what approach is used to estimate source concentration, the results will usually be subject to uncertainty. Some general approaches to the problem of uncertainty are discussed in Chapter 11.

8.7 DATA MANAGEMENT

8.7.1 Preprocessing and Postprocessing

The process of preparing and assembling input data for a computer-based numerical model is referred to as preprocessing, while the process of examining and presenting simulation results is termed postprocessing. The volume of input data required by a multidimensional flow and transport model, for a field site of even moderately complex hydrogeology, can be prohibitively large. Efficient management of these data, and their translation into forms which can be used by a simulation program, are an essential part of any modeling effort of significant size. Postprocessing of simulation results is equally important, if the maximum information is to be extracted from the model output and the results are to be presented in easily understood graphic or tabular form.

In general, preprocessing involves five basic steps:

1. Designing horizontal and vertical discretization schemes; assembling spatial discretization data such as node spacing and layer thickness, and setting up appropriate boundary conditions.
2. Assigning hydraulic and transport parameters to the nodes or cells. When uniform parameters or simple zonations are not adequate, a computer program for performing spatial interpolation may be used to obtain model nodal values from measured data points.
3. Establishing a suitable temporal discretization scheme and setting up initial conditions, if the problem requires transient simulation.
4. Assembling information on sinks and sources, including locations, specified flow rates or specified hydraulic connections with the aquifer, and specified solute concentrations.
5. Selecting a solution option and appropriate solution parameters.

Information prepared and assembled during the preprocessing stage is generally arranged in one or more files containing a series of numeric or character records which can be read by a particular model code. Preprocessing is usually considered *manual* when input files are prepared according to the instructions given in the user's manual for the model code, which may involve the use of text editors, spreadsheet programs, or other software for general data analysis and presentation. Preprocessing is usually described as *automated*

when input files are prepared using computer programs or subroutines which have been specifically developed to simplify the input process of an existing model. However, the distinction between manual and automated preprocessing is clearly a matter of arbitrary definition, as model users generally apply all data processing tools available to them, and often write their own programs to handle particular input needs.

Manual preparation of input files has certain advantages. For beginners, it forces them to become familiar with the input/output structures and various options of a model code. This can be a frustrating process, especially if the input structure and file formats of the model code are not well organized and designed. However, the user may gain a better understanding of the model code and its operation, and a better appreciation of the various options; these gains can often outweigh the frustration and time lost in manual preparation of input files. For experienced users, the flexibility and speed of manual input file preparation can be enhanced by using files created for a previous model as templates. Manual preprocessing can also be made more convenient through the use of commercially available software for data analysis and presentation. One of the primary disadvantages of manual file preparation is that changing the model grid after all input files have been created can be both difficult and tedious without the help of specialized software. A further disadvantage is that the detection of a simple typographical or format error in a manually created input file can sometimes require significant time and effort.

Automated preprocessors are designed to facilitate preparation of input files by adding interfaces more user-friendly to model programs. Early preprocessors were typically no more than a series of question and answer sessions; apart from freeing the user from dealing directly with the format of input files, this type of preprocessor did little to enhance model development. More recent preprocessors combine computer-aided design (CAD) and preprocessing capabilities, and can be of significant help in the preparation of input files. Many of these processing software packages, such as ModelCad (Rumbaugh, 1993); PM (Processing MODFLOW) (Chiang and Kinzelbach, 1993); and MODIME (Modular Integrated Modeling Environment) (SSPA, 1994) facilitate model grid design by graphically superimposing the grid on a digitized base map of the project area. The input and editing of aquifer parameter distributions and sink/source locations can also be done directly on a graphical display of the digitized base map. As preprocessing software packages become more flexible and powerful, it is certain that they will find more and more use in simulation projects. Most preprocessing software packages are likely to remain external to main simulation codes for ease of development and maintenance, although there is also a tendency to build preprocessing capabilities into the simulation

codes themselves, as in the case of the FLOWPATH code (Franz and Guiguer, 1990).

Postprocessing involves analysis and presentation of model simulation results. Most numerical codes save simulated head or concentration distributions as a sequence of two- or three-dimensional arrays in text files. To analyze or process these data and display the results graphically, one must generally use either commercially available software for data analysis and presentation, or specifically designed postprocessors. The following are some of the most useful kinds of information or information display which can be derived from the output of a flow or transport model through postprocessing:

1. Residuals and relevant statistical measures describing the differences between observed and calculated heads or concentrations at observed points; this information is required during model calibration.
2. Hydrographs or concentration breakthrough curves at well locations of interest, if the simulation is transient; these are used both for calibration and for presenting the model response at specific locations.
3. Head and concentration contour maps for all or part of a model layer, or for a specified vertical cross section.
4. Local flow and solute mass budgets for regions enclosed within specified boundaries or for certain sink/source locations (while most flow and transport models provide a summary of global flow and mass budgets, it is often desirable to examine flow and solute mass budgets on a local basis).
5. Pathlines, arrival times, and extent of capture zones at selected times.

For most modeling applications, the time and effort spent on preprocessing and post processing of data far exceed that spent on other project activities. The continuing efforts to develop more intuitive and more user-friendly preprocessing and postprocessing capabilities have prompted both interest and concern in the groundwater modeling community. While a user-friendly interface increases efficiency and boosts productivity (e.g., van der Heijde, 1992), one can argue (e.g., National Research Council, 1990) that the availability of such interfaces may encourage the inappropriate use of models by unqualified personnel. There are clearly merits to both views; however, whether perceived as desirable or not, the continued development and expanding application of automated preprocessing and postprocessing software appears to be inevitable. As with any developing technology, minimizing misapplication will be largely in the hands of the user community. The most important principle is to treat the user-friendly interface as a means to enhance efficiency and productivity, rather than as an excuse to use the model as a black box.

8.7.2 Geographic Information Systems (GIS)

Rapid development in computer hardware and software technology has resulted in new and improved tools, which could drastically change the way groundwater modeling is done today. One of these tools is GIS, or *Geographic Information System*, which is essentially a computer software package for storage, manipulation, and display of data characterized by geographic coordinates. A GIS, which combines data base management, geostatistical analysis, and graphical display, can serve as an integrated environment in which field data are analyzed and checked, and the conceptual model formulated and updated. Linked by an interfacing program, the field data and conceptual model can be converted directly by the GIS to the input data files for a groundwater model. The model simulation results can, in turn, be retrieved by the GIS for analysis and graphical presentation. While GIS tools are not widely used in groundwater modeling today because of their relatively high costs and extensive hardware requirements, it is expected that they will become an indispensable part of modeling application in the future.

Harris et al. (1989) describe the use of a GIS system with a three-dimensional finite-element flow, energy, and solute transport code (CFEST) in flow and transport simulation of the San Gabriel Basin in California. Orzol and McGraph (1992) discuss the modification of the USGS MODFLOW code for linking with the commonly used GIS package ARC/INFO. Van der Heijde (1992) discusses the general procedures and special considerations involved in the use of the GIS system in groundwater modeling. Maidment (1993) presents a summary on the state of GIS for surface water and groundwater modeling.

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GROUNDWATER
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Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

ATTACHMENT 4

A PRACTICAL GUIDE TO GROUNDWATER AND SOLUTE TRANSPORT MODELING

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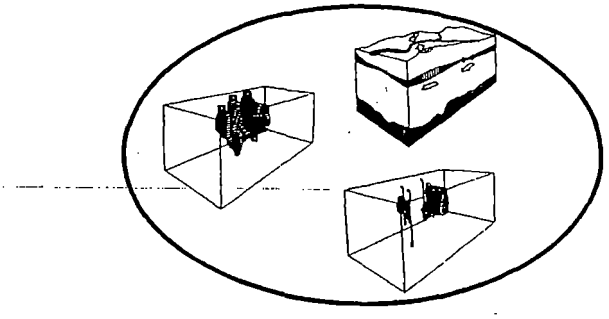
model calibrated to a contaminant plume of a certain size may not accurately simulate smaller or larger plumes, such as is required to show plume remediation. Since dispersion is more limited in the vertical than in the horizontal plane, it is better to approximate a transport problem by a two-dimensional horizontal-plane model than by a two-dimensional vertical-plane model, solely from the standpoint of dispersion simulation.

Errors originating in simplification of the natural system are often mistakenly balanced by modifying dispersion coefficients or other model parameters. For example, vertical contaminant migration due to density effects may be poorly represented by increasing vertical dispersivity. Clean surface recharge may also be mistakenly invoked to generate the same apparent predicted concentration distribution as for a density-driven plume. The distorted model will produce different conclusions than a realistic model, even though the distorted model may provide "realistic" predictions for one set of conditions and point in time. The errors introduced by preparing a simplified model should be assessed in comparison with errors or uncertainties built into the model.

8 Applying Numerical Models



Numerical modeling is the most commonly used form of groundwater modeling analysis. The strength of numerical models is their ability to tie together data and physical principles into a coherent and useful picture of a site. Their weakness is their ability to hide gaps, errors, or misunderstandings inside a cloak of technical respectability. A model contains many levels of information at different scales, and different users will extract different results to fit their needs. Models are one of the best accompaniments to a characterization program in evaluating site conditions.

This chapter explains the steps usually required to prepare a realistic site-specific model (Figure 8.1). The steps of developing a numerical model, together with the related ramifications to the model predictions, are presented in the order they are normally undertaken. The emphasis of the following sections is on the practical everyday uses of models rather than on experimental, research-oriented, or untested approaches to modeling analyses. Many points that have been examined in previous chapters are reflected in this chapter to form one comprehensive review of model application. Additional guidance is provided in ASTM Standard D 5447-93 (ASTM 1993).

8.1 COMPILING DATA

The first step of a model study consists of collecting and evaluating relevant data on the flow system under investigation. Input data for the model are used for:

- Problem definition (material properties and geometry of hydraulic units)

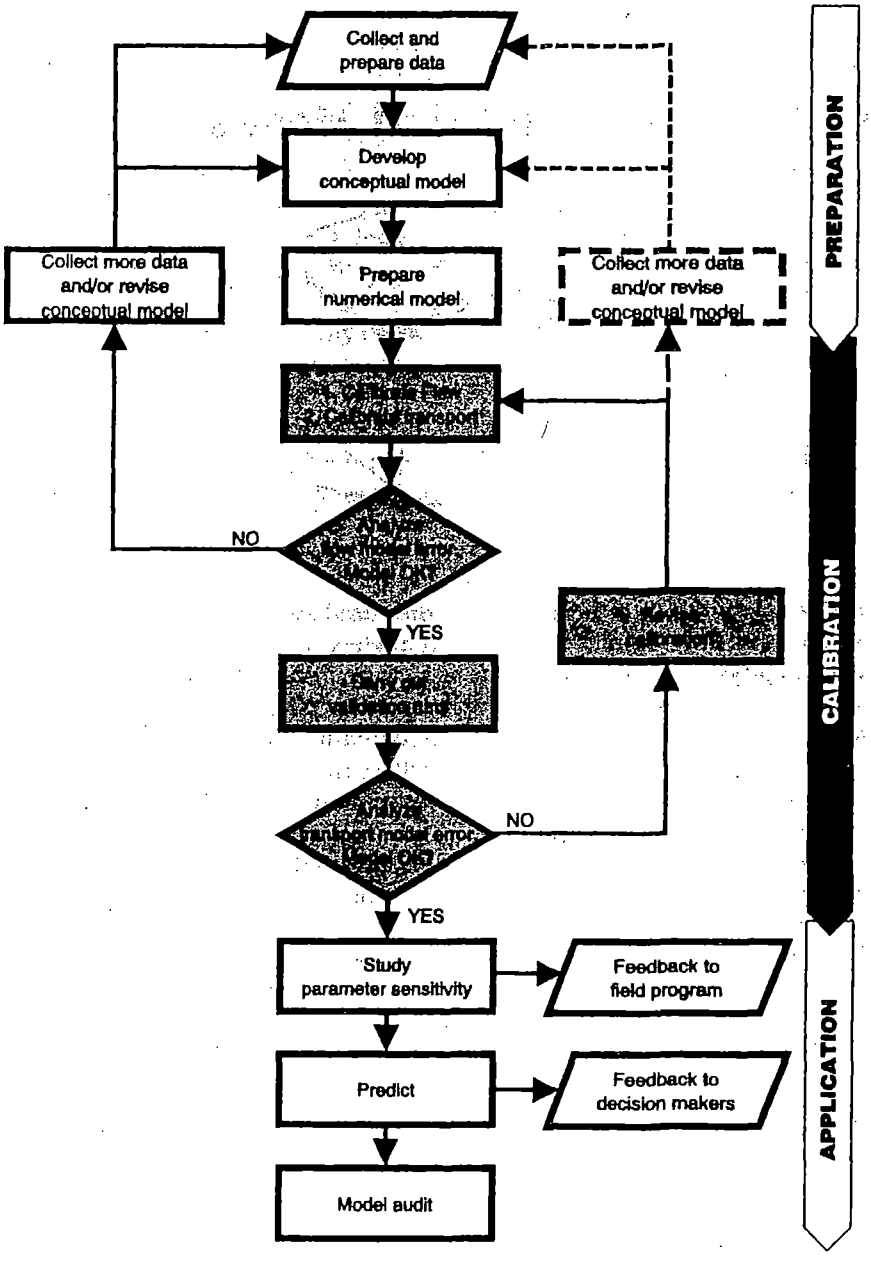


Figure 8.1 The groundwater modeling process.

- Numerical requirements (initial conditions, boundary conditions, and transient conditions)
- Modeling requirements (calibration, validation, and definition of alternate scenarios).

Input data for a variably saturated, variable-density, flow and transport model are summarized in Table 8.1. Data are separated into data defining the physical framework of the flow system and data defining groundwater recharge and discharge. An additional list of data needs and likely sources is given in subsection 8.4.5. Typical values for many of these input parameters are summarized in Appendixes B, C, and D.

Related groups of data should be summarized in the form of maps or plots, at a suitable scale and level of detail, overlaid on a suitable base map. A set of maps (see Figure 8.2) may comprise:

- Topographical map at a suitable scale depending on the groundwater system to be modeled. A simplified topographical map or map of significant site features can serve as the base map on which other data are overlaid.
- Hydrological map and/or vegetation map showing all surface water bodies such as lakes, streams, ponds, or channels. Springs, wetlands, and swamps can be included. The map should provide a clear picture of the surface drainage system. Morphological features can also be plotted on this map.
- Map indicating all wells and boreholes made for geological surveys, water supply, and site characterization. A suitable well or borehole identification will relate these elements to a separate database comprising related data such as well coordinates, screened interval, and other well design data.
- Geological map(s) together with cross-sectional plots or fence diagrams (Figure 8.3). Geological maps should show information such as faults, thickness and elevation of strata, depth of stream channels and changes in rock types. This information then can be related to groundwater occurrence and movement, possibly allowing identification of the hydrostratigraphy.
- Maps showing measured groundwater head distributions at different times. The interpreted potentiometric surface and monitor well locations are included on these maps.
- Land use map showing agricultural areas, recreational areas, industrial areas, irrigation canals and so forth. This map can be used in conjunction with the hydrological map to delineate recharge/discharge areas.
- Concentration distribution maps in horizontal and vertical planes, for several times, together with source locations. Historic land use maps or aerial photographs can also help with source identification.

TABLE 8.1 Typical Model Input Data

	Physical Framework
Aquifer type	Topography
	Geology
	Stratigraphy
	Aquifer geometry (Base, thickness, lateral extent)
	Lithological variation within the aquifers
Aquifer characteristics	Hydraulic conductivity/anisotropy
	Porosity
	Conductivity/Pore pressure relationship
	Saturation/Pore pressure relationship
	Specific yield
	Specific storage
	Dispersivity (for transport modeling)
	Soil bulk density (for transport modeling)
Aquifer boundaries	Location
	Prescribed head
	Prescribed flux
	Semipermeable boundary (Leakage factor, head in adjacent system)
	Hydrological Boundary Conditions
	INITIAL WATER LEVELS/GRADIENTS
Internal inflow/outflow	Recharge/discharge (Area, rate, duration)
	Extraction or injection wells (Location, rate, duration)
	Exchange with surface water (Surface water elevation, base of surface water, leakage factor)
	Groundwater Quality Boundary Conditions
Land use	Industrial, agricultural
Aquifer environment	Groundwater chemistry
	Background values of solutes/contaminants
	pH
Source	Location
	Area
	Source concentration
	Volume
	Duration
Contaminant	Solubility
	Density
	Viscosity
	Henry's law constant
	Octanol-water partition coefficient
	Diffusion coefficient
	Adsorption isotherm
	Decay rate
	Biodegradability

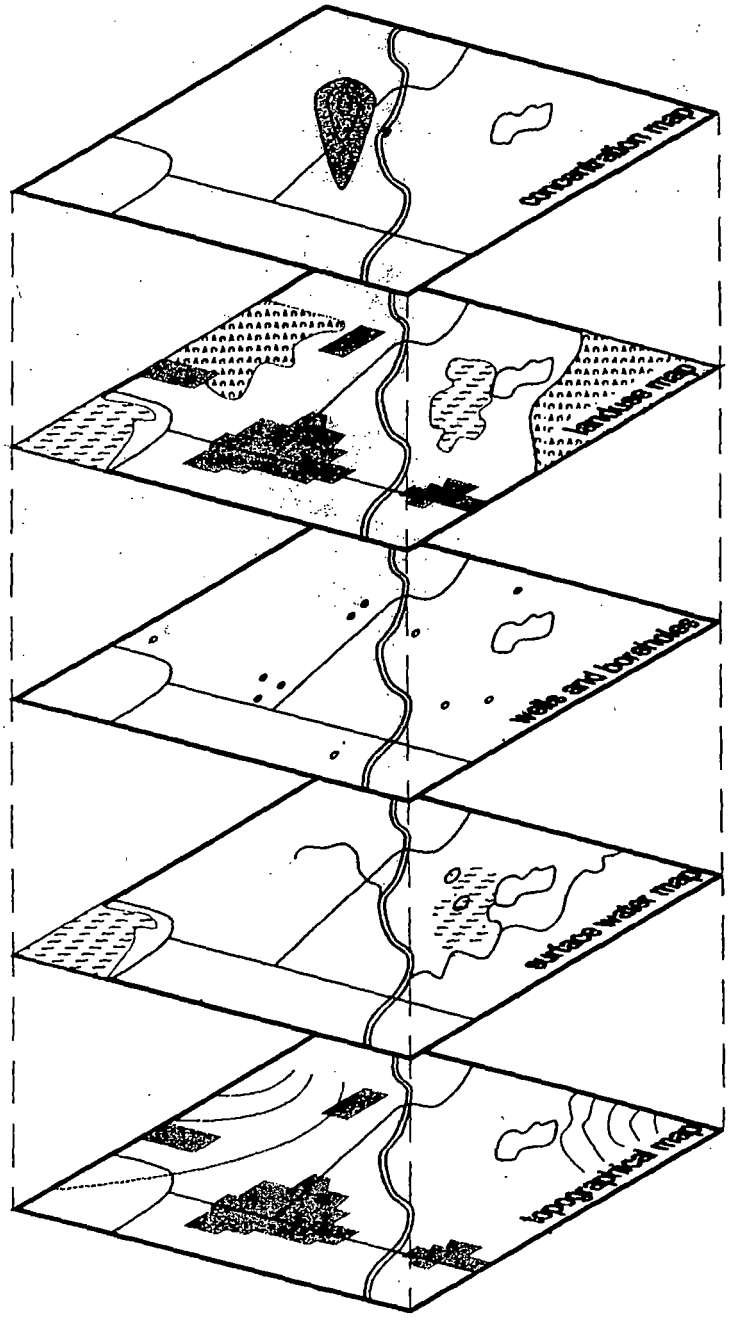


Figure 8.2 Preparation of data in maps linking data in information layers.

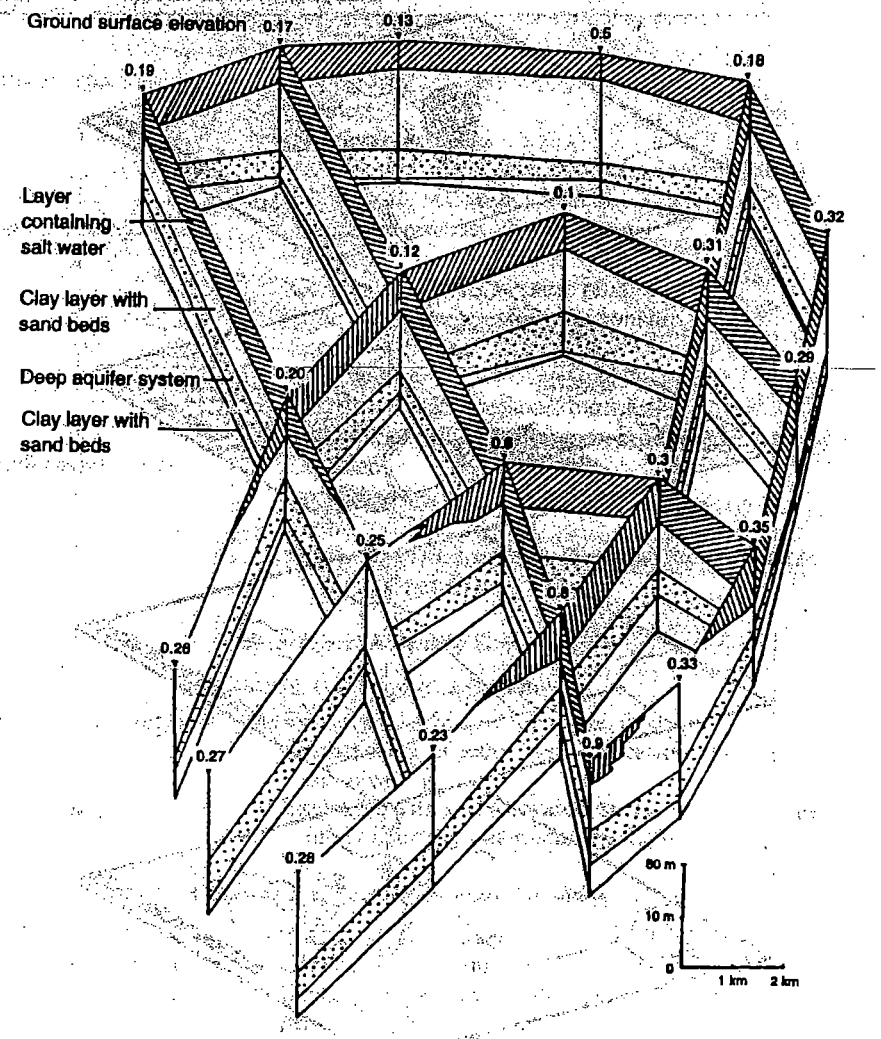
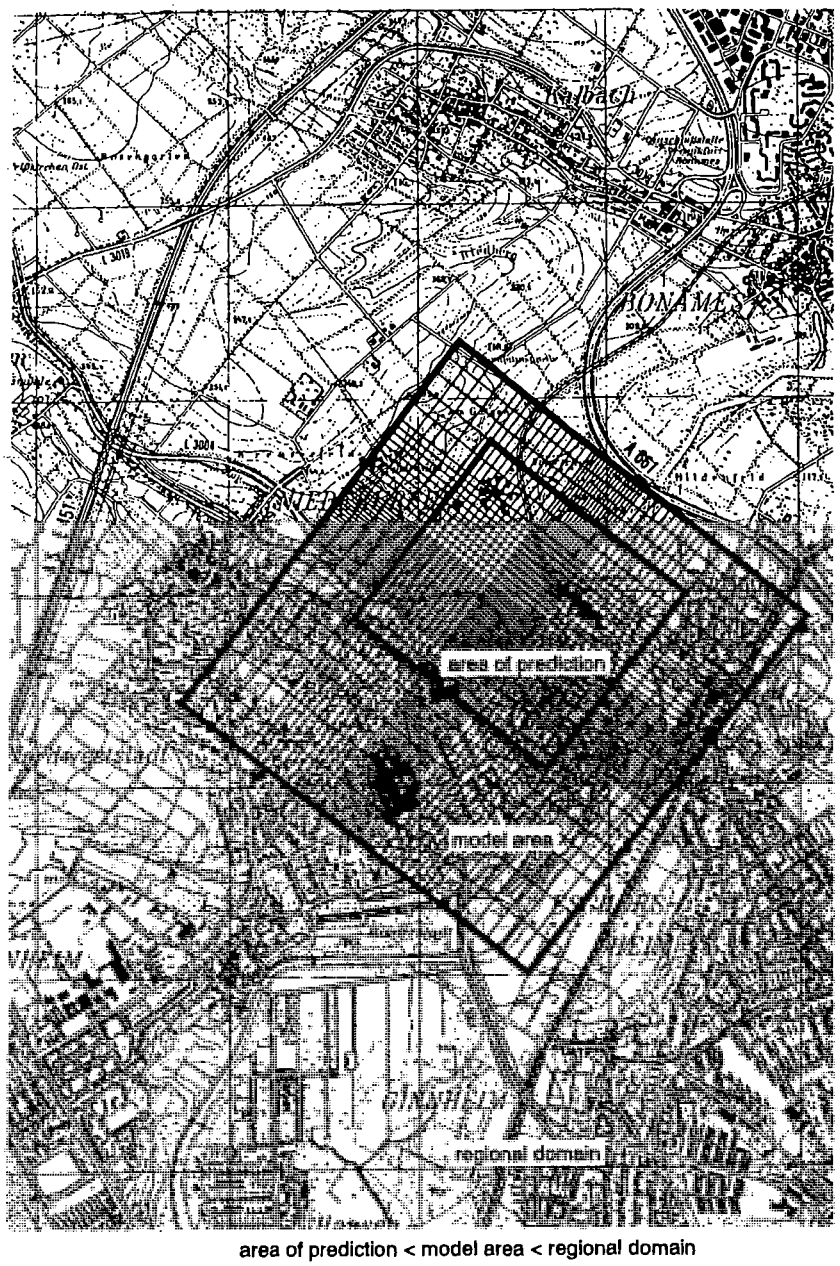


Figure 8.3 Fence diagram to illustrate stratigraphy.

- Additional maps might include structure-contour maps of aquifer base elevations or aquifer and aquitard isopachs, elevations of top and base of hydrogeologic units, thickness of units, hydraulic conductivity or transmissivity distributions, evaporation and rainfall contour maps, vertical hydraulic gradients and flow-direction maps.

Figure 8.2 shows a set of such maps overlaid as layers.

Model design, data collection and review often occur at three scales (Figure 8.4). At a regional scale, information is gathered to identify and define suitable model boundaries. Data at a local scale are reviewed to estimate the area within which accurate predictions are required. The actual size of the model



area of prediction < model area < regional domain

Figure 8.4 Collecting input data at different scale.

domain then ranges somewhere between the area of prediction and the regional boundary. Once the model domain is designed, future data evaluation concentrates on this area.

8.2 DEVELOPING A CONCEPTUAL MODEL

The second step in a modeling study is developing a conceptual model. A conceptual model is an idealization of the real world that summarizes the current understanding of site conditions and how the groundwater flow system works. It embodies all of the important features of the flow system, while incorporating simplifying assumptions. The three purposes of developing a conceptual model are:

- Develop a better understanding of site conditions, and be able to communicate this understanding.
- Define the groundwater problem for development of a numerical model.
- Aid in selecting a suitable numerical model.

The development of an appropriate conceptual model is the key to a successful modeling study. The conceptual model will evolve with the addition of new data; the first conceptual model may not be the last. The term "conceptual model" is sometimes used to refer to the entire conceptual model, model selection, and model setup process. These other aspects of developing a site-specific model are discussed in the following sections of this chapter.

The conceptual model is the point where all relevant ideas are incorporated into an idealization of reality. Concepts that initially appear farfetched can be saved for future consideration if the first conceptualization fails. A conceptual model cannot be approached logically, step by step. However, the following questions may be used to get the process started:

- Is the physical system understood well enough to be cast into to a model?
  - Have field data been interpreted and cast into a useful format?
  - Is there consensus about the flow system and geologic setting?
- What are the technical (and also perhaps political, economic, or legal) issues to be addressed?
  - Can they be answered by modeling analyses?
  - Do the technical issues require a high degree of accuracy in predictions?
- What are the physical and chemical processes that should be incorporated?
  - Confined and/or unconfined conditions?

Saturated and/or unsaturated flow?
Miscible or immiscible transport?
Geochemical reactions or decay?
Density-dependent flow and transport?

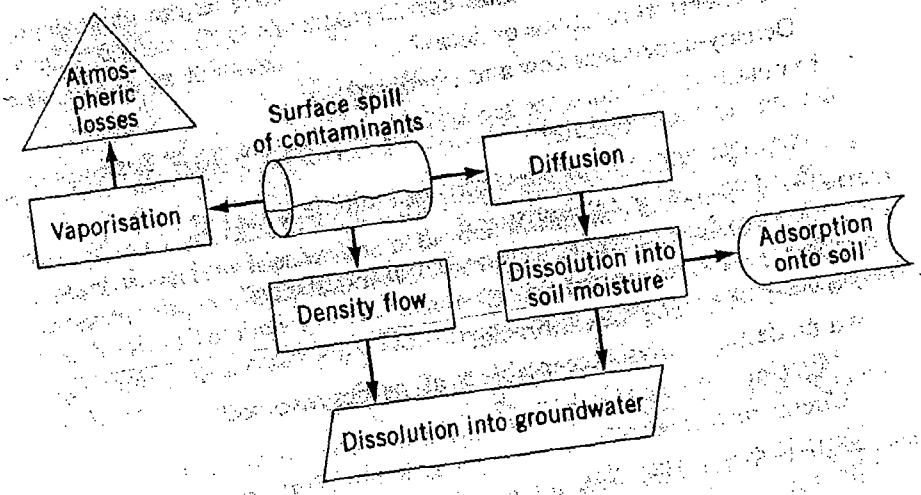
- How many dimensions are needed in the model, and which dimensions are the most appropriate?
  Heterogeneity and/or anisotropy or multiple aquifers?
  Point or areal sources or sinks?
  Density-dependent or bouyant flow and transport?
  Remediation to be simulated?
- How will the model be calibrated?
- Is a modeling analysis acceptable to all parties involved?
  Modelers and technical team?
  Clients or reviewers?
  Regulatory personnel?
- Will modeling be cost-effective at this site?
  Could analytical or mass-balance models suffice?
  Will additional field data be required anyway and make a modeling analysis superfluous?

Once you consider questions like these you can develop the conceptual model. The three steps in developing a conceptual model are (1) explore and summarize the key mechanisms governing groundwater flow and chemical-species transport at the site, (2) develop the assumptions and simplifications required to make the real situation tractable to analysis, and (3) establish the framework of the model (number of dimensions, type of model).

The conceptual model itself may take various forms. Different approaches are suited to different flow systems and model objectives. Figure 8.5 illustrates different types of conceptual models:

- Flow diagram indicating the interactions between different elements of the problem (Figure 8.5*a*).
- Mass-balance summary (Figure 8.5*b*).
- Geological cross sections labeled with key processes (Figure 8.5*c*).
- Three-dimensional diagram summarizing site conditions (Figure 8.5*d*).

This stage in a modeling analysis is difficult because it involves collating and interpreting many disparate pieces of information, together with various opinions and experience from other sites, into a cohesive picture capable of being analyzed numerically. Inexperienced modelers may need to seek experience with comparable sites from other modelers, hydrogeologists, hy-



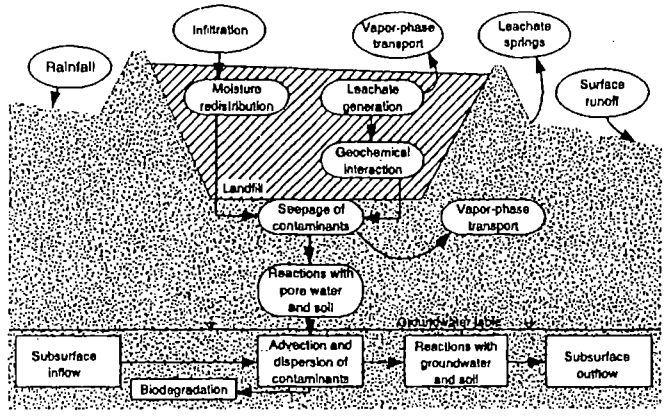
Mass balance for flow			
Water in storage	=	7.4×10^8	m^3
Change of water in storage	=	-3.4×10^2	m^3/d
Inflow from rivers	=	1.5×10^8	m^3/d
Inflow at wells	=	2.2×10^2	m^3/d
Inflow by infiltration	=	2.2×10^7	m^3/d
Inflow by boundaries	=	2.8×10^7	m^3/d
Outflow at rivers	=	0.0	m^3/d
Outflow at wells	=	0.0	m^3/d
Outflow by evaporation	=	0.0	m^3/d
Outflow at boundaries	=	2.0×10^8	m^3/d
Error	=	-4.4×10^5	m^3/d

Figure 8.5 Conceptual model examples.

drologists, or project managers. Factors that modelers often overlook during this process of model development include:

- Effect on transport calculations of selecting too few dimensions (see Chapter 7).
- Neglect of significant transport in bedrock, or other low permeability units, despite negligible flow (see example case in Appendix A.5).
- Underestimation of pollutant source quantities based on observed dissolved concentrations (see subsection 8.4.5).
- Neglect of density effects (see Section 3.3).

PROCESSES IN CROSS SECTION



THREE-DIMENSIONAL SCHEMATIC DIAGRAM

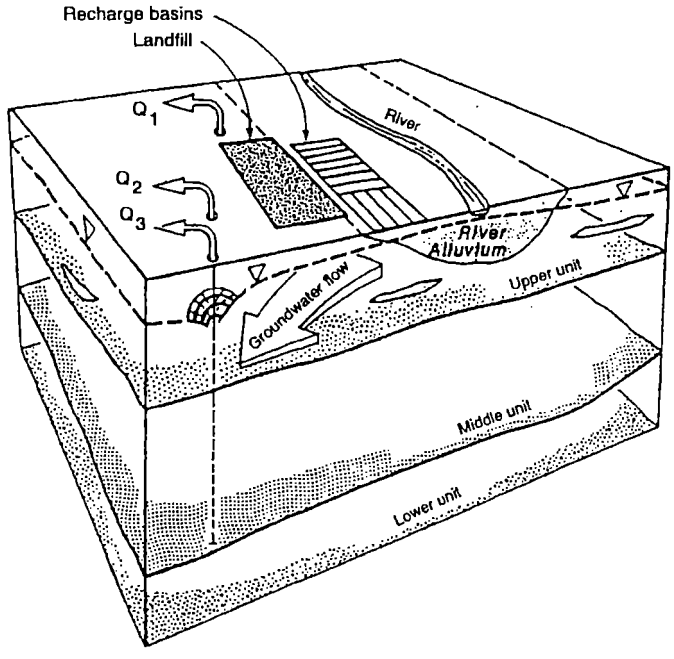


Figure 8.5 (*Continued*)

The observed behavior of a contaminant plume may demonstrate the key factors to consider when developing a conceptual model. For example:

- A plume with multiple high-concentration zones may indicate a pulsing source, multiple sources, aquifer heterogeneity, and so on.
- A plume with high concentrations at depth may indicate density effects, clean surface recharge, or fracture flow.
- A plume that has not migrated substantially may contain chemical species strongly retarded due to sorption, chemical reactions, decay, degradation, or volatilization.
- A plume in which bacteria density correlates with chemical concentration may indicate a biodegradable contaminant.
- A plume that migrates at the base of an aquifer or along the slope of the bedrock, rather than in the direction of groundwater flow may indicate density effects.

Review of the modeling approach that you developed, with all interested parties, on completion of the conceptual model helps to ensure overall model acceptance.

8.3 SELECTING A MODEL CODE

The third step in the modeling process is to select a code. The process of selecting an appropriate numerical model is discussed in various papers and reports, including van der Heijde and Park (1986) and Bond and Hwang (1988). The selection process cannot be entirely rationalized, and often involves nonscientific issues, but general considerations are listed in the following subsections. The selection approach discussed in these sections has been developed with and for the U.S. Environmental Protection Agency (EPA), and has been successful in gaining model acceptance in many projects in the United States.

8.3.1 Defining Model Objectives

Objectives criteria are used to distinguish models designed for general studies from those designed for detailed analysis. In groundwater transport the objectives of general studies are to:

- Develop understanding of the main factors governing contaminant transport at a particular site.
- Rank sites by means of quick, simplified analyses.

Such studies may be used in the initial stages of a modeling study, or when site-specific data are sparse, or when general, worst-case or relative con-

clusions only are required. Models selected on the basis of these objectives include analytical models, analog models, empirical models, mass-balance models, and simple numerical models.

The objectives of detailed modeling studies are to:

- Identify data gaps and guide the field program (including placement of monitoring wells).
- Predict the present and future concentrations in groundwater.
- Compare the effectiveness of remedial alternatives.
- Characterize source areas and release history.
- Optimize remediation, engineering design, or monitoring network.
- Characterize uncertainty and/or perform cost/benefit analyses to support management decisions.
- Assist with negotiations or litigation by providing a focal point for discussions.

Numerical models are often selected for application in a detailed modeling study because they can simulate a more realistic and detailed picture of the site. The predicted results are correspondingly more specific and reliable. Detailed studies may be regional or local in scale. A regional study may address questions such as basinwide water management, travel time to and concentrations at distant points, and the effects of external factors (e.g., other well fields, other pollutant sources, neighboring flow systems, and water bodies) on flow and transport. A local study might address questions such as planning a monitoring network, flow and transport adjacent to pollution sources, and designing remedial alternatives. Local models generally produce high-resolution results in time and space, whereas regional models may define general trends and may incorporate local models into a wider framework.

8.3.2 Selecting a Model on Technical Criteria

Technical criteria are used to match site characteristics with a model of appropriate capabilities. The chosen model should be capable of adequately representing the governing flow and hydrogeology processes and hydrostratigraphy of the site, as far as is known. These governing processes may include:

- Flow conditions (confined/unconfined, horizontal/vertical, saturated/unsaturated).
- Stratigraphy.
- Variations in time and space of boundary conditions (e.g., recharge and discharge) and contaminant sources.
- Porous, fractured, or karstic media.

- Spatial variations of material properties (both natural and engineered properties).
- Single or multiphase flow.
- Flow or transport affected by fluid density or temperature.

8.3.3 Implementing the Model

If several selected models satisfy the objectives and technical selection criteria, then one can use implementation criteria to narrow the choices. Implementation criteria involve the following considerations:

- Has the model been peer-reviewed and is it available for further review, or is it a research model or closely held proprietary model unavailable to public agencies or third parties?
- Is the model well-documented and are users supported?
- Has the model been verified against results from analytical solutions, laboratory results, other verified models, and field data?
- Has the model been applied successfully at similar sites?
- Is the model easy to implement on available computer systems and relatively easy to use by the modeler(s)?
- Does the model have a good track record of acceptability to reviewing/regulatory agencies, and do peer reviewers and experienced users regard the model favorably?

The objectives, technical and implementation criteria can be summarized as four key questions:

- Can the model adequately simulate site conditions?
- Can it satisfy the objectives of the study?
- Is the model verified and reasonably well field tested?
- Is the model well-documented, peer reviewed, and available?

Sometimes application of all these criteria will disqualify all known models. Usually the choice of the "best" model consists in a compromise among the criteria listed above. Today a considerable number of practical models are available. Hence model selection should be based on the selection criteria rather than on subjective arguments such as familiarity with, or availability of, a specific model.

8.4 SETTING UP A MODEL

Model setup is the fourth step in the modeling process. Model setup and calibration often constitute 50% to 70% of the total modeling effort. Model setup

entails selecting the model domain, discretizing data in space and time, defining boundary and initial conditions, and assembling and preparing model input data. Choice of the domain and discretization affect the physical and numerical resolution and level of effort (cost) of the modeling study.

8.4.1 Selecting a Model Domain

Selection of the optimum model domain involves balancing the following factors:

- The domain should cover the entire area of interest, including areas that may be affected by future chemical-species transport, and should encompass the effects of internal disturbances (e.g., aquifer pumping or injection, or seepage from impoundments). Future transport can be roughly estimated by calculating transport velocities and retardation factors or by analytical solution. If the entire chemical-species plume is not included in the model domain, overall mass balances will not be available.
- The boundaries of the domain should take advantage of natural groundwater boundaries such as rivers, lakes, drains, groundwater divides, edge of aquifer, boundary between adjacent pumping centers, coastline, groundwater recharge/discharge area or boundary location distant (in hydrologic terms) from the area of interest. Note that rivers, lakes, and drains are not always groundwater boundaries and that groundwater divides may move over time or with depth.
- The model domain should be oriented parallel to the primary groundwater flow direction (at least in the area of primary interest, e.g., highest concentration plume) to reduce numerical dispersion (see also Section 6.3).
- Available data should adequately define conditions throughout the domain selected.
- Domain size should be minimized to reduce computational effort.

Frequently the ideal domain size for a groundwater flow problem is much larger than the domain required for a groundwater transport model. In this case a sequence of two models (see also subsection 7.2.6), a problem-oriented finite-element grid (see also Section 6.3), or a compromise between the two domain sizes may be selected.

8.4.2 Discretizing a Model in Space and Time

This section discusses the factors that commonly affect the selection of model discretization. Table 8.2 lists these factors and how they generally affect selection of the model grid.

TABLE 8.2 Common Factors Affecting Model Discretization

Factor	Aspect of Model Discretization Affected
Modeling objectives	Domain size and areas of finer resolution
Area and duration of interest	Domain size
Location of sources and sinks	Finer discretization zone
Heterogeneity and anistropy	Orientation and refinement of grid
Particle velocity and retardation	Cell size and domain size
Natural boundaries	Limit on cell size to simulate boundary
Numerical stability	Limit on ratio of cell sizes
Numerical accuracy	Limit on cell sizes
Computational effort	Limit on total number of cells
Resolution of flow field	Finer discretization where high gradient
Resolution of concentration distribution	Finer discretization where high gradient

We can select the time and space discretization, that is, the time steps and cell (element) sizes, for numerical models by optimizing the following aims (listed in order of importance):

- Enhance model solution stability and convergence.
- Increase model resolution.
- Minimize numerical dispersion.
- Minimize computational requirements for memory, storage, and run-time.

How to meet these aims by the choice of appropriate model discretization is discussed in this section.

Model solution stability and convergence can be improved by the selection of time steps and calculation-mesh cell sizes that are consistent with one another (see Chapters 5 and 6 for related stability considerations). For example, it can be shown that for a two-dimensional, unsteady flow problem, stability will be ensured by the following condition (Bear and Verruijt 1987):

$$0 < dt < \frac{S}{2T} \frac{dx^2\, dy^2}{(dx^2 + dy^2)} \tag{8.1}$$

where

S = storativity in [1/L]

T = transmissivity (of aquifer or cell, depending on dimensions of model) in [L^2/T]

dx = cell width in the x-direction in [L]
dy = cell width in the y-direction in [L]

Other criteria are presented below.

Accurate predictions require selecting a cell size sufficiently fine to represent local variations in hydraulic head or concentrations, and defining time steps small enough to represent temporal variation of conditions. While variable cell sizes allow for greater flexibility, highly variable cell sizes can introduce a loss in accuracy (Bear and Verruijt 1987) and stability.

Numerical dispersion, or unnatural spreading of a chemical-species plume, occurs due to the neglect of higher-order terms in the Taylor series expansion of the finite-difference or finite-element formulation of the governing equations. It can also occur due to inappropriate space or time discretization. The potential for numerical dispersion varies with alternate schemes for approximating time and space derivatives. We can minimize unwanted dispersion by selecting appropriate calculation-mesh cell size, mesh orientation, and size of time steps. The following subsections discuss how to make appropriate selections. We test for the existence of numerical dispersion by applying a finer mesh and time step and comparing the coarse and fine-scale predicted results. Computational efforts are minimized by reducing, as far as possible, the total number of calculation cells and time steps in a given calculation. Minimizing computational effort is counter to accuracy requirements, and a compromise must be made, with accuracy in the area of interest being the primary goal.

In discretizing a model, the orientation of the model, space discretization and time discretization must be considered. These selections are discussed in the following subsections.

Orienting the Model Grid

The orientation of the model grid, which most often naturally accommodates all of the factors influencing choice of orientation, is a mesh-oriented parallel with the large-scale geologic features. In nearly all cases the optimum model orientation does not coincide with primary compass directions, roads, or plant boundaries. The following factors affect mesh orientation (in order of importance):

- Hydrologic, hydrogeologic, and geologic features at the site. Representation of key features such as rivers, streams, impoundments, faults, and other natural boundaries can be simplified through appropriate orientation of the mesh. For example, a fault zone that affects groundwater flow is best represented by cells oriented parallel and perpendicular to the fault.
- Predominant groundwater flow direction. Numerical dispersion due to the groundwater velocity being split into components parallel to the calculation-mesh axes is minimized if the mesh is oriented along the

direction of predominant groundwater flow. If the flow direction varies within the model domain, alignment of the calculation mesh with the flow direction in the primary area of interest is optimum.

- Anisotropy of hydraulic properties. Since the hydraulic conductivity is expressed in the model as components aligned with the axes of the calculation mesh, inaccuracies are reduced by choosing a mesh oriented coincident with the conductivity tensor.

Choosing the Model-Grid Cell Size

Selecting the model grid is comparable to selecting a net for fishing; the openings in the net must match the size of the "fish" (heterogeneities and predictive details) to be captured. This is illustrated in Figure 8.6, which shows an example borrowed from atmospheric modeling. Figure 8.6 shows a sequence of three different discretizations of world topography. Using a grid of 500 by 500 km, mountain ranges are barely identified. However, a grid of 100 by 100 km allows land elevations to be distinguished in far greater detail.

Consider the following factors when choosing a model grid:

- Degree of heterogeneity in hydraulic or transport parameters, and in boundary conditions.
- Model domain size.
- Predicted resolution required to meet modeling objectives.
- Restrictions imposed by computational resources.

Similar constraints apply to vertical discretization, with the added consideration of stratification due to density effects, recharge, and shallow or deep sources or sinks of water or contaminant. In general, the accuracy of the predicted results improves with finer calculation meshes, but computational time and space requirements increase correspondingly.

For transport problems, as demonstrated in Section 6.2, the cell size that minimizes numerical dispersion can be calculated using the cell Peclet number (Pe) which is the ratio of the advective to diffusive terms in the transport equation. The cell Peclet number is defined as the dimensionless ratio (Equation 6.11)

$$\mathrm{Pe} = \frac{v_x\, dx}{D_x} \tag{8.2}$$

where

v_x = particle velocity in the x-direction in [L/T]
dx = cell size in the x direction in [L]
D_x = dispersion coefficient in [L^2/T].

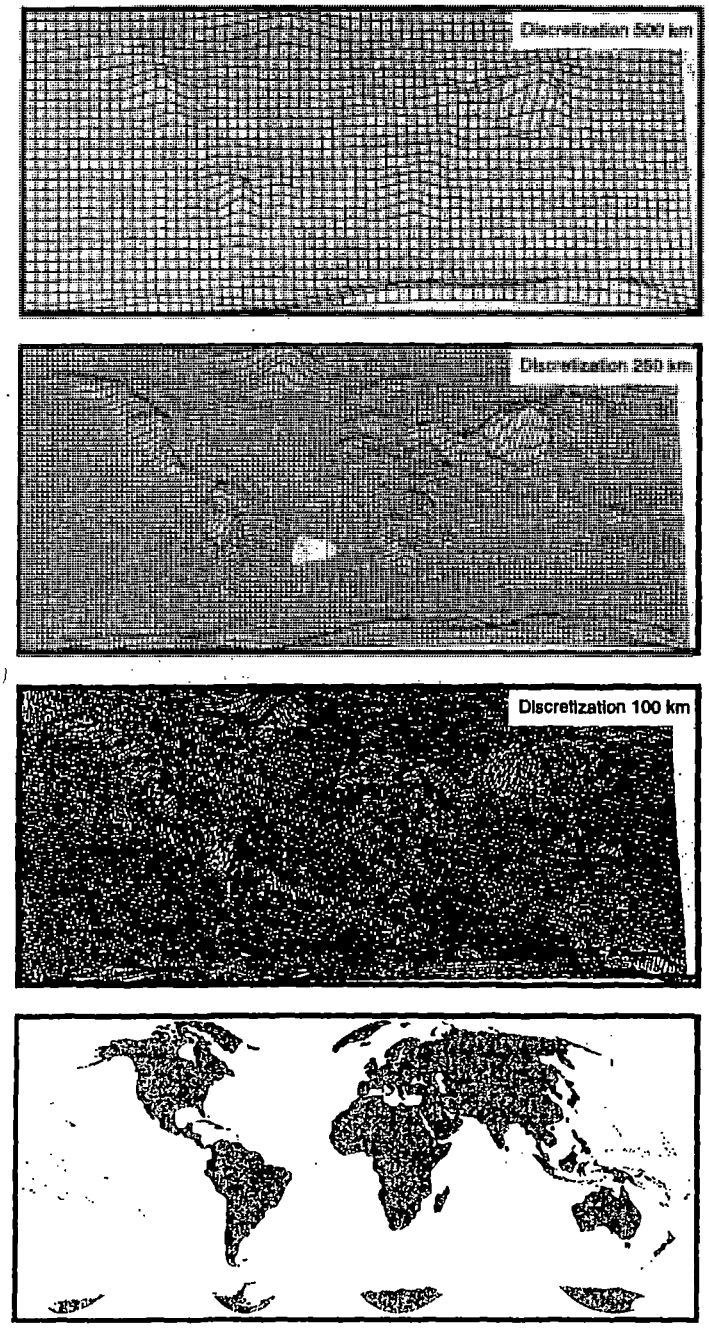


Figure 8.6 Discretization of the world topography using decreasing grid spacing (after FAZ, June 1, 1994).

Since the dispersion coefficient can be defined empirically as

$$D_x = v_x a_x \tag{8.3}$$

where a_x is longitudinal dispersivity in the x-direction [L], the Peclet number can be rewritten and the Peclet criterion can be defined as

$$\text{Pe} = \frac{dx}{a_x} \leq 2 \tag{8.4}$$

To ensure numerical stability and minimize numerical dispersion, the cell Peclet number should ideally be no greater than two (Pinder and Gray 1977); that is, the cell size should be no larger than twice the dispersivity. Since the dispersivity is a measure of the characteristic length of heterogeneities of the system, this criterion also makes physical sense. In practice, the Peclet number constraint is often relaxed outside the area of interest, where lower predictive accuracy is acceptable.

For unsaturated flow the Peclet number criterion has been shown to be nearer 0.5 (El-Kadi and Ling 1993), implying the need for a finer mesh in unsaturated analyses. Confusingly, the term "Peclet number" is also sometimes used to describe the ratio of dispersive to diffusive components of the transport equation (Bear 1979).

The use of geostatistical methods, such as kriging, to estimate data between data points is discussed in subsection 8.4.5. If model input data are being kriged to provide interpolations, then the associated semivariogram may be used to select an appropriate cell size. The cell size can be related to the range of the semivariogram:

$$dx < 0.3 \text{ range of semivariogram} \tag{8.5}$$

Mesh resolution should be finer in areas of large hydraulic or concentration gradients, and in the vicinity of features of particular interest (pumping or injection wells, rivers, sources of contamination, etc.). The mesh may be relatively coarse in areas of small gradients, in areas of less interest, or in areas where data are sparse.

The appropriate ratio of the lengths of the cell sides (cell aspect ratio) is calculated by comparing the travel time across the cell in each direction. Ideally the ratio of travel times should be unity, though ratios of up to 10:1 may be used without introducing significant error. In general, the greater the variability in cell size, the greater is the computational effort required in generating a convergent solution. Gradual variation in cell sizes, for example, cell size increases between adjacent cells no greater than a factor of 1.5, will facilitate model convergence, increase stability, and reduce inaccuracy. A

sample calculation mesh and the criteria used in generating it are shown in Figure 8.7 and Table 8.3.

Space discretization can be more flexible in FE models than FD models because FE models are not constrained to a rectilinear grid and can conform more closely with the geometries and flow directions in the model. A carefully designed FE model can satisfy most space discretization requirements using fewer elements than an FD model. In FE models each node and element must be numbered. The FE numbering scheme has no effect on model accuracy, but it does strongly affect the storage requirements and execution time. The bandwidth of the coefficient matrix is linked to the node numbering scheme, and model execution time is proportional to the square of the bandwidth:

$$\text{Bandwidth} = 2 \times (\text{largest difference between node numbers for any element} + 1) \tag{8.6}$$

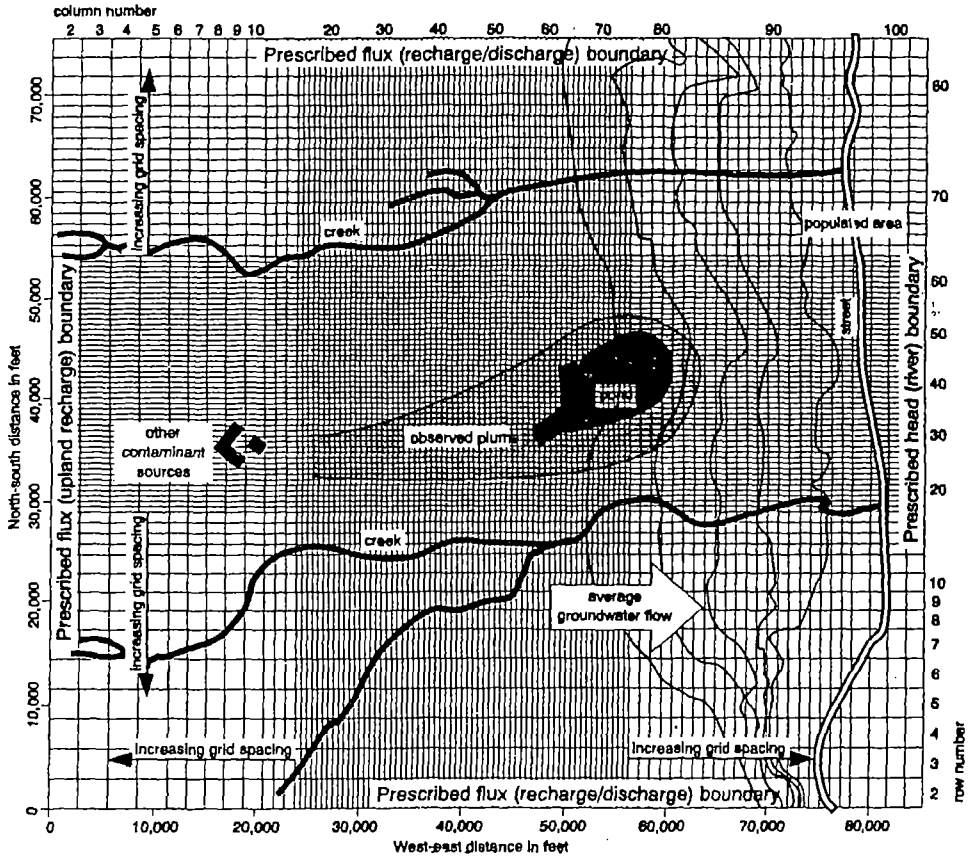


Figure 8.7 Example calculation grid-grid selection criteria.

TABLE 8.3 Example Calculation Grid–Grid Selection Criteria for Figure 8.7

Category of Criterion	Criterion	Purpose of Criterion	Model Grid
Orientation of calculation grid	Along primary flow direction	Minimize numerical dispersions due to grid orientation	Yes
Location of boundaries	Using suitable boundary conditions Far from area of prediction Encompasses plume	Prevent boundary effects from influencing solution	Yes
Cell aspect ratios	$1{:}10 \leq \Delta x{:}\,\Delta y \leq 10{:}1$	Minimize convergence problems	$0.125 \leq \Delta x{:}\,\Delta y \leq 8{:}0$
Cell Peclet number	$Pe \leq 10$	Minimize numerical dispersion due to space discretization	$Pe < 5$ in area of plume (Pe up to 37 at boundaries)
Cell Courant number	$\Delta t \leq$ travel time across cell	Minimize numerical dispersion due to time discretization Maximize numerical stability	Not relevant in steady-state calculation
Concentration gradients and hydraulic gradients	Fine cells near ponds and sources	Resolve relatively steep gradients	Yes
Smoothness of discretization	$0.5 < (\Delta x_n{:}\,\Delta x_{n+1}) < 2.0$	Minimize convergence problems	Yes
Total number of calculation cells	Minimize number of calculation cells	Minimize calculation time	8282 cells

The bandwidth can be minimized by numbering nodes sequentially along the shortest grid distance, as shown in Figure 8.8. In this example, numbering nodes sequentially along the horizontal yields a bandwidth of 9, whereas numbering vertically yields a bandwidth of 5. Numbering is more complex in real field studies. An example of grid numbering for a field application is also presented in Figure 8.8. Element numbering may also be done so as to group consecutive elements in the data input file. For example, in layered models, element numbering may follow material layers so that consecutive elements have the same hydraulic properties.

Selecting Time Step Size

Two kinds of time interval are used in models: stress periods (during which boundary conditions are constant and between which boundary conditions vary) and time steps (during which model calculations are made). This section discusses time step selection. Time steps are required for transient calculations. Factors affecting choice of time step include stability considerations, numerical dispersion in transport calculations, time variation of boundary conditions and time-related modeling objectives. In general, the smaller the time step, the more accurate are the predicted results. Too small a time step results in excessive computation time, while too large a time step results in an excessive number of iterations required to reach a mass-balanced solution, and possibly numerical dispersion or instability.

Criteria for selecting time steps for both flow and transport calculations are presented below. A typical approach to selection of spatial and temporal discretization is to use the dispersivity to select mesh size, and to use this mesh size to calculate an appropriate time step.

The time step (dt) that minimizes numerical dispersion can be calculated using the cell Courant number (Co): the ratio of the advective to time-dependent terms in the transport equation. The cell Courant number, and Courant criterion, are defined as the dimensionless ratio (see also Equation 6.8):

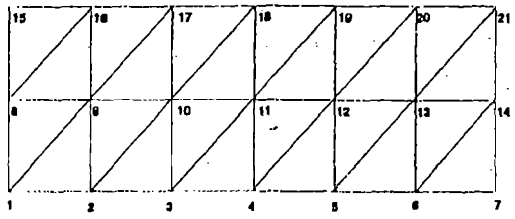
$$\mathrm{Co} = \left| v \frac{dt}{dx} \right| < 1 \tag{8.7}$$

To minimize numerical dispersion and maximize numerical stability, the cell Courant number should be no greater than unity for the smallest cell (Pinder and Gray 1977). This criterion can be interpreted physically as a restriction that transport of a particle across a cell should occur in one or more time steps (Bear and Verruijt 1987). For nonlinear problems the choice of time step is critical because the stability of the solution is even more sensitive to time step size than is the case for linear solutions. Alternate forms of the Courant number have been proposed for unsaturated flow problems (El-Kadi and Ling 1993).

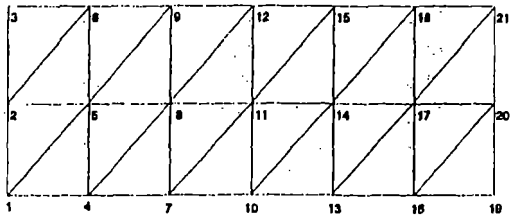
COMPARISON OF BANDWIDTH FOR DIFFERENT NUMBERING SCHEMES

Bandwidth = largest difference between node numbers for an element plus one

Bandwidth = 9



Bandwidth = 5



EXAMPLE NODAL NUMBERING SCHEME

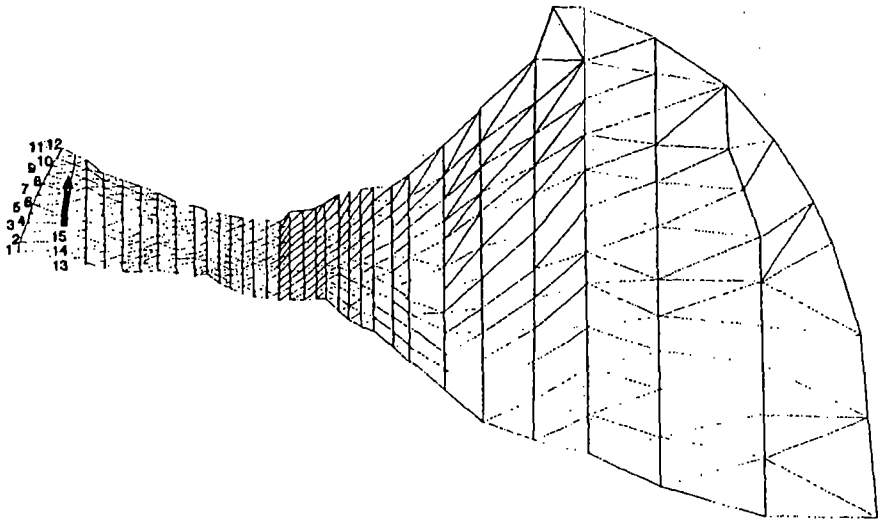
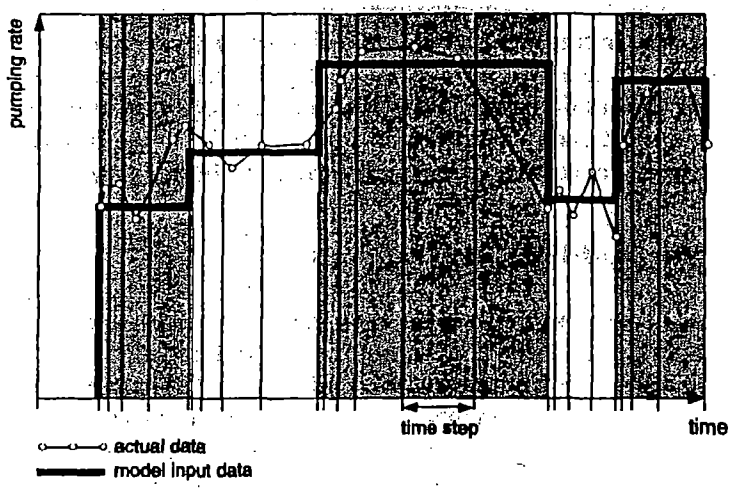


Figure 8.8 Dependency of bandwidth on nodal numbering scheme.

WELL DISCHARGE



HEAD

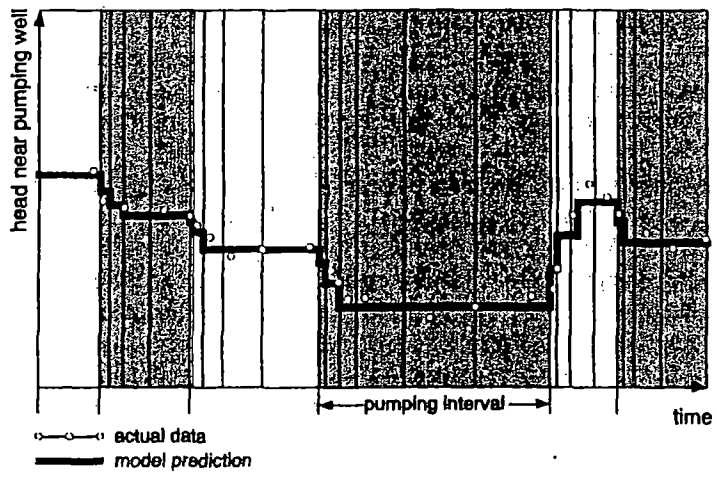


Figure 8.9 Example of time discretization.

Errors arise in the model predictions due to truncation, as discussed in subsection 8.4.1. These errors must decrease over the course of the calculations or the predictions will become unstable. For an unsteady flow problem, solved by an explicit finite-difference approximation, Equation 8.1 presents the condition for stability linking time step and cell size. In addition, time steps should be small during times of changing hydraulic or concentration boundary conditions, such as initiation of water withdrawal (Figure 8.9) or initiation of contaminant sources. Gradual variation of time steps throughout the calculations will facilitate model convergence and improve model stability.

8.4.3 Defining Model Boundary Conditions

A model boundary is the interface between the model calculation domain and the surrounding environment. Boundaries occur at the edges of the model domain and at other points where external influences are represented, such as rivers, wells, leaky impoundments, or chemical spills and so forth. Boundary conditions are expressions of the effect of the external world on the model domain, and they are required to complete the description of a flow or transport problem. The mathematical expression of the boundary condition is required for a well-posed problem. There are three major types of boundary conditions, all of which may vary with time (Table 8.4). These boundary conditions each have differing degrees of constraint upon the model solution, and implications to the ease of developing a balanced model solution.

Understanding Hydrologic Boundary Conditions

Commonly encountered hydrologic boundaries, include surface water bodies, seepage faces, water table, impermeable boundaries, recharge boundaries, and local sources or sinks. Each of these boundary conditions is discussed below.

Surface water bodies, such as rivers, lakes, canals, sea coast, impoundments, and drains, can be represented by prescribed pressure or prescribed flux boundaries. A prescribed pressure (or head) boundary is appropriate if the model calculation cells are small enough to define the volume of the surface water body accurately, the head in the surface water is known, and any intervening hydraulic resistance can be defined. This boundary condition is useful when the groundwater or surface water levels are subject to fluctuation, since the resulting flux between groundwater and surface water will be calculated rather than prescribed. A prescribed flux boundary is appropriate if the flux between surface and groundwater is known.

Seepage faces adjacent to surface water bodies or along drains, tunnels, and the like, may be represented by specification of the head or pressure in the surface water in combination with model cells of high hydraulic conductivity (relative to the aquifer) in the air adjacent to the potential seepage face. Seepage flow may be controlled by the presence of a semipermeable layer, in which case a third-kind boundary condition is applicable.

TABLE 8.4 Model Boundary Conditions

Boundary Type	Boundary Name	Common Applications	Constraints on Solution	Effects of Boundary Condition on Solution
First kind, or Dirichlet boundary	Prescribed pressure, hydraulic head or concentration	Lakes, rivers, springs, constant-head wells, seepage faces	Most constrained	Easiest to solve
Second kind, or Neumann boundary	Prescribed flux of head or concentration	Impermeable boundary, water divide, streamline, infiltration, evaporation, sinks, and sources	Moderately constrained	Moderately difficult to solve
Third kind, or Cauchy boundary	Semipermeable or head-dependent flux	Leaky rivers, drains, seepage faces	Least constrained	Most difficult to solve

The water-table location may be a prescribed pressure boundary in models restricted to vadose-zone or confined-aquifer simulations. The water table is the surface at which pore-water pressure is equal to atmospheric pressure (usually assumed to be zero pressure) or the hydraulic head is equal to the elevation head.

Impermeable boundaries, such as groundwater divides, boundaries parallel to flowlines, or impermeable fault zones, can be represented by a special case of the prescribed flux condition—a zero-flux boundary. Low-permeability zones, such as aquitards, slurry walls, and liners, can be represented by low-permeability cells, with the provision that at least two model calculation cells are used to represent such a zone. This is because of the permeability averaging that occurs in calculating flow between neighboring cells.

Recharge boundaries, such as precipitation, infiltration, evaporation, transpiration, and other areal sources, can be represented by a prescribed flux boundary. This type of boundary may be subject to other constraints, such as excessive infiltration leading to ponding and/or runoff, evaporation being a function of the depth to the water table, or transpiration related to the depth of the root zone.

Local sources and sinks, such as injection or extraction wells, and point sources can be represented by a prescribed flux boundary. However, to approximately represent a point source or sink in a two-dimensional, vertical-plane model, a flux cannot be prescribed without distorting the predictions. An alternative is to use a prescribed head boundary in which a translation from the head in the well to the equivalent head in the model cell (as discussed by Beljian 1988) is performed. In the case of an extraction or injection well open to several aquifers, there are three possible approaches:

- Represent the cased volume of the well by small, highly permeable cells and specify the source/sink cell at the pump elevation.
- Calculate the contribution from each aquifer proportional to the transmissivity of each zone and specify several prescribed flux cells.
- Use an empirical analytical solution to divide pumpage between layers.

In the case of a deep groundwater basin, perhaps up to 3000 m deep, the base boundary condition selected should reflect known conditions and the purpose of the model. For example, a layer of lower permeability, suggested by field data or lithologic or structural characteristics, could be used to define a shallower effective depth of the model (location of zero-flow boundary). In the absence of such a layer, a fictitious depth of the aquifer base would be specified. Boonstra and de Ridder (1981) suggest using, as a first approximation, an effective depth of one-fourth to one-eighth of the average distance between major streams draining the basin.

Figure 8.10 shows an example of the effects of a prescribed head versus a prescribed flux boundary. The model focused on the unsaturated zone. In this case two alternate model domains were used to illustrate that a local model

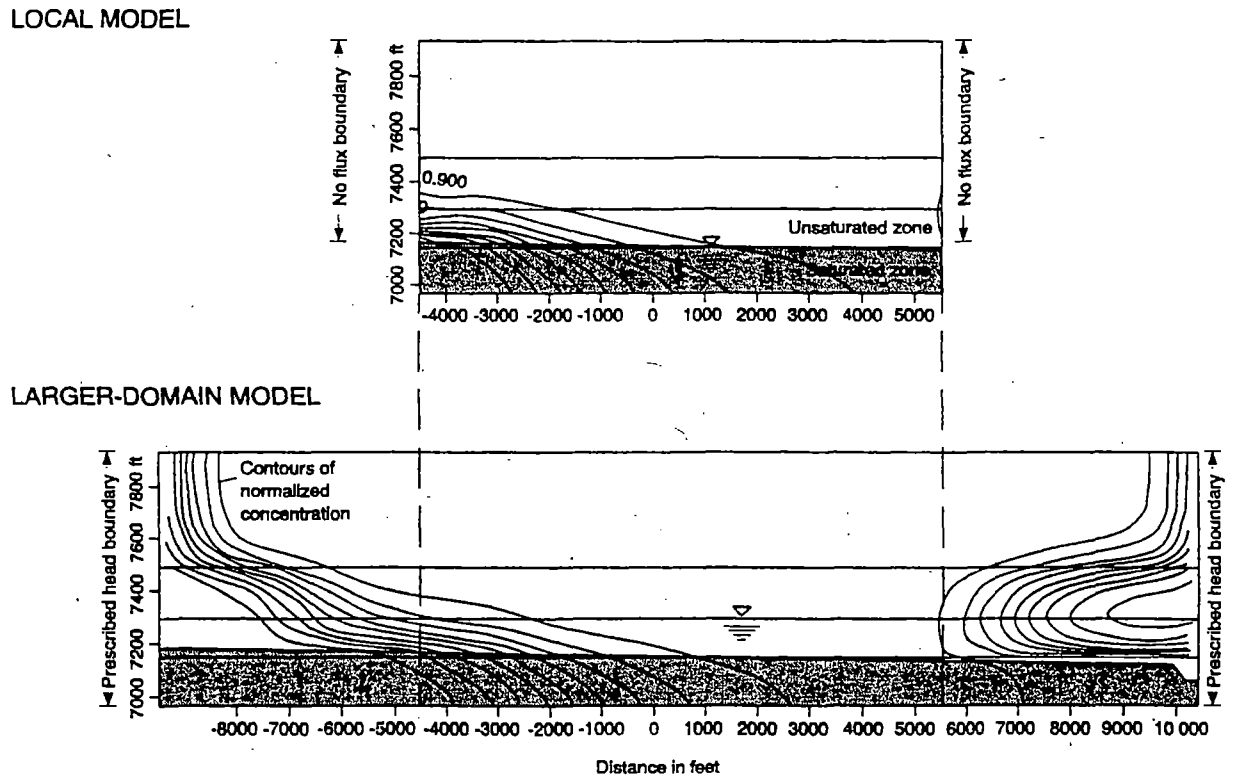


Figure 8.10 Investigating the effect of alternate boundary conditions in flow modeling.

with zero lateral flow boundary conditions in the unsaturated zone produced the same flow predictions as a larger-domain model with prescribed head boundary conditions.

Understanding Transport Boundary Conditions

Commonly encountered concentration boundaries include leaky structures, leachate generation, immiscible contaminant sources (NAPLs), groundwater recharge/discharge, evaporative discharge, injection wells, and surface water bodies. Each of these boundary conditions is discussed in Section 3.4 as well as below.

Leaky structures, such as leaking landfills, ponds, drains, or infiltration beds, can be represented by a prescribed flux boundary with associated concentrations. Similarly injection wells can be simulated by a prescribed flux boundary with a constant concentration in the injected water.

Some materials (residual NAPL, mine waste, etc.) generate contaminated leachate, acting as passive polluters of through-flowing groundwater. Such sources may be represented as prescribed concentration boundaries, with a possible upper limit on the chemical mass that can be mobilized.

Immiscible contaminant sources, such as solvents, coal tar, or paints, can be represented in several ways, two of which are (1) in a multiphase model the contaminant source can be represented explicitly; or (2) in a solute model the source can either be represented by a prescribed concentration boundary or as a prescribed flux boundary, with concentration equal to the pure-phase solubility of the chemical species (or use Raoult's law for solubility in a mixture). If the prescribed concentration boundary is chosen, there is no check on the total mass of solute entering the model domain. The source then is assumed to be infinite during the span of the model simulation. This is often a reasonable assumption because the mass available is usually much greater than the mass dissolving. If the prescribed flux boundary is chosen, then the rate of dissolution of the chemical species must be estimated. The dissolution rate is often a model calibration parameter because chemical to groundwater mass transfer coefficients, immiscible source contact area, and the effect of the groundwater velocity on dissolution rates are poorly understood (Cherry 1990; Mercer and Cohen 1990).

In most situations the effects of a source rather than the source itself may be known or measured. In this case the source may be represented as a black box (effective source) by a prescribed mass flux or concentrations at some downgradient point. The danger of this approach is that the effects of the source may not be fully known and the source thereby misrepresented.

Groundwater recharge/discharge, such as metal-laden recharge from mining areas, geothermal upwelling, recharge from agricultural irrigation or feed lots, or recharge from domestic or industrial areas, can be represented by a prescribed flux boundary with known concentrations.

Surface water bodies can result in clean or contaminated recharge or act as sinks for polluted water. Usually the surface water body can be well repre-

sented by a prescribed head and concentration boundary (see the discussion at the beginning of this subsection). However, in the case of a perched surface water body a prescribed-flux or third-type boundary is appropriate.

Specification of inappropriate boundary conditions will constrain the model solution and result in incorrect predictions (Franke et al. 1987; Frind and Hokkanen 1987; see Figure 8.11). Due to the lack of complete field data about boundary conditions, assumptions will be necessary. These assumptions must be supported by site data and the results of the model. Not all combinations of boundary conditions are acceptable; some will lead to unstable or nonunique solutions. For example, for a steady-state problem, specified-flux conditions on all boundaries will not lead to a unique solution.

Common misuses of boundary conditions include the assumption of a zero-flux boundary at an aquifer/bedrock boundary when in reality significant transport may occur in the bedrock, or specification of prescribed head (or pressure) conditions at many model boundaries resulting in a highly constrained model solution.

8.4.4 Defining Initial Conditions

The initial conditions describe the distribution of heads (or pressures) and concentrations throughout the model domain at the start of the simulation. Errors in initial conditions will propagate through a transient solution, causing unrealistic predictions. The initial conditions for a steady-state simulation are important mainly to save computational effort in reaching a solution. However, the initial conditions for a transient problem strongly influence the predicted results. The initial conditions supplied to a transient run should be the result of a steady-state flow or transient flow and transport simulation of background flow and transport conditions, which will give a mass-balanced starting point. Background conditions will include many of the features of the domain such as long-term pumpage, agricultural recharge, metal-laden upland recharge, or salt-laden geothermal upwelling.

Initial conditions for the unsaturated zone might be the pore pressures calculated based on a steady infiltration or evaporation rate. The assumption of hydrostatic pressures in the unsaturated zone often leads to unrealistic suction pressure and relative hydraulic conductivity, which translates into incorrect water fluxes in the vadose zone.

Specification of an observed concentration distribution (e.g., an interpreted plume based on observed concentrations) as initial conditions for a transient transport simulation often leads to erroneous predictions because a field program rarely measures the highest-occurring concentrations and differing interpolations would lead to widely varying predictions. Therefore it is better to use estimated source terms as a starting point for the transport model, even if the source is not well defined. These sources can be used to calibrate the transport model, and to predict the starting conditions for subsequent model

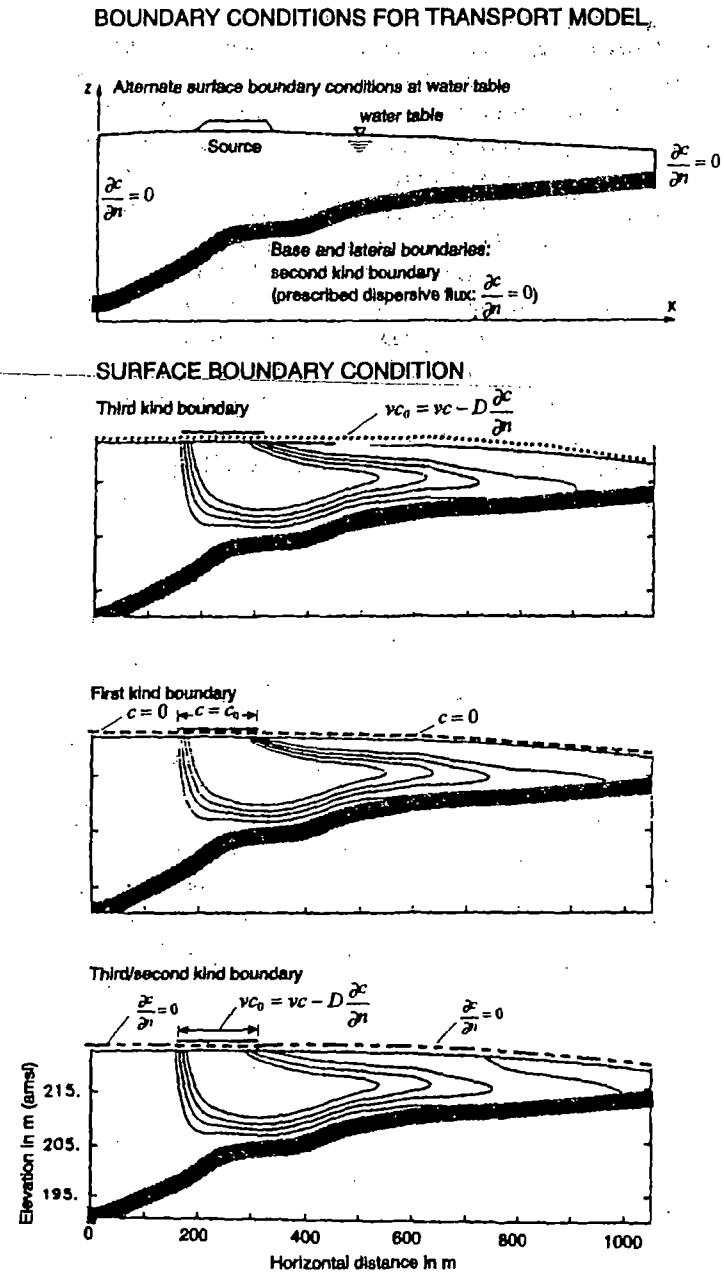


Figure 8.11 Investigating the effects of alternate boundary conditions in transport modeling (after Frind and Hokkanen 1987).

simulations. If the source term is unknown, one approach to source estimation would be to use an analytical inverse model (e.g., Domenico and Robbins 1985; Dale and Domenico 1990) to estimate sources based on the observed chemical plume. Other options are discussed in the next section.

8.4.5 Preparing Model Input Data

The selection and preparation of model input data are discussed in the following sections. The choice of individual model input parameters, based on literature data, is discussed in Appendixes B and C. The following three subsections discuss the types of input data required, how to estimate unknown or uncertain data; and a checklist for model calculations.

Listing Required Input Data

Input data for the model are used for:

- Problem definition (material properties and geometry).
- Numerical requirements (initial conditions, boundary conditions, time-stepping constraints, and spatial discretization).
- Modeling requirements (calibration targets, validation targets, and definition of alternate hypotheses and scenarios).

Input data for a typical, variably saturated variable-density, flow and transport model are summarized in Table 8.1. Typical values for many of these input parameters are presented in Appendixes B, C, and D. Data sources for a site model are listed in Table 8.5.

Estimating Uncertain Data

Field data provide local estimates of conditions, whereas a model requires input of data distributed over the entire model domain. Either model input data are zoned (with homogeneous values within each zone) or they have a continuum of input values. Overall model realism reflects the methods used to estimate model input data from field, laboratory, and literature data. Estimation methods range from use of parameter estimation or inverse models to trial and error methods. Inverse models range from analytical models for contaminant problems (e.g., Domenico and Robbins 1985; Dale and Domenico 1990) to three-dimensional, numerical parameter estimation models (e.g., Hill 1993). The advantages of inverse methods are:

- Results include the mean value and the parameter variance.
- Subjectivity is removed from the calibration process.
- All of the field data are honored.

TABLE 8.5 Common Data Sources for a Site Model

Model Input Parameters	Potential Source of Data[a]
Hydraulic conductivity	Slug, pump and packer tests, and published data
Distribution of hydrogeologic units	Boring logs, geophysics, and geologic maps
Specific storage	Slug and pump tests
Specific yield	Pump tests and porosity data
Recharge/discharge	Precipitation, soil properties, streamflow, pumpage records, elevation, vegetation maps, land use
Unsaturated soil properties	Permeameter tests
Initial water levels, gradients	Field water levels
Background concentrations	Field concentrations
Porosity	Soil analysis
Molecular diffusion coefficient	Published data
Dispersivity	Tracer tests, other field-tested models, or published data
Adsorption distribution coefficient	Batch and column tests, empirical equations for organics, and published data
Soil bulk density	Soil analysis
Density and viscosity	Published data
Contaminant sources	Material inventory, storage, and use, leachate tests, aerial photographs, etc.

[a] Other possible sources include published data on neighboring areas or similar sites.

The disadvantages of inverse methods are:

- Extensive time and computational effort are involved.
- Intuition based on training, experience, and knowledge of the site and other soft data are neglected.
- The model output may be erroneous, unstable or nonunique if the input data are sparse or of differing quality.

The trial-and-error method involves variation of uncertain input data, taking into account observed data, intuition, and analogies with other sites. The trial-and-error approach is the most widely used approach in practical applications. The advantages of this method are:

- Data of varying accuracy may be used appropriately.
- Soft data, such as inferences based on site-specific data, may be reflected in the input data.
- Less effort is required to understand results.

The disadvantages of the trial and error method are:

- The resulting model reflects, in part, the experience (or inexperience) of the modeler.
- There is no simple criterion for assessing satisfactory completion of this task (see Section 8.5).
- The uncertainty in the input data, and its reflection in accuracy of predictions, is not quantitatively defined.

There are a range of methods that fall between the inverse and trial-and-error methods and can be used to augment trial-and-error and parameter estimation. These techniques include geostatistics and kriging, indicator kriging, semiprobabilistic methods, search theory, equivalent media approximations, and observed plume evaluation. An explanation of each of these methods follows.

Geostatistics provides a method for characterizing a parameter distribution. Interpolated data, and the probable accuracy of estimation, can be calculated by kriging the observed data. This method, in its original form, uses the assumption that all data are spatially correlated, with the closest pairs of data being the most strongly correlated. Kriging can be readily undertaken if the data set is relatively homogeneous and smoothly varying, but kriging carries with it advantages and disadvantages similar to inverse modeling. In addition the variogram analysis accompanying kriging will provide information on the correlation length of the data, which is helpful in selecting model cell sizes (see subsection 8.4.2). Reducing interpolation uncertainty through collecting additional data can be assessed in advance, giving a rationale for extending or terminating a field program. Kriging should not be used to interpolate groundwater concentration data because the contaminant distribution typically violates two key assumptions of the kriging method: isotropic and stationary random fields.

Semiprobabilistic methods may involve use of pieces of other methods to suit the problem at hand. The following description provides an example of the application of a semiprobabilistic method. The problem involved model investigation of the effect of sand lenses as preferential flow paths for contaminant migration at a new facility. To identify all the sand lenses by means of a field-drilling program is very costly, but a deterministic modeling approach is not appropriate, since no contamination field data existed to corroborate a trial-and-error method to parameter estimation. Although other methods can be devised to suit differing needs, the method used to estimate model input data for this site was as follows:

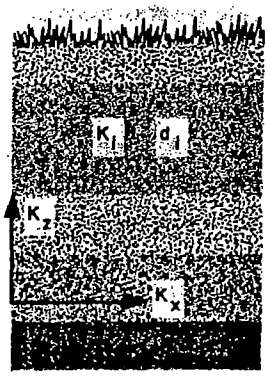
- Borings across the site intercepted multiple sand lenses of varying thicknesses. These lenses did not correlate between borings.

- Sand lens geometry was observed, in road cuts, to follow a pattern of similar length-to-thickness ratios. This ratio was used to select the model cell aspect ratios.
- The typical thicknesses of the sand lenses were known from the existing borehole data. These data were used to generate a histogram of lens thickness versus number of occurrences of that thickness (probability distribution).
- A simple computer program was used to randomly generate material property distributions based on the sand lens thickness histogram, with a cutoff on the number of lenses based on the observed ratio of sand to other material in the site borings.
- A deterministic model for groundwater flow and contaminant transport was run for each random distribution (sometimes called a *realization*).
- Based on the predictions of travel distance from the deterministic model, a histogram of travel distance versus number of occurrences of that travel distance was prepared. Hundreds of realizations would be required to generate a set of results representative of the possibilities at this site. However, random- and stratified-sampling methods can reduce the number of runs needed by about an order of magnitude (McKay et al. 1979). This reduces the number of runs to a manageable number. It was shown that the possibility of a sequence of interconnected sand lenses providing a long travel path was remote.

Search theory or indicator kriging can be used to reduce uncertainty in parameters such as location of a geologic boundary, location of a plume boundary (helpful for capture-zone analyses), and location of a source boundary. That is, parameters, or targets, in which a present/absent or yes/no answer is required. The result of such an analysis is the probability of targets of different shapes and sizes existing, dependent on the distribution of drill-hole data (Savinskii 1965). This method could also be used to evaluate the worth of collecting additional field data (answering in advance the questions of how much the uncertainty will be reduced, and that density of drill holes are appropriate) as is described by Freeze et al. (1990).

Equivalent homogeneous media assumptions can sometimes be used for all or part of the model domain. The problem is to calculate the effective hydraulic conductivity that would result in the same predicted groundwater flow as the actual heterogeneous medium. A variety of methods for different, steady-flow situations are presented in Figure 8.12. These results use the assumption that all data are equally valid. There are no equivalent rules for transient flow, except that the effective hydraulic conductivity has a value that lies between the arithmetic mean and the geometric mean. For example, in an aquifer pumping test the effective hydraulic conductivity at early times approaches the arithmetic mean, whereas at late times the effective hydraulic conductivity approaches the harmonic mean (Gorelick and Hernandez 1990).

LAYERED SYSTEM

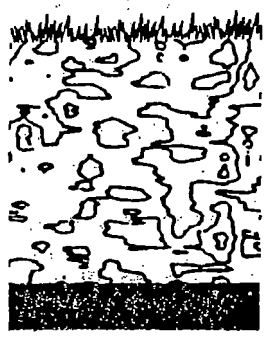


Arithmetic mean $$K_A = \sum_{i=1}^{n} \frac{K_i d_i}{\sum_{i=1}^{n} d_i} = k_x$$

Harmonic mean $$K_H = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{k_i}} = k_z$$

Geometric mean $$K_G = (K_1 K_2 \ldots K_n)^{1/n}$$

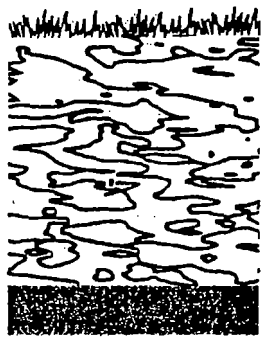
HOMOGENEOUS AQUIFER WITH ISOTROPIC CORRELATION LENGTHS



Two-dimensional uni-directional flow $$K_x = K_y = K_G$$

Three-dimensional uni-directional flow $$K_x = K_y = K_z = K_G(1+\frac{\sigma_Y^2}{6})$$

HOMOGENEOUS AQUIFER WITH ANISOTROPIC CORRELATION LENGTHS



Two-dimensional uni-directional flow

$$K_x = K_G \exp\left[\sigma_Y^2\left(\frac{1}{2} - \frac{1}{1+\lambda_x/\lambda_y}\right)\right]$$

$$K_y = K_G \exp\left[\sigma_Y^2\left(\frac{1}{2} - \frac{\lambda_x/\lambda_y}{1+\lambda_x/\lambda_y}\right)\right]$$

Infinite domain uni-directional flow $$K_H < K < K_A$$

where

λ_x, λ_y = correlation length in x and y

σ_Y^2 = variance of K

Figure 8.12 Equivalent homogenous medium approximations of hydraulic conductivity.

The source term supplied to a model is often controversial and uncertain. As a first approximation, the mass in an observed solute plume may be calculated by estimating the aquifer volume corresponding to a certain groundwater concentration, and then calculating the mass of contaminant in water and soil in that volume, using the following formulas:

$$M_w = C_w n V \tag{8.7}$$

$$M_s = C_s(1 - n)\rho_b V \tag{8.8}$$

where

M_w = mass of contaminant in groundwater in [M]
M_s = mass of contaminant on soil in [M]
C_w = concentration in groundwater in [M/L^3]
C_s = concentration on soil in [M/M]
n = porosity in [l]
V = volume of ground being considered in [L^3]
ρ_b = bulk density in [M/L^3]

This mass, however, is unlikely to adequately represent the source volume because the monitoring program will probably not measure the highest concentrations in the plume, a NAPL plume may exist, and the mass on the soil will be uncertain. In the case of one extensively monitored, contaminated site, only a few percent of the total volume of the contaminant source was calculated from the observed concentrations.

Despite the great interest in immiscible-phase or nonaqueous phase liquid (NAPL) modeling, practical models are still not available (see Table 9.3). In groundwater modeling classes at the Waterloo Centre for Ground Water Research, Cherry (1990) presented the following conclusions about immiscible contaminants:

- Dissolution of immiscible phases is not well understood, particularly in the vicinity of groundwater pumps (i.e., at high groundwater velocities), and as the residual immiscible mass is depleted.
- NAPL may be present in the groundwater despite saturated, dissolved concentrations not being observed because (1) contaminated and clean waters have mixed in the vicinity of wells, (2) the source is heterogenously distributed, (3) the source is becoming depleted, or (4) dispersion has occurred in the aquifer.
- Detailed vertical monitoring may be the key to delineating residual NAPL zones.
- The ratios and relative concentrations of chemicals in the NAPL and groundwater can provide insight into the source characteristics.

Checking Model Calculations

The following general list for checking model calculations is intended to maintain consistency and credibility in model predictions:

- Plot all input data provided to model and check for accuracy and consistency.
- Check the Courant number for appropriate time step size.
- Check the Peclet number for appropriate cell sizes.
- Check model stability and convergence behavior; corrections to previous solutions should ideally monotonically decrease with time after each change in stress.
- Check model flow and solute mass balances.

When checking the model solution's behavior, note that a well-converged flow solution is prerequisite to a well-behaved transport solution. If the model solution oscillates, diverges, or gives unrealistic results for reasons unrelated to factors identified in the list above, then underrelaxing the solution may damp oscillations (at the cost of greater computational effort). If the relaxed model solution still gives inaccurate results, then a sequence of runs in which the model input data are successively simplified will usually lead to a satisfactory solution. For example, a sequence of runs may involve:

- Remove density and viscosity effects.
- Remove unsaturated zones (or change unsaturated characteristics to resemble saturated coefficients).
- Simplify or remove some boundary fluxes.
- Simplify model layering.
- Remove anisotropy.
- Remove heterogeneity.
- Decrease number of dimensions.

At some point in this sequence the model solution will usually improve. The missing complexities can then be added back individually, leading to the source of the problem. Alternative approaches to model problem solving are suggested in Section 8.5.

A poor flow mass balance will usually result in a correspondingly poor solute mass balance. The size of an acceptable mass-balance error depends on the situation being simulated and on the objectives of the modeling analysis. However, flow mass errors of less than 1% and solute mass errors of less than 5% are commonly accepted.

A model embodies many disparate pieces of data and assumptions. While individual assumptions may appear reasonable, the net effect of the assumptions may be unrealistic.

8.5 CALIBRATING THE MODEL

Model calibration, step 5 of preparing the model, is the process of varying uncertain model input data over likely ranges of values until a satisfactory match between simulated and observed data is obtained. Calibration is needed to account for unmeasured, unknown, or unrepresented conditions or processes and uncertainty in measured input data. If an inverse model is used for parameter estimation, then this step of the modeling process is partially automated. However, if data need to be extrapolated, or the field data are unsuited to automatic calibration, then the more traditional approach of trial-and-error calibration is needed. Model testing prior to application in predictive mode is often split into two processes: calibration and validation. The field data are split into two data sets, often for different time periods at one site; one data set is used for calibration and the other for validation. Model validation is described in Section 8.6.

The general approach to the numerical modeling process is illustrated in Figure 8.1. The model calibration step requires the greatest effort. The calibrated model may evolve in a variety of ways. For example, the modeling analysis might begin with incomplete field data, and the model may be refined as additional data become available.

Alternatively, the model input data could contain some well-defined parameters and some highly uncertain data. Varying these uncertain data is the starting point for the calibration process. Typically parameters should only be varied within measured or likely ranges, parameter distributions should be limited to geologically feasible hypotheses, and parameter values in areas beyond the extent of field data can be varied the most.

It is quite possible that alternate, equally statistically valid, model calibrations can be developed from a single dataset (Brooks et al. 1994). This is most likely to happen in cases where minimal data are available and/or the model extends significantly beyond the area adequately characterized by field data. An example of the model calibration process is described in Appendix A.5. Model calibration usually involves most of the following steps, in the order listed:

- Specify calibration criteria and calibration and/or validation protocol. This step is often required in controversial or sensitive modeling applications. General expectations for model predictions should be developed before calibrating the model. Calibration criteria compare model-prediction errors with key components of the model mass balance. That is, a discrepancy between predicted and observed heads is compared to a key hydraulic gradient, as described below. Or, a discrepancy between predicted and observed concentrations is compared to the variability in observed concentrations in neighboring monitor wells. Model performance criteria might be:

  Paired-data testing (i.e., comparing predicted and observed values for corresponding locations in time and space). Common examples of

such testing are *variance* between predicted and observed data should be less than about 10% of the range of observations, *standard deviation* between predicted and observed data should lie in the range 0.7 to 1.0 depending on the number and quality of data points, and *bias* between predictions and observations should be random rather than systematic.

Averaged paired-data testing (i.e., the predicted and observed values are averaged over space and/or time and then compared).

Frequency distribution testing (i.e., predicted and observed cumulative frequency distributions are compared).

- Calibrate the flow model before calibrating the flow and transport model (except in cases of density-coupled transport, in which the flow and transport calculations are inextricably bound together). Accept only convergent, stable, and well-balanced model calculations.
- Simulate natural background conditions (regional recharge, geothermal upwelling, natural mineralogic dissolution, etc.), predict the water levels and concentrations corresponding to preexisting conditions, and compare predictions with any available observations.
- Modify model assumptions and/or uncertain input data, within reasonable bounds, to obtain a realistic simulation. Specify model input data in ranges of values. Note the accuracy of these data so that changes made during the calibration procedure will concentrate on the most uncertain data while remaining within realistic bounds.
- Predict transient flow and transport conditions for the period of development up to the present (this process is also known as *history matching*). Ideally the transient calibration period should be as long as, or longer than, the period of future predictions to which the calibrated model will be applied.
- Evaluate the model predictions versus historical observations (Figure 8.13) The model evaluation should use as many pieces of information as possible (i.e., not just water levels and concentrations but also spring levels, river in/outflows, vertical hydraulic gradients, river concentrations, and any other relevant descriptive data).
- Decide whether additional model refinement is needed based on the calibration criteria you specified.
- Examine "calibrated" model input and output and evaluate whether:

  Input data individually and jointly make sense.

  Site-specific data cover the area predicted to be of concern.

  Model output indicate initial conceptualization was appropriate.

During model calibration model input data or assumption modifications should start by varying the most uncertain parameters or those that are the most critical to the predictions (e.g., hydraulic conductivity for flow or adsorption distribution coefficient for transport). It is a waste of effort to sys-

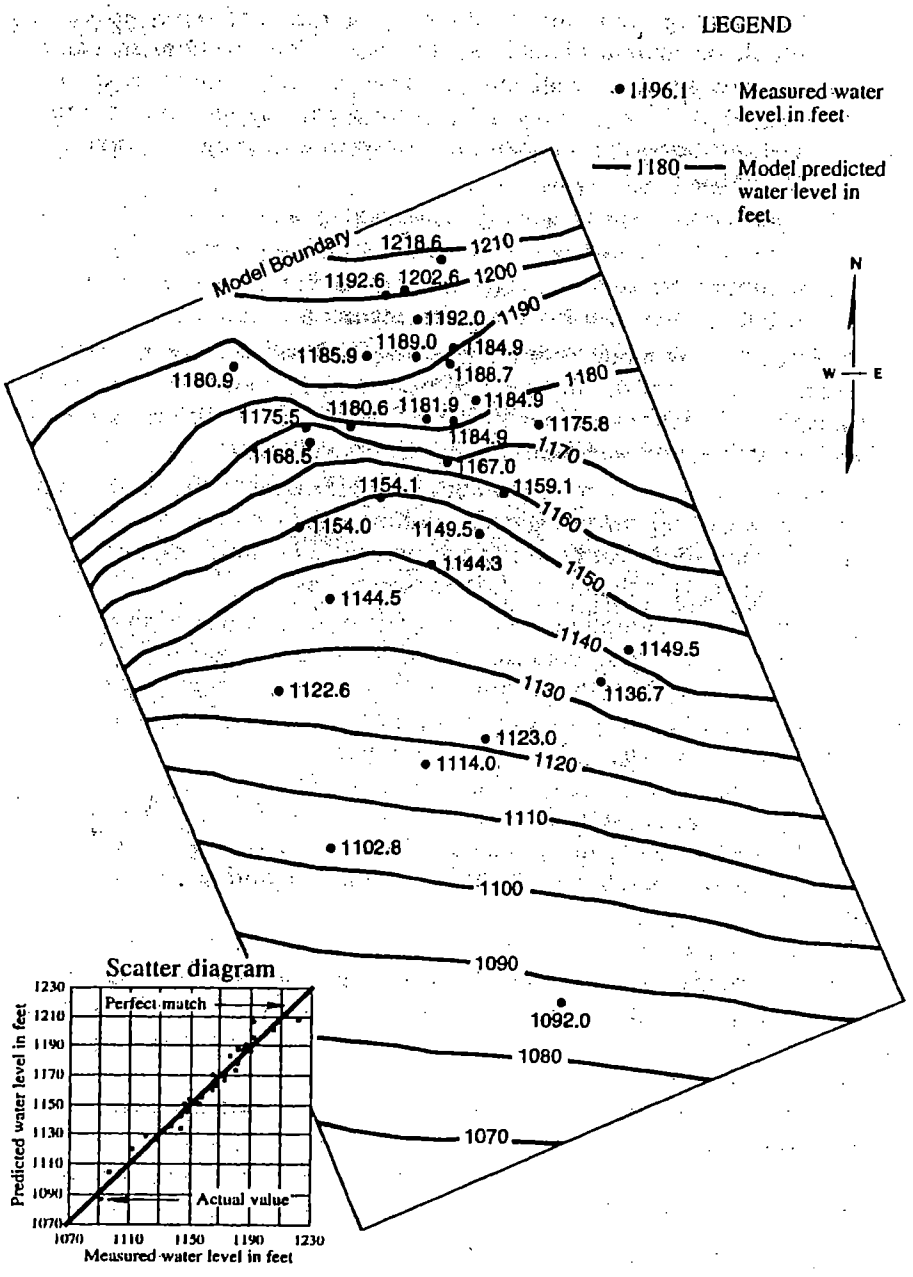


Figure 8.13 Measured and predicted groundwater levels.

tematically vary all parameters during calibration; instead, the most promising combinations of parameters should be tested and refined.

If the model predictions fail to meet expectations, then a model error analysis may be used to assess overall predictive bias (see Section 8.6), or, for more extreme cases the entire underpinnings of the model may have to be re-evaluated. This requires an assessment of modeling assumptions and errors.

There are five general sources of model errors:

- Mathematical model errors. These errors involve the physical and mathematical basis of the computer code, and its numerical framework. This framework should be appropriate to the situation to be simulated, with inherent assumptions honored.
- Conceptual errors. Among these are misconceptions about the governing mechanisms, boundary conditions, sources, and dimensionality of the problem (see Section 8.2 and Chapter 7).
- Input data errors. Input errors include mistakes in data entry (which should be checked by plotting all inputs with a nonsmoothing contour-plotting package), sets of assumptions about data that in combination do not make sense, measurement error, and levels of heterogeneity that either have not been identified and characterized in the field or cannot be represented in the model. The last two sources of error are unavoidable and should be taken into account when interpreting the model predictions.
- Numerical errors. Examples include truncation errors due to truncation of the Taylor series expansion of the finite-difference or finite-element formulation of the governing equations, roundoff errors due to the precision of numbers stored by the computer, and numerical dispersion due to discretization (see Section 6.2).
- Interpretation errors. These can take the form of misunderstanding of the predicted results particularly in the case of models without post-processors capable of providing meaningful summaries, analysis, and graphic output), misconception of the expected results (e.g., prediction of contaminant migration at an angle to groundwater flow), and comparison of spatially and temporally averaged model predictions with point observations.

If consideration of all of these sources of error does not reveal the source of the problem, then the process of model simplification described in Subsection 8.4.5 on checking model calculations can be used to assess the model and probably uncover problem areas. The discrepancies between model predictions and observed values can uncover factors missing from the model; see Section 8.7 for an expansion of this idea.

As is the case with weather forecasting, a model that does not meet calibration criteria is still useful. It can be used to assess the interactions of the mechanisms governing groundwater flow and transport at a site, to help to

develop a better overall understanding of site conditions, and to assess the need for additional data. It cannot, however, be used to predict remedial design or to show regulatory compliance.

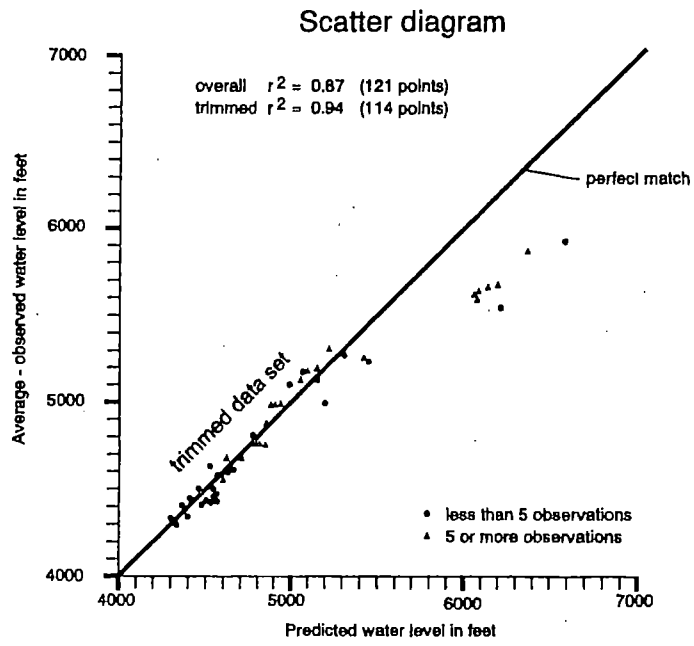
8.6 ANALYZING MODEL ERROR

The objectives of model error analysis are to quantity how well the model simulates the physical system and to identify problem areas in the model. The most basic measure of model error is the difference between predicted and observed values. It has been stated in U.S. Congressional Hearings that groundwater transport models cannot be expected to give better than order of magnitude accuracy. In specific model applications the predictive capability of the model depends on:

- The nature, extent, and reliability of data available with which to define the model.
- The intrinsic capabilities of the mathematical model to represent site conditions.
- The realism of the conceptual model used to prepare the site-specific model.
- The degree of bias introduced by assumptions made during the modeling process.
- The time and volume over which model predictions are used, as compared to the calibration and validation volumes and time periods.

The degree of uncertainty in data considered useful by geologists and hydrogeologists is often orders of magnitude greater than the data accuracy imposed by design constraints. Therefore there is often a discrepancy between the accuracy of data supplied to a model and the accuracy expected from model predictions. Once again, the model purpose will partly determine the level of acceptable error in a model.

The method typically used to quantity model error is to compute the difference between predicted and observed values (residual) at a point (e.g., monitoring location) and evaluate these differences. Other methods of comparing predicted and observed data have been described in Section 8.5 and in ASTM Standard D 5490-93 (ASTM 1993). The residuals may be illustrated as scatter diagrams of predicted values versus observed values (Figure 8.14) or histograms of residuals (Figure 8.14). The scatter diagrams, together with the computed coefficient of determination, indicate where the greatest discrepancies occur and whether there are a few major discrepancies or general disagreement between predictions and observations. Ideally a histogram of residuals should be normally distributed around zero, with an average residual of zero. If the histogram is skewed, the implication is that the model consistently over- or underpredicts the variable of concern. The histogram



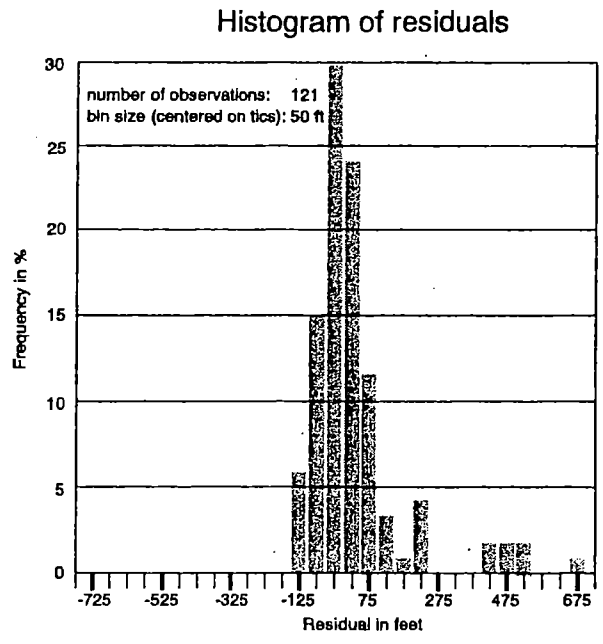


Figure 8.14 Model error analysis.

also indicates the percentage of residuals that are unacceptably large. A map of the residuals (which may be contoured) can be used to relate errors in prediction to the hydrogeolgic setting and indicate problem areas. Ideally residuals should be randomly distributed.

Other measures of the goodness of fit between predictions and observations (Loague and Green 1991) include

Maximum error (ME),

$$\mathrm{ME} = \max \left\{ |P_i - O_i| \right\}_{i=1}^{n} \tag{8.9}$$

Root mean square error (RMSE),

$$\mathrm{RMSE} = \left[\sum_{i=1}^{n} \frac{(P_i - O_i)^2}{n} \right]^{\frac{1}{2}} \left[\frac{100}{\bar{o}} \right] \tag{8.10}$$

Coefficient of determination (CD),

$$\mathrm{CD} = \frac{\sum_{i=1}^{n} (O_i - \bar{o})^2}{\sum_{i=1}^{n} (P_i - \bar{o})^2} \tag{8.11}$$

Modeling efficiency (EF),

$$\mathrm{EF} = \frac{\sum_{i=1}^{n} (O_i - \bar{o})^2 - \sum_{i=1}^{n} (P_i - O_i)^2}{\sum_{i=1}^{n} (O_i - \bar{o})^2} \tag{8.12}$$

Coefficient of residual mass (CRM),

$$\mathrm{CRM} = \frac{\sum_{i=1}^{n} O_i - \sum_{i=1}^{n} P_i}{\sum_{i=1}^{n} O_i} \tag{8.13}$$

where

O = observed value
$\bar{o}$ = mean observed value
P = predicted value
n = number of values

ME, RMSE, and CRM tend to zero and CD and EF tend to one for perfect predictions. RMSE, EF, and CRM are relative measures useful to compare the calibration of alternative model runs. ME and CD are directly useful and more commonly used.

The degree of model error that is acceptable depends on several factors:

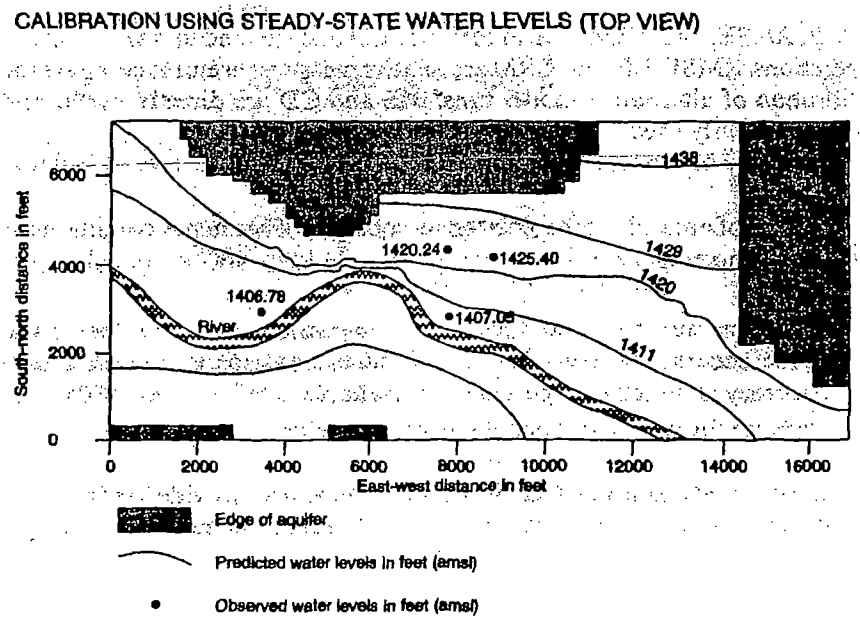
- Degree of natural heterogeneity or complexity of boundary conditions. A highly heterogeneous or complex domain may not be modeled precisely.
- Location, number, and accuracy of measurements. The "perceived model accuracy is inversely proportional to available data, actual model accuracy is directly proportional to available data" (Pinder 1990).
- Purpose for which the model has been developed.

Model sensitivity analysis (see Section 8.8) may also provide feedback on model error/predictive uncertainty due to uncertainty in model inputs.

8.7 VALIDATING THE MODEL

Model validation (sometimes called verification), the seventh step, is the process of demonstrating that the calibrated model is an adequate representation of the physical system. Model validation is a shortcut to gaining greater confidence in model predictions in the absence of uncertainty analyses. Validation is more common in hydrologic modeling for which time-history data are often available. ASTM (1993) discusses validation at length, including concepts of global versus local validation. However, ASTM (1993), while recommending validation to reduce the problem of nonunique solutions, also states that a calibrated but unvalidated model may be used for predictive analyses in conjunction with a careful sensitivity analyses. This step is rarely addressed in limited modeling studies because of the extra data and modeling effort required. However, in the absence of validation a model is untested beyond the exact conditions used in the calibration, and use of the model to make other than general predictions is questionable. A successfully calibrated model cannot be relied upon to provide accurate predictions in all cases (Freyberg 1988). Four possible approaches to model validation, the fourth method requiring the least effort on the part of the modeler, are listed below.

- Successfully predict alternate conditions. An example of this method is to calibrate using steady-state data and validate using transient data (Figure 8.15). This method has the advantage that the comparison data sets are independent, but additional parameters (such as storage parameters and fluxes) come into play in the transient calculations that may invalidate an



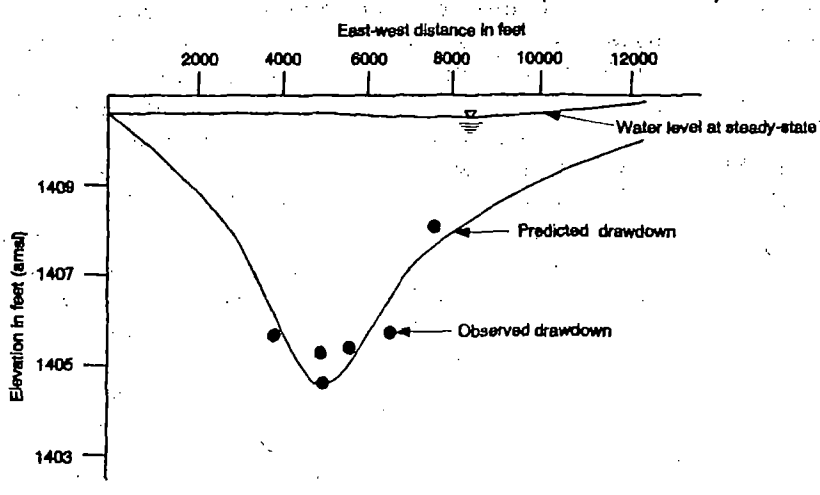


Figure 8.15 Validation using transient data.

apparently realistic steady-state model. An alternate approach is to use a part of a transient data set in calibrating the model, and the remainder of the data set, possibly subject to differing boundary conditions, in validating the model. This is the most commonly applied approach to model validation.

- Successfully predict existing conditions. In this method model validation uses comparison data not employed in the calibration process (Figure

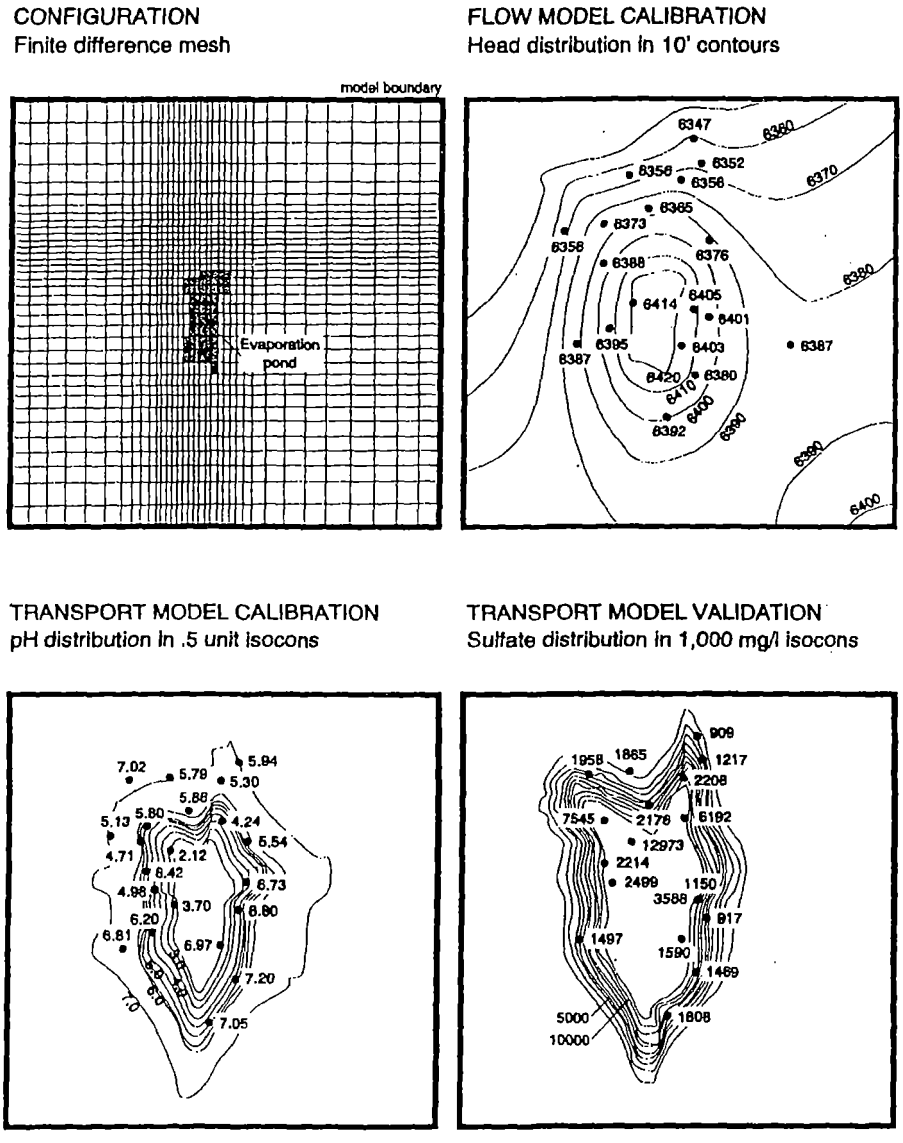


Figure 8.16 Validation using comparison data not employed in model calibration.

8.16). This method is the most useful but can only be used if there are sufficient data to describe the entire domain satisfactorily using half the available data; this is rarely the case. Even when there are sufficient data, this method is not ideal, since the calibration and validation data sets may not be entirely independent.

- Compare the model predictions with the results of other models of the identical situation (Figure 8.17). This method validates the numerical

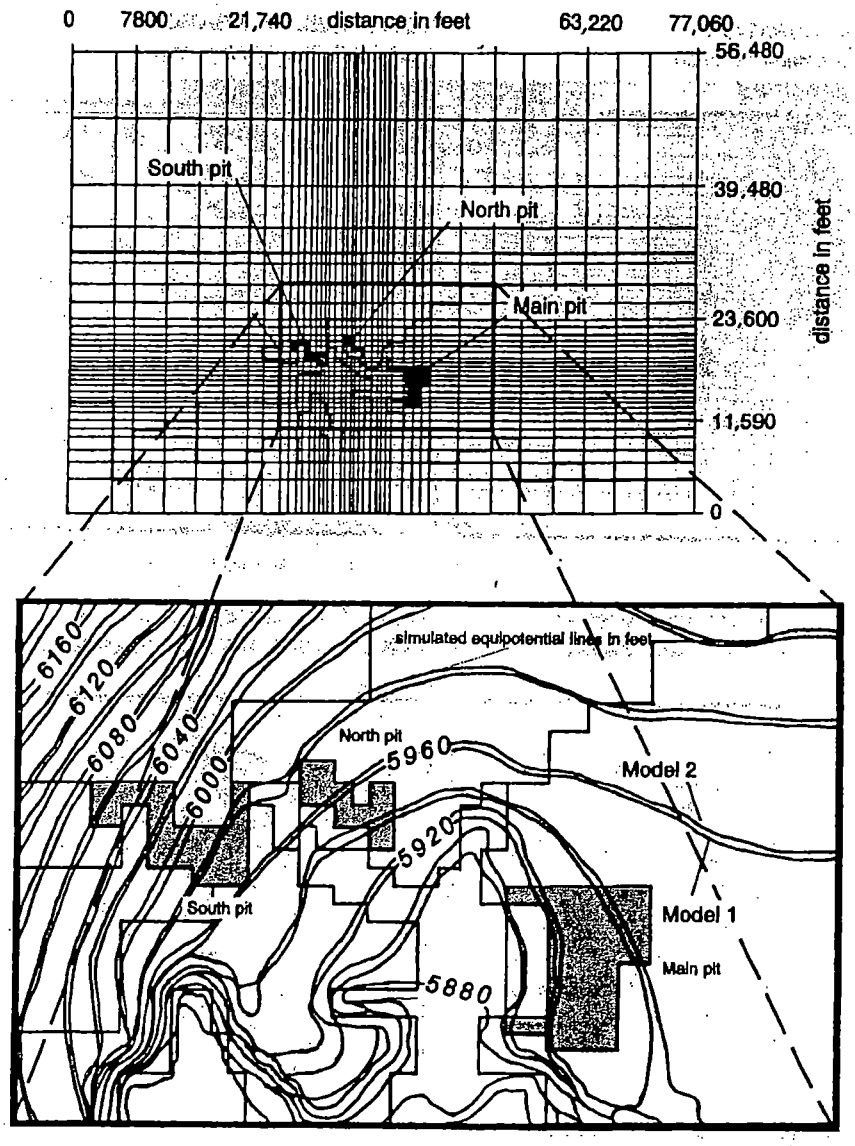


Figure 8.17 Validation by comparing different models applied to mine dewatering and reclamation.

model but not the conceptual model, since the same assumptions, boundary conditions, and input data are employed in the two sets of model simulations.

- Predict conditions for locations beyond the existing monitoring network or at future times, to validate the model when additional fieldwork is undertaken (Figure 8.18). This is the most persuasive argument in favor of a realistic model, and the most practical method for validating a transport model. This method requires that modeling analyses and fieldwork be conducted in parallel over a period of time.

In each of the above methods, you must consider measurement errors, precision, and completeness of the comparison data set when evaluating the model.

8.8 DEMONSTRATING MODEL SENSITIVITY

Model sensitivity analysis is the eighth step in the modeling process. The purpose of sensitivity analyses is to demonstrate the model responses to variations in uncertain input parameters. The model response to these variations is of interest because the range in the resulting predictions illustrates the level of model prediction uncertainty and, given a sensitivity case with results as statistically valid as the calibrated model, the nonuniqueness of the calibrated input data set. A systematic sensitivity analysis provides sufficient data to rank the input parameters in terms of their influence on the predicted results. The results of a sensitivity analysis can be used to:

- Identify sensitive input parameters for the purpose of guiding additional field data collection and, perhaps, focussing calibration efforts
- Define parameters to be used in uncertainty analysis.

A typical sensitivity analysis involves the following steps:

1. Assemble the input data together with their ranges of uncertainty. These ranges of values should have been assessed during the model calibration stage. The ranges may be based on the greatest extremes estimated from the:
   - Variation of observed values
   - Uncertainty in measurement
   - Range in literature values for similar conditions
   - Range in historical conditions
2. Rerun the calibrated model with each of the input parameters (ideally including grid size see Figure 8.19, and time step) individually varied to their maximum and minimum values. An ad hoc sensitivity analysis

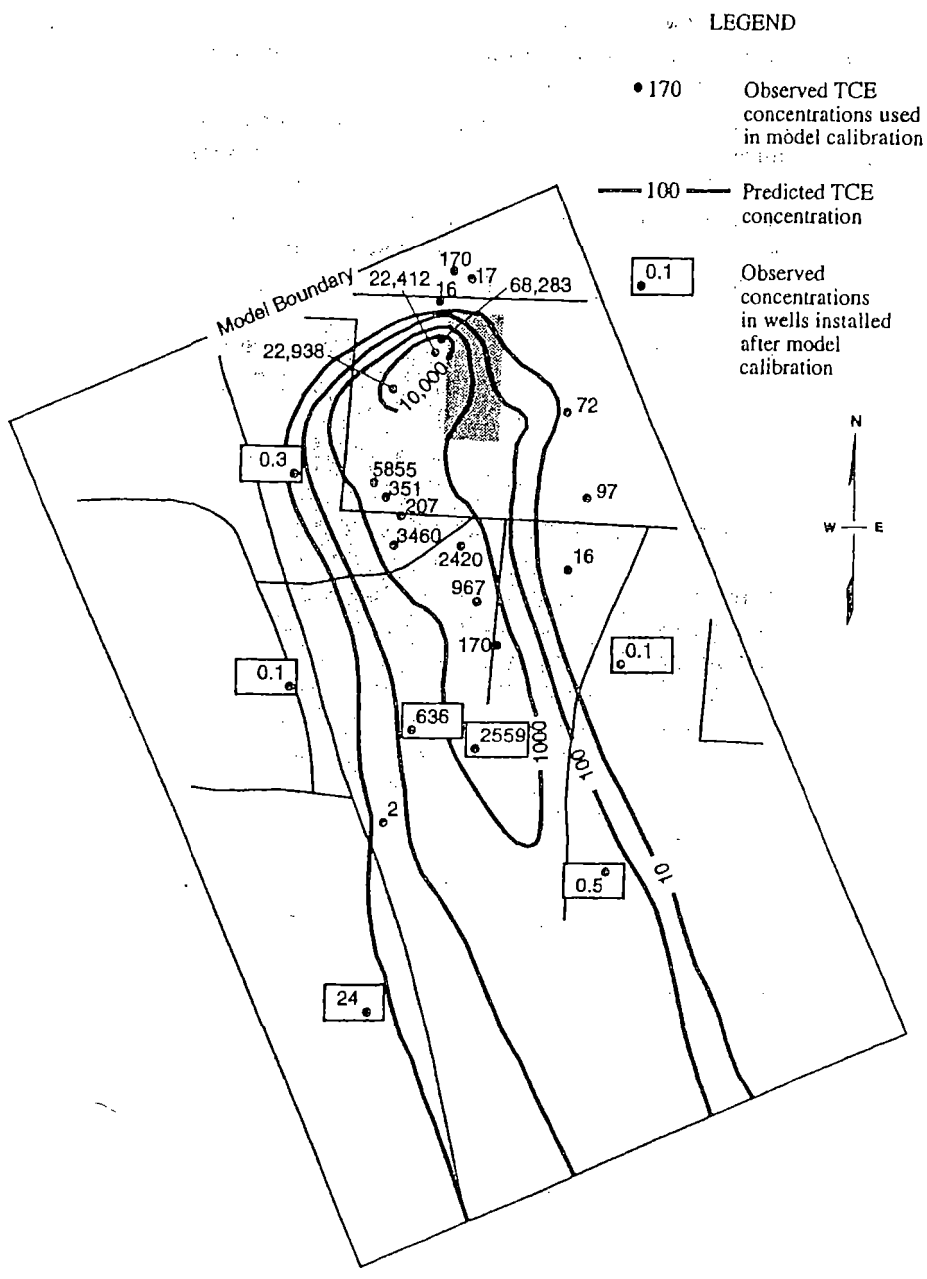
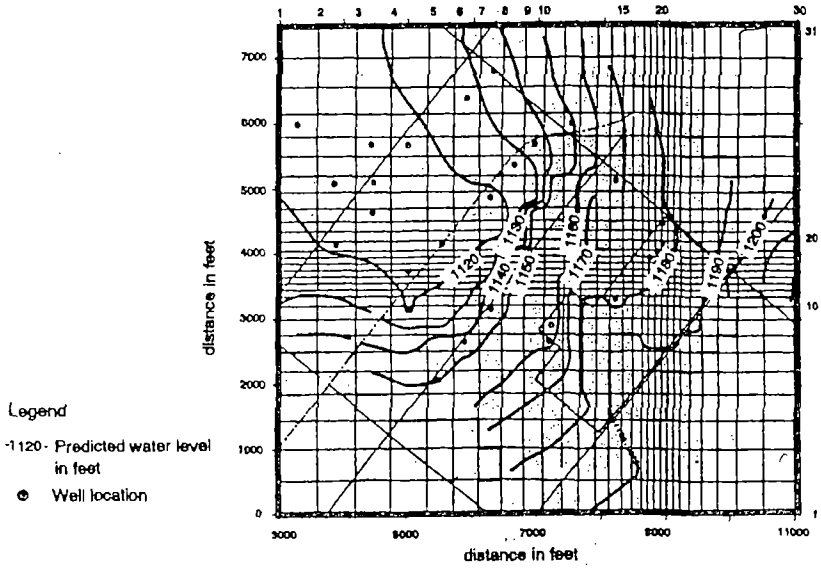


Figure 8.18 Model validation by performing model audit.

COARSE-GRID MODEL (930 cells)



FINE-GRID MODEL (4,672 cells in area of the above model)

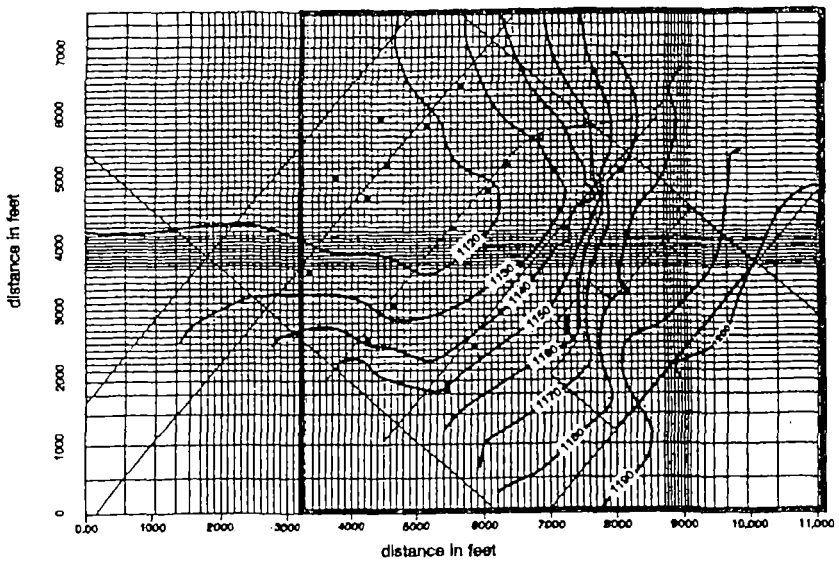


Figure 8.19 Sensitivity of predictions to grid size.

will have been undertaken during the calibration process, but a more rigorous sensitivity analysis is based on the calibrated model. It is not advisable to vary several parameters at once to achieve the "worst case," since this case is usually both highly unlikely and leads to noncalibrated model predictions.

3. Compare the predicted results and interpret in terms of ranking of input data and uncertainty in predictions. The results of sensitivity analyses can be normalized so that the effects of different parameters can be compared quantitatively. For example, a sensitivity index S can be defined

$$S = \frac{|dh|}{(dP/P)} \tag{8.14}$$

where

S = normalized sensitivity index which is a measure of the average change in the predicted variable per fractional change in the input parameter

$|dh|$ = difference in predicted variable, at one or more key locations, between the base case and sensitivity case

dP = change in input parameter value

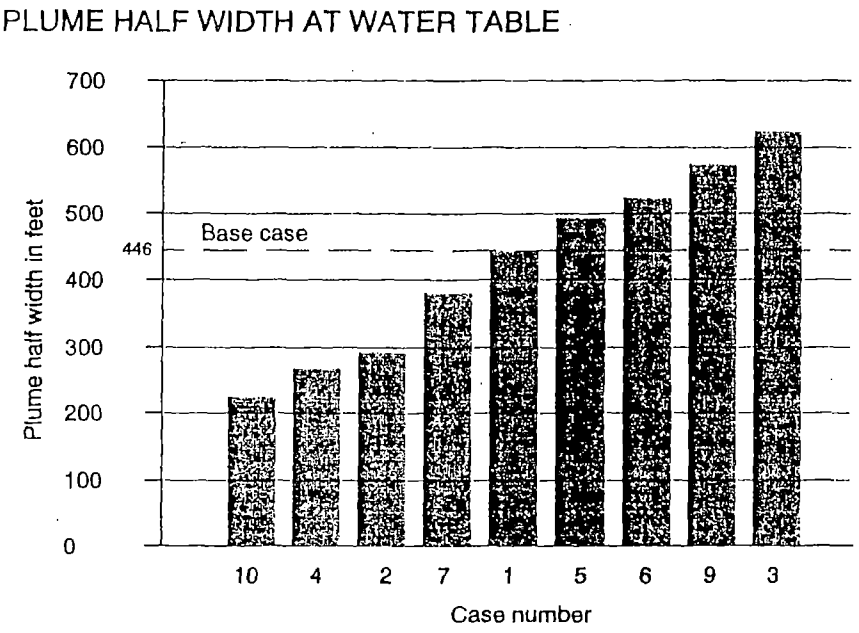
P = initial input parameter value

The S values so calculated can be numerically ranked. These results will be biased if the ranges of input parameters analyzed are so conservative as to be unrealistic.

Several ideas for presenting sensitivity analyses are shown in Figures 8.20 and 8.21. For example, the results shown in Figure 8.20 suggest that the input parameters should be ranked in importance in this order: dispersivity, hydraulic conductivity, and anisotropy of hydraulic conductivity. The results in Figure 8.21 suggest that ranges in predicted concentrations, for current conditions, of up to an order of magnitude may arise through variation over likely ranges of the input parameters. As a result predicted concentrations are considered to be accurate to an order of magnitude.

The results of the sensitivity analyses occasionally indicate that the calibration parameter data set is not unique. In this case further runs, error analysis, or a search for additional data to help select between the cases is warranted.

Some parameter estimation models (Hill 1993) are also designed to automate the sensitivity analysis process. Viewing interim results sometimes changes the course of a modeling study, but in general model sensitivity analysis is an easily automated step.



SUMMARY OF CASES EXAMINED					
Case No.	Parameter	Plume half width in feet	Case No.	Parameter	Plume half width in feet
10	Low dispersivity	227	3	High conductivity	628
4	Low vertical to horizontal conductivity	266	9	High dispersivity	577
2	Low conductivity	295	6	1 : 100 Horizontal to vertical conductivity	522
7	Low conductivity interbeds	383	5	1 : 10 Horizontal to vertical conductivity	494

Figure 8.20 Example of summarizing sensitivity analysis.

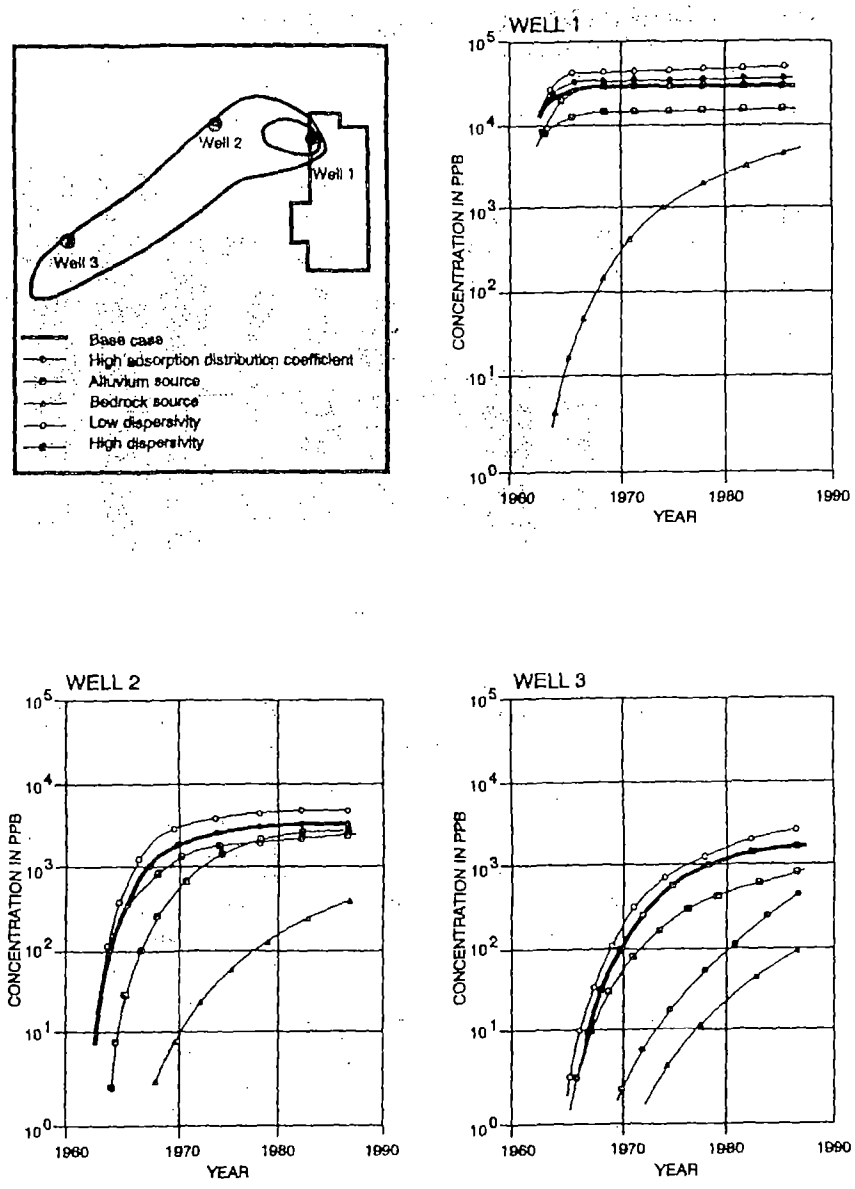


Figure 8.21 Example of illustrating transport parameter sensitivity analysis.

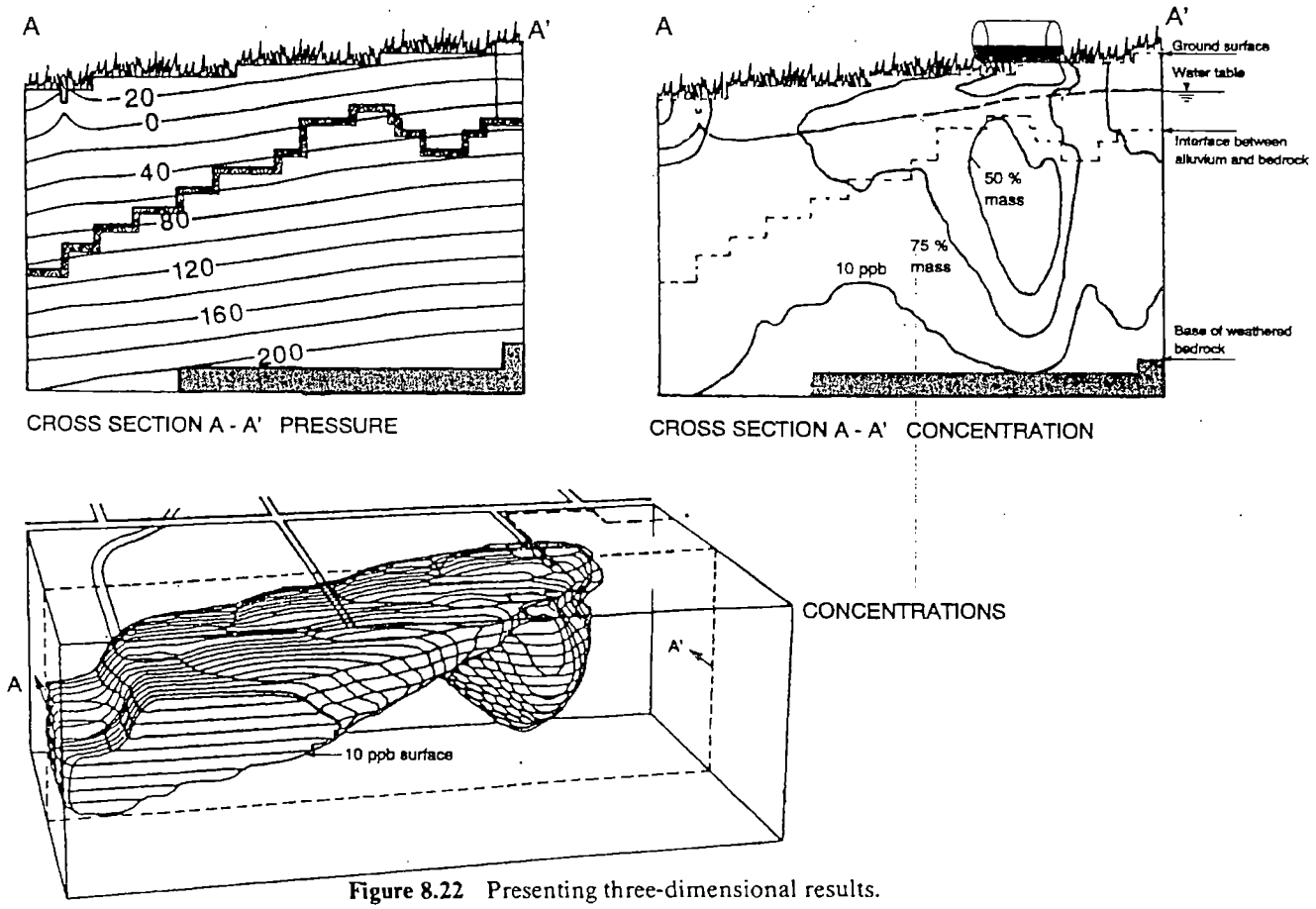


Figure 8.22 Presenting three-dimensional results.

8.9 CONDUCTING MODEL PREDICTIONS

Step nine in the model process is conducting the model predictions. The model predictions are often considered the main purpose of the modeling exercise, although there may be many more purposes to a model than straightforward prediction. The assumptions to be used in the predictive analyses should be well defined. The designer, or end user, of the model results may not understand this aspect of the model. For example, the assumption of a continuing source is not specific enough. Assumptions about the location, concentration, volume, phase(s), accompanying species, and possible depletion of the source also need to be addressed.

If remedial schemes are to be simulated, then the model may be used to predict the capture zones for different schemes, concentrations at any extraction points, time for remediation, concentration time histories at water-supply or compliance points, contaminant mass balances, and ranking of alternate remedial schemes in terms of effectiveness.

Since model predictions are used to generate specific predictive values more often than relative results, concern in the reliability of the predictions is warranted. As model predictions extrapolate over time, the time period of model predictions should be comparable to the period of model calibration. For example, a model that has been calibrated over a 1-year period against water-quality data is not likely to be reliable for 100-year transport predictions. An uncertainty analysis would be helpful in quantifying model reliability under these conditions.

The interpretation of the model predictions should include the modeler's assessment of where the model is more or less accurate and the relative degree of uncertainty in the predictions.

8.10 PRESENTING MODEL RESULTS

Often the last step in the modeling process is writing the report. The presentation of the model results greatly influences the utility of the predictions. It is useful to present tables summarizing the path of development of the model (Table 8.6). The predicted results themselves may be presented in many forms, including three-dimensional graphics (Figure 8.22), concentration time histories (Figure 8.23), vertical cross sections (Figure 8.24), vector illustrations of groundwater flow (Figure 8.25), or comparison of alternate remedial schemes (Figure 8.26). Ideally each figure should be self-explanatory. This means that an explanatory map, site-specific features, and an explanation of the case simulated should be included. Presentation of normalized results (e.g., contours of concentration relative to a source concentration of 1.0) may be used to unify predictions for many contaminants onto one figure.

Due to recent advances in computer hardware and software, there are few restrictions on the model users' imagination for presenting model results. Color plots, as shown in Figures 8.27 to 8.30, may convey the results of complex models in a more informative way for a general audience.

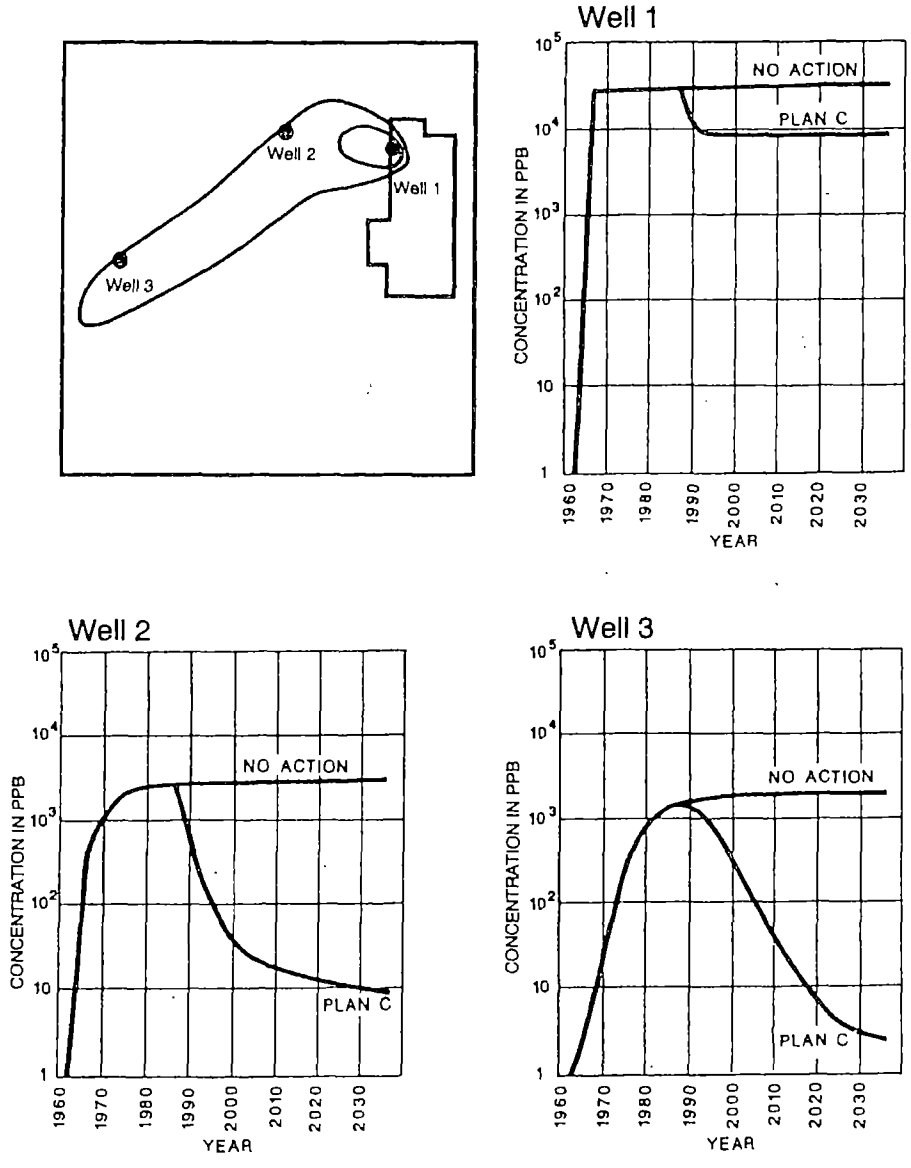


Figure 8.23 Presenting predicted concentration time histories.

TABLE 8.6 Summary of Model Runs

Description and Date	Reason for Performing Calculation	Data Summary[a] K_x (ft/day)	K_y (ft/day)	K_z (ft/day)	n (1)	S_s (1/ft)	K_d (ml/g)	D_L (ft/day)	D_T (ft/day)	Assumptions Summary	Comments on Results	Suggested Next Stage
Case 8 16/10/92	Calibrate flow model	—	—	—	—	—	—	—	—	Zero recharge	Hydraulic heads even lower than case 7. Same conclusions. Mass imbalance 3%.	Revise alluvium hydraulic conductivities to reflect results of new pump tests.
Case 9 17/10/92 flow only	Update model hydraulic properties and investigate influence of hydraulic conductivity on predicted flow patterns	50.0 50.0 0.005 0.05	50.0 50.0 0.005 0.05	0.5 0.5 0.005 0.05	0.28 0.28 0.01 0.01	$5\ E^{-4}$ $5\ E^{-4}$ $1\ E^{-4}$ $1\ E^{-4}$	0.0 0.0 0.0 0.0	100.0 100.0 100.0 100.0	10.0 10.0 10.0 10.0	Recharge assumption as for case 6	Water table drops indicating that surface recharge is not compatible with the revised alluvium K values. Mass balance 1%.	Using best-case hydrodynamics case run transport calculations.

Not applicable

Case 10 26/10/92 hydro-dynamics and transport	Using best-case hydro-dynamics to date. predict and compare TCE concen-trations	7.0 5.0 0.005 0.05	5.0 7.0 0.005 0.05	0.05 0.05 0.005 0.05	0.28 0.28 0.01 0.01	$5\ E^{-4}$ $5\ E^{-4}$ $1\ E^{-5}$ $1\ E^{-5}$	0.0 0.0 0.0 0.0	100.0 100.0 10.0 10.0	10.0 10.0 10.0 10.0	Source 2 (for TCE) only: source assumed active 1961 to 1974 incl. Total of 10,000 gals. assumed disposed.	Predictions show deep migration of plume. Influence of alluvium anisotropy and geometry of alluvium/ bedrock inter-face can be noted. Depth of plume reflects supplied base head boundary conditions.	Revise bedrock rock-type configuration to reflect latest subcrop map. Also refine base head boundary conditions.

[a] Data listed are presented in the following order: (1) General alluvium. (2) high-conductivity alluvium. (3) tertiary bedrock. (4) Precambrian bedrock.

Note:

K_x = horizontal hydraulic conductivity along x-axis in [ft/day]
K_y = horizontal hydraulic conductivity along y-axis in [ft/day]
K_v = vertical hydraulic conductivity in [ft/day]
n = porosity in [1]
S_x = specific storativity in [1/ft]
K_d = adsorption distribution coefficient in [ml/g]
D_L = longitudinal dispersion coefficient in [ft/day]
D_T = transverse dispersion coefficient in [ft/day]

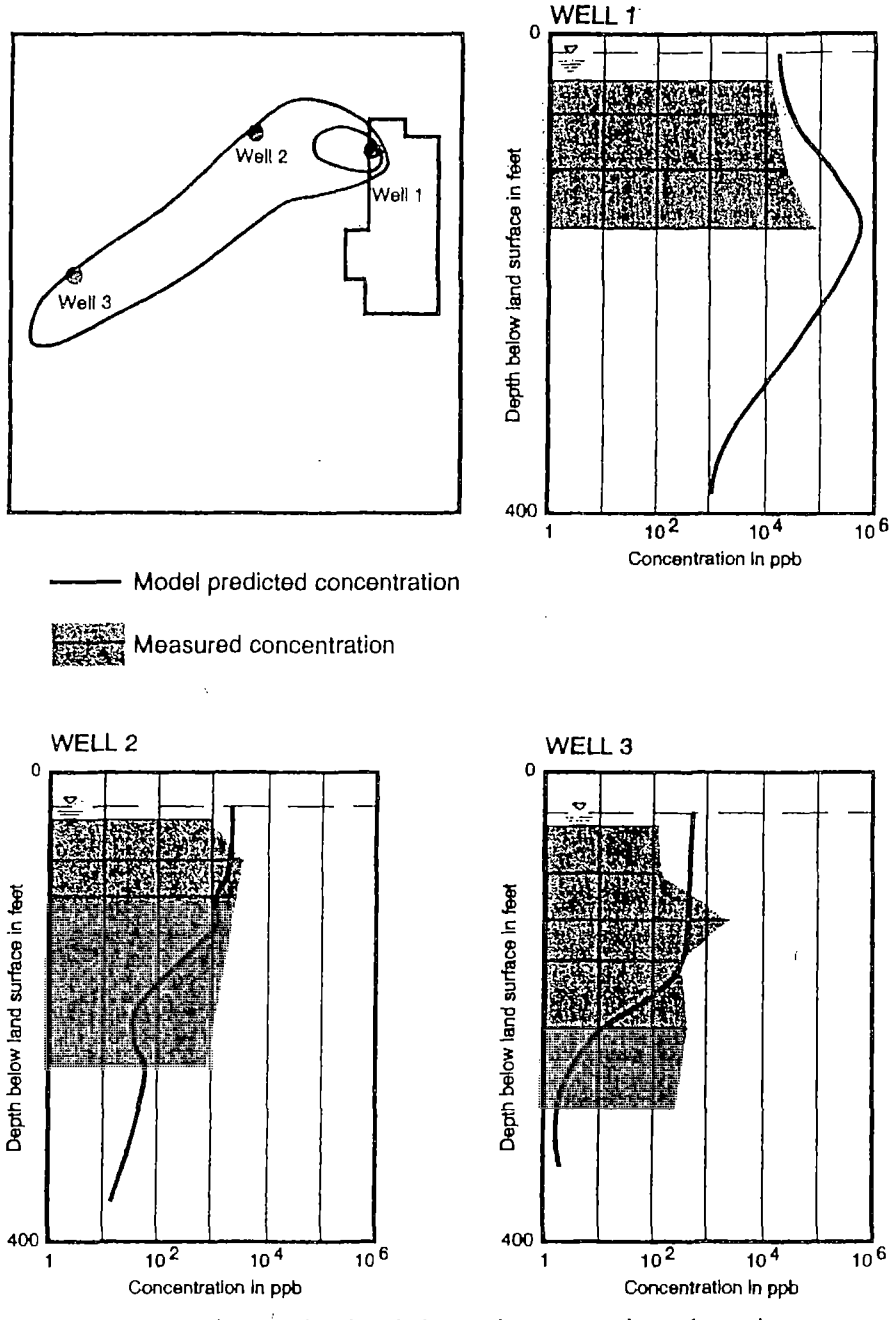


Figure 8.24 Presenting predicted and observed concentrations along plume center line.

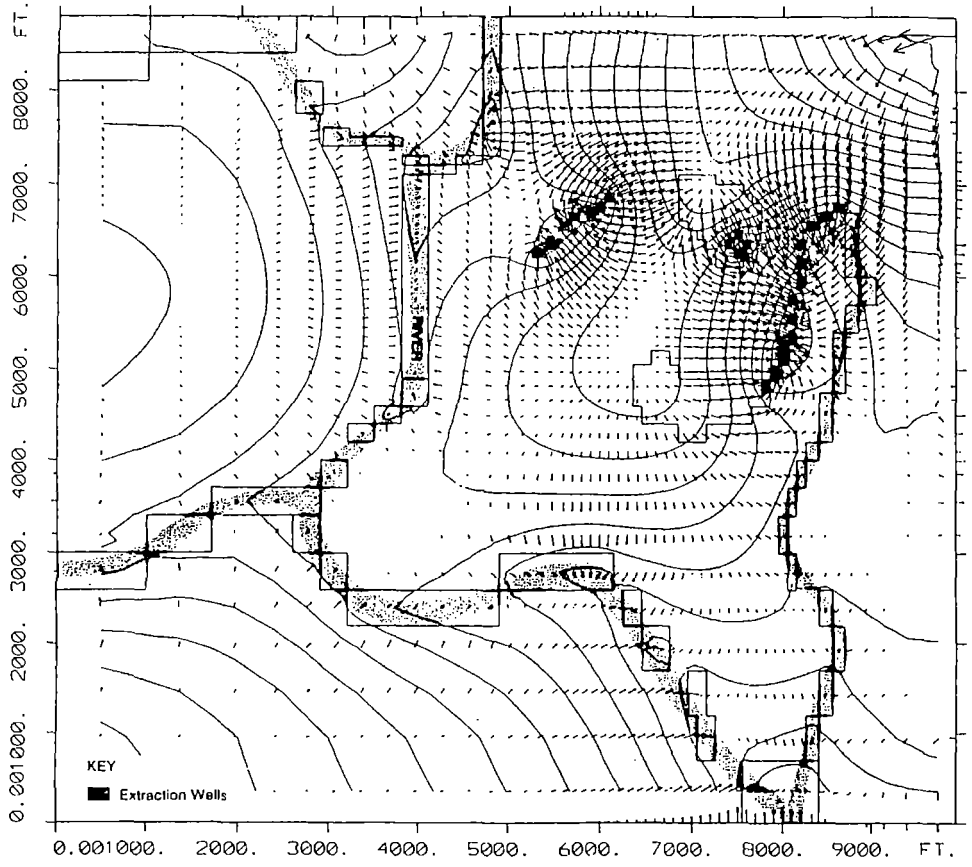


Figure 8.25 Predicted flowfield with remedial wells.

8.11 AUDITING THE MODEL

Occasionally it is possible to conduct a model audit, that is, to compare the model predictions of the future to the actual outcome. Audits can reveal areas of differing model accuracy. Often the actual stresses applied to the system vary from the idealistic stresses simulated in a model; however, a successful audit adds substantially to model credibility. An example of an audit study is presented by Konikow (1986), and another example is illustrated in Figure 8.18.

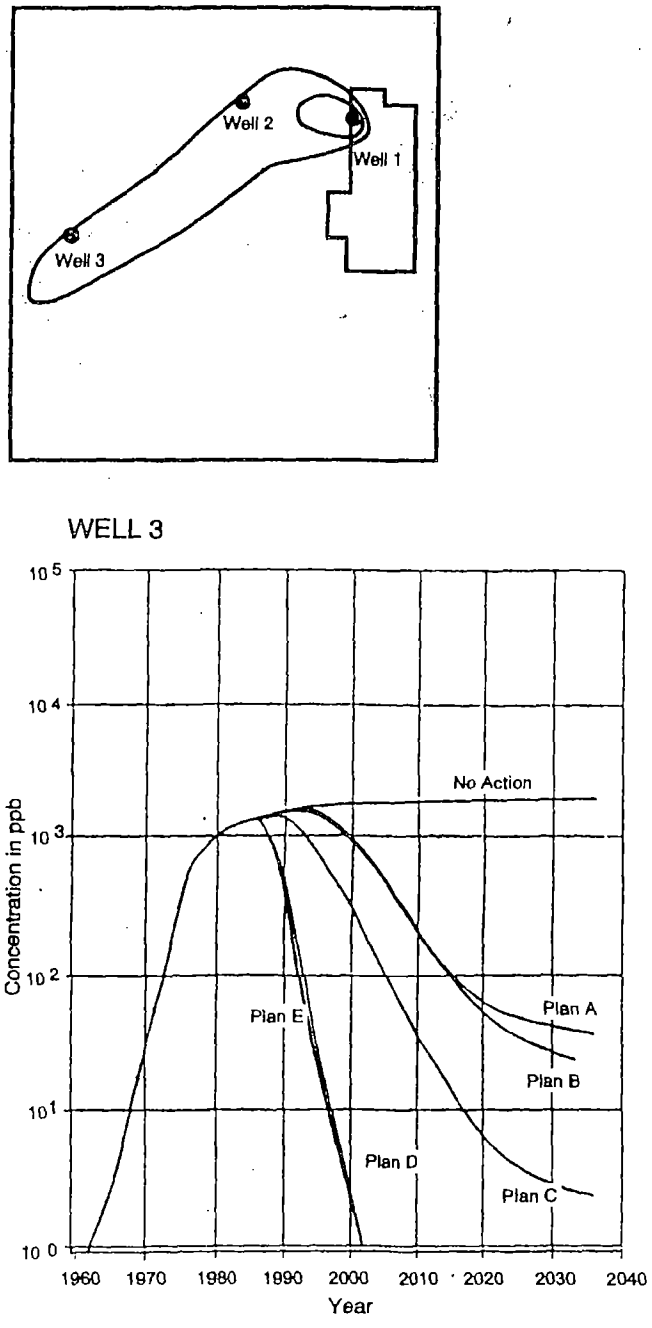


Figure 8.26 Presenting predicted concentrations for alternative remedial plans.

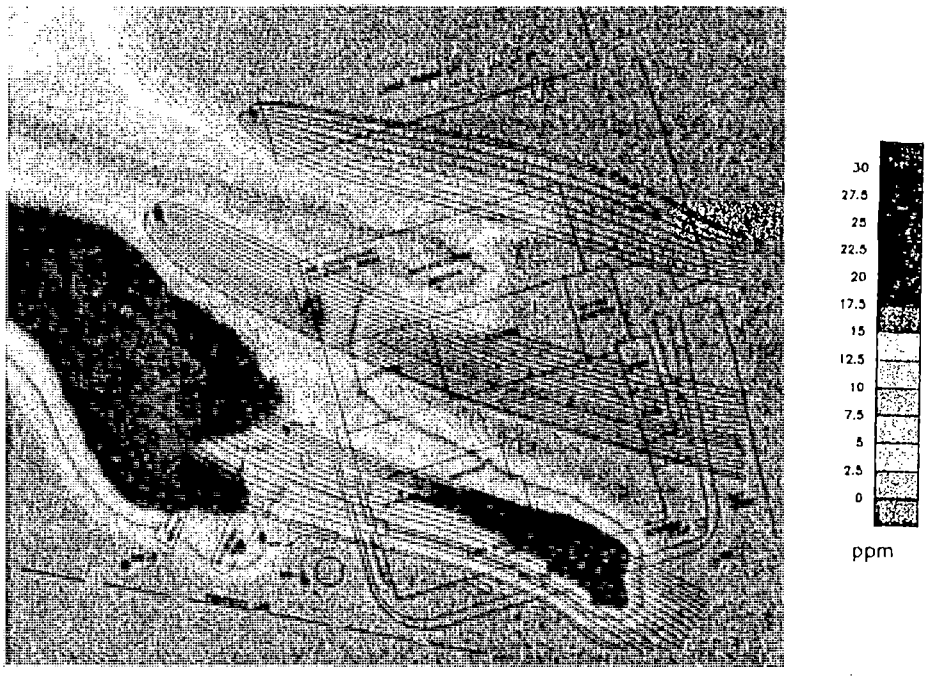


Figure 8.27 Incomplete capture of contaminant plume (provided by T. Franz, Waterloo Hydrogeologic, Inc.).

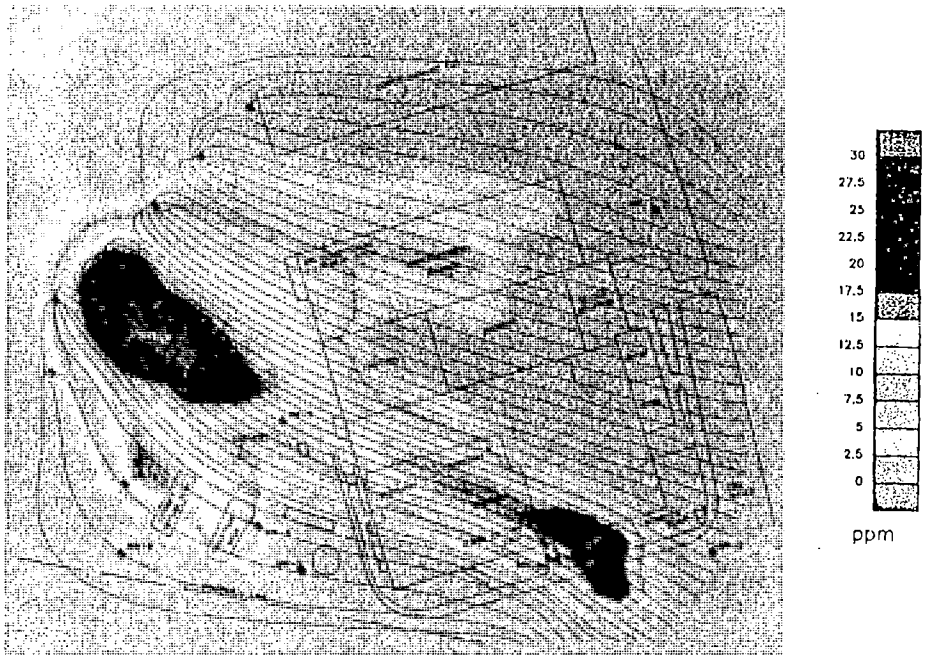


Figure 8.28 Complete capture of contaminant plume (provided by T. Franz, Waterloo Hydrogeologic, Inc.).

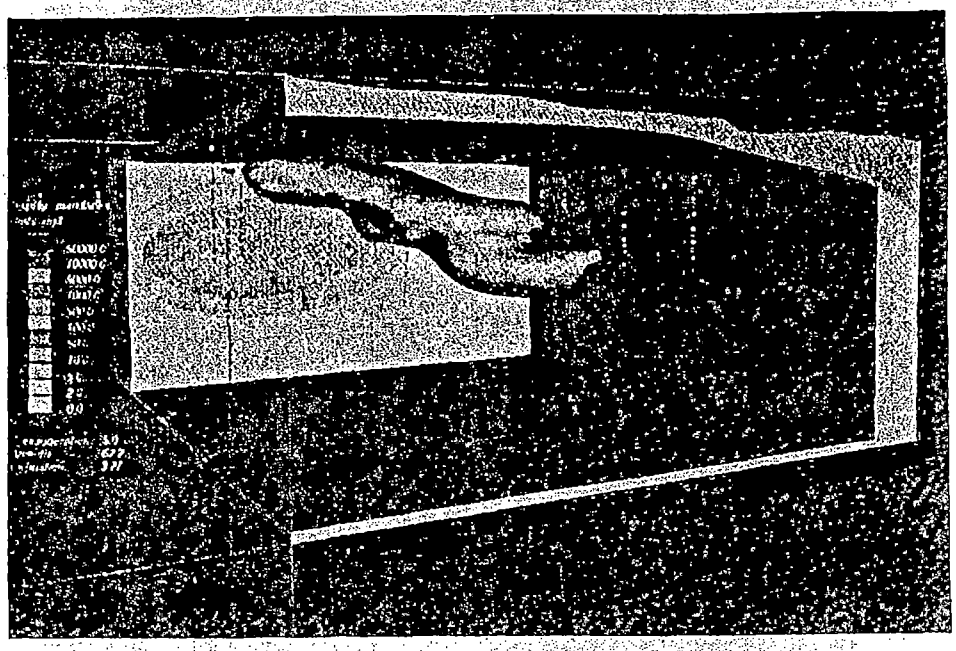


Figure 8.29 Predicted drawdown around oil-spill recovery wells.

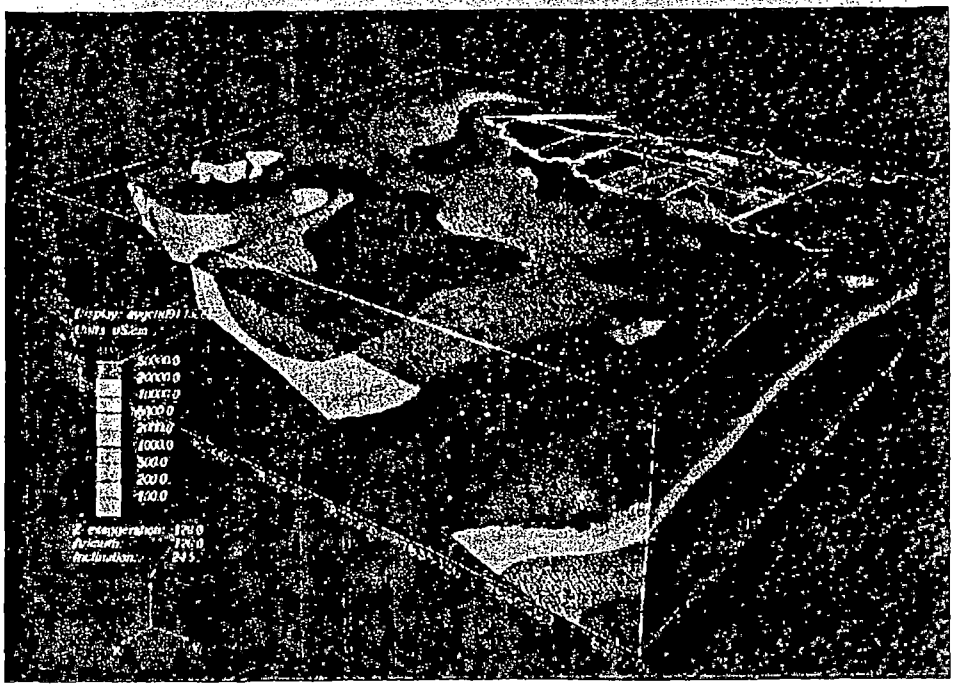


Figure 8.30 Observed concentrations in three dimensions.

TABLE 8.7 Summary of Common Modeling Errors

Error	Typical Result	Remedy
	Data Errors (90% of Errors Fall in This Category)	
Inconsistent units	Infeasible results	Plot all input data and use QA/QC
Misaligned formatted input	Wild results	Check mass balances and plot input and output
Infiltration rate greater than saturated vertical hydraulic conductivity or less than minimum unsaturated hydraulic conductivity in unsaturated flow models	Improbable hydraulic gradients and unstable behavior	Understand physical meaning of specified unsaturated moisture curves
	Conceptual Errors (The Most Difficult Errors to Identify)	
Mismatch between assumptions (e.g., extrapolation of model base to above extrapolated model water table)	Unrealistic predictions	Plot input and output data
Indiscriminate use of fixed value boundaries	Overconstrained model	Replace fixed head with flux boundaries
Misuse of general-head boundaries	These are essentially damped fixed-value boundaries	See above
Specification of all flux boundaries for steady-state run	Nonunique solution	Have at least one boundary cell with a fixed value
Use of fixed-head or flow boundaries in model cells much larger than the feature being simulated	Exaggerated recharge or discharge	Use properly sized model cells, or replace fixed value by fixed-flux boundaries
Unfocused model	Rough predictions	Plan in advance and coordinate with others involved
No simplification, e.g., high degree of heterogeneity	Unnecessarily complex, unwieldy model	Study data for patterns and spend time to develop defensible conceptual model on paper first

TABLE 8.7 (*Continued*)

Error	Typical Result	Remedy
	Code Errors (The Most Difficult Errors to Solve)	
Nondocumented or unrecognized model code limitations	Unrealistic or inconsistent results	Check predictions against expected results; solve by thorough understanding of code, talking to code author or using an alternate code
	Application Errors	
Model boundaries aligned with site boundaries/roads/compass directions, etc.	Potentially unrealistic results	Use natural boundaries
Model boundaries within influence of sources or sinks	Under- or overpredictions	Use appropriate model domain
Model cells and aspect ratios time steps inconsistent	Numerical dispersion	Check Peclet number, Courant number, and stability criteria when setting up advective-dispersive transport models
Model depth = depth of measurements	Biased or incomplete predictions	Use natural model boundaries
Failed model	Inconsistent or no results	Plan alternate approaches from the start
Uncertainty about use of model		Conduct failure analysis (cause/effect/avoidance of failure) or economic analysis (cost of model analysis vs. cost of decisions based on model analyses) or seek assistance

Supplying observed or interpreted concentrations as model input	Unrealistic redistribution due to unbalanced initial conditions	Specify sources rather than observations
Supplying observed or interpreted heads as starting point for transient run	Unrealistic redistribution due to unbalanced initial conditions	Use predicted, steady-state flow pattern as initial conditions
Application beyond range of validation	Poorly defended predictions	Validate model consistent with intended application
Supplying extreme changes in material properties	Unstable results	Use several model cells across interface or define intermediate-value zone
Contaminant plume partially exits model boundary	In predicted remediation the plume may be drawn back through the boundary at unrealistic concentrations, resulting in invalid results	Use larger domain
	Interpretation Errors	
Smoothed contours of predictions hide small-scale problems	Acceptance of incorrect results	Plot unsmoothed data initially
Use of unstable, unconverged or unbalanced results	Acceptance of results containing systematic error	Check mass balances, solution behavior, and plot interim results

8.12 AVOIDING COMMON MODELING ERRORS

The most reliable way of learning the best approach to conduct a modeling analysis is to make many experiments and/or mistakes and learn from them. However, in the interest of saving time and effort, this section provides a compilation of the most commonly noted modeling errors, and ways to avoid them (see Table 8.7). The types of error have leeen divided into five categories for ease of reference:

- Data errors
- Conceptual errors
- Code errors
- Application errors
- Interpretation errors

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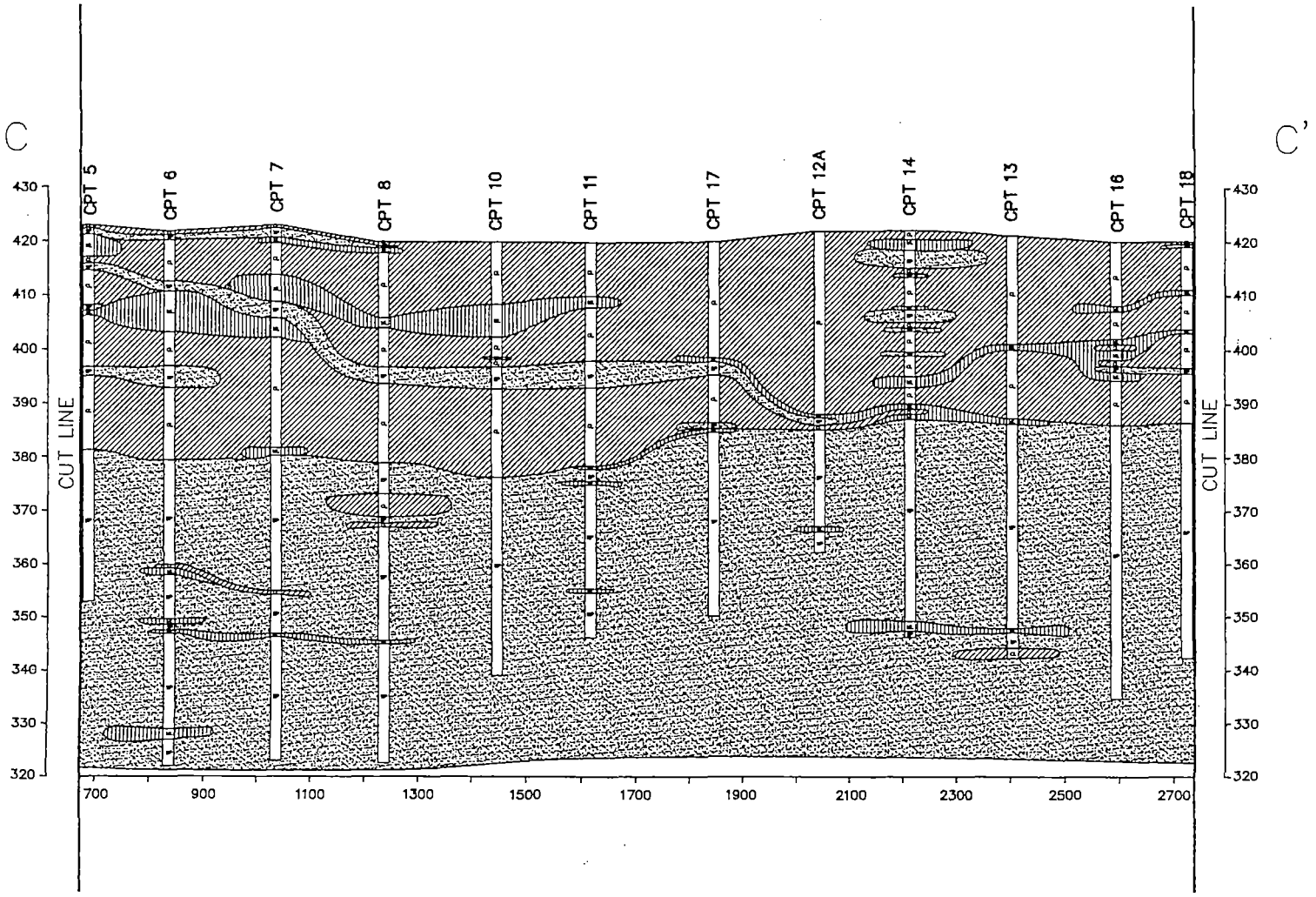
GSI Job No. G-2561
March 31, 2002

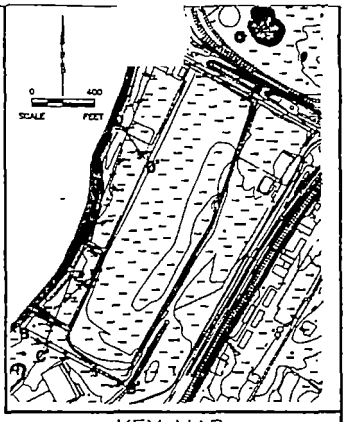
GROUNDWATER
SERVICES, INC.

Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

ATTACHMENT 5





KEY MAP

LEGEND

CLAY

SILT

SAND

CPT–1 CONE PENETROMETER TEST NUMBER

VERTICAL SCALE 1"=10'
HORIZONTAL SCALE 1"=100'

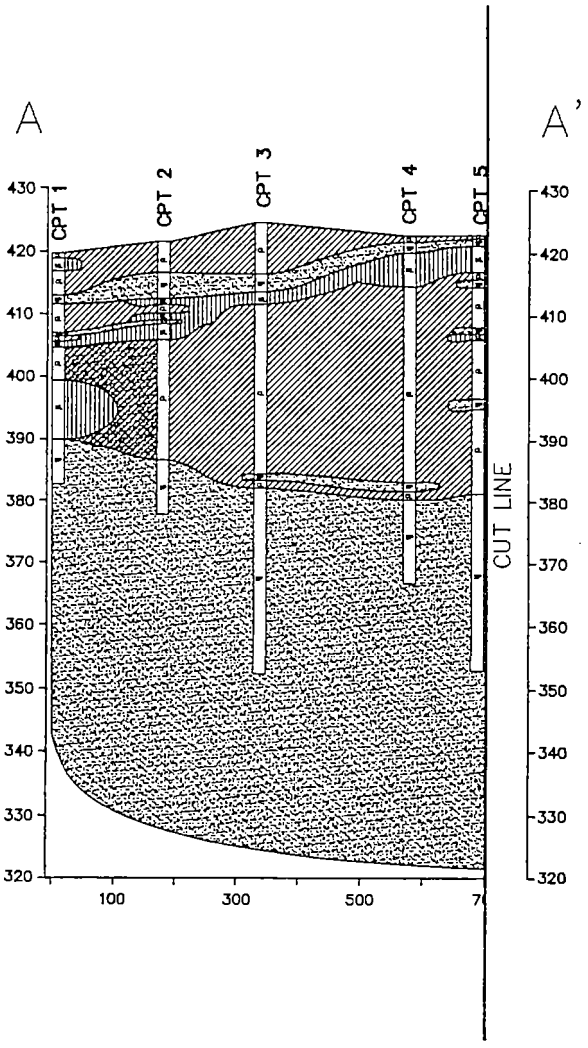
KRUMMRICH GROUNDWATER MIGRATION CONTROL PREDESIGN INVESTIGATION
SAUGET, ILLINOIS

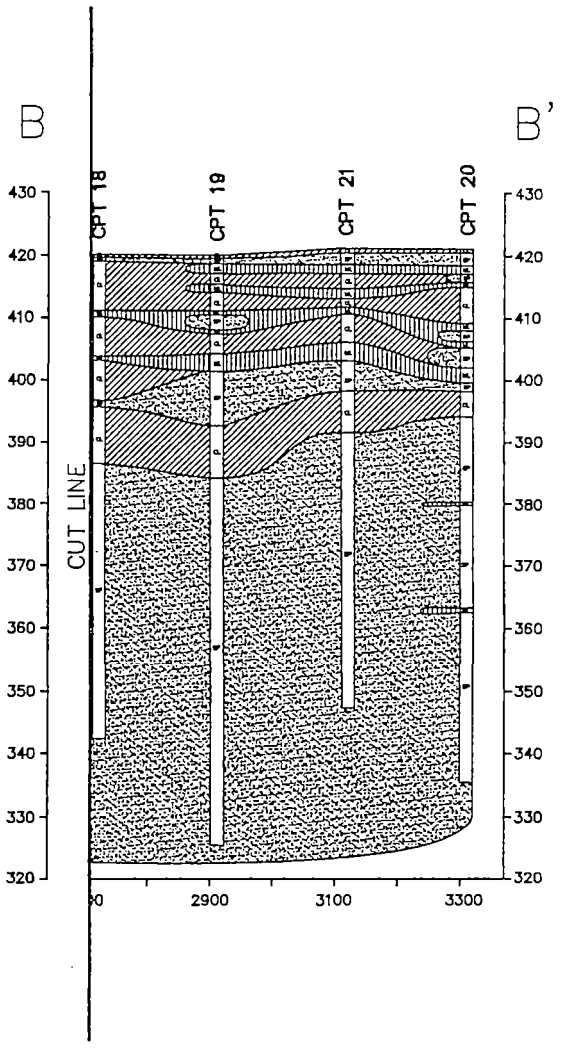
Geological Crossection

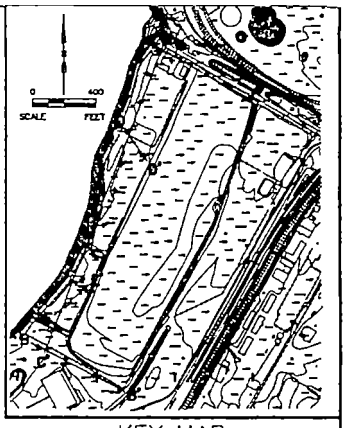
Date	Project Number	Drawing Number
9/12/01	2320010058.00	5
Drawn by: DJD	Design by: MRF/SJS	Checked by: MRF

URS

Attachment 5.1
Geologic Cross-Section,
URS, 2001







KEY MAP

LEGEND

- CLAY
- SILT
- SAND

CPT-1 CONE PENETROMETER TEST NUMBER

VERTICAL SCALE 1"=10'
HORIZONTAL SCALE 1"=100'

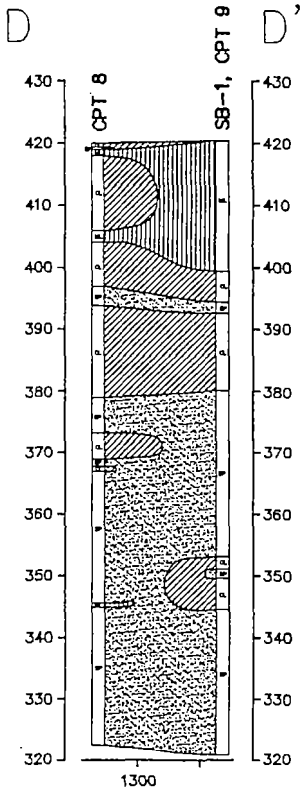
KRUMMRICH GROUNDWATER MIGRATION CONTROL PREDESIGN INVESTIGATION
SAUGET, ILLINOIS

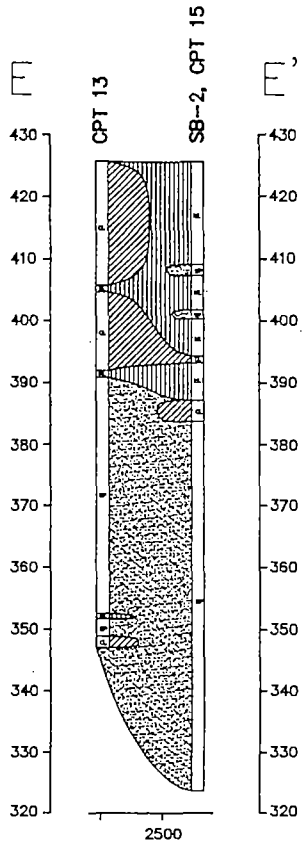
Geological Crossection

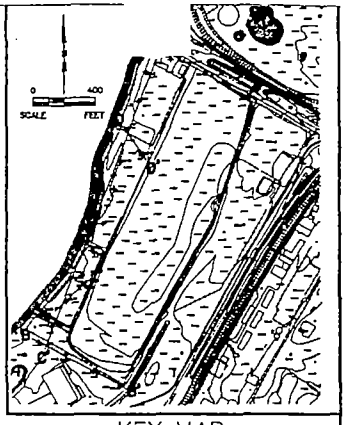
Date	Project Number	Drawing Number
9/12/01	2320010058.00	5
Drawn by: DJD	Design by: MRF/SJS	Checked by: MRF

URS

Attachment 5.2
Geologic Cross-Section,
URS, 2001







KEY MAP

LEGEND

CLAY
SILT
SAND
CPT-1 CONE PENETROMETER TEST NUMBER

VERTICAL SCALE 1"=10'
HORIZONTAL SCALE 1"=100'

KRUMMRICH GROUNDWATER MIGRATION CONTROL PREDESIGN INVESTIGATION
SAUGET, ILLINOIS

Geological Crossection

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URS

Attachment 5.3
Geologic Cross-Section,
URS, 2001

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March 31, 2002

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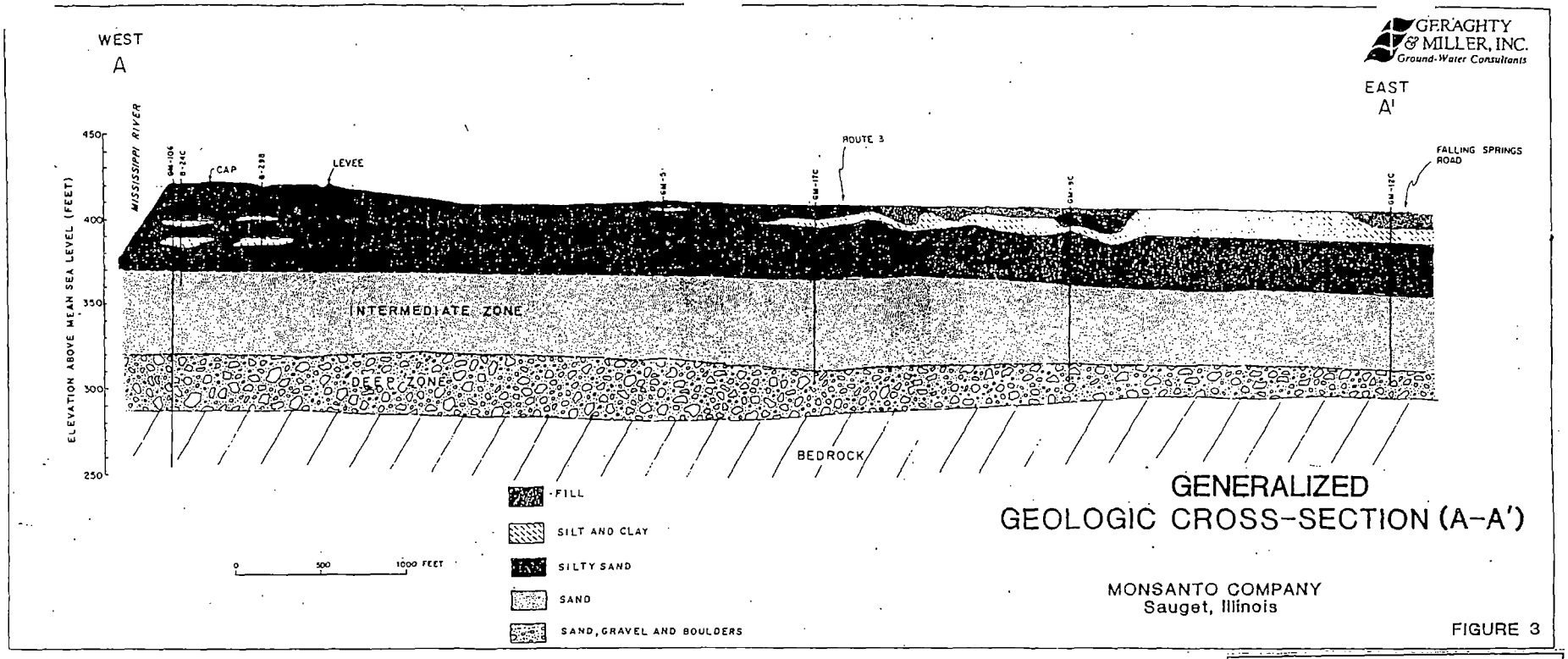
Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

ATTACHMENT 6



GENERALIZED
GEOLOGIC CROSS-SECTION (A-A')

MONSANTO COMPANY
Sauget, Illinois

FIGURE 3

Attachment 6
Geologic Cross-Section,
Geraghty & Miller,
Date unknown

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March 31, 2002

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Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

ATTACHMENT 7

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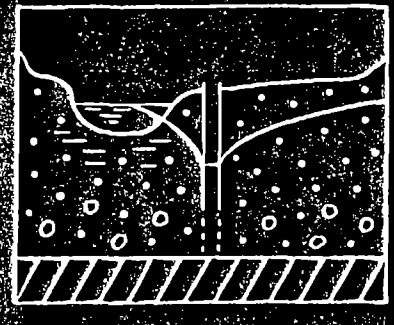
Ground-Water Development in East St. Louis Area, Illinois

by R. J. SCHICHT



ILLINOIS STATE WATER SURVEY

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REPORT OF INVESTIGATION 51

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

by R. J. SCHICHT

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1965

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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.

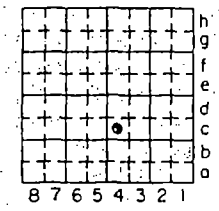
This report presents a quantitative evaluation of the ground-water resources of the East St. Louis area and is based on all data on file at the State Water Survey and in other published reports. The geohydrologic characteristics of the ground-water reservoir are given along with an analysis of past, 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

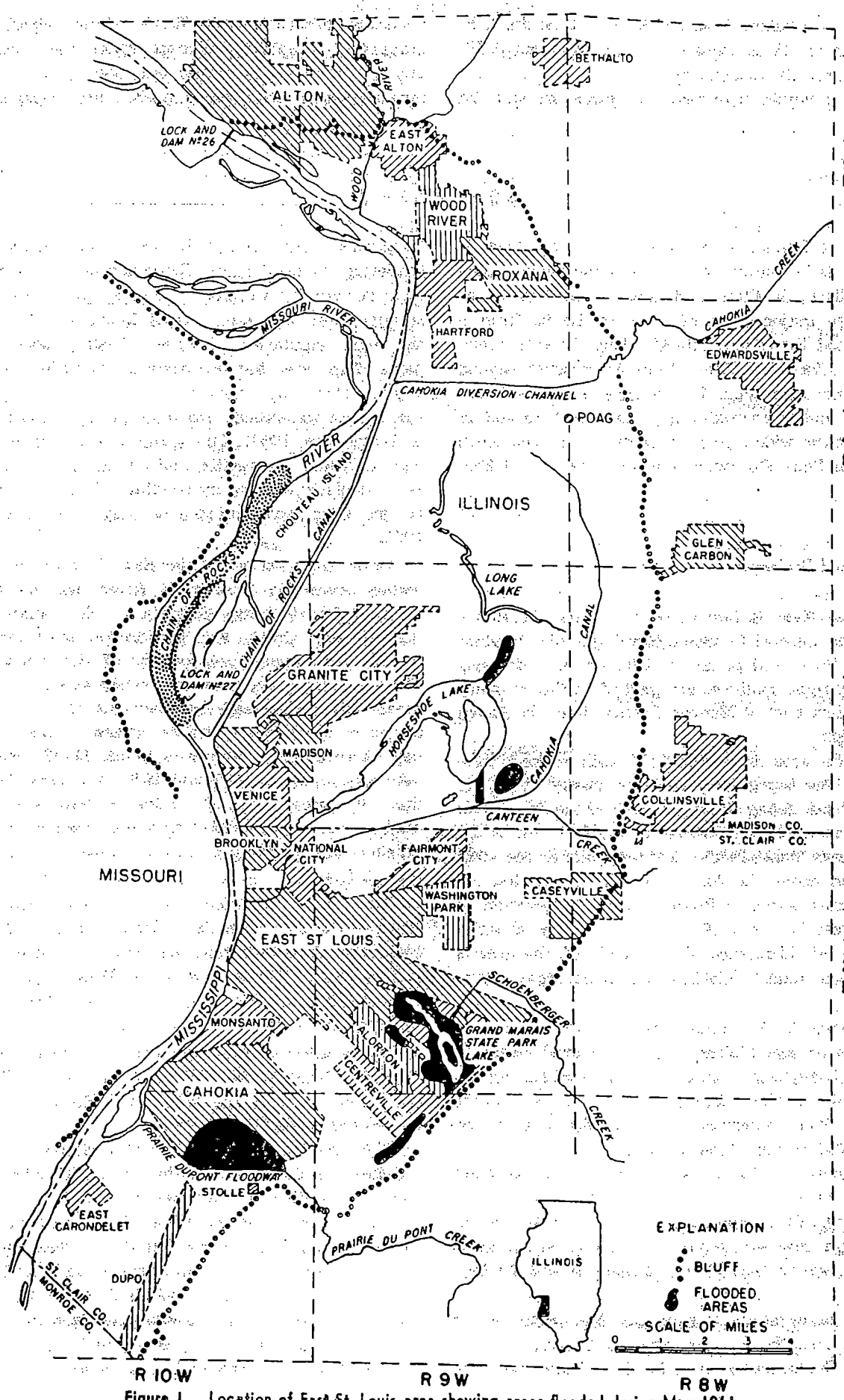


Figure 1. Location of East St. Louis area showing areas flooded during May 1961

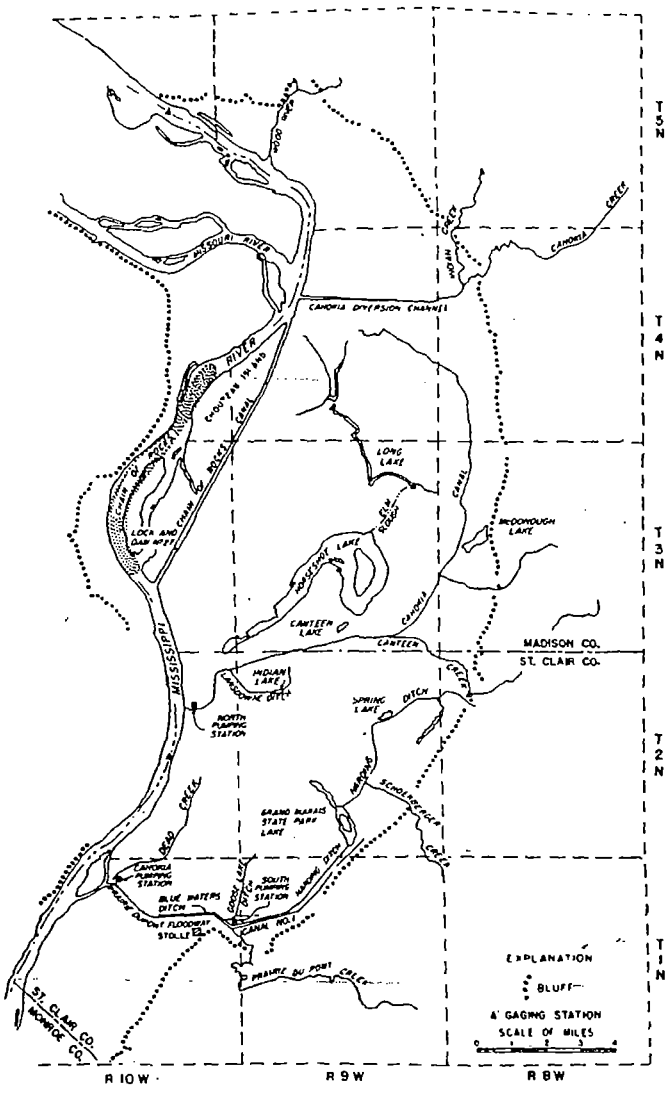


Figure 2. Drainage system and locations of stream-gaging stations

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 1. 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	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

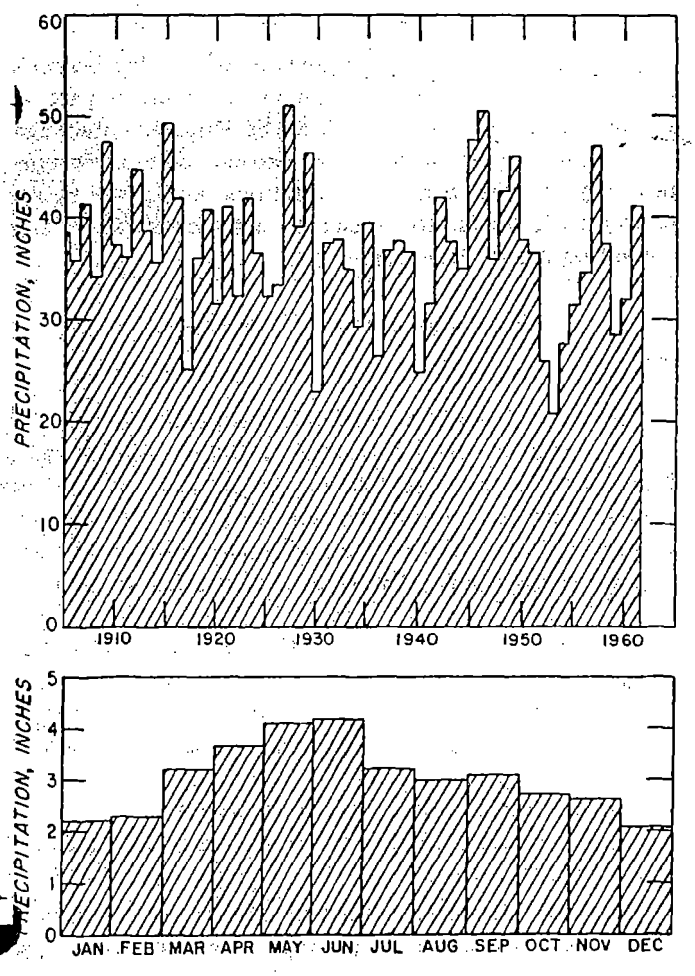


Figure 3. Annual and mean monthly precipitation at Lambert Field

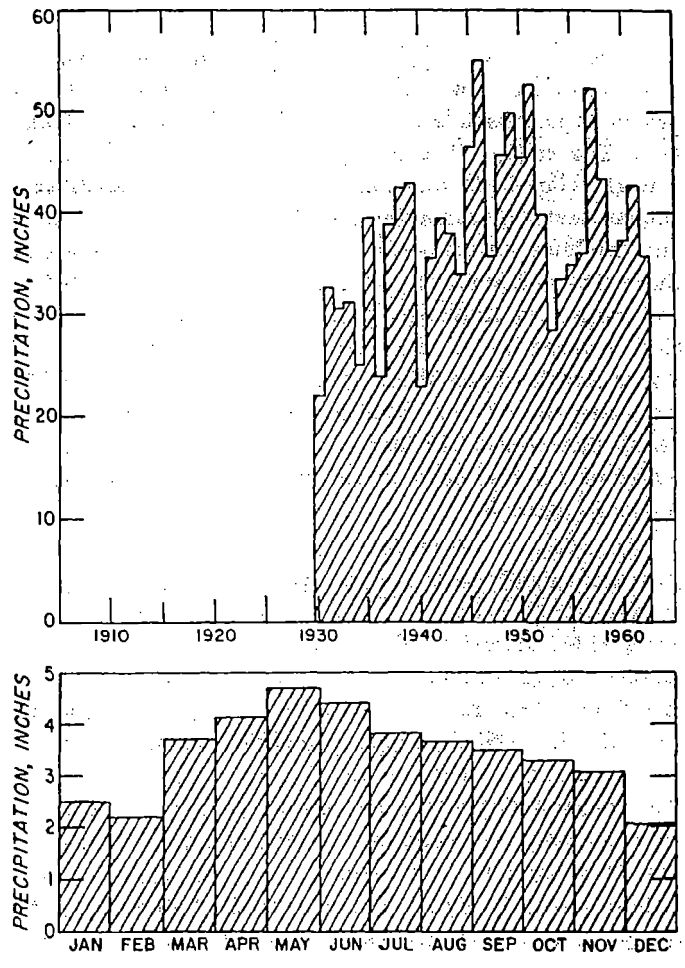


Figure 4. Annual and mean monthly precipitation at Edwardsville

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-

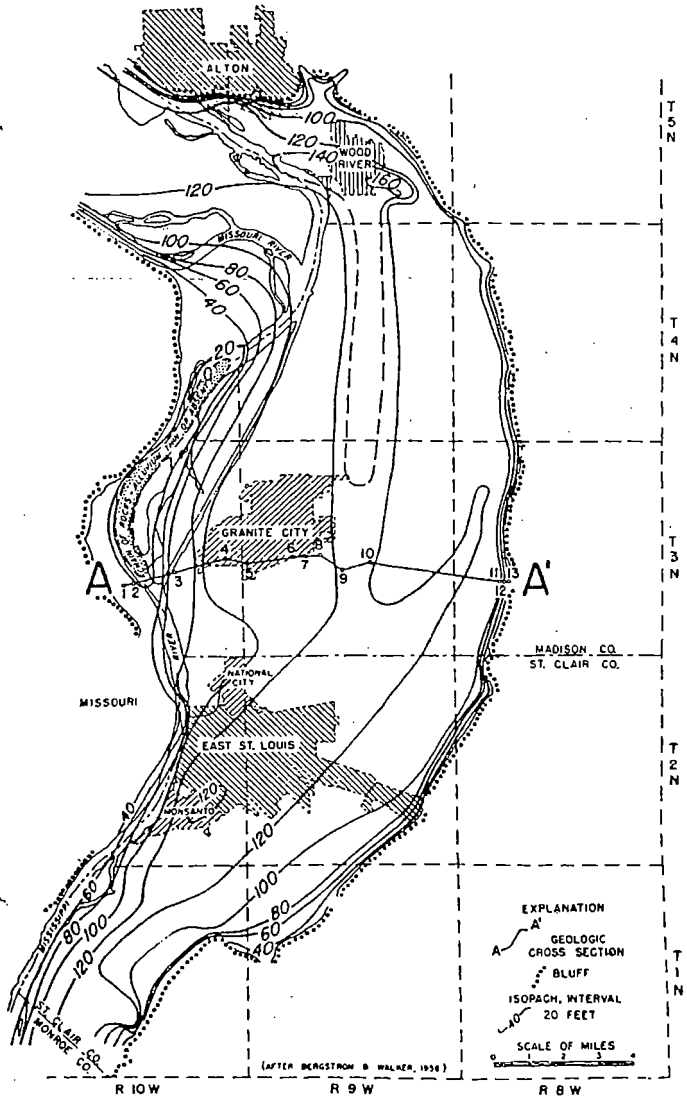


Figure 5. Thickness of the valley fill

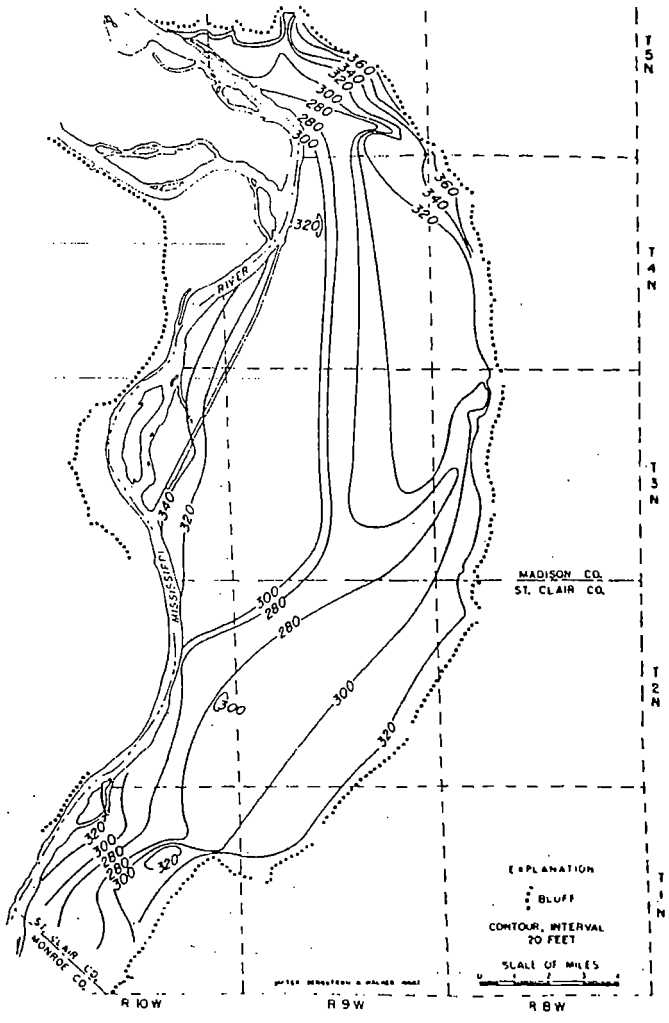


Figure 6. Bedrock topography

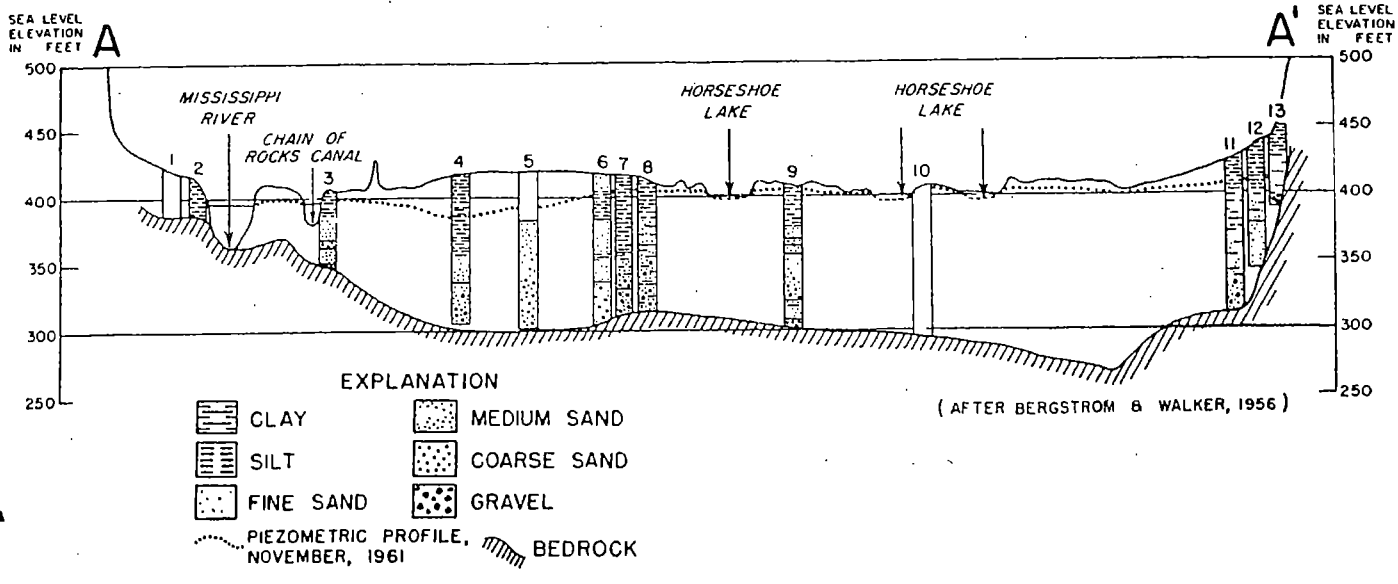


Figure 7. Geologic cross section and piezometric profile of the valley fill

Table 6. Logs of Selected Wells*

Illinois State Geological Survey test 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 Oil Company well 2 (1952)—150 feet N, 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 ¾ 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, T3N, R10W, Madison Co. Illinois Geological Survey sample set 23406. Studied by R. E. Bergstrom. Est. elev. 422 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

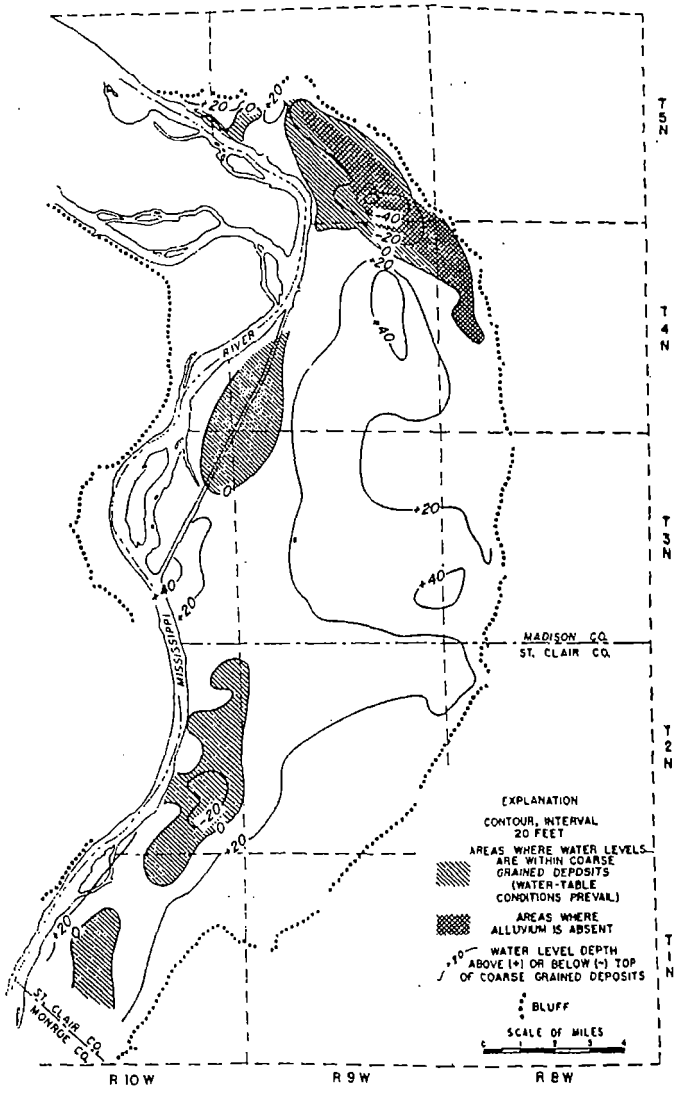


Figure 8. Location of areas where water-table conditions prevail

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

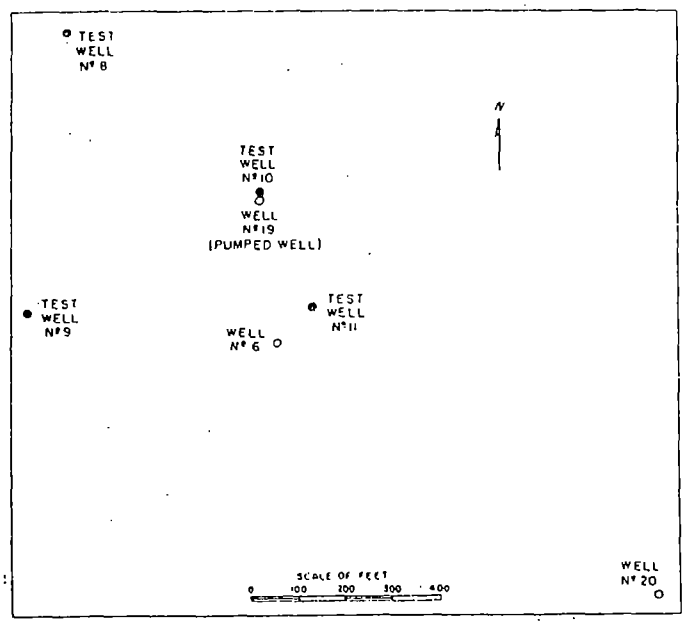


Figure 9. Location of wells used in aquifer test 1

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

Test Well 8

Formation	From (ft)	To (ft)
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)		

Test Well 9

Formation	From (ft)	To (ft)
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

Test Well 10

Formation	From (ft)	To (ft)
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

Test Well 10 (Continued)

Formation	From (ft)	To (ft)
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

Test Well 11

Formation	From (ft)	To (ft)
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

Well 20

Formation	From (ft)	To (ft)
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

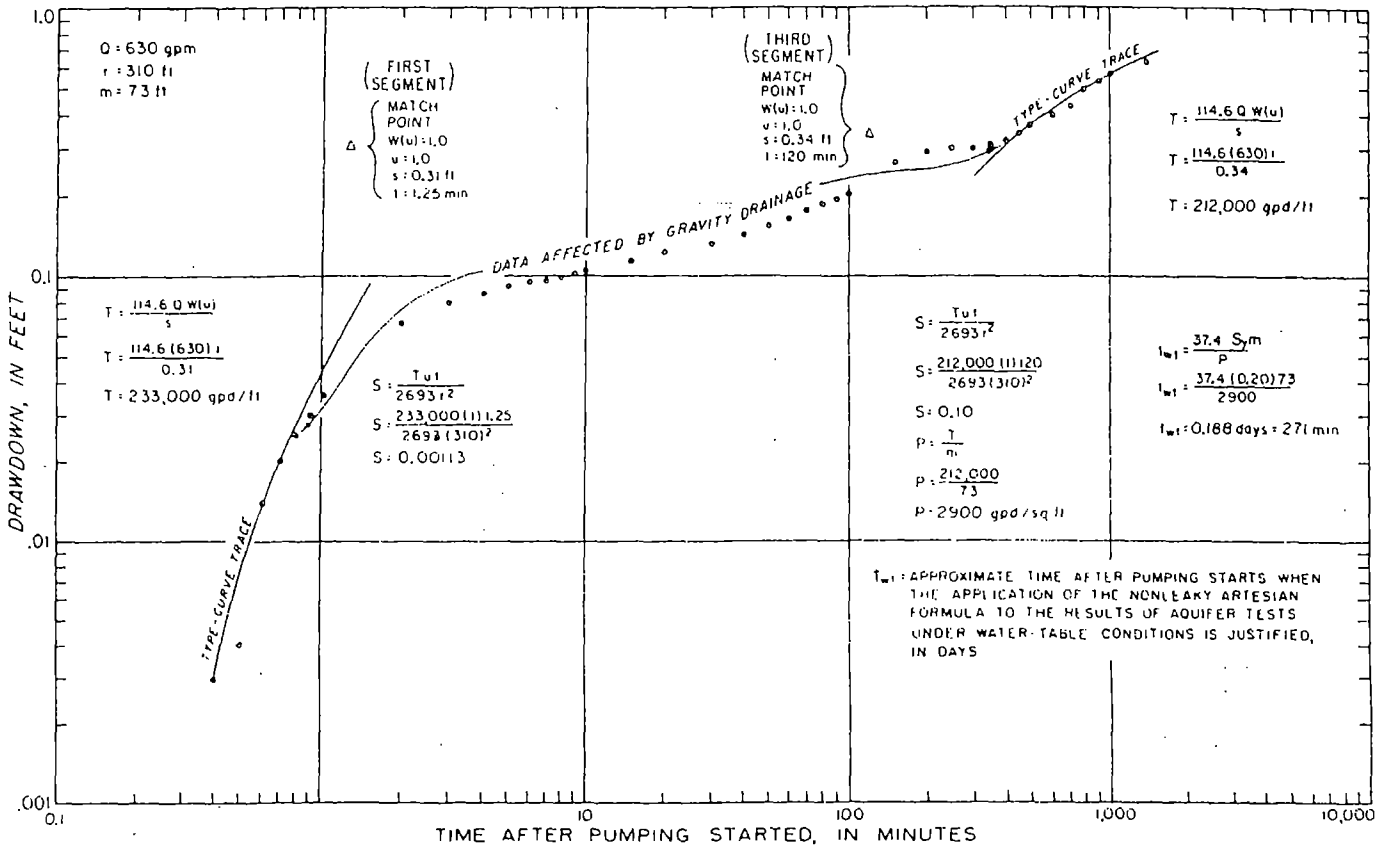


Figure 10. Time-drawdown data for well 6, aquifer test 1

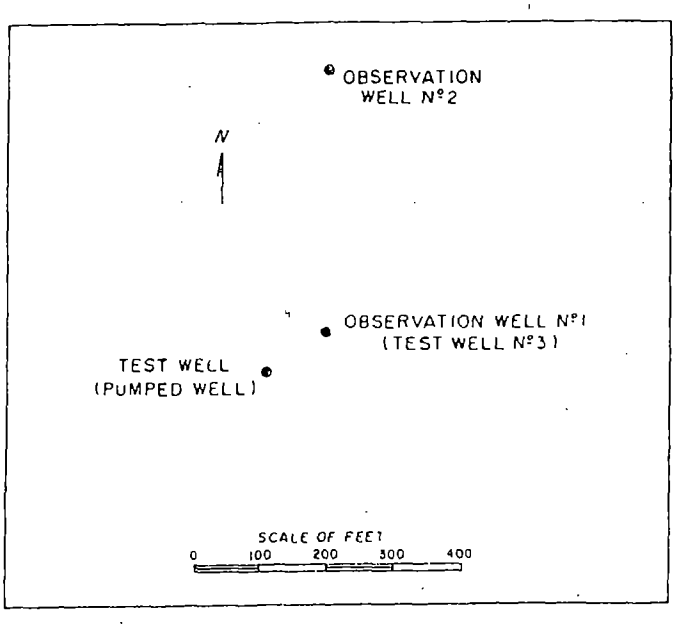


Figure 11. Location of wells used in aquifer test 2

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

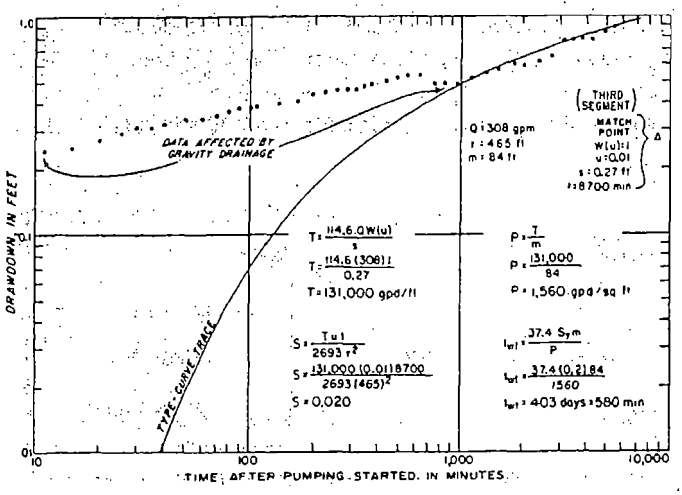


Figure 12. Time-drawdown data for observation well 2, aquifer test 2

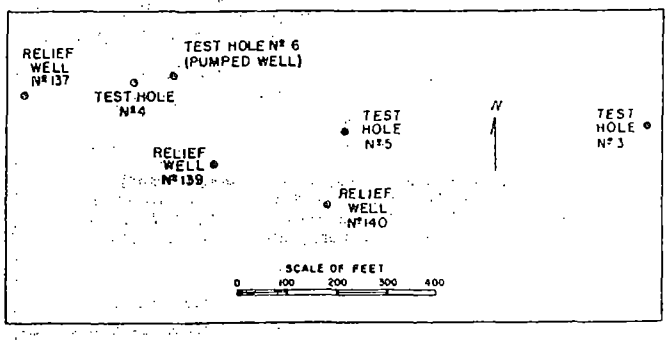


Figure 13. Location of wells used in aquifer test 3

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 (ft)
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

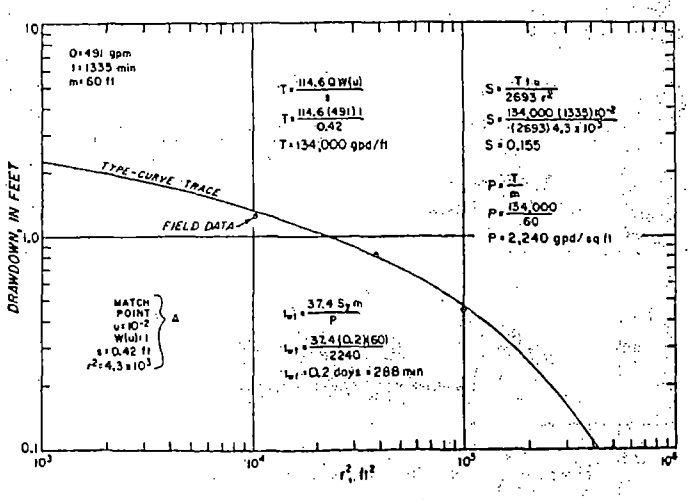


Figure 14. Distance-drawdown data for aquifer test 3

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):

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

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

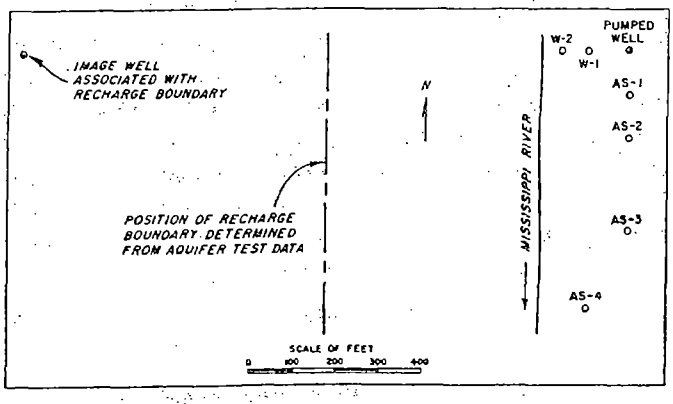


Figure 15. Location of wells used in aquifer test 4

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 (ft)	To
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-1		
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

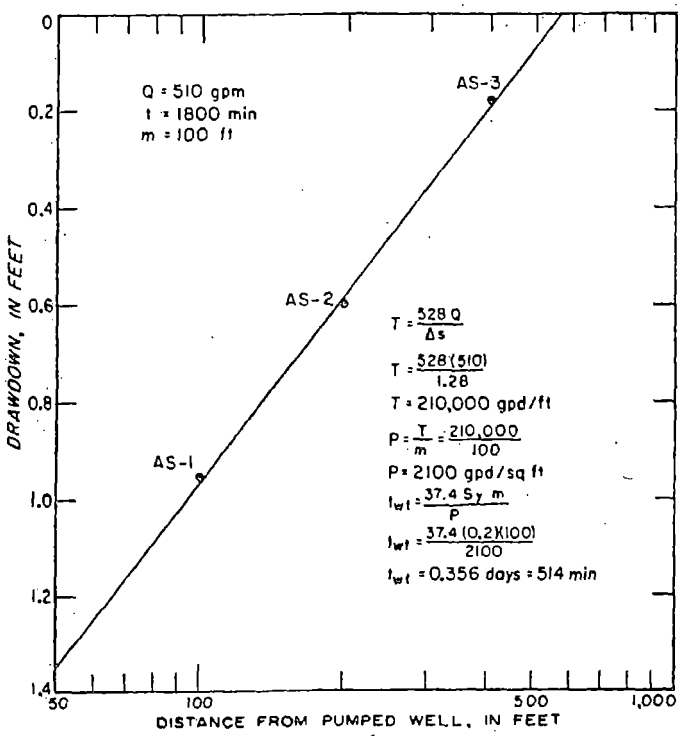


Figure 16. Distance-drawdown data for aquifer test 4

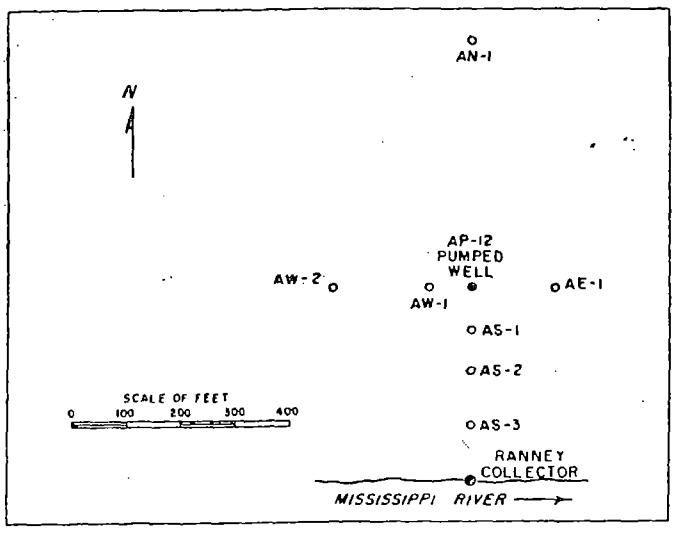


Figure 17. Location of wells used in aquifer test 5

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.

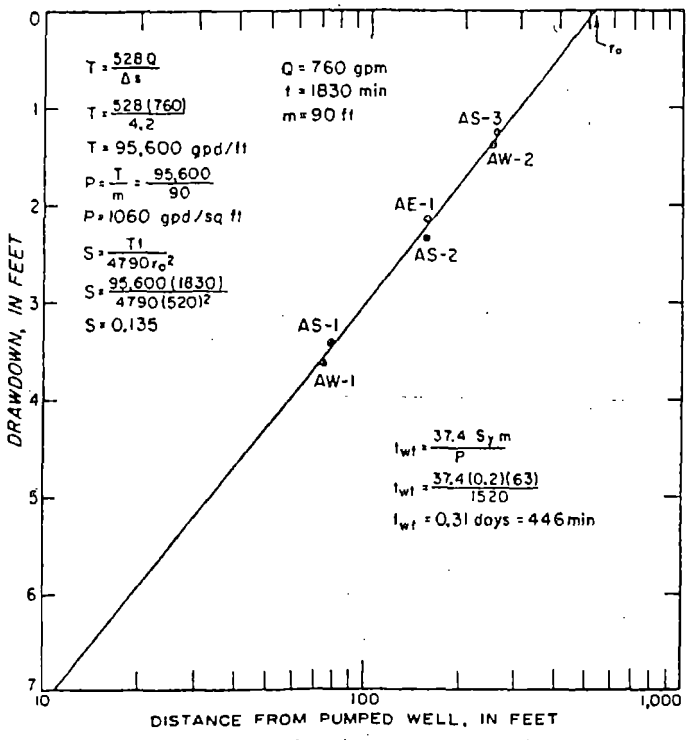


Figure 18. Distance-drawdown data for aquifer test 5

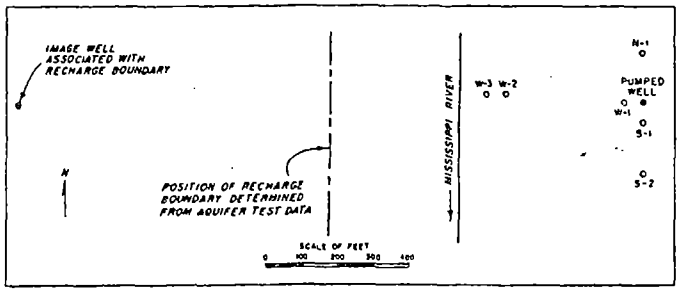


Figure 19. Location of wells used in aquifer test 6

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

Formation	From (ft)	To (ft)
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

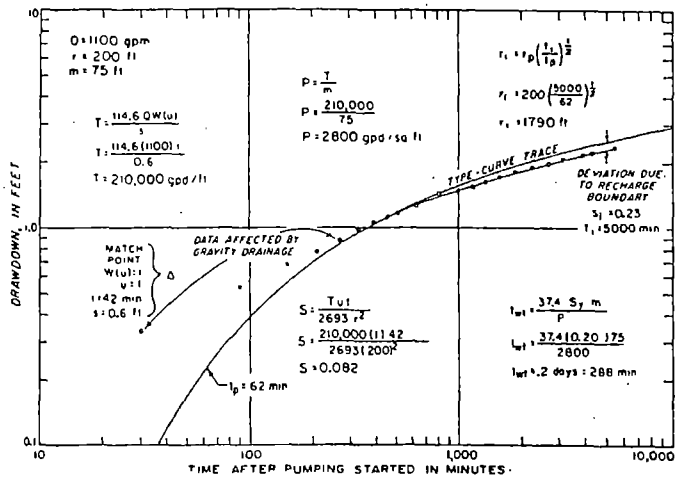


Figure 20. Time-drawdown data for well S-2, aquifer test 6

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

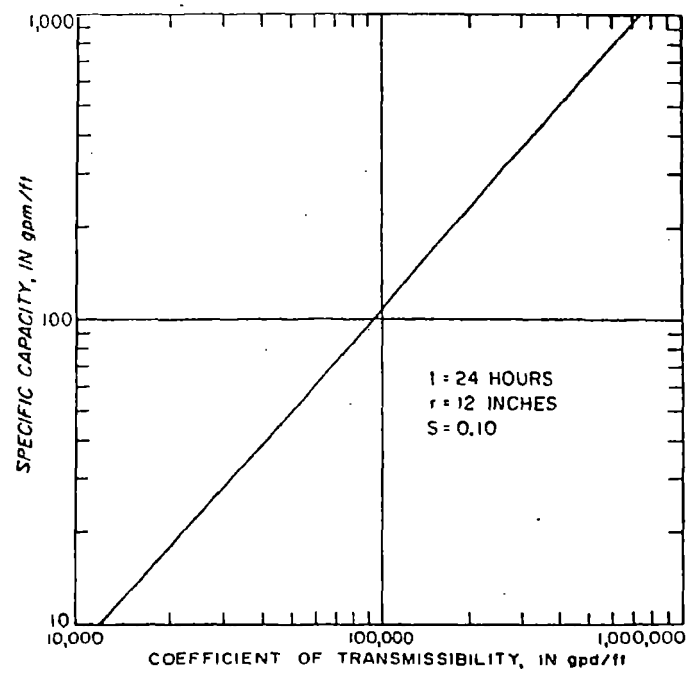


Figure 21. Theoretical relation between specific capacity and the coefficient of transmissibility

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 × 40	Porous concrete	5/54	5/54	0.2
Amos Bonham	Thorpe		30 × 40	Porous concrete	10/54	4/55	0.45
Herbert Bischoff	Thorpe	60	30 × 40	Porous concrete	1/54	5/54	0.5
V. W. Eckmann	Thorpe	48	30 × 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
22.1b1	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
26.8g1	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-13.1c1	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
29.3h1	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 Stockyards Co.	110	18	40	5/61	360	37	1248	8.18	152.5	200,000	73	2740
1.3a5	National Stockyards 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						
MAD—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
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 transmissibility (gpd/ft)	Estimated saturated thickness (ft)	Coefficient of permeability (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
14.1f	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

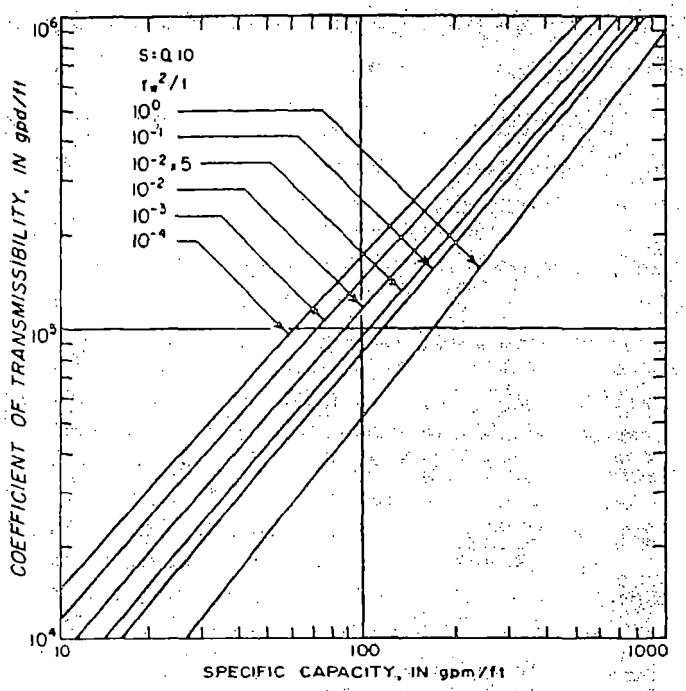


Figure 22. Coefficient of transmissibility versus specific capacity for several values of well radius and pumping period

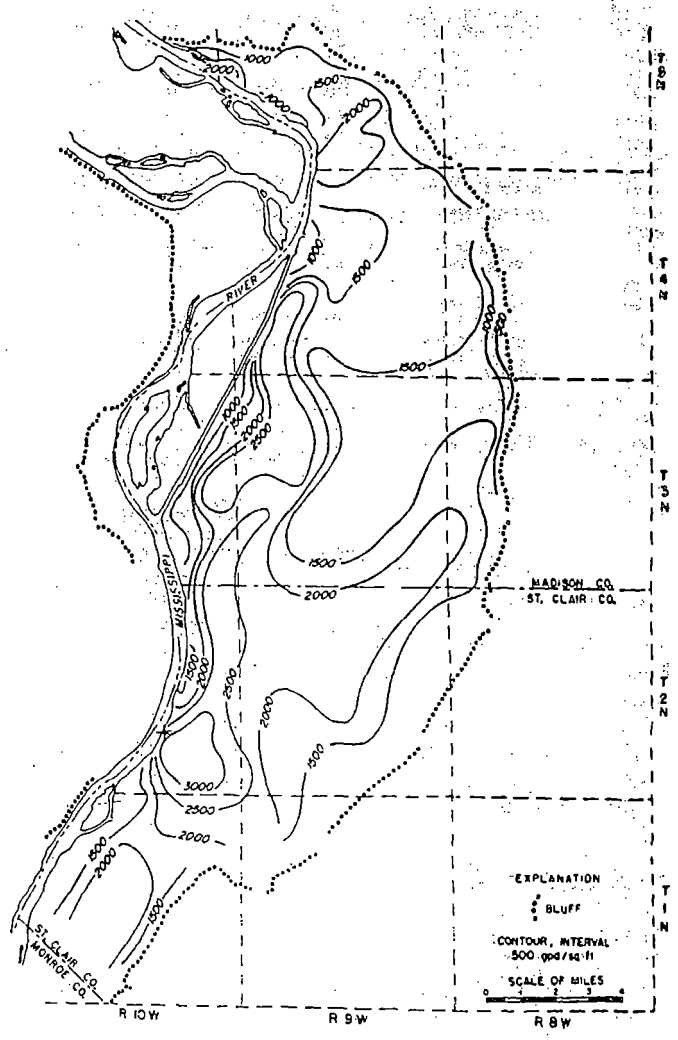


Figure 23. Coefficient of permeability of aquifer

Table 17. Pumping Center Specific-Capacity Data

Pumping center	Pumpage in 1961 (mgd)	Average drawdown (ft)	Specific capacity (gpd/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.

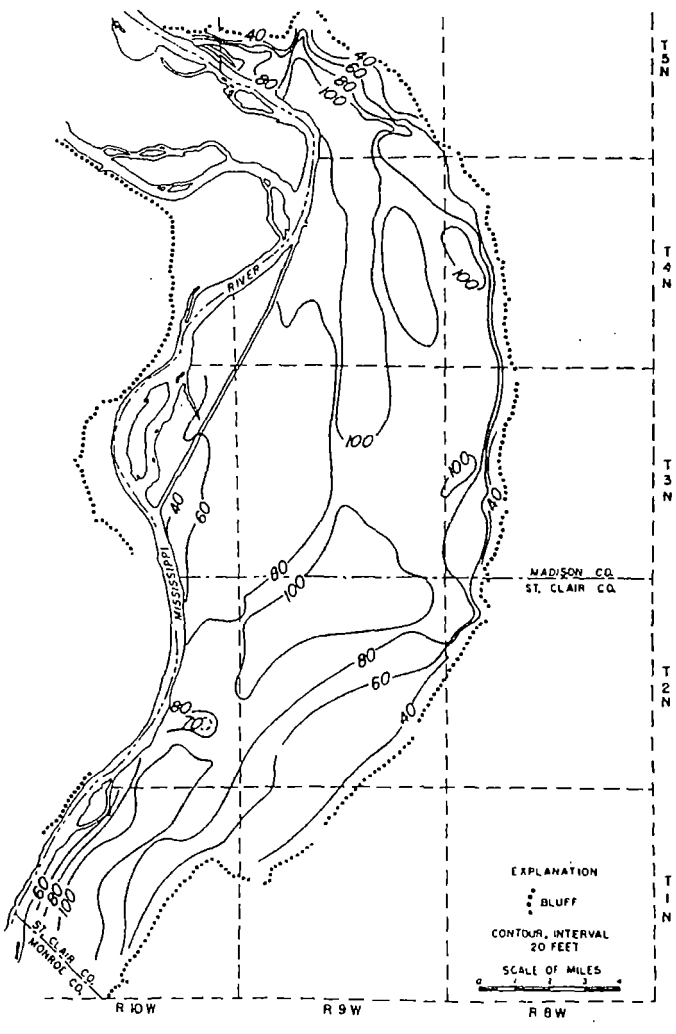


Figure 24. Saturated thickness of aquifer, November 1961

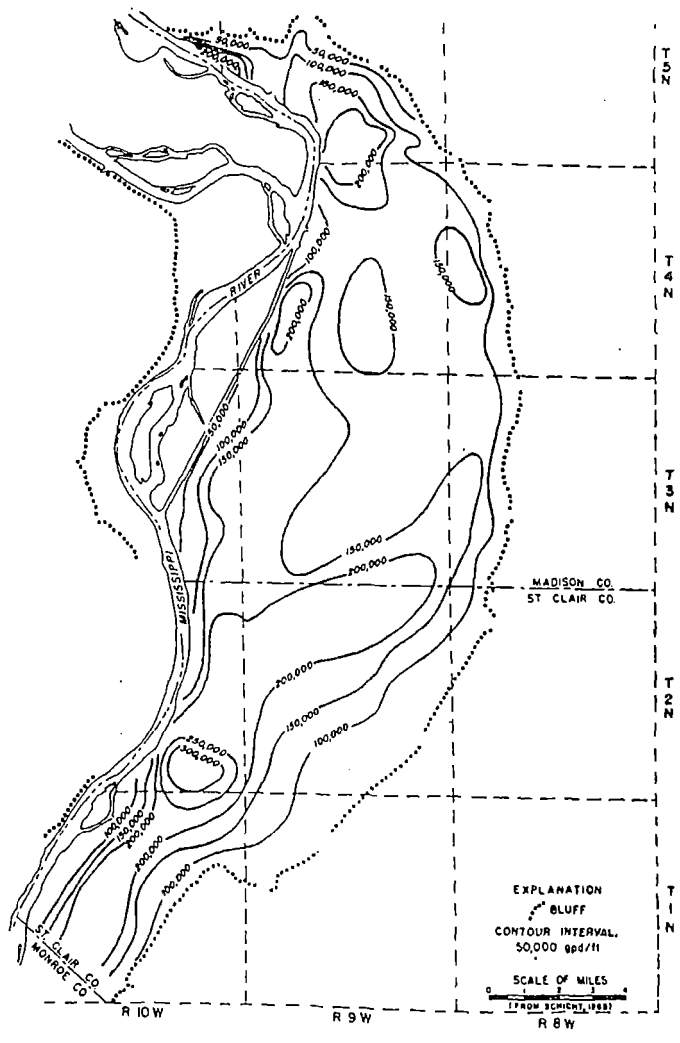


Figure 25. Coefficient of transmissibility of aquifer

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 × 2½	9.5
108	0-73	24	35	24	Everdur Johnson	60 100	6.0
100	0-63	14	37	12	Slotted pipe	¼ × 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,

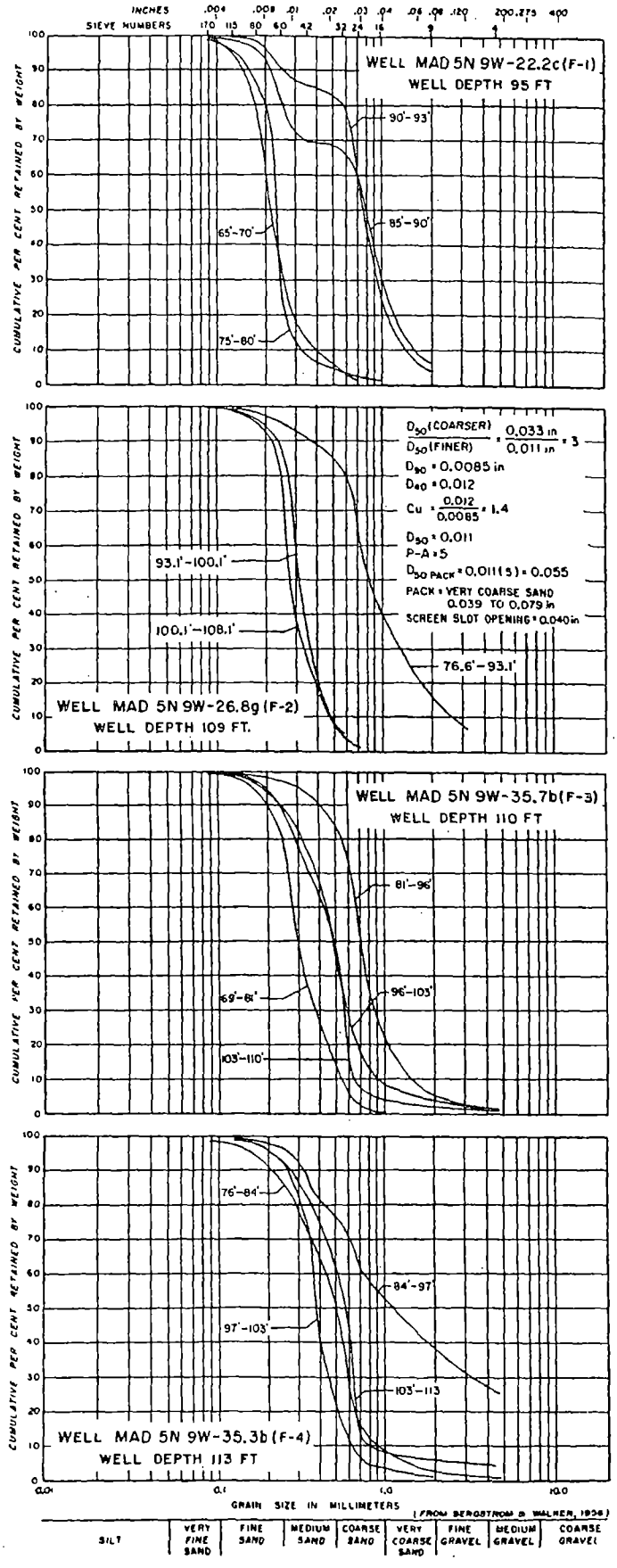


Figure 26. Mechanical analyses of samples from wells

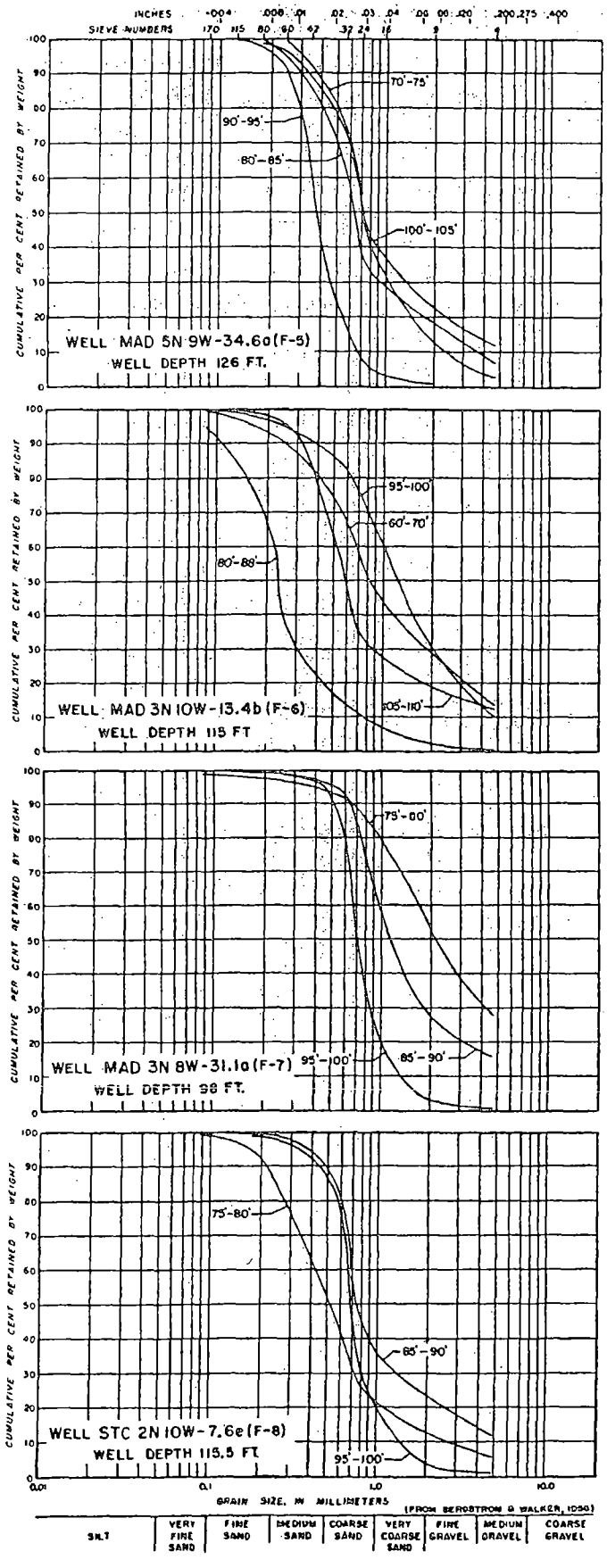


Figure 27. Mechanical analyses of samples from wells

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 C 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

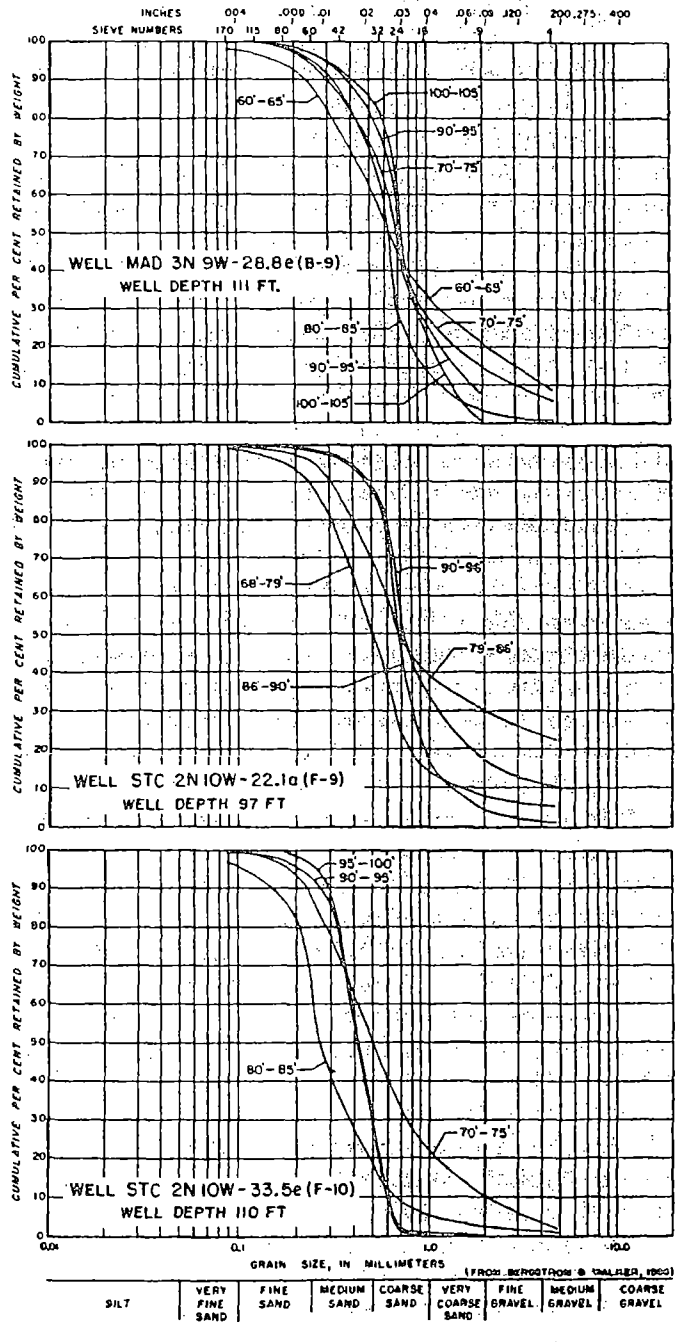


Figure 28. Mechanical analyses of samples from wells

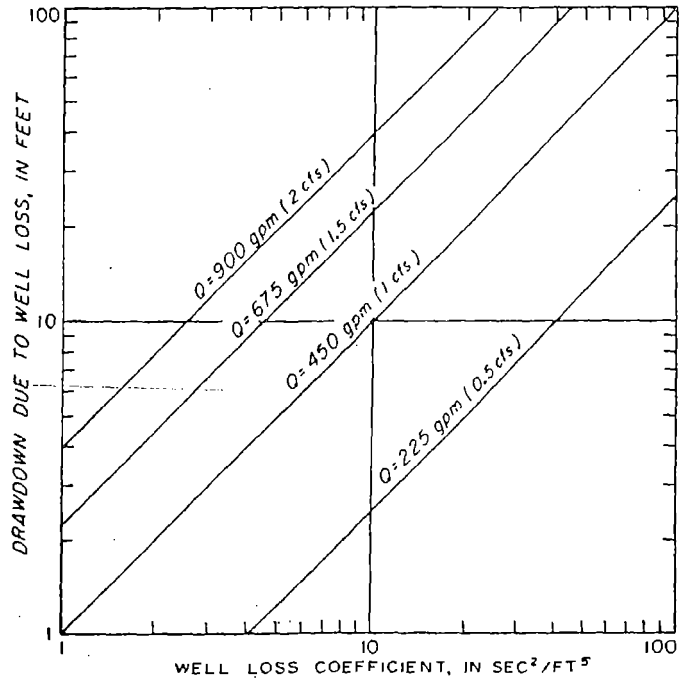


Figure 29. Relation between well-loss constant and drawdown due to well loss

and sometimes impossible to restore the original capacity if the well-loss constant is greater than 40 sec²/ft⁵.

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 sec²/ft⁵		Well-loss coefficient of 5 sec²/ft⁵		Well-loss coefficient of 10 sec²/ft⁵	
		Drawdown* (ft)	Specific capacity* (gpm/ft)	Drawdown* (ft)	Specific capacity* (gpm/ft)	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

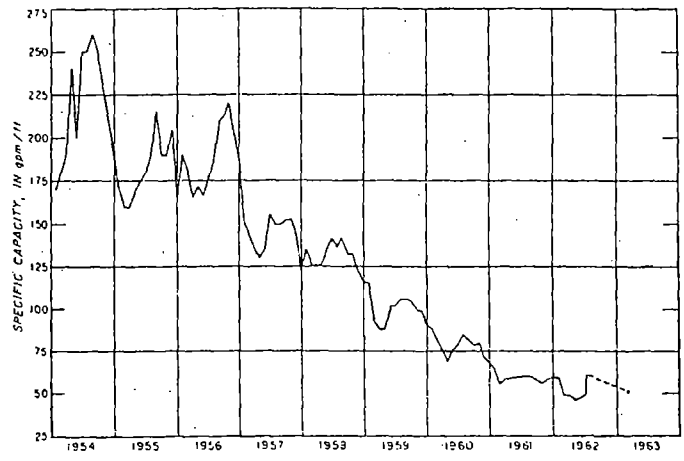


Figure 30. Specific-capacity data for collector well, 1954 to March 1963

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):

$$L_s = Q/A_e V_c(7.48) \qquad (9)$$

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

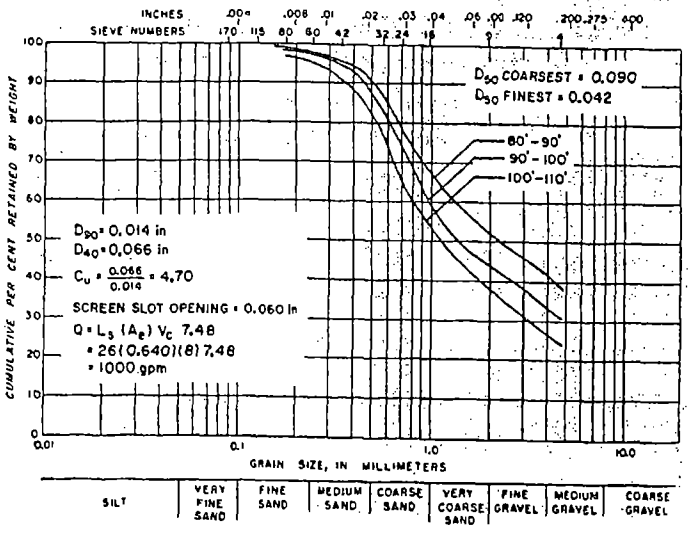


Figure 31. Mechanical analyses of samples for test hole

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-

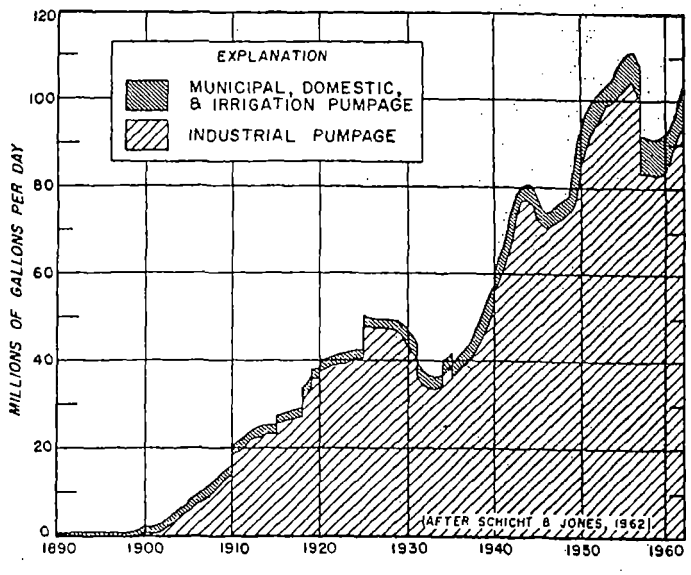


Figure 32. Estimated pumpage from wells, 1890 through 1962, subdivided by use

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

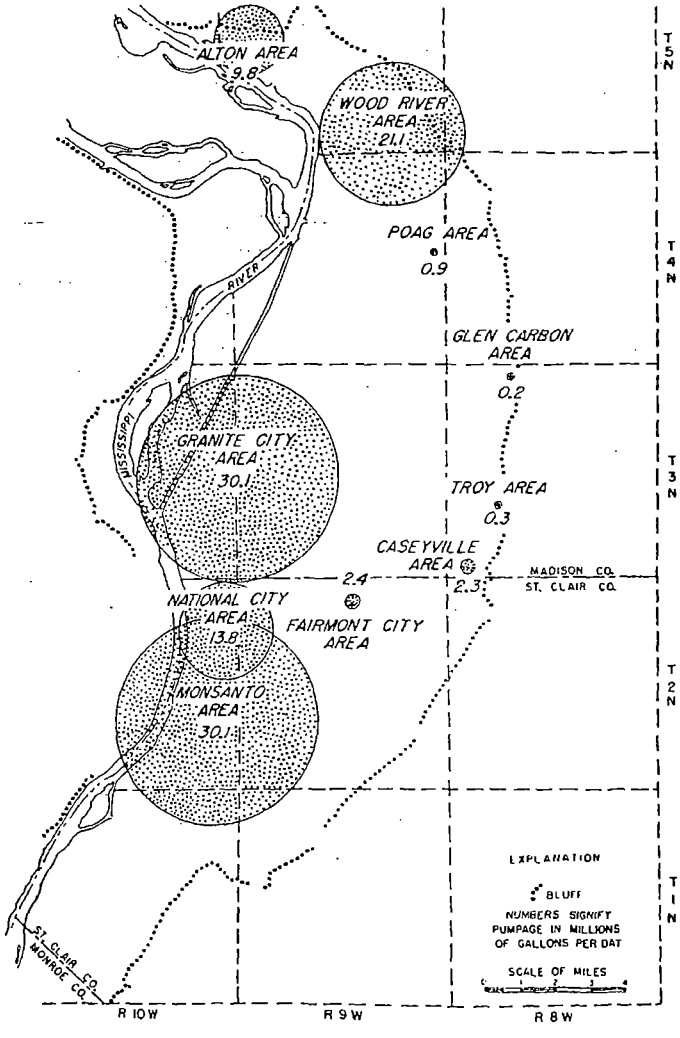


Figure 33. Distribution of estimated pumpage in 1956

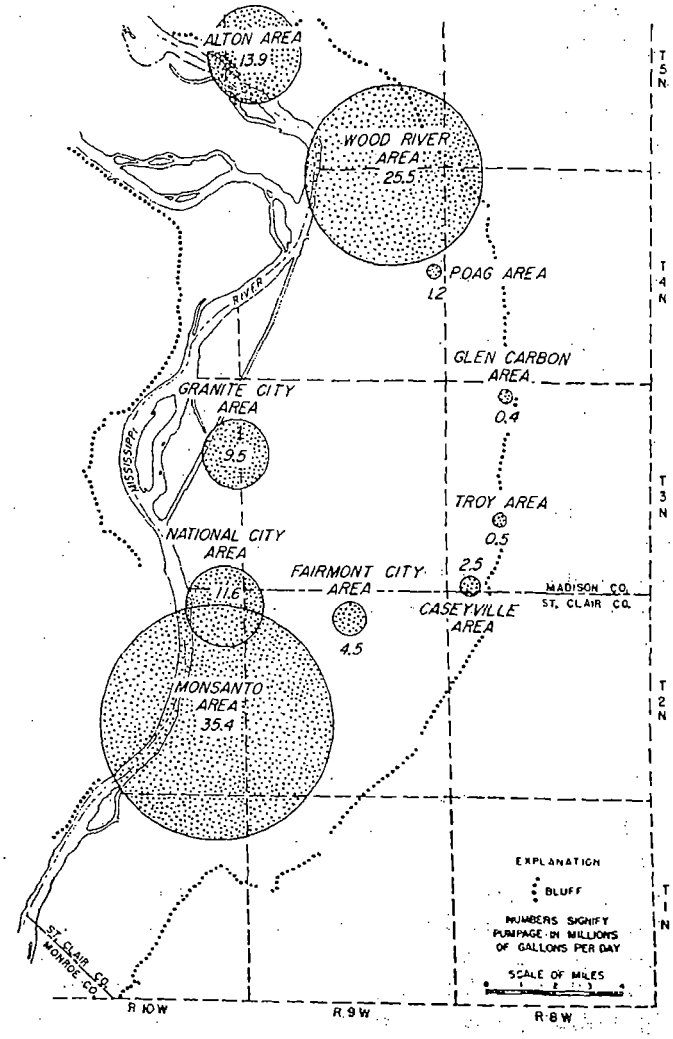


Figure 34. Distribution of estimated pumpage in 1962

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-

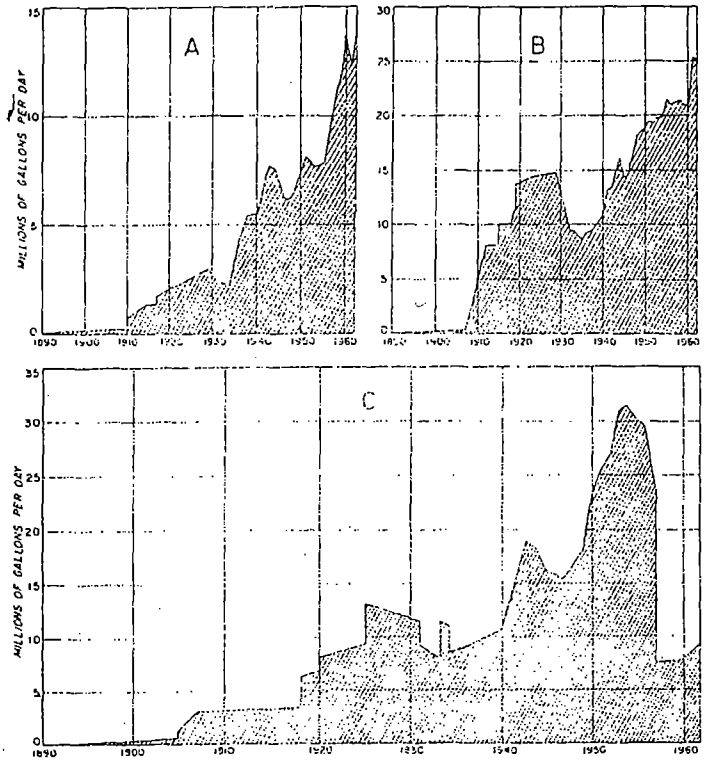


Figure 35. Estimated pumpage, Alton area (A), Wood River area (B), and Granite City area (C), 1890-1962

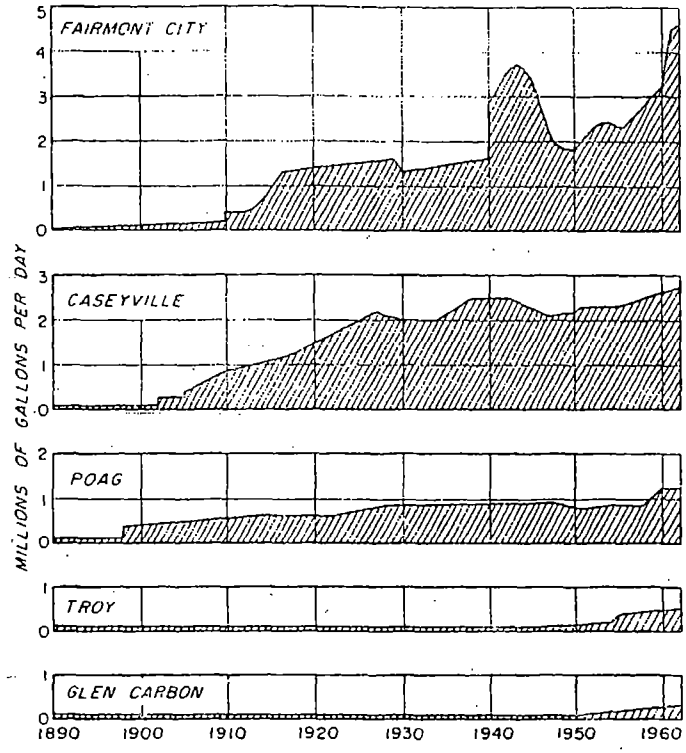


Figure 37. Estimated pumpage, Fairmont City, Caseyville, Poag, Troy, and Glen Carbon, 1890-1962

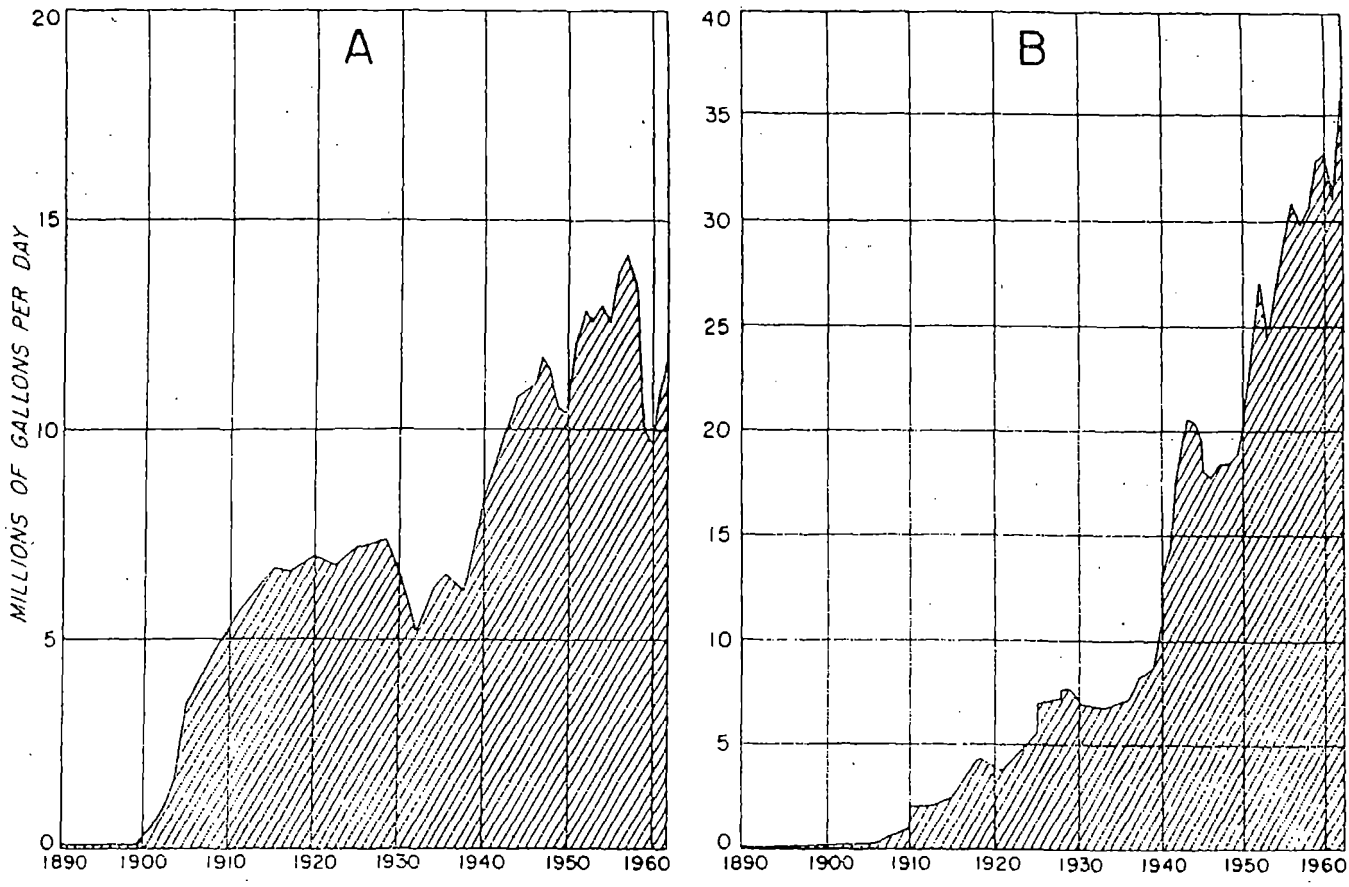


Figure 36. Estimated pumpage, National City area (A) and Monsanto area (B), 1890-1962

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.

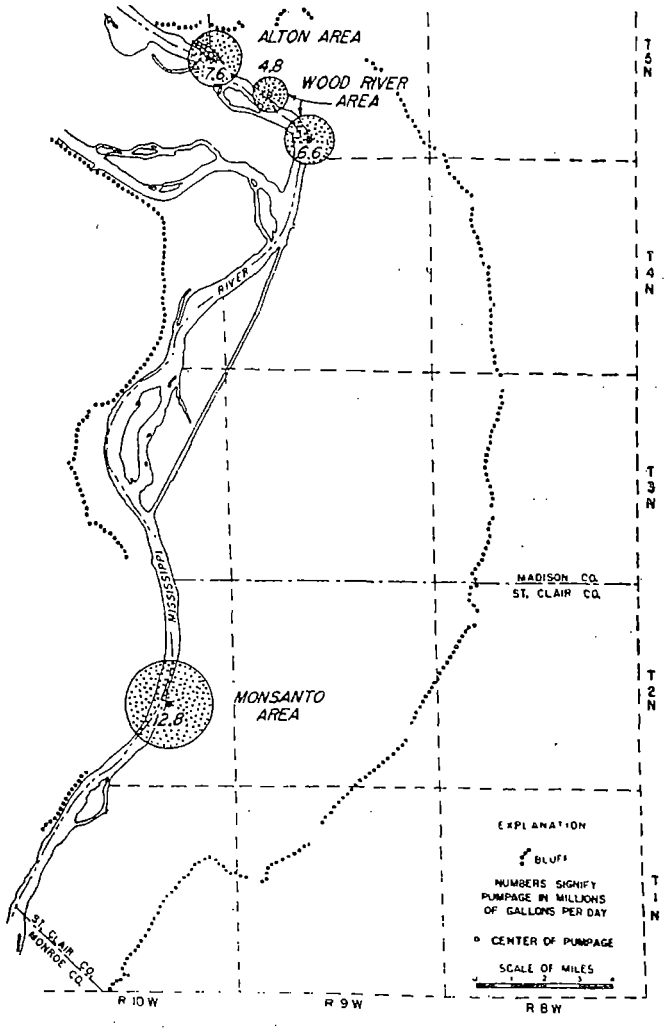


Figure 38. Distribution of estimated pumpage from wells near Mississippi River in 1962

Table 22. Distribution of Pumpage from Wells near Mississippi River

(Pumpage in million gallons per day)

Pumping center	1956		1960		1961		1962	
	From all wells in center	From wells near river	From all wells in center	From wells near river	From all wells in center	From wells near river	From all wells in center	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 ad-

dition, 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

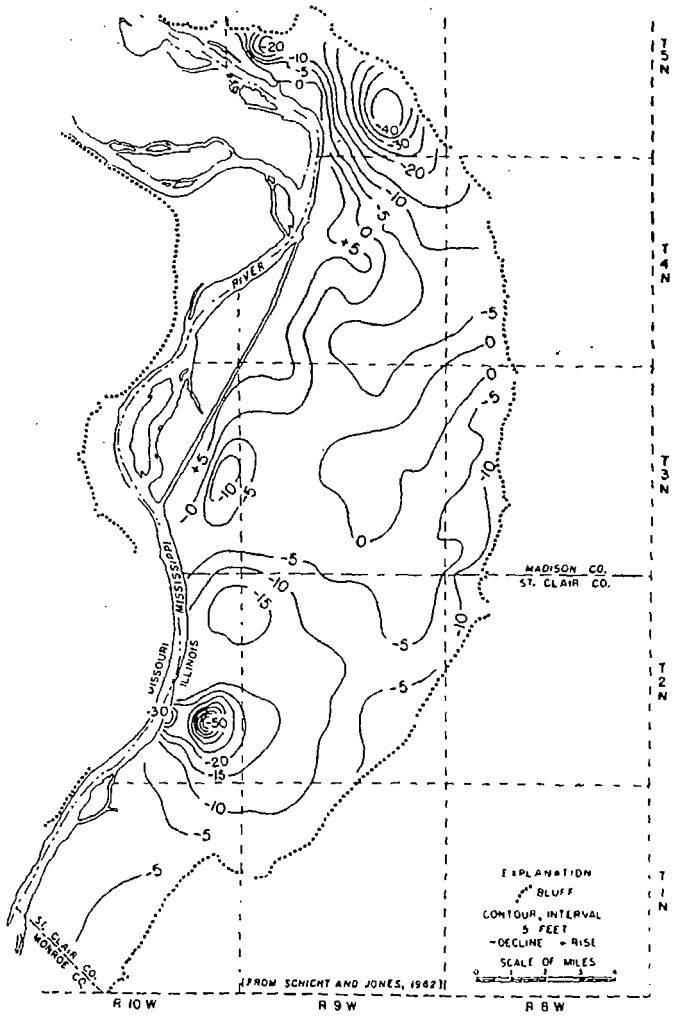


Figure 39. Estimated change in water levels, 1900 to November 1961

have not changed appreciably in the Horseshoe Lake area.

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 th northern reach of Chain of Rocks Canal. Immediatel 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

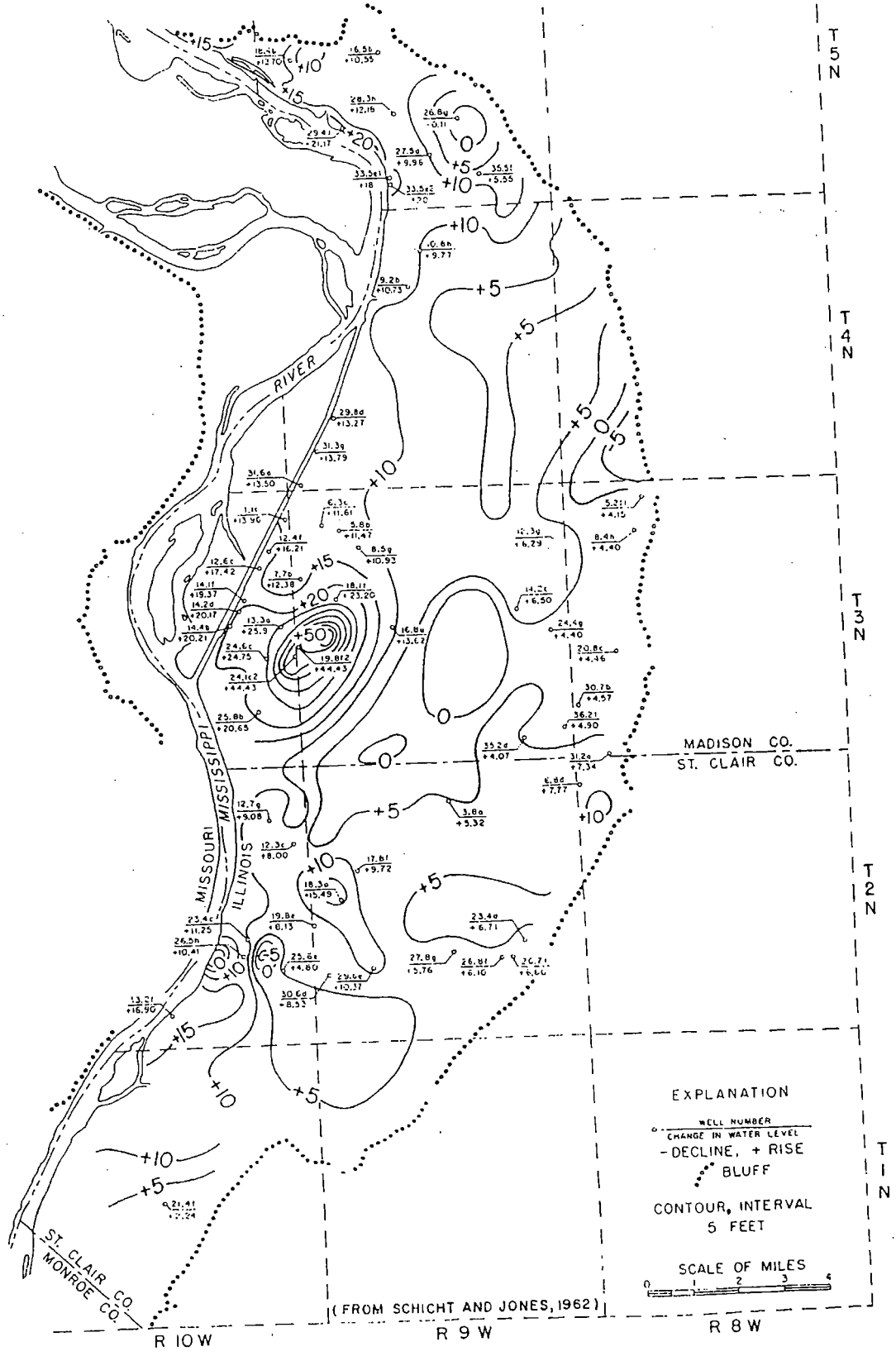


Figure 40. Estimated change in water levels, December 1956 to November 1961

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

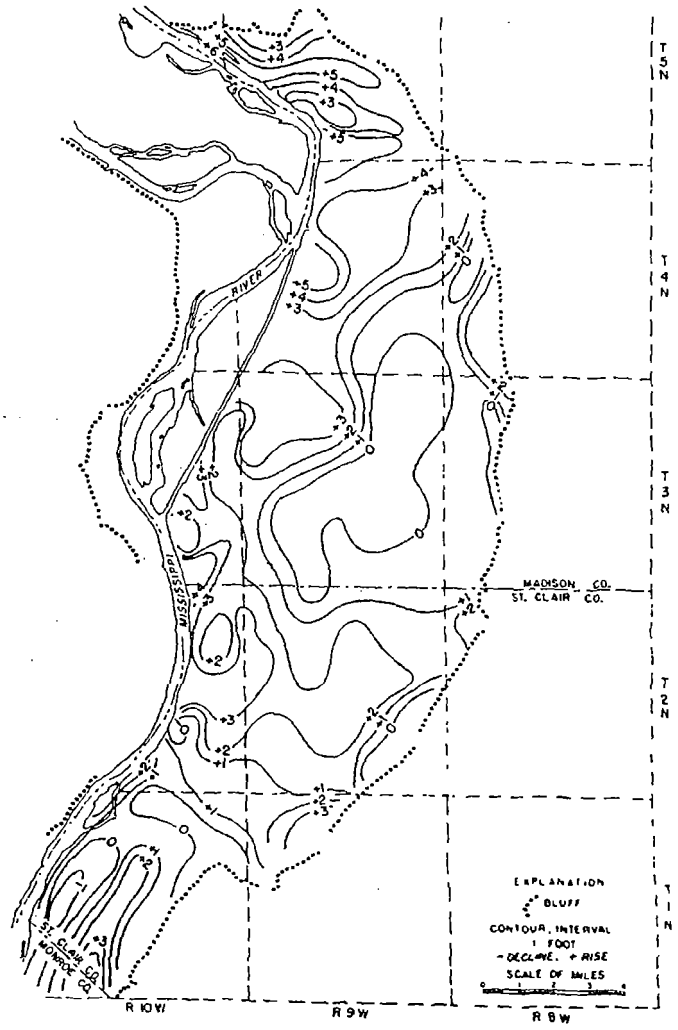


Figure 42. Estimated change in water levels, June 1961 to June 1962

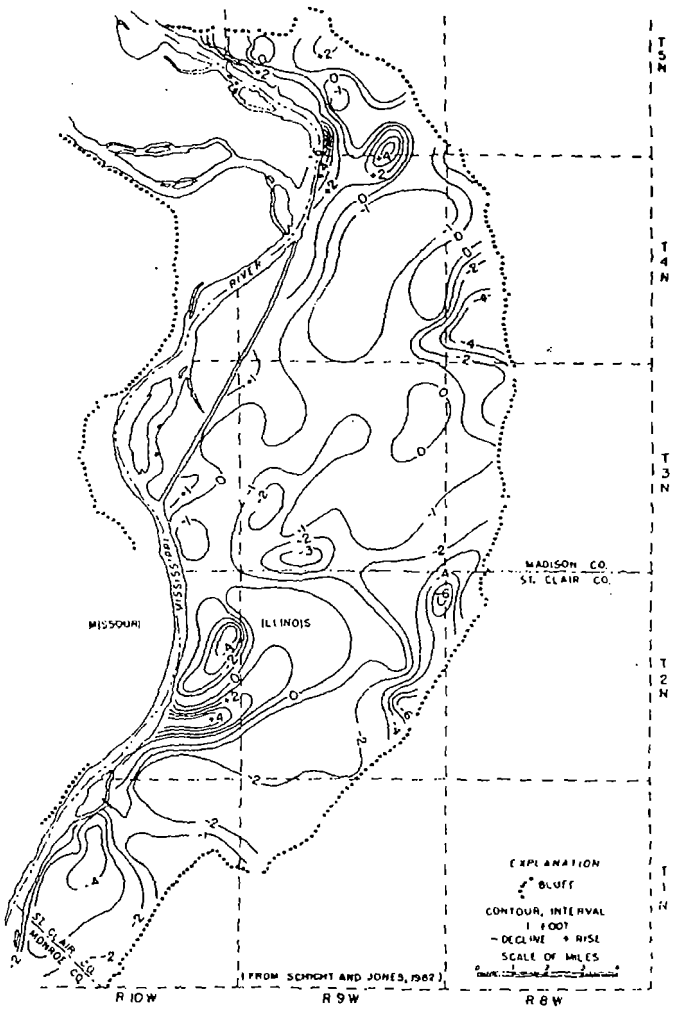


Figure 41. Estimated change in water levels, June to November 1961

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 norm The hydrograph of water levels in well MAD 3N8 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

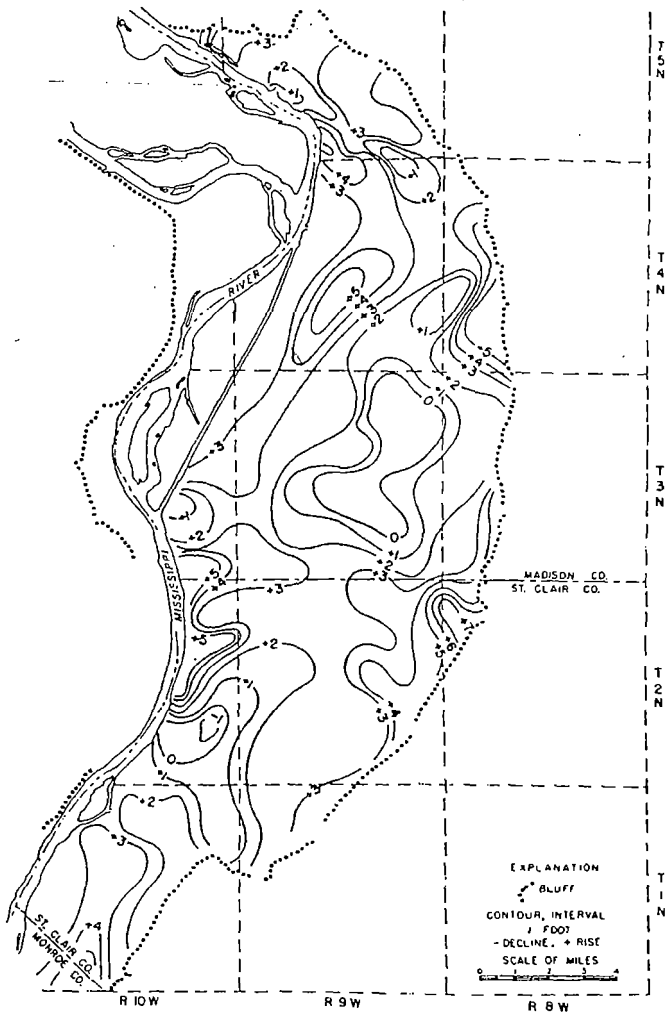


Figure 43. Estimated change in water levels, November 1961 to June 1962

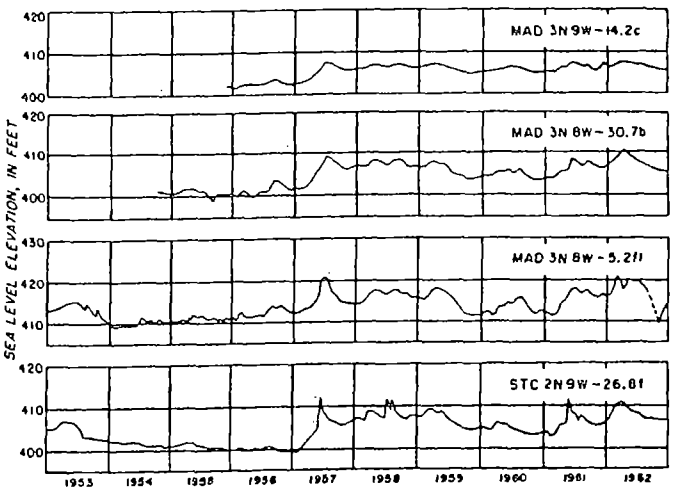


Figure 44. Water levels in wells remote from major pumping centers, 1953-1962

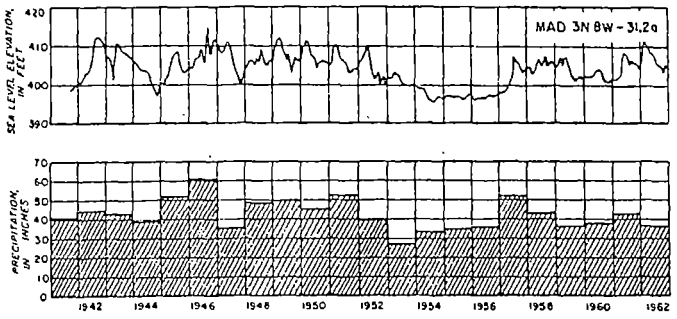


Figure 45. Water levels in well MAD 3N8W-31.2a and annual precipitation at Edwardsville, 1941-1962

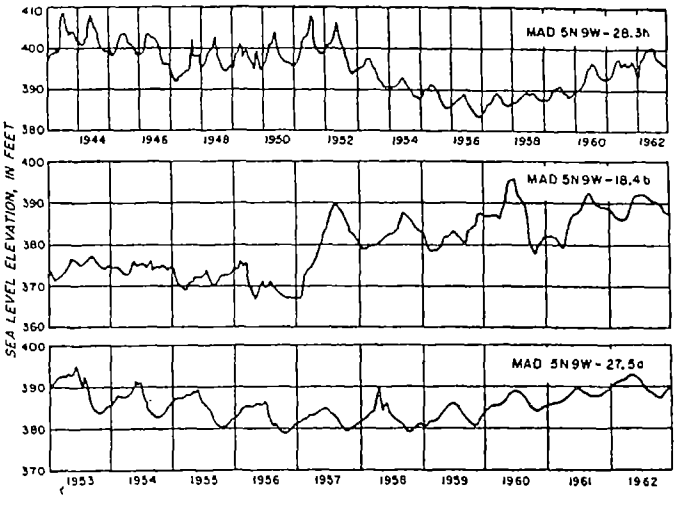


Figure 46. Water levels in Alton and Wood River areas

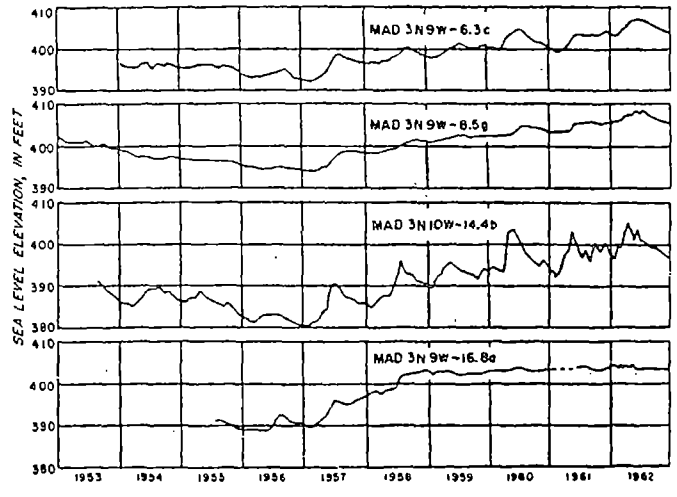


Figure 47. Water levels in Granite City area, 1951-1962

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.

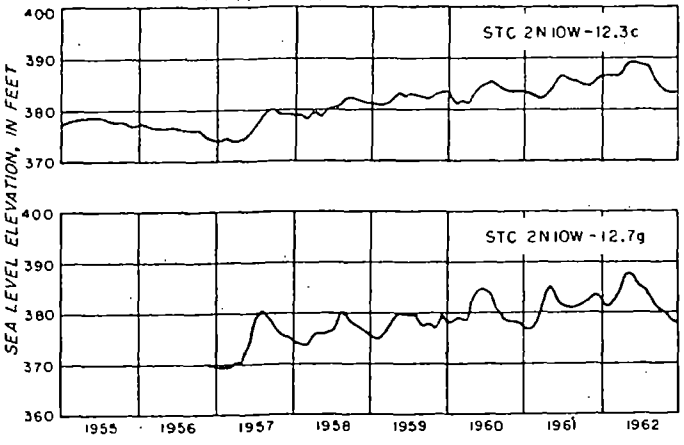


Figure 48. Water levels in wells in National City area, 1955-1962

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.

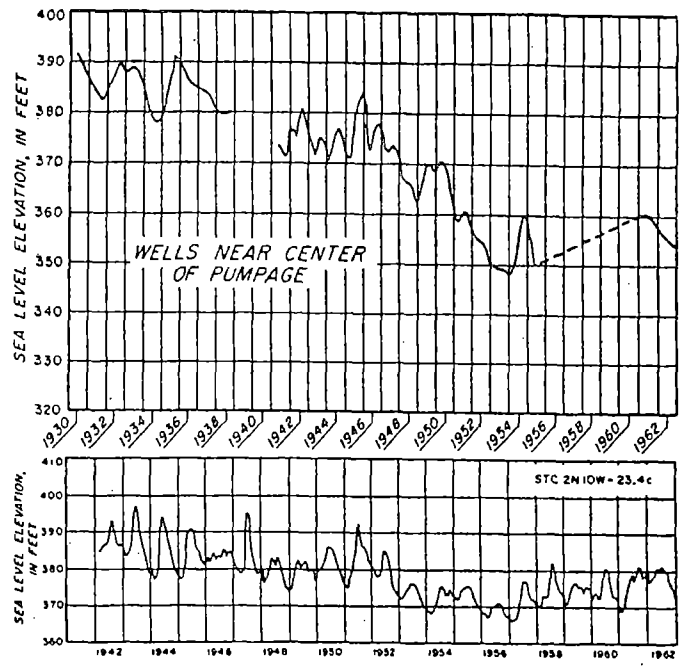


Figure 49. Water levels in wells in Monsanto area

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

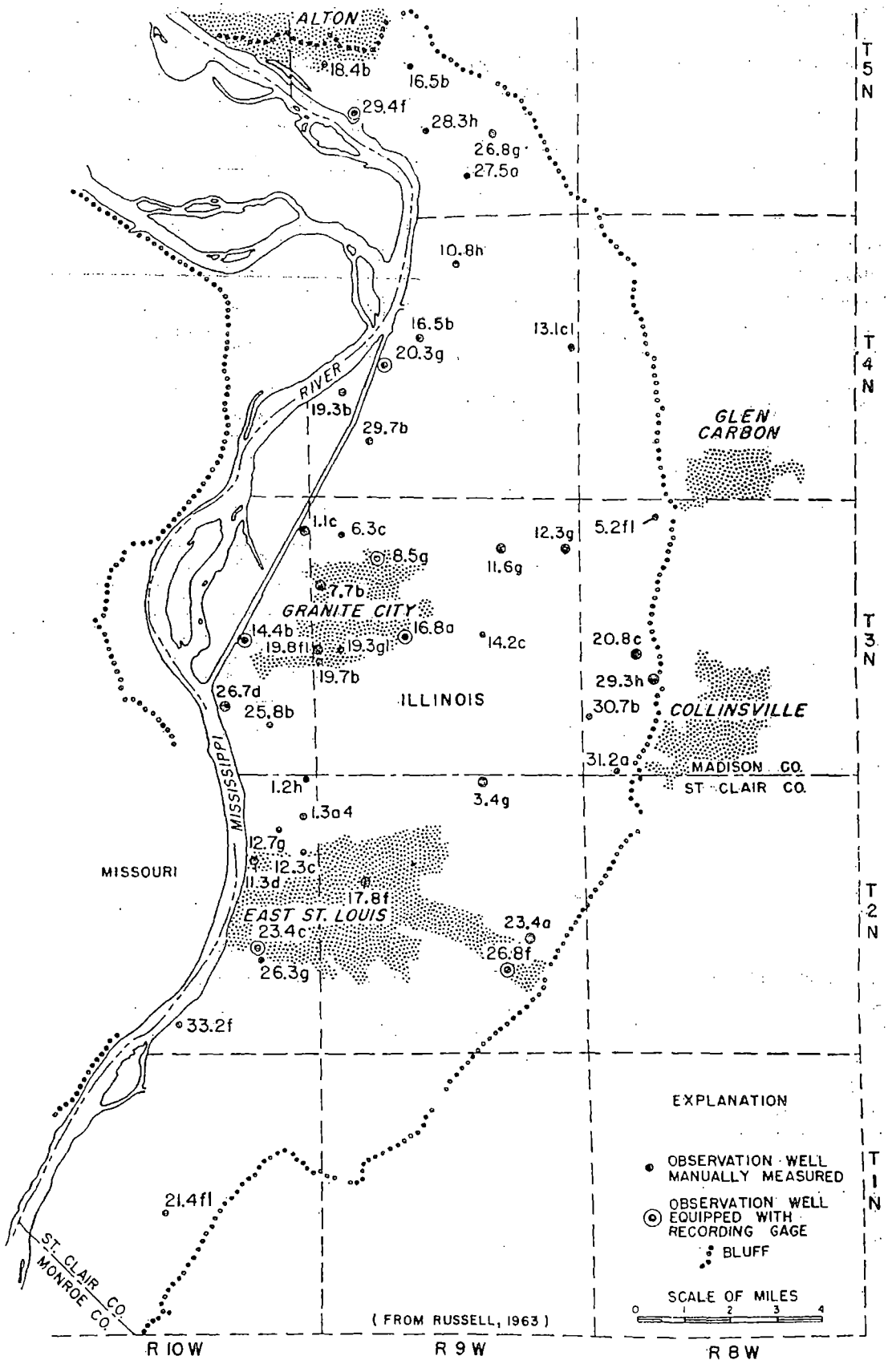


Figure 50. Locations of observation wells

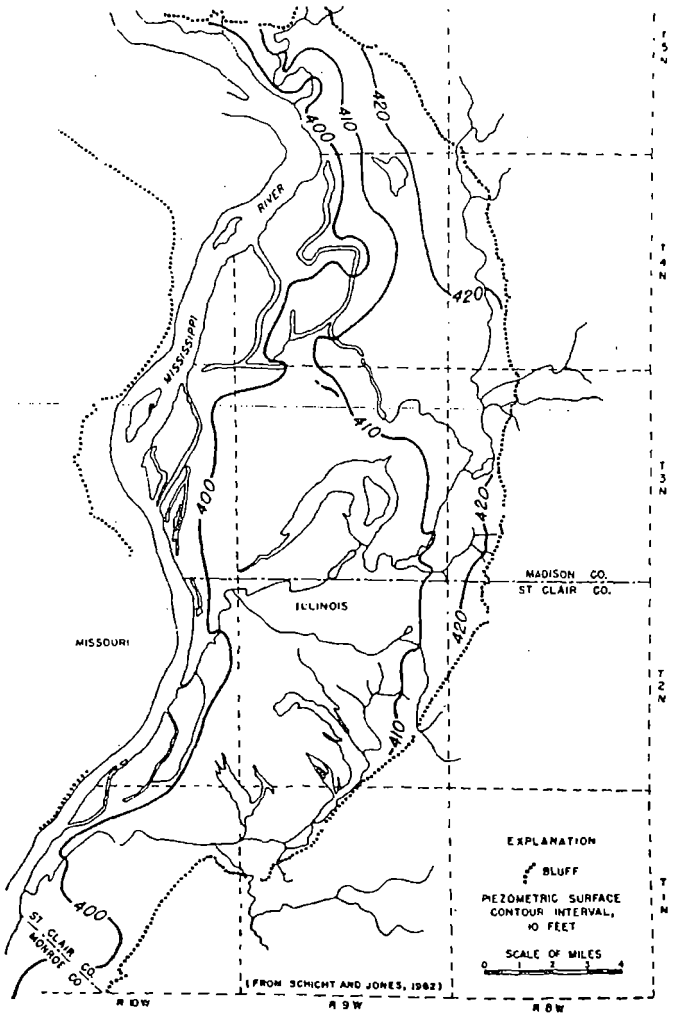


Figure 51. Drainage system and estimated elevation of piezometric surface about 1900

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

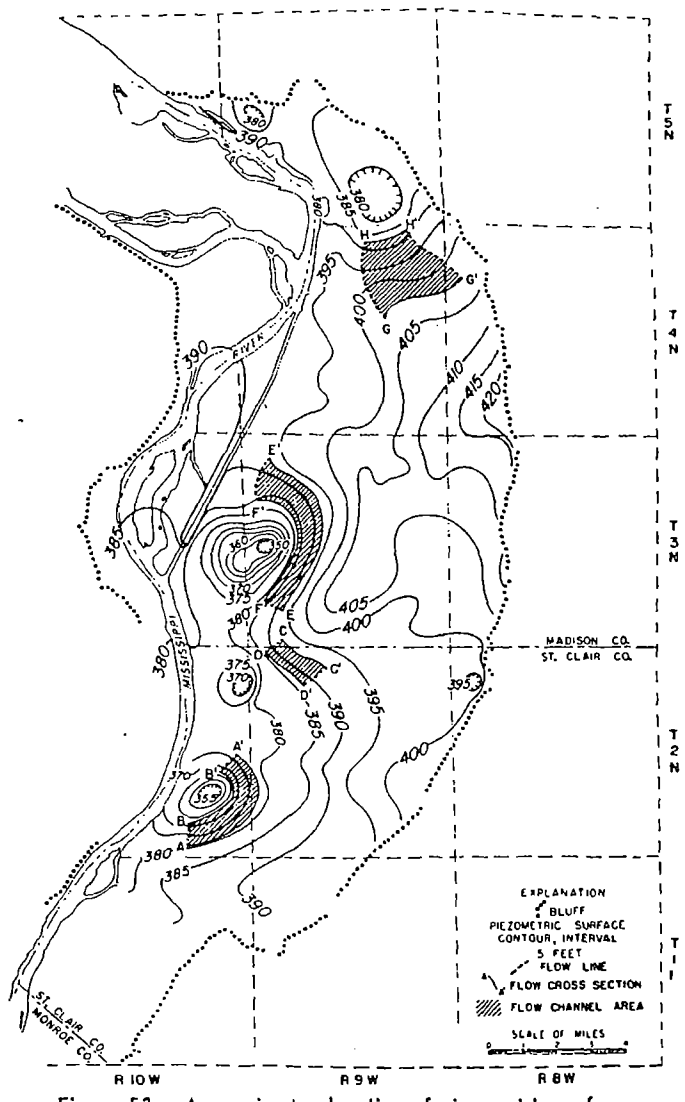


Figure 52. Approximate elevation of piezometric surface, December 1956

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
13.6c1	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
33.5e1	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
4N10W-					
36.5f	415	2.75	412.25	—0.77	+1.89
3N8W-					
5.2f1	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
19.3g1	417.74	15.78	401.96		
19.8f1	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
20.8d1	416.68	13.57	403.11	—0.13	+2.10
20.8d2	414.71	11.89	402.82	—0.16	+1.86
20.8e1	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
12.6c	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
24.1c1	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)

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
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
7.6c	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
1.3a1	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
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
26.1g1	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
27.2h1	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.

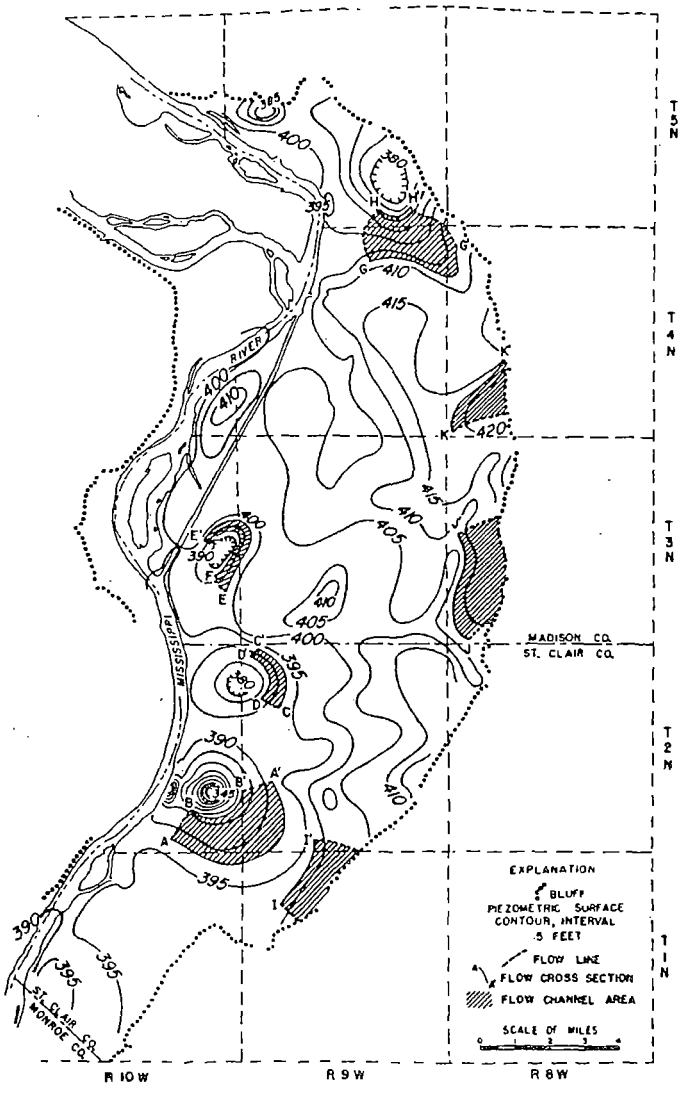


Figure 53. Approximate elevation of piezometric surface, June 1961

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.

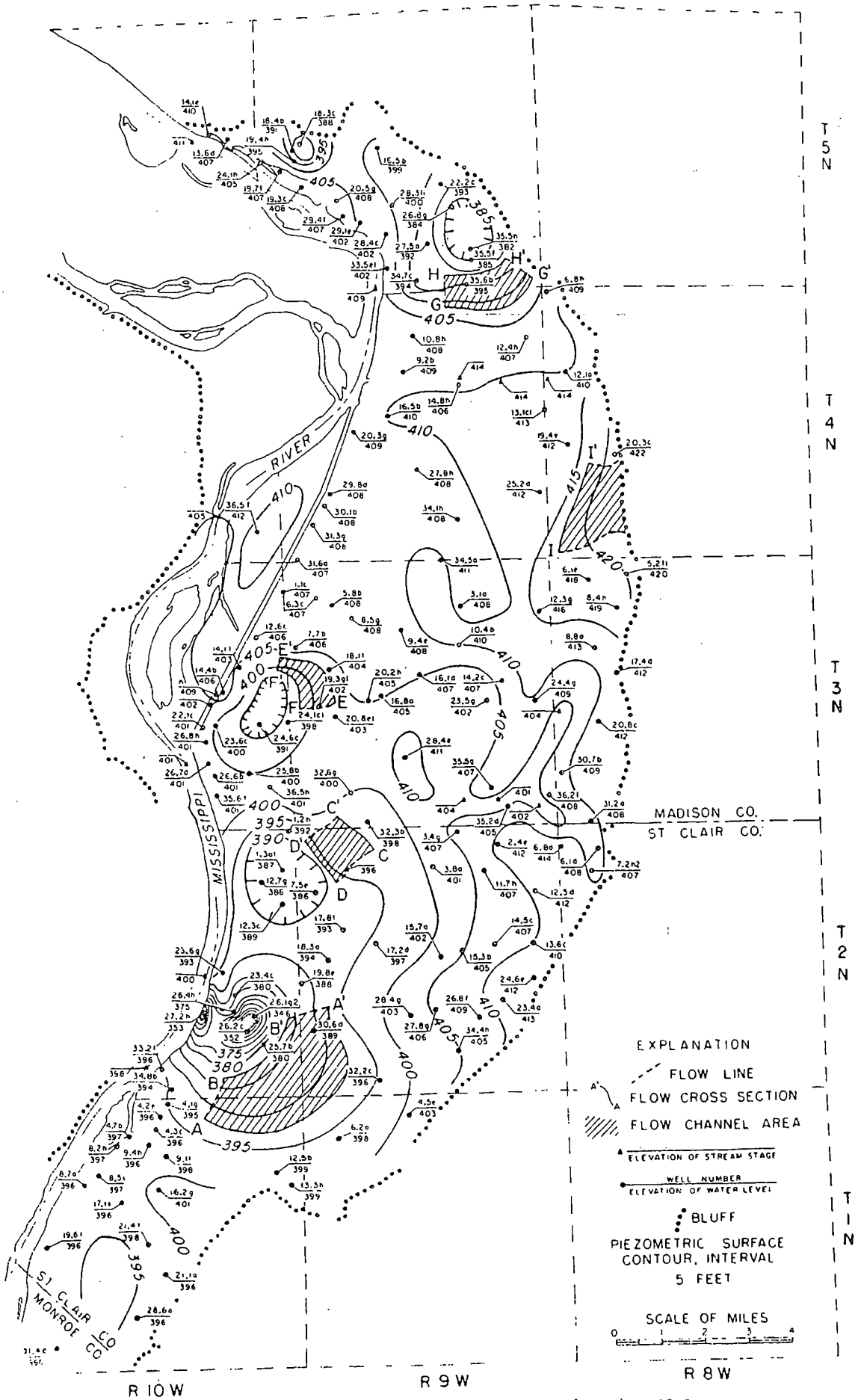


Figure 54. Approximate elevation of piezometric surface, June 1962

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):

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

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_l = 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 f 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 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.

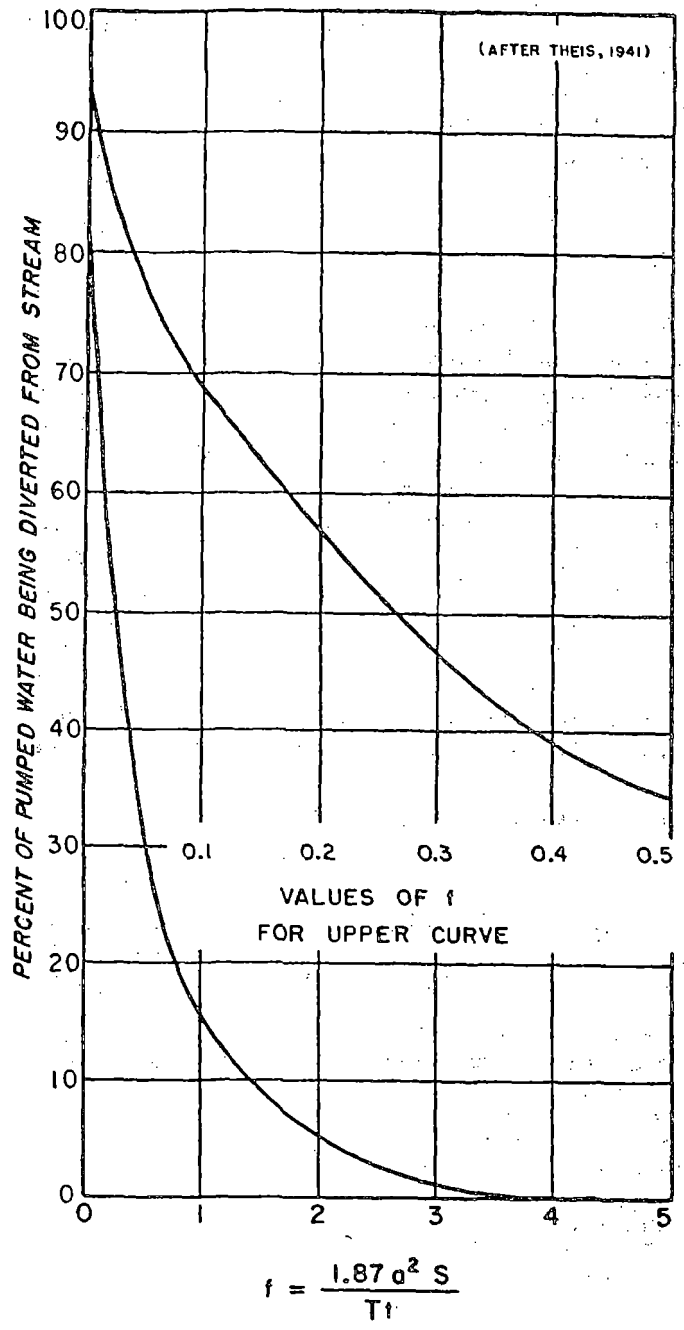


Figure 55. Graph showing the relationship between percent of pumped water being diverted from a stream and the factor 'f'

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)	River 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:

$$I_t = I_h(\mu_a/\mu_t) \qquad (21)$$

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

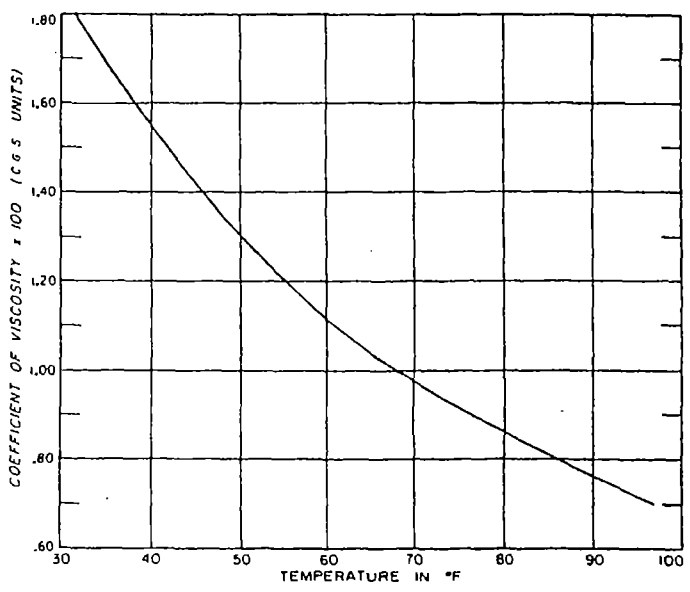


Figure 56. Graph showing relationship between coefficient of viscosity and temperature

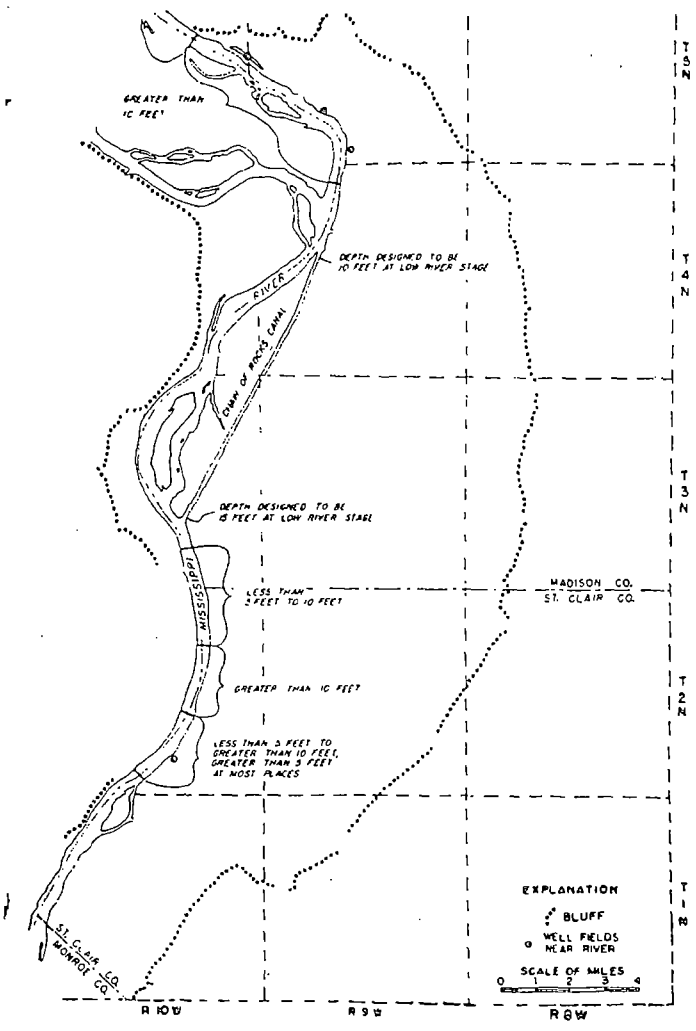


Figure 57. Estimated depths of Mississippi River and locations of well fields near river

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.

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

where:

R_i = potential recharge by induced infiltration, in gpd

I_t = 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.

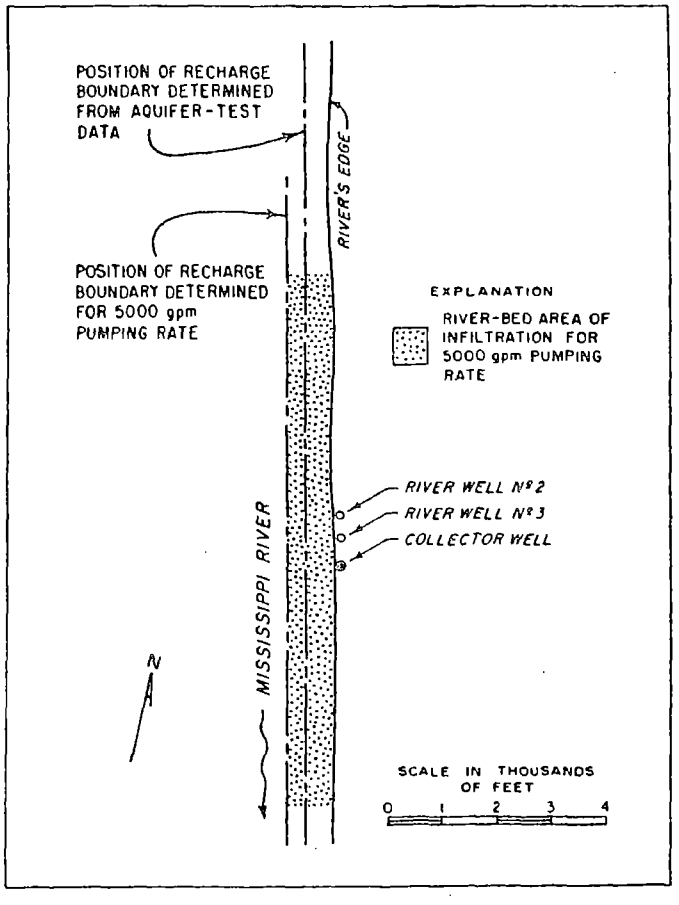


Figure 58. River-bed area of infiltration for Shell Oil Refinery well field

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 belo amounts of induced infiltr dry periods when streamflo surface water are low. Prof be used to determine the a river. Potential recharge b determined by substituting

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

Location of aquifer-test site	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 (1 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.

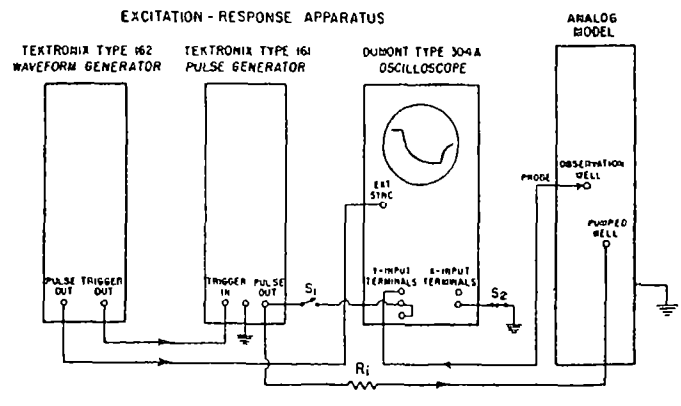


Figure 59. Schematic diagram of electric analog computer

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_i (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.

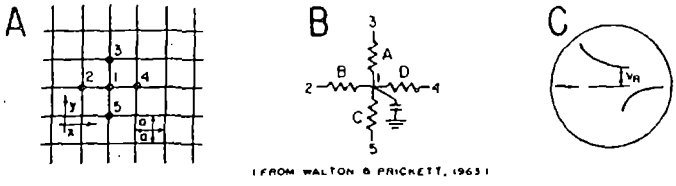


Figure 60. Finite-difference grid (A), resistor-capacitor net (B), and pumping rate oscilloscope trace (C)

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_{1-5} = 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:

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

$$h = K_2V \qquad (26)$$

$$Q = K_3I \qquad (27)$$

$$t_d\ =\ K_4t_s \qquad (28)$$

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):

$$(K_3K_4)/K_1 = 1 \qquad (29)$$

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):

$$R = K_3/(K_2T) \qquad (30)$$

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:

C = 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 $K_4 = 1.826 \times 10^5$ days/sec and when $t_d = 5$ years, the pulse duration, t_s, 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 equa-

tions 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

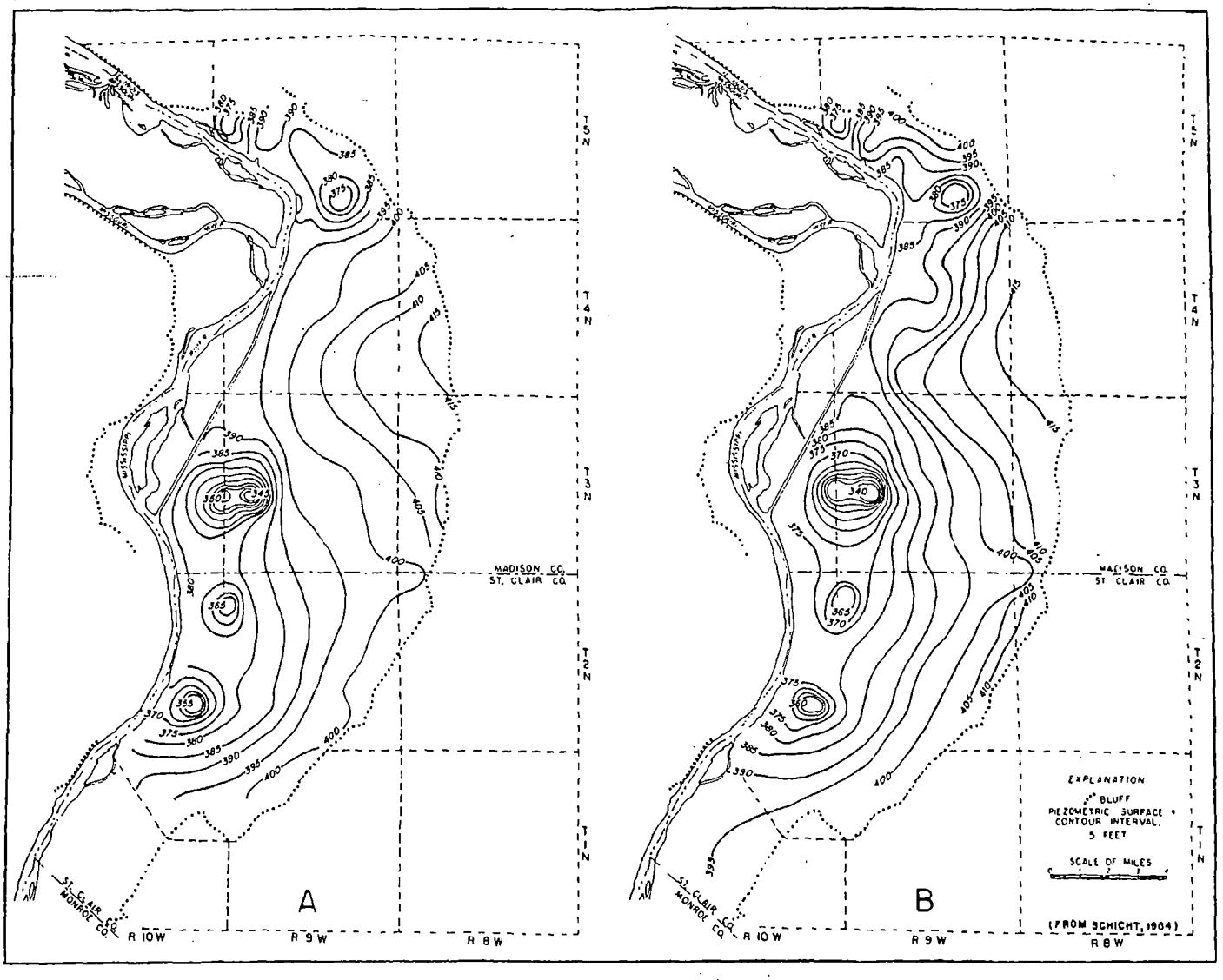


Figure 61. Elevation of piezometric surface, December 1956, actual (A), based on analog computer results (B)

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 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 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

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.

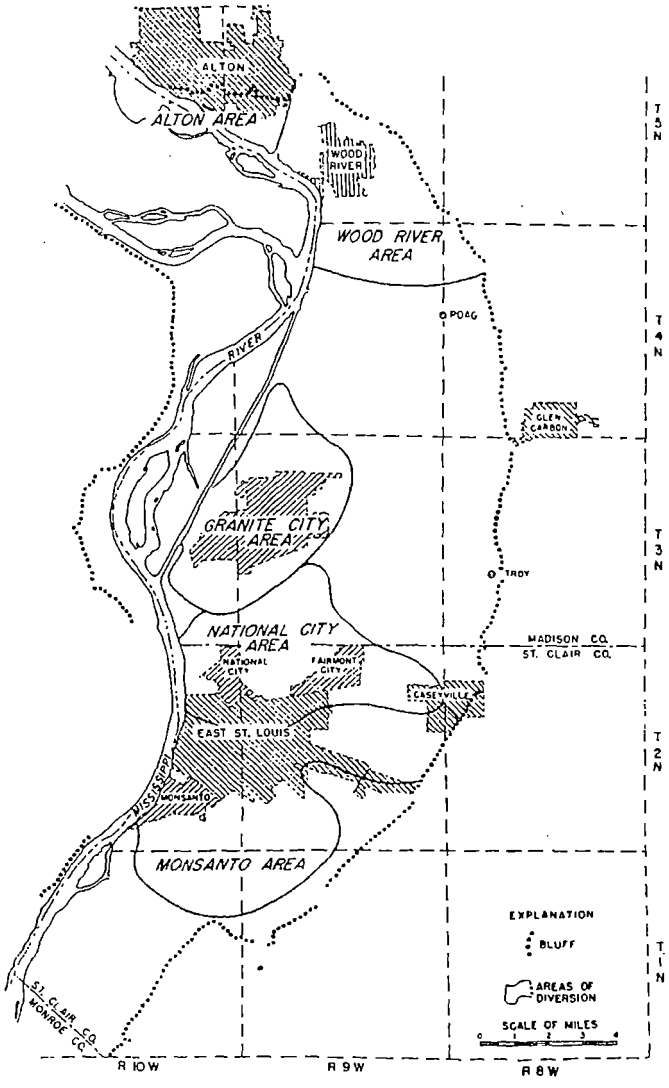


Figure 62. Areas of diversion in November 1961

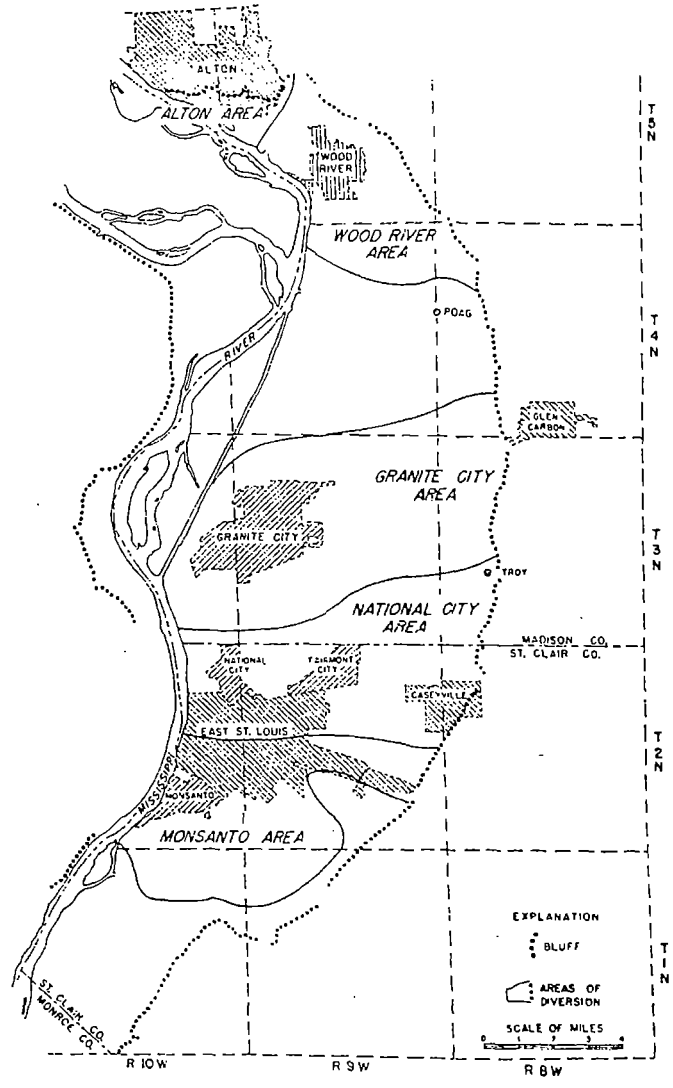


Figure 63. Areas of diversion in December 1956

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

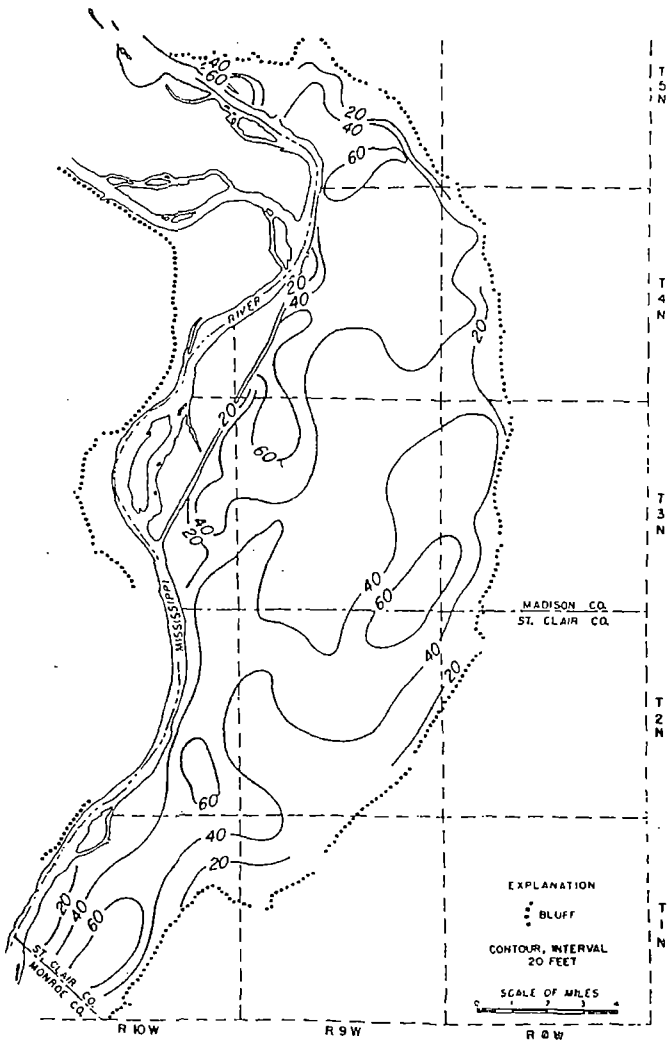


Figure 64. Thickness of medium sand and coarser deposits in lower part of valley fill

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

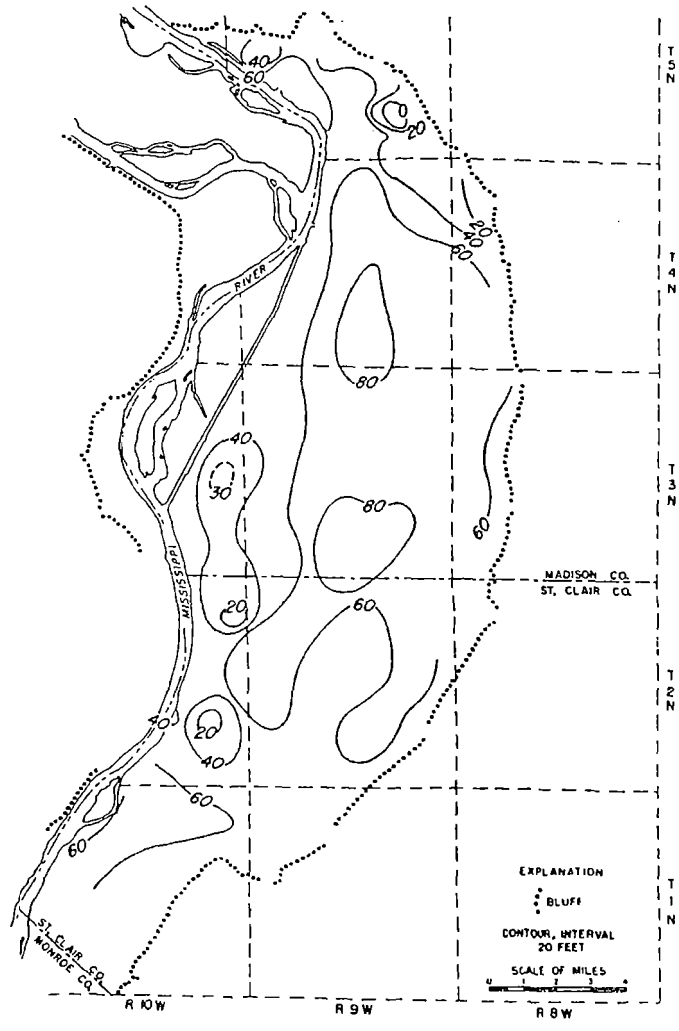


Figure 65. Estimated available drawdown to top of medium sand and coarser deposits in June 1962

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

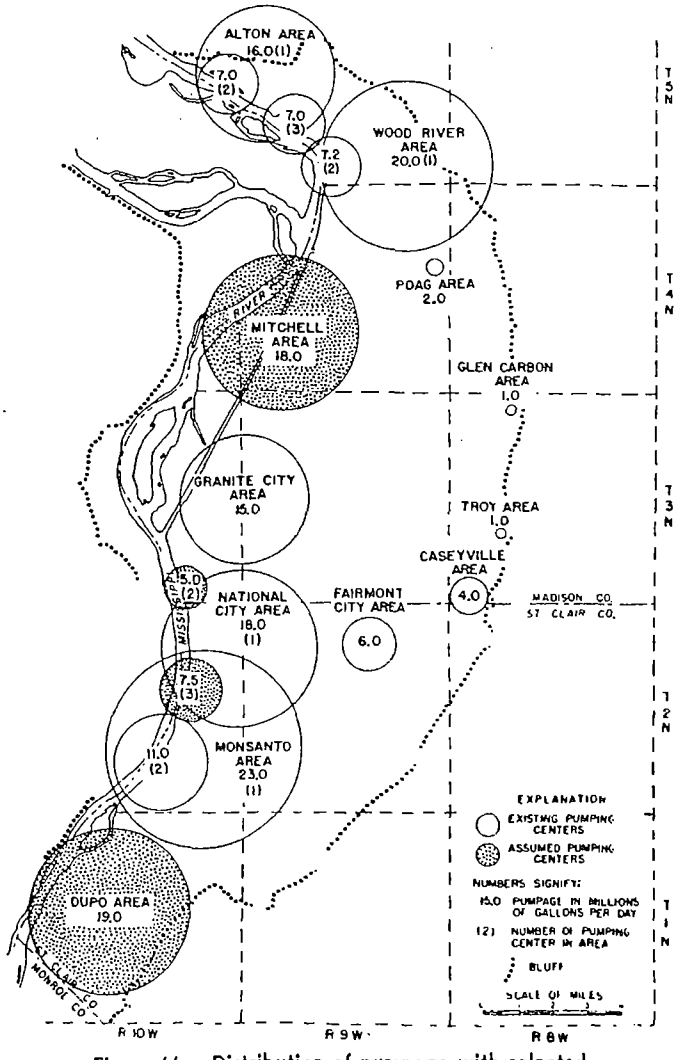


Figure 66. Distribution of pumpage with selected scheme of development

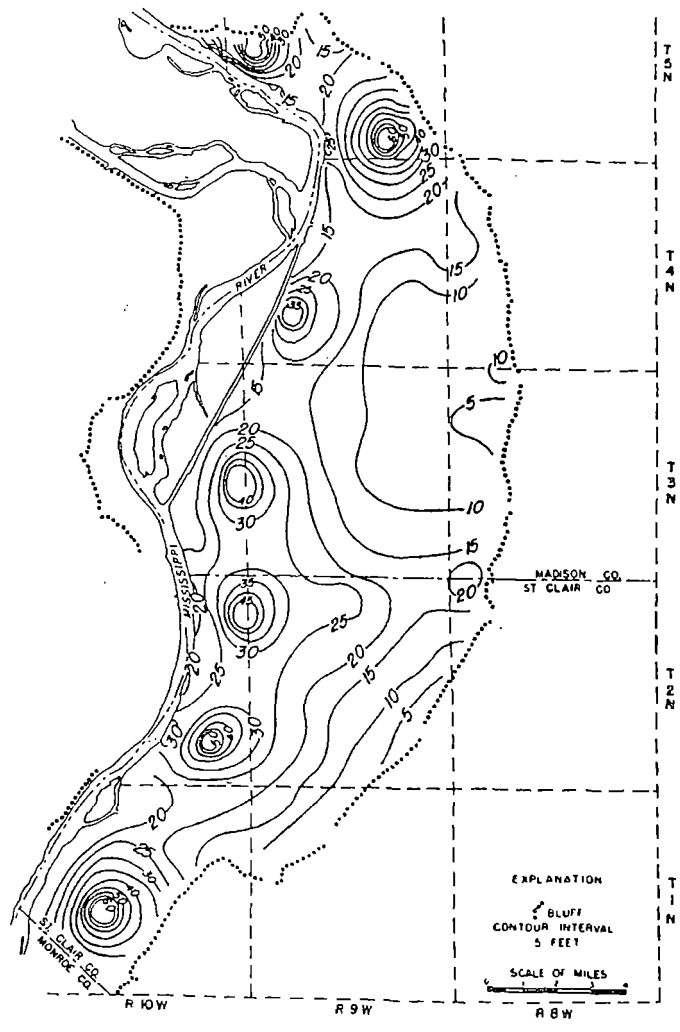


Figure 67. Water-level declines with a selected scheme of development

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

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.

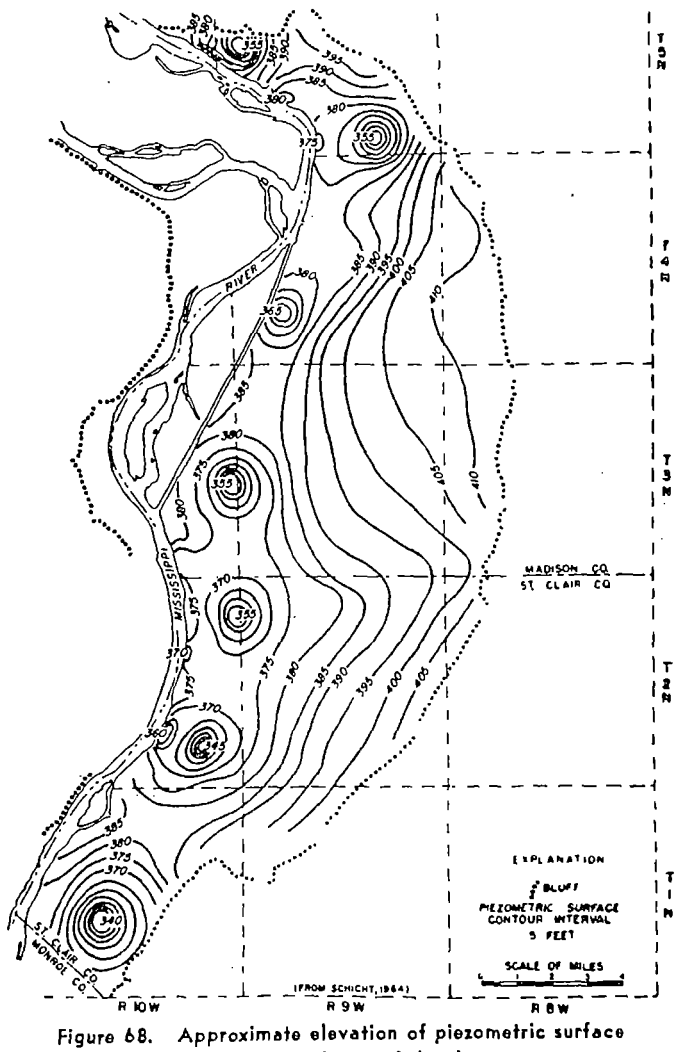


Figure 68. Approximate elevation of piezometric surface with a selected scheme of development

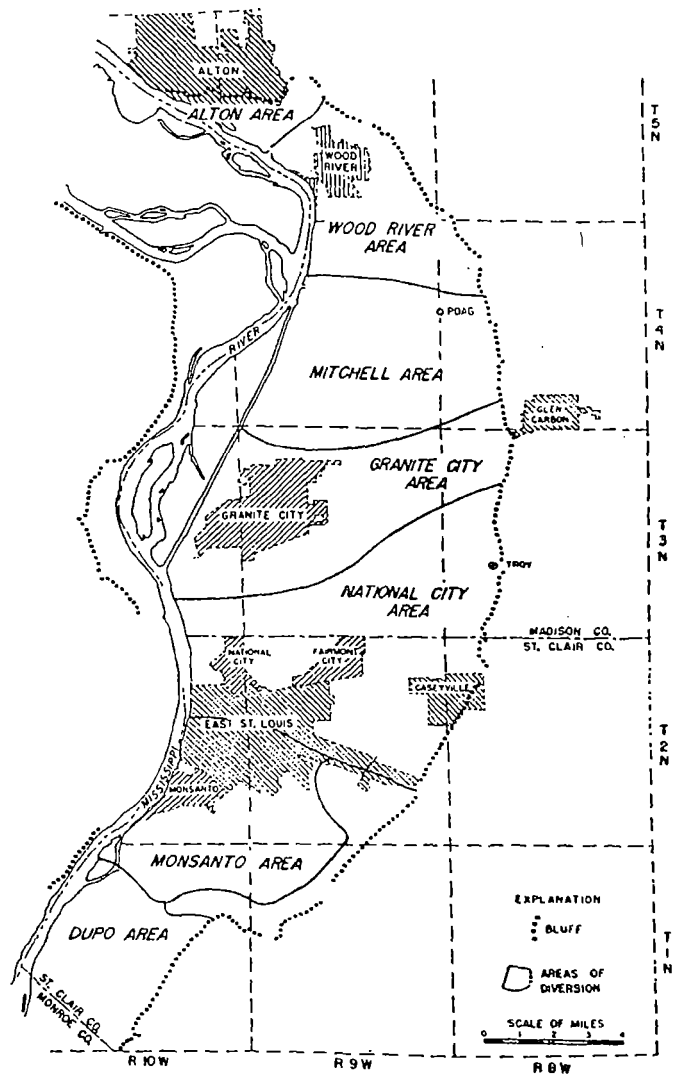


Figure 69. Areas of diversion with selected scheme of development

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

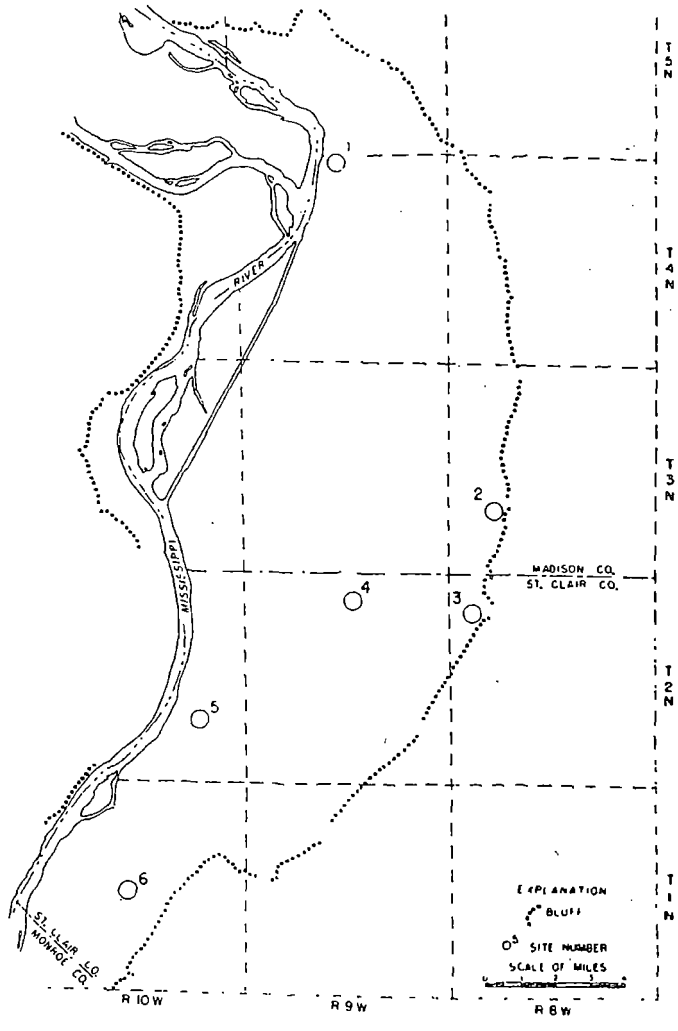


Figure 70. Sites where ground-water samples are periodically collected

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-26.7c	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		56
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	56
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-16.3a1	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-16.5b1	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-19.3h1	26	1/ /44		0.2						446		26			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- 5.2f1	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- 8.4h1	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-20.8c1	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-30.7b1	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
3N8W-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- 6.4a1	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- 1.1d1	52	7/22/52		6.4						304	61.7	6			357	384		57
3N10W-1.1d2	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-7.2h1	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		
1N10W-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- 7.6e1	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						306	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 (Cl)	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	110	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-13.7g1		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-26.3d1	95	6/24/43		8.0						374		29			533	668		60
2N10W-26.3d2	110	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.

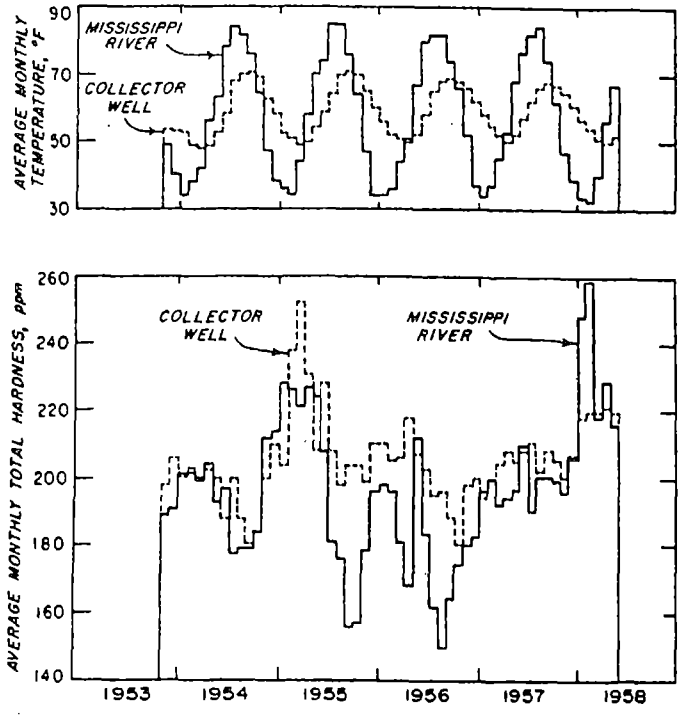


Figure 71. Chemical and temperature data for collector well and Mississippi River, 1953-1958

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-	165-	388.8-	572-	956-	1400-	59-	Well 3
		3.6	260	496.4	872	1140	1894	61	6/25/56 to 1962
	3	1.6-	150-	371.3-	408-	820-	1202-	58-	Well 1
		6.8	220	485.5	776	1100	1761	64	10/31/49-5/28/56
Hartford (V)	1	2.3-	5-	43.4-	204-	247-	336-	55.5	Well 2
		12.2	30	129.8	384	482	580	61	7/5/47 to 1962
Virginia Carolina Chemical Co.	4	2.3-	2-	1.4-	272-	331-	353-	55-	7/10/44 to 1962
		50	22	1130.1	456	1560	2075	60	
Missouri and Pacific R. R.	6	5.4-	9-	52.2-	364-	420-	473-	57-	South well
		15.0	28	235.5	428	532	731	62	10/28/59-9/3/58
	6	0.4-	2-	55.6-	76-	129-	240-	57-	North well
		20.8	93	374.2	504	733	963	62.5	3/10/44-10/7/54
Hartford (V)	1	0.7-	4-	39.3-	146-	227-	346-	55.5	Well 1
		16.4	21	129.4	400	501	590	57	3/31/50 to 1962
Troy (V)	2	0.1-	3-	103.1-	264-	396-	438-	56-	Well 1
		0.6	11	177.1	310	464	563	57.5	1/25/54 to 1962
American Zinc Co.	5	9.6-	20-	254.5-	280-	521-	624-		Well 2
		50.4	119	744.2	392	1080	1596		11/22/44-11/15/48
	5	15 -	43-	260.4-	236-	503-	766-		Well 7
		23	118	422.7	320	638	1150		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	Discharge (ft/sec)	Laboratory number	Temperature (°F)	Turbidity	Iron (Fe)	Manganese	Fluoride (F)	Boron (B)	Silica (SiO_2)	Chloride (Cl)	Sulfate (SO_4)	Nitrate (NO_3)	Ammonium (NH_4)	Calcium (Ca)	Magnesium (Mg)	Sodium (Na)	Alkalinity (as $CaCO_3$)	Total hardness (as $CaCO_3$)	Total dissolved minerals
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	6.9	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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DRAFT
GSI Job No. G-2561
March 31, 2002

GROUNDWATER
SERVICES, INC.

Interim Groundwater Remedy Design Basis

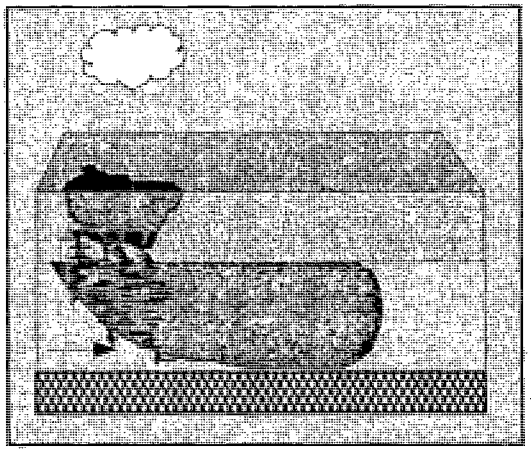
Sauget Area 2
Sauget, Illinois

ATTACHMENT 8

SOURCE EVALUATION STUDY

Sauget Area 1

Sauget and Cahokia, Illinois



Submitted to
Solutia Inc.

May 21, 2001

Groundwater Services, Inc.
2211 Norfolk, Suite 1000, Houston, Texas 77098

May 21, 2001

GROUNDWATER
SERVICES, INC.

EXECUTIVE SUMMARY

An evaluation of the source of impacted groundwater at Site I within Sauget Area 1 was performed to evaluate two questions:

1) *What is the dominant source mechanism at the site?*
2) *What is the effect of an intensive pump-and-treat system on the lifetime of the source?*

Dominant Source Mechanism

Two source mechanisms that have the potential to be active at the Sauget site are: 1) *leaching* of unsaturated source materials, and 2) residual Dense Nonaqueous Phase Liquid *(DNAPL) dissolution* (see Figure 1). Six general indicators were evaluated to assess which of these two source mechanisms are primarily responsible for the observed plumes associated with Site I.

The analysis indicated that *DNAPL dissolution* is a major source mechanism at Site I based on an analysis of six different indicators. The following indicators support the conclusion that trapped residual DNAPL is present: dissolved constituent concentrations increase with depth, site constituents extend from the water table to the bottom of the water-bearing interval, and three constituents are found at concentrations that suggest the possible presence of non-mobile residual DNAPL. Some leaching of unsaturated waste/soil materials may also be occurring, as some constituents did not show increasing concentrations with depth. Overall, however, **DNAPL dissolution** appears to be the dominant source mechanism at Site I.

Source Conceptual Model

If DNAPL dissolution is the dominant source mechanism at Site I, it is likely that the DNAPL in the unconsolidated valley fill deposits is present as thin vertical fingers and small horizontal pools throughout the entire vertical extent of the water-bearing unit. Only a small fraction of the total DNAPL mass can ever be removed by pumping any "free-phase" DNAPL pools, if they are found. The rest of the DNAPL is immobile, and will serve as a long-term continuing source of constituents to groundwater.

The current natural mass removal rate via dissolution from the Site I source zone was estimated to be 7000 kg/yr assuming uniform source concentrations throughout the source zone.

Effect of Pumping

As shown by DNAPL dissolution expressions, increasing the flow rate through a DNAPL source zone will significantly decrease the concentration of constituents in the extracted groundwater. For example, if the flowrate through a DNAPL source zone is increased by a factor of 8.9 (to 1500 gpm) due to intensive pumping, the resulting concentration is likely to decrease by a factor of 3.6 while pumping is active, resulting in an overall increase in the mass removal rate of only 2.5 times. Therefore, an intensive pump-and-treat system at Site I with 8.9 times the natural flowrate through the source area (an achievable pumping rate if there is no reinjection) would result in an initial mass removal rate of 17,500 kg/yr.

A planning level source lifetime calculation was done to estimate the relative performance of various remediation schemes. This analysis, while not providing high-confidence estimates of the absolute time to cleanup, does indicate that with an assumed mass of 410,000 kg of VOCs + SVOCs in the saturated zone below Site I, intensive pumping over a 10 to 30 year period does not appear to have an appreciable effect on overall source lifetime (i.e., ≤ 10% reduction). Similar limitations are expected for Sites G/H/L as well.

INTRODUCTION

As requested by Solutia Inc. (Solutia), Groundwater Services, Inc. (GSI), has completed a study of hydrogeologic, source, and fate and transport data from the Sauget Area 1 located in Sauget and Cahokia, Illinois. The study was conducted to: 1) help determine what type of source mechanisms are responsible for dissolved constituents found in the affected groundwater, and 2) determine the feasibility of remediating this source area by aggressive pumping. This letter report summarizes the results of the study.

PROJECT BACKGROUND

An extensive RI/FS study of Sauget Area 1 is now being conducted by Solutia. Data from two groundwater monitoring well transects indicates the presence of dissolved constituents migrating west in groundwater from the vicinity of one of the six source areas in Area 1 (i.e., Site I) at concentrations exceeding Illinois Class II groundwater standards.

Source Site I

Site I originally was a sand and gravel pit which received industrial and municipal wastes from 1931 to 1957. Site I is approximately 19 acres in area and underlies a large, fenced, controlled-access, gravel covered truck parking lot and the Sauget City Hall and associated parking lots (Sauget Area 1 EE/CA and RI/FS Support Sampling Plan). Soil samples collected from Site I have indicated elevated levels of volatile organic compounds (e.g., benzene, chlorobenzene); semi-volatile organic compounds (e.g., naphthalene, trichlorobenzene); pesticides; herbicides; PCBs; and metals.

Hydrogeology

Sauget Area 1 is located in the Mississippi River floodplain in an area referred to as the American Bottoms. The geology of the area is described as consisting of unconsolidated valley fill deposits (Cahokia Alluvium) overlying glacial outwash material (Henry Formation). In general, the permeability of the unconsolidated material increases with depth, with the outwash material being comprised of medium- to coarse-grained sand and gravel. The hydrogeologic conceptual model divides the unconsolidated water-bearing unit into three horizons: the shallow horizon (generally 15-30 ft deep), the middle horizon (generally 30-70 ft deep), and the deep horizon (generally 70-110 ft deep).

These unconsolidated deposits are underlain by limestone and dolomite bedrock.

Study Constituents

For this study, two classes of constituents were evaluated. The two constituent classes were selected based on prevalence and concentration in groundwater, and include:

- Volatile Organic Compounds (chlorinated and non-chlorinated), and
- Semi-Volatile Organic Compounds (chlorinated and non-chlorinated).

SOURCE MECHANISMS

Knowledge of which source mechanisms are active at a site is important for developing an accurate conceptual model of constituent fate and transport, and for developing appropriate remedial responses. Two source mechanisms that have the potential to be active at the Sauget site are leaching of unsaturated source materials and residual DNAPL dissolution (see Figure 1).

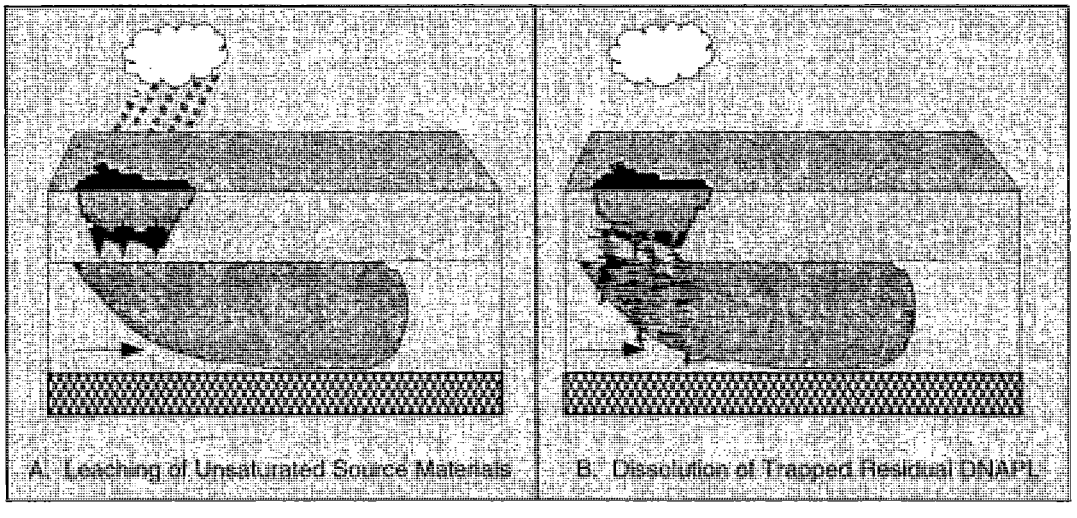


FIGURE 1. Two Potential Groundwater Source Mechanisms

Leaching of unsaturated source materials (see Panel A in Figure 1) results from infiltration of rainfall through near-surface source materials such as waste materials in the source areas and contaminated unsaturated soils. Residual DNAPL dissolution (see Panel B in Figure 1) occurs when soluble organic

constituents dissolve from trapped residual DNAPL fingers and pools that entered the subsurface when the source area was active.

EVIDENCE OF SOURCE MECHANISMS

The available groundwater data were evaluated to help assess the likelihood that the two most likely source mechanisms are present at the site. The following six indicators were used:

General Indicators of Strong DNAPL Dissolution Processes:

- Indicator 1: Concentrations are generally increasing with depth.
- Indicator 2: Constituents are found deep in the water-bearing unit.
- Indicator 3: Concentrations are above 1% of the pure-phase solubility.
- Indicator 4: Results of EPA Quick Reference Fact Sheet "Estimating Potential for Occurrence of DNAPL at Superfund Sites," (Newell and Ross, 1992).

General Indicators of Strong Soil Leaching Processes:

- Indicator 5: Leachate concentrations (as indicated from TCLP tests of unsaturated waste materials) are greater than groundwater concentrations in the shallow horizon.
- Indicator 6: Concentrations in the shallowest horizon are greater than in deeper horizons.

To assess these indicators, groundwater constituent data were compiled (see Tables 1-2). Data from the groundwater transect (well AA-I-S1 for Site I) were used to evaluate constituent concentrations in the shallow horizon (< 30 ft deep) vs. middle horizon (30-70 ft deep) vs. deep horizon (70-110 ft deep) (see Table 1). Note that only the transect monitoring well closest to the source area was evaluated.

RESULTS

Indicator 1: An evaluation of groundwater data for Site I shows that the sum of *maximum* detectable VOCs + SVOCs in groundwater concentrations from the deep horizon is 47.5 mg/L, compared to only 22.1 mg/L in the shallow horizon (see Table 1). This trend is also seen in the majority of the individual VOC and SVOC constituents. For example, the maximum chlorobenzene concentration increases from 8.7 mg/L in the shallow horizon, to 20 mg/L in the middle horizon, and to 34 mg/L in the deep horizon. Of the five constituents with maximum concentrations greater than 1 mg/L, three (chlorobenzene, 1,2,4-trichlorobenzene, and 1,4-dichlorobenzene) have their maximum

concentrations in the middle or deep horizon. The other two, cis/trans 1,2-dichloroethene and 4-chloroaniline, have the maximum concentration in the shallow horizon.

Indicator 2: Site constituents are found throughout the entire depth of the unconsolidated unit, from the water table surface to locations over 100 ft deep.

Indicator 3: Three site constituents (chlorobenzene, 1,4 dichlorobenzene, and fluoranthene) are found in concentrations that exceed 1% of each respective pure-phase solubility (see Table 1).

Indicator 4: Based on site historical data and observed groundwater concentrations, the EPA Fact Sheet "Estimating Potential for Occurrence of DNAPL at Superfund Sites," shows a "**High-Moderate**" Potential for DNAPL at Site I.

Indicator 5: Of the six constituents where a comparison could be made, five had higher concentrations in the groundwater than in the leachate from waste materials, suggesting that leaching was not responsible for the highest groundwater concentrations at Site I (see Table 2).

Indicator 6: As described above, only two of the five constituents with concentrations greater than 1 mg/L (cis/trans 1,2-dichloroethene and 4-chloroaniline) have their maximum concentrations in the shallow horizon.

KEY POINT: SITE I SOURCE MECHANISMS

DNAPL dissolution is a major source mechanism at Site I based on an analysis of the evaluated indicators. Dissolved constituent concentrations increase with depth, site constituents extend from the water table to the bottom of the water-bearing interval, and three constituents are found at concentrations that suggest the possible presence of non-mobile residual DNAPL.

Some leaching of unsaturated waste/soil materials may also be occurring, as some constituents such as 4-chloroaniline did not show increasing concentrations with depth. Overall, however, DNAPL dissolution appears to be the dominant source mechanism at Site I.

POTENTIAL FOR SOURCE REMEDIATION

Conceptual Model of Source

The following discussion summarizes our conceptual model of the DNAPL source located in the saturated zone beneath Site I:

- DNAPL is present as "fingers" and "pools" in the saturated zone extending from approximately 15 to 110 ft below the surface (see Figure 2 for a conceptual figure).

  Supporting Information: "Once penetration of the capillary fringe occurs, downward movement will continue until all the CHC (chlorinated hydrocarbon) solvent is present as suspended fingers (ganglia) in the porous media and/or as pools of CHC perched on low-permeability zones. Once a pool starts to form on top of a low-permeability layer somewhere above the bottom of the aquifer, a continued supply of CHC will cause (1) enlargement of the pool, (2) penetration of the layer, and/or (3) spawning of new downward-moving fingers at the perimeter of the layer." Johnson and Pankow (1992)

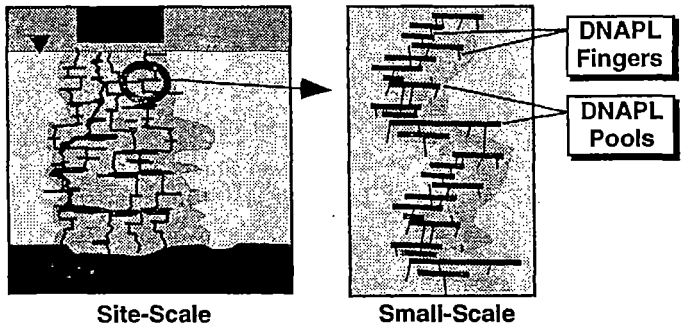


FIGURE 2. Conceptual Model of DNAPL Source Zone

- Small horizontal pools of DNAPL are present throughout the entire vertical extent of the saturated zone, and not just at the bottom of the unit.

  Supporting Information: "In granular aquifers, small horizontal zones of residual or free-phase DNAPL need not be caused by particularly low permeability zones such as silt or clay. A minor contrast in grain size distribution and hence permeability, as from a coarse sand layer to a fine sand, causes variation in DNAPL entry pressure. A DNAPL will accumulate on the finer-grained layer while spreading laterally until it reaches the edge of the layer, or until the height of the free-product accumulation on the layer exceeds the entry pressure for the layer." Pankow and Cherry (1996)

- Much or most of the DNAPL mass is present in the trapped residual state that cannot be recovered by pumping.

  Supporting Information: "Note that after the continuous NAPL body has been converted to a residual form, the individual NAPL blobs are held very tightly in the porous media by capillary forces. Wilson and Conrad (1984) evaluated the force required to mobilize and completely sweep away residual blobs in porous media in terms of the hydraulic gradient a pumping system would have to generate to either 1) begin blob mobilization, or 2) mobilize all blobs in a porous medium. This relationship, presented as a graph of hydraulic conductivity vs. required hydraulic gradient, indicates that mobilization of NAPL blobs by pumping will occur only in very coarse porous media with a very high hydraulic gradient.

The rest of the blobs will stay trapped in the porous media, serving as a long-term source of dissolved contaminants." (Wiedemeier et al., 1999)

(Note: For the unconsolidated alluvial fill deposits at this site, Wilson and Conrad's analysis indicates that a gradient of 0.5 ft/ft would be required to begin to mobilize NAPL blobs. This is 500 times the current hydraulic gradient, and impossible to effect over the entire source area without extensive pumping and re-injection).

- It is extremely unlikely that any DNAPL that may be present at the site is still mobile or will become mobile under current conditions.

  Supporting Information: "Once the release of DNAPL into the subsurface ceases, subsurface movement of DNAPL also ceases soon thereafter, perhaps within weeks or months at solvent sites. The resulting immobile DNAPL then exists in the DNAPL source zone as "residual" non-aqueous liquid and also possibly as "free-product" accumulations ponded on lower permeability layers within aquifers, or on the tops of aquitards. The free-product DNAPL will not become mobile again unless a release of more DNAPL causes further accumulation in the same zones, or unless there are changes in pressure in the surrounding water phase due to groundwater pumping or injection." Pankow and Cherry (1996)

- The presence of pools that can be pumped is relatively rare at DNAPL sites, and if such pools are found and pumped, only a small fraction of the total DNAPL in place is removed.

  Supporting Information: "In field investigations of sites where extensive solvent contamination exists, pools of free-product solvent are only rarely found, even when their existence is not in doubt." "It is the author's experience that chlorinated solvents with their high densities form thick pools only rarely." Pankow and Cherry (1996)

  "Therefore, in a practical sense, NAPL removal translates to recovery of a small percentage of NAPL at a site (i.e., whatever continuous NAPL can be collected)." Wiedemeier et al., (1999)

- The presence of DNAPL pools and fingers will only occupy a small fraction of the available pore space in the source zone.

  Supporting Information: "However, heterogeneity has a marked influence on the direction of DNAPL migration. A random distribution of permeability and displacement pressure will result in a highly erratic pattern of DNAPL flow..." "The remarkable sensitivity of DNAPL penetration to the capillary-hydraulic properties can be expected to result in highly complex, seemingly chaotic saturation distributions in the subsurface." "Even in the idealized case of a perfectly homogeneous medium, DNAPL can be expected to penetrate in the form of narrow, elongated distributions in which the mean saturation of DNAPL is small." Pankow and Cherry (1996)

- The source will persist for a long time.

  Supporting Information: "These calculations suggest that zones of residual DNAPL and especially pool DNAPL can persist in the subsurface and contribute to groundwater contamination for decades to centuries." "For most chlorinated solvents, the rate of

dissolution of pools will be sufficiently slow that the DNAPL source zones will cause significant contamination of the groundwater for centuries or more." Pankow and Cherry (1996)

- Pumping can increase the rate that mass is removed from the source, but the removal efficiency will be much less than the removal efficiency for natural attenuation.

  Supporting Information: "However, the increase in mass removal (by pumping) will not be in proportion to the increase in the groundwater pumping rate because of limitations on the DNAPL dissolution kinetics, and because of further dilution with clean water from outside the source zone." Pankow and Cherry (1996)

KEY POINT: CONCEPTUAL MODEL FOR SOURCE

Most of the Site I DNAPL in the unconsolidated valley fill deposits is present as thin vertical fingers and small horizontal pools throughout the entire vertical extent of the water-bearing unit. Only a small fraction of the total DNAPL mass can ever be removed by pumping free-phase pools, if they are found. Under current conditions, the rest of the DNAPL is immobile, and will serve as a long-term continuing source of constituents to groundwater.

ESTIMATED NATURAL DISSOLUTION RATE

Natural Groundwater Flushing Rate

Separate hydraulic conductivity and hydraulic gradient data were developed for the shallow, middle, and deep horizons of the unconsolidated deposits.

The hydraulic conductivity estimates developed for the model were based on: 1) literature reports, and 2) preliminary analysis of RI/FS slug test data. The literature reference (Ritchey and Schicht, 1982) reported that the hydraulic conductivity for the unconsolidated material used for water supply in the American Bottoms area ranged from $5x10^{-2}$ to $1.4x10^{-1}$ cm/sec.

The analysis of RI/FS slug test data from Site I wells showed the following hydraulic conductivities:

Horizon	Site I (well ST-I-S) (cm/sec)
Shallow	$4.5x10^{-3}$
Middle	$5.1x10^{-2}$
Deep	$1.3x10^{-1}$

Using the data from the literature report, slug test results, and calibration work, the following hydraulic conductivities were used in the model:

- Shallow Horizon: $1x10^{-2}$ cm/sec
- Middle Horizon: $1x10^{-1}$ cm/sec
- Deep Horizon: $1x10^{-1}$ cm/sec

Using RI/FS potentiometric surface maps provided by Roux Associates, Inc., the following hydraulic gradients were used in the model:

- Shallow Horizon: 0.001 ft/ft
- Middle Horizon: 0.001 ft/ft
- Deep Horizon: 0.001 ft/ft

These values yield the following representative values for groundwater Darcy velocity at the site:

- Shallow Horizon: 10.4 ft/yr
- Middle Horizon: 104 ft/yr
- Deep Horizon: 104 ft/yr

As shown by the data, the shallow horizon of the unconsolidated deposits is less permeable, and has a much lower groundwater velocity than the more coarse-grained middle and deep horizons.

The hydrogeologic conceptual model divides the unconsolidated water-bearing unit into three horizons: the shallow horizon (generally 15-30 ft deep), the middle horizon (generally 30-70 ft deep), and the deep horizon (generally 70-110 ft deep). Therefore the assumed saturated thicknesses for the shallow, middle, and deep units were: 15 ft, 40 ft, and 40 ft, respectively. When a 1400 ft wide source zone is assumed (the width of Site I perpendicular to groundwater flow), a naturally-occurring groundwater flushing rate of 168 gpm is obtained (3.1 gpm for the shallow unit, 82.5 gpm for the middle unit, and 82.5 gpm for the deep unit).

Natural Mass Removal Rate

The average total VOC + SVOC concentrations from the transect well closest to Site I (well AA-I-S1) are 13.3 mg/L, 21.9, mg/L, and 19.9 mg/L for the shallow, middle, and deep horizons, respectively. For this planning-level calculation, it was assumed that these concentrations extended throughout the entire width of the Site I source zone, a potential overestimation

(however, if all other source removal calculations use the same assumptions, the relative results will be accurate). Therefore, the mass removal rate under natural conditions was estimated by multiplying average VOC + SVOC concentrations for each horizon by the flow for each horizon, and converting to a mass rate of kg/yr leaving the source zone (3.78 L/gal; 1440 min/day; 365 day/yr; 10^{-6} kg/mg). This calculation resulted in the following naturally-occurring mass removal rate totaling approximately 7000 kg/yr from all three horizons:

- Shallow Horizon: 82 kg/yr
- Middle Horizon: 3,613 kg/yr
- Deep Horizon: 3,271 kg/yr

TOTAL: 6,966 kg/yr

KEY POINT: NATURAL MASS REMOVAL RATE

The natural mass removal rate from the Site I source zone is estimated to be 7000 kg/yr assuming uniform source concentrations throughout the source zone.

Assumed Flowrate From An Intensive Pump-and-Treat System

Three methods were evaluated to provide a planning-level estimate of the flowrate from an intensive pump-and-treat system at Site I (see Appendix A). First, an empirical well yield relationship (Driscoll, 1986) based on transmissivity, expected drawdown, and assumptions for other variables in the nonequilibrium (Jacob) equation was used. The second method was based on typical well yields from regional water supply wells as reported by Schicht (1965):

> "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."

The third method was based on evaluating specific capacity (well yield divided by drawdown) provided by Schicht (1965).

These calculation approaches suggest that an intensive pumping system for Site I could yield 1000–2500 gpm. For the purpose of this project, a value of 1500 gpm was used.

KEY POINT: GROUNDWATER FLOWRATE FROM INTENSIVE PUMPING

An intensive pump-and-treat system was assumed to have a yield of 1500 gpm.

EFFECT OF PUMPING GROUNDWATER

Because most of the DNAPL is trapped and cannot be removed by direct pumping, a groundwater pump-and-treat system will generally not remove DNAPL directly, but instead will slowly dissolve the DNAPL trapped in fingers and pools. While this dissolution process is relatively slow and inefficient, it will remove DNAPL mass.

Dissolution Kinetics for DNAPL Fingers and Pools

Several analyses have been performed to evaluate the effect of increased pumping rates on the DNAPL dissolution rate for both fingers and pools. In a key paper written by Hunt et al. in 1988, the authors developed relationships for the kinetics of dissolution in NAPL source zones. They evaluated laboratory studies and mass transfer approaches used in the chemical engineering literature, and derived dissolution expressions for residual NAPL ganglia (also called "fingers" or "blobs"). They concluded that:

"Ganglion lifetimes are weakly dependent on flow velocity such that to decrease the lifetime from 100 years to 10 years requires a three order of magnitude increase (x1000) in flow velocity."

In other words, increasing the groundwater pumping rate will increase the finger dissolution rate, but only slightly based on this relationship:

$$\frac{\text{mass transfer rate with pumping}}{\text{mass transfer rate without pumping}} = 10^{\frac{\log 10\left[\frac{Q_{pumping}}{Q_{natural}}\right]}{3}}$$

Using this NAPL dissolution relationships reported by Hunt et al. (1988), a 1500 gpm pumping system (a 8.9 times increase in the natural flow rate through the system) would result in a 8.9 fold *increase* in water flushed through the system, but a 4.3 fold *decrease* in effluent concentrations, resulting in a net increase in mass removed only by a factor of 2.1:

$$\frac{\text{mass transfer rate with pumping}}{\text{mass transfer rate without pumping}} = 10^{\frac{\log 10\left[\frac{1500\text{ gpm}}{168\text{ gpm}}\right]}{3}} = 2.1$$

The same type of concentration reduction is expected when higher groundwater flowrates are used to dissolve NAPL pools. Dissolution kinetic relationships developed by Johnson and Pankow (1992) indicate that the mass

transfer rate (and pool lifetime) changes with the square root of groundwater velocity:

$$\text{Pool Dissolution Time (yrs)} = 2.43\text{x}10^{-5}\ \rho\ C_{sat}\ [\,l_p^3/D_v\, v_d\,]^{0.5}$$

where:

ρ	=	DNAPL density (g/m^3)
C_{sat}	=	saturation concentration (g/m^3)
l_p	=	length of pool in direction of groundwater flow (m)
D_v	=	vertical dispersion coefficient (m^2/s)
v_d	=	Darcy velocity for groundwater (m/day)

Therefore, increasing the groundwater flowrate over a pool by a factor of 8.9 would result in an initial concentration *decrease* by a factor of 3.0 (approximately the square root of 8.9), and the overall *increase* in the mass removal rate by only a factor of 3.0.

Note that these theoretical expressions are supported by lab and field data (e.g., see Pankow and Cherry, 1996). Because source zones include a mixture of pools and fingers, it was assumed in this study that increasing the groundwater flowrate through the source zone by a factor of 8.9 (by pumping) would increase the mass transfer by a factor of 2.5 (the mid-point of finger value of 2.1 and pool values 3.0) when pumping was started. This is because groundwater concentrations decrease by a factor of 3.6 due to mass transfer effects. Note that after pumping is stopped, the concentrations would rebound and increase by a factor by the same amount (in the case of these calculations, by a factor of 3.6).

KEY POINT: EFFECT OF PUMPING GROUNDWATER ON CONCENTRATIONS

As shown by DNAPL dissolution expressions, the mass removal rate from a DNAPL source zone is only weakly dependent on the groundwater pumping rate. For example, if the flowrate though a DNAPL source zone is increased by a factor of 8.9 due to intensive pumping, the mass removal rate will only increase by a factor of 2.5 (a representative value for effects of pumping on DNAPL finger and DNAPL pool dissolution) because concentrations in the recovered groundwater would be reduced by a factor of 3.6 due to mass transfer effects.

Mass Removal Rate of Intensive Pump-and-Treat System

Under an intensive pumping scenario with an increase in natural flow (from 168 gpm to approximately 1500 gpm), the groundwater concentrations being removed from the source are expected to fall to between one-third to one-fifth

of the observed concentrations under lower flow, natural conditions. Assuming a middle value of post-pumping concentrations that are 3.6 times smaller than the natural concentrations, the initial VOC + SVOC effluent concentrations from an intensive groundwater pump-and-treat system are estimated to be: 3.7 mg/L, 6.0 mg/L, and 5.5 mg/L for the shallow, middle, and deep units, respectively.

Therefore, under an intensive pump-and-treat scenario where 1500 gpm are being flushed through the Site I source zone (an 8.9-fold increase in the flushing rate), the initial mass removal rate is predicted to only increase by a factor of 2.5, from 7000 kg/yr to 17,500 kg/yr due to mass transfer effects related to DNAPL dissolution. Note that this is only the initial mass removal rate for the intensive pumping case, and that this concentration will drop slowly over time as mass is removed from the system.

KEY POINT: MASS REMOVAL RATE FROM INTENSIVE PUMPING

An intensive pump-and-treat system was estimated to have an initial mass removal rate of 17,500 kg/yr, accounting for both the increased flowrate through the system and decreased concentrations in groundwater.

Effect of Pumping on Source Lifetime

Estimating source longevity is a process involving considerable uncertainty, as the original mass in place, mass removal rate, and the change in the mass removal rate over time must all be known. While absolute estimates have a high level of uncertainty, the relative comparison of remediation alternatives can be made with more confidence. In the analysis below, the absolute values for source lifetime should be considered highly uncertain, while the relative comparisons should be considered more accurate.

Estimated Source Mass

A range of estimates of source mass were developed, assuming that the entire saturated zone below Site I is affected by DNAPL. Then the calculated mass removal rates for natural attenuation and an intensive pump-and-treat system were used to estimate source longevity.

Source mass is a function of source volume, the porosity, the residual saturation of DNAPL in the source zone, and the fraction of source volume containing DNAPL. At Site I, the estimated source volume is 1400 ft by 95 ft by 500 ft, or 66,500,000 ft^3. Residual saturation (the fraction of open pore space occupied by DNAPL) values are typically assumed to be between 0.01 and 0.15

(see Pankow and Cherry, 1996), and a value of 0.05 was used for this analysis. A porosity of 0.35 was considered representative of the unconsolidated alluvial deposits at the site. Finally, it was assumed that 1% of the aquifer volume contains residual DNAPL.

Based on these assumptions, a planning-level estimate for the volume of DNAPL under Site I was estimated to be 87,000 gallons. Assuming an average density of 1.25 (based on an average of the density of chlorobenzene and 1,4-dichlorobenzene, two of the most commonly-found site constituents), the estimated mass of DNAPL is approximately 410,000 kg. Note that the actual mass may be more or less, but for the purpose of performing relative calculations of source longevity this value appeared to provide reasonable results.

KEY POINT: SITE I SOURCE MASS ESTIMATE

A planning-level source mass estimate of 410,000 kg of VOCs+SVOCs was estimated for the DNAPL source zone below Site I. There is considerable uncertainty in this estimate, with the actual mass potentially being higher or lower than 410,000 kg.

Source Decay Model

A simple source model, originally developed as part of the BIOSCREEN model (Newell et. al. 1996, EPA/600/R-96/087) and now being included as part of the BIOCHLOR model (Aziz et al., 2000, EPA/600/R-00/008) was used to estimate the lifetime of the groundwater source at Site I under different remediation options.

In this simple box model, the source zone is considered to be located in a box containing some mass of dissolvable contaminants. The rate at which contaminants leave the box is estimated from the rate at which flowing groundwater removes contaminants from the box. The time required to achieve a cleanup standard can then be estimated by comparing the mass of contaminants in the box vs. the time required to remove contaminants from the box. To more closely match real-site conditions, the source concentration is assumed to decay over time, in proportion to the remaining source mass (Wiedemeier et al., 1999). With this assumption, the source concentration over time can be described using:

$$C_t = C_{so} \exp^{(-k_s t)}$$

where:

C_t = Source concentration at time t (mg/L)
C_{so} = Observed source concentration at t = 0 (mg/L)
t = Time (years)
k_s = Source decay coefficient (1/year)

(Note that this decay coefficient **is not** related in any way to first-order decay coefficients reported in the literature for natural attenuation, as the literature values typically represent decay half-lives from 0.1 to 10 years and represent biodegradation of dissolved contaminants in the plume *once they have left the source.* The source decay coefficient values represent how quickly a source zone is being depleted, and will usually have much longer half-lives, typically tens or hundreds of years.)

The source decay coefficient, representing how quickly the source is being depleted, can be derived using estimates of the source mass and rate that contaminants leave the source (Newell et al., 1996):

$$k_s = \frac{Q \cdot C_{so}}{M_o}$$

where:

Q = Groundwater flowrate through source zone (L/year)
C_{so} = Observed source concentration at time = 0 (mg/L)(or kg/L)
M_0 = Dissolvable mass in source at time = 0 (mg)(or kg)

This model assumes that the only mass leaving the source zone is dissolved in the water flowing through the source zone. Note that Q and C_{so} are related; the thickness of the source zone should be matched with an appropriate average concentration for that entire depth horizon.

With a first-order source decay term, the source concentration at any time can be derived, providing the time required to reach any concentration:

$$t = -\frac{1}{k_s} \ln\left(\frac{C_t}{C_{so}}\right)$$

where:

t = Time required to reach concentration C_t (years)

Five Source Lifetime Cases

For this analysis, five different cases were evaluated using the source lifetime described above:

Case 1: Natural attenuation only (initial removal rate of 7000 kg/yr)

Case 2: 1 year of intensive pump-and-treat (initial removal rate of 17,500 kg/yr), followed by natural attenuation

Case 3: 5 years of intensive pump-and-treat (initial removal rate of 17,500 kg/yr), followed by natural attenuation

Case 4: 10 years of intensive pump-and-treat (initial removal rate of 17,500 kg/yr), followed by natural attenuation

Case 5: 30 years of intensive pump-and-treat (initial removal rate of 17,500 kg/yr), followed by natural attenuation

With this approach (see Appendix B), the following times to cleanup were estimated:

		Estimated Time to Cleanup (years)	% Reduction from Natural Attenuation Only
Case 1	Natural Attenuation Only	488	-
Case 2	1 Yr of Intensive Pump-and-Treat + Natural Attenuation	486	0.4% reduction
Case 3	5 Yrs of Intensive Pump-and-Treat + Natural Attenuation	480	2% reduction
Case 4	10 Yrs of Intensive Pump-and-Treat + Natural Attenuation	472	3% reduction
Case 5	30 Yrs of Intensive Pump-and-Treat + Natural Attenuation	441	10% reduction

Figure 3 shows a comparison of source concentrations vs. time for two of the six cases.

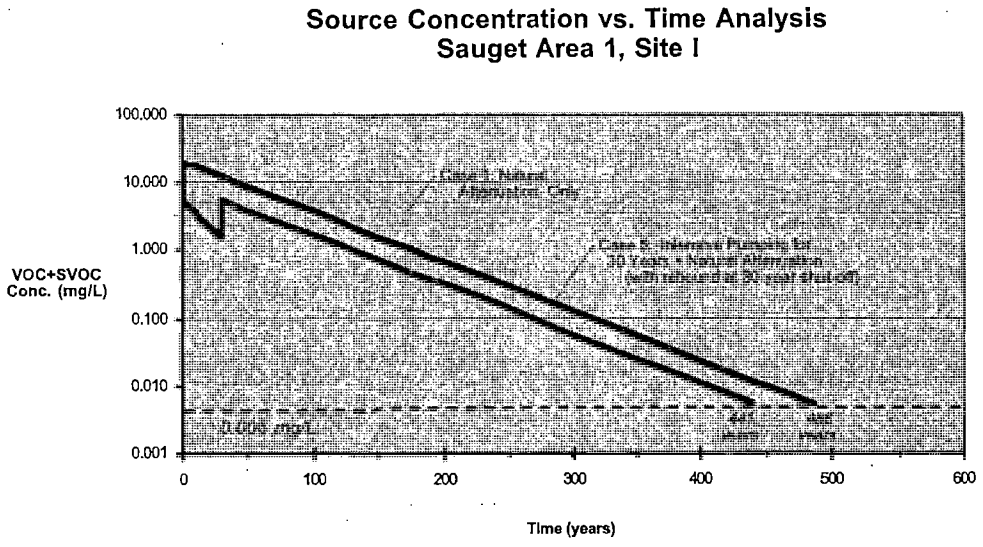


FIGURE 3. Source concentration vs. time graphs for Case 1 (Natural Attenuation Only) and Case 5 (30 Years of Intensive Pump-and-Treat + Natural Attenuation). For Case 1, concentrations start at 20 mg/L and decline as a first order decay relationship over time. For Case 5, the source concentration starts at 20 mg/L, but concentrations are reduced by a factor of 3.6 due to mass transfer effects caused by the almost 9 times increase in groundwater flow through the source zone. After 30 years, pumping is stopped, groundwater flow is restored to natural conditions, and mass transfer effects cause an increase in concentration by a factor of 3.6 (the "rebound" effect). Overall, the source modeling exercise shows that with the source assumptions described in the text, the time required to restore groundwater is reduced only slightly by 30 years of intensive pumping, from 488 years to 441 years (10% reduction).

Sensitivity Analysis

The source lifetime analysis has several areas of uncertainty, and should be used to evaluate relative differences between remediation alternatives rather than to provide an absolute source lifetime estimate. Significant sources of uncertainty include:

- *The assumption that concentrations observed in well AA-I-S1 extend throughout the entire 1400 ft source width of Site I.* If some sections of the 1400 ft source width of Site I are lower concentration, the following impact on the source lifetime is expected: 1) for the natural attenuation case, the overall source lifetime estimates will not change as both the removal rate and the mass in the source are functions of the source width; and 2) for the

pumping case, some reduction in overall source lifetime is expected as source mass is dependent on source width but removal rate is not (it is dependent on pumping rate and expected concentrations).

- *The assumptions that the source is represented by a residual DNAPL saturation of 0.05 and that 1% of the source zone is impacted by DNAPL residual.* These assumptions have a great deal of uncertainty (the literature reports that residual saturations can be as high as 0.50), and were selected in part to yield source lifetimes in the range of several hundreds of years to match the source conceptual model discussed above. If the source is much smaller than the estimated 410,000 kg of VOCs+SVOCs, then the impact of a pumping system will be greater, and greater than a 1% to 10% reduction in source lifetime will be realized. If the source mass is only 41,000 kg (an unlikely event based on the persistence of the source to date), then an intensive pump-and-treat system is predicted to reduce the source lifetime by from 49 years (natural attenuation alone) to 22 years (intensive pumping). Conversely, if the mass is greater, a pump-and-treat system will have less of an effect.

Other, potentially less significant sources of uncertainty are:

- *The assumption that concentrations under a pumping scenario will be smaller than concentrations observed under natural flow conditions.* While there is uncertainty in the actual amount, it has been demonstrated in lab studies and the field that increasing the flowrate through a DNAPL source zone will result in lowered concentrations (for example, see Pankow and Cherry, 1996). Therefore we expect some concentration reduction with a pump-and-treat scenario.

- *The assumption that the flow throughout each interval is uniform and that the concentration in each interval can be calculated by averaging each sample point.* These assumptions were used in the mass removal calculation. While there may be some uncertainty in these assumptions, the large number of vertical samples reduces the potential error.

- *The assumption of a first-order decay relationship for the source dissolution rate.* This assumption is based on observations about source decay, and is now used in two EPA peer-reviewed models, BIOSCREEN and BIOCHLOR. While the exact source concentrations curve may not be exactly first order, it will almost certainly fit a first-order decay curve better than assuming constant source concentrations until the source is exhausted. (Note that the use of the first order decay model for the source

does not mean that literature-based first-order decay constants for dissolved constituents were used. A source decay constant is based on removal rate and initial source mass, while a biodegradation rate is based on how fast concentrations decay after they leave the source. This study used a source decay approach, and did not use biodegradation rates to estimate source lifetime).

- *No availability effects related to desorption of constituents at low concentrations have been considered.* Slow desorption of non-available fraction of constituents sorbed to aquifer materials will likely reduce the efficiency of any flushing technology. More pronounced effects may be observed for intensive pumping scenarios.

Additional Analysis

A similar analysis was performed for Sites G/H/L using the same calculation approach as was used for Site I (Appendix B). Two cases were performed, and show little impact from a five-year intensive pumping program:

		Estimated Time to Cleanup (years)	% Reduction from Natural Attenuation Only
Case 6	Natural Attenuation Only – Sites G/H/L	434	-
Case 7	5 Yrs of Intensive Pump-and-Treat + Natural Attenuation	427	2% reduction

An evaluation of other constituents present in Sites G/H/I/L groundwater, such as herbicides, pesticides, dioxins, and metals indicates that some constituents will like achieve cleanup goals faster than the VOCs + SVOCs analyzed for this source report, and others may take longer. Ratios of the maximum observed concentrations at Area 1 vs. the Illinois Class I standard for representative constituents provide a general indication of how quickly various constituents may achieve cleanup goals:

CONSTITUENT (Constituent Class)	MAXIMUM CONCENTRATION IN GROUNDWATER (ug/L)	ILLINOIS CLASS I STANDARD (ug/L)	RATIO OF MAX. CONC. / ILLINOIS CLASS I STD.
Chlorobenzene (VOC)	34,000	100	340
2,4-dichlorobenzene (SVOC)	14,000	75	187
Alpha-BHC (Herbicide)	72	0.03	2400
2,4-D (Pesticide)	190	70	2.7
Total PCBs (PCB)	12	0.5	24
Cu (Metal)	3000	650	4.6
Ni (Metal)	7800	100	78
Pb (Metal)	3600	7.5	480
Zn (Metal)	33,000	5000	6.6

On the basis of this general evaluation, alpha-BHC may take longer to achieve cleanup goals than the VOCs+SVOCs, while 2,4-D may take less time. Other factors, such as the mass of each constituent in the source zone and the constituent-specific fate and transport process will determine the ultimate time required to remediate the Area 1 source zones.

KEY POINT: ESTIMATED SOURCE LIFETIMES FOR SEVEN CASES

A planning level source lifetime calculation was done to estimate the relative performance of various remediation schemes. This analysis, while not providing high-confidence estimates of the absolute time to cleanup, does indicate that with an assumed mass of 410,000 kg of VOCs + SVOCs in the saturated zone below Site I, intensive pumping over a 1 to 30 year period does not appear to have an appreciable effect on overall source lifetime (i.e., ≤ 10% reduction). Similar limitations are expected at Sites G/H/L as well.

CONCLUSIONS

Based on the overall groundwater source evaluation at Site I of Sauget Area 1, DNAPL dissolution appears to be the dominant source mechanism. Planning level source lifetime calculations indicate that intensive groundwater pumping will not have an appreciable effect on the overall source lifetime at Site I or at Site G/H/L.

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GROUNDWATER SERVICES, INC.

TABLE 1
SUMMARY OF SITE I GROUNDWATER CONCENTRATIONS BY DEPTH AND COMPARISON TO CONSTITUENT SOLUBILITY
Sampling Period: November to December 1999

Solutia Inc.
Area 1, Sauget and Cahokia, Illinois

Detected Constituent	0-30 ft Depth Conc. (mg/L)	30-70 ft Depth Conc. (mg/L)	70+ ft Depth Conc. (mg/L)	Solubility (mg/L)
VOCs				
1,1-Dichloroethane	0.96	<0.5	<1.0	5,060
1,1-Dichloroethene	0.032 J	<0.5	<1.0	2,250
Benzene	**0.62**	0.19	0.14 J	1,750
Chlorobenzene	8.7	20.0	**34.0**	***472***
cis/trans-1,2-Dichloroethene	**1.2**	0.31	0.001 J	3,500 (7)
Ethylbenzene	**0.87**	0.27	0.074	169
Tetrachloroethene	<0.5	<0.5	0.001 J	200
Toluene	0.018 J	**0.086 J**	0.00089 J	526
Vinyl Chloride	**0.97**	0.32	0.0012 J	2,760
Xylenes, total	<0.5	0.023 J	0.014	186
SVOCs				
1,2,4-Trichlorobenzene	<0.01	<0.5	**2.7**	300
1,2-Dichlorobenzene	0.13	0.32 J	**0.5**	156
1,3-Dichlorobenzene	0.11	**0.29 J**	0.150 J	NA
1,4-Dichlorobenzene	4.4	**10 D**	9.7 D	***73.8***
2,4,5-Trichlorophenol	<0.01	<0.5	0.0018 J	1,200
2,4-Dichlorophenol	<0.01	0.042	0.047 J	4,500
2-Chlorophenol	0.0055 J	0.039	**0.052**	22,000
2-Methylnaphthalene	<0.01	<0.5	0.0013 J	NA
2-Methylphenol (o-cresol)	<0.01	0.003 J	<0.4	26,000
4-Chloroaniline	**4.1 D**	1.7 D	0.018	5,300
Acenaphthene	<0.01	<0.5	0.00033 J	4.24
Carbazole	0.0014 J	0.013	**0.013**	7.48
Di-n-butylphthalate	<0.01	0.00034 J	0.00051 J	11.2
Dibenzofuran	<0.01	0.019 J	<0.4	NA
Diethylphthalate	<0.01	0.0051 J	<0.4	1,080
Fluoranthene	<0.01	0.022 J	<0.4	***0.206***
Hexachlorobenzene	<0.01	<0.5	0.001 J	6.2
N-Nitrosodiphenylamine	0.0053	**0.028**	0.02	35.1
Naphthalene	0.0042 J	0.024	**0.066**	31
Phenanthrene	<0.01	**0.089 J**	0.0013 J	NA
Phenol	<0.01	0.0044 J	<0.4	82,800
bis(2-chloroethyl)ether	0.0011 J	<0.5	<0.4	17,200
bis(2-ethylhexyl)phthalate	0.00069 J	<0.09	<0.072	0.34
Total Detected Conc. (mg/L)	**22.1**	**33.8**	**47.5**	

Notes:
1) Table includes only those compounds detected in at least one groundwater sample for each constituent class. Comparison to solubility includes groundwater sampled at any depth in source area monitoring well.
2) Groundwater samples included are from nearest source area monitoring well only (i.e., AA-I-S1).
3) J = Estimated value. D = Diluted sample. NA = Not available.
4) **Bold** type denotes maximum groundwater concentration by depth.
5) **Underlined bold italics** type denotes maximum groundwater concentration exceeds 1% of constit. solubility.
6) Lowest solubility of cis/trans-1,2-Dichloroethene pair indicated.
7) Solubility data from Illinois Tiered Approach to Corrective Action Objectives (TACO).
8) For comparison purposes, non-detectable concentrations are taken as the detection limit shown.

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TABLE 2
SUMMARY OF SITE I GROUNDWATER CONCENTRATIONS BY DEPTH VERSUS TCLP WASTE DATA
Sampling Period: November to December 1999

Solutia Inc.
Area 1, Sauget and Cahokia, Illinois

Detected Constituent	0-30 ft Depth Conc. (mg/L)	30-70 ft Depth Conc. (mg/L)	70+ ft Depth Conc. (mg/L)	Max. TCLP Conc. (mg/L)	Media With Max. Conc.
VOCs					
1,1-Dichloroethane	0.96	<0.5	<1.0	NA	-
1,1-Dichloroethene	**0.032 J**	<0.5	<1.0	<0.02	GW
Benzene	**0.62**	0.19	0.14 J	0.14	GW
Chlorobenzene	8.7	20.0	**34.0**	8.9	GW
cis/trans-1,2-Dichloroethene	1.2	0.31	0.001 J	NA	-
Ethylbenzene	0.87	0.27	0.074	NA	-
Tetrachloroethene	<0.5	<0.5	0.001 J	0.29	-
Toluene	0.018 J	0.086 J	0.00089 J	NA	-
Vinyl Chloride	**0.97**	0.32	0.0012 J	<0.04	GW
Xylenes, total	<0.5	0.023 J	0.014	NA	-
SVOCs					
1,2,4-Trichlorobenzene	<0.01	<0.5	2.7	NA	-
1,2-Dichlorobenzene	0.13	0.32 J	0.5	NA	-
1,3-Dichlorobenzene	0.11	0.29 J	0.150 J	NA	-
1,4-Dichlorobenzene	4.4	**10 D**	9.7 D	1.3	GW
2,4,5-Trichlorophenol	<0.01	<0.5	0.0018 J	**1.4**	TCLP
2,4-Dichlorophenol	<0.01	0.042	0.047 J	NA	-
2-Chlorophenol	0.0055 J	0.039	0.052	NA	-
2-Methylnaphthalene	<0.01	<0.5	0.0013 J	NA	-
2-Methylphenol (o-cresol)	<0.01	0.003 J	<0.4	0.014 J	-
4-Chloroaniline	4.1 D	1.7 D	0.018	NA	-
Acenaphthene	<0.01	<0.5	0.00033 J	NA	-
Carbazole	0.0014 J	0.013	0.013	NA	-
Di-n-butylphthalate	<0.01	0.00034 J	0.00051 J	NA	-
Dibenzofuran	<0.01	0.019 J	<0.4	NA	-
Diethylphthalate	<0.01	0.0051 J	<0.4	NA	-
Fluoranthene	<0.01	0.022 J	<0.4	NA	-
Hexachlorobenzene	<0.01	<0.5	0.001 J	<0.05	-
N-Nitrosodiphenylamine	0.0053	0.028	0.02	NA	-
Naphthalene	0.0042 J	0.024	0.066	NA	-
Phenanthrene	<0.01	0.089 J	0.0013 J	NA	-
Phenol	<0.01	0.0044 J	<0.4	NA	-
bis(2-chloroethyl)ether	0.0011 J	<0.5	<0.4	NA	-
bis(2-ethylhexyl)phthalate	0.00069 J	<0.09	<0.072	NA	-

GW Conc. Greater	**5**
TCLP Conc. Greater	**1**

Notes:
1) Table includes only those compounds detected in at least one groundwater sample for each constituent class. Comparison to TCLP waste concentration includes groundwater sampled at any depth in source area monitoring well.
2) Groundwater samples included are collected from nearest source area monitoring well only (i.e., AA-I-S1).
3) J = Estimated value. D = Diluted sample.
4) **Underlined bold** type denotes maximum groundwater concentration or TCLP concentration.
5) TCLP waste data from unsaturated waste samples.
6) NA = Not analyzed.
7) For comparison purposes, non-detectable concentrations are taken as the detection limit shown.

May 21, 2001

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APPENDIX A
DESIGN PUMPING RATE OF HYDRAULIC CONTAINMENT WELLS
Groundwater Alternative D, Intensive Pumping, Sites G, H, I, and L
Sauget Area 1, Sauget and Cahokia, Illinois

PROBLEM: What is estimated pumping rate and number of wells for intensive pumping system for Site I + Sites G/H/L plume?

ASSUMPTIONS:

K = 0.1 cm/sec for middle, deep horizon

b = 80 ft (40 ft middle horizon, 40 ft deep horizon)

i = 0.001 ft/ft

Available drawdown (s) = 15 ft (thickness of shallow unit) (this equals thickness of shallow saturated horizon)

METHOD: Use three different methods to develop a basis for flowrates for an intensive pumping system for the combined Site I + Site G/H/L plume areas.

Method 1. First, an empirical well pumping rate relationship (Driscoll, 1986) based on transmissivity, expected drawdown, and assumptions for other variables in the nonequilibrium (Jacob) equation was used. For this site, a transmissivity of 170,000 gpd/ft was calculated (based on an assumed hydraulic conductivity of 0.1 cm/sec and a saturated thickness of 80 ft for the combined middle and deep horizons).

For unconfined units:

$$\frac{Q}{s} = \frac{T}{1500}$$ (Eqn. 3, Q in gpm, s in ft, T in gpd/ft)

$$Q(gpm) = \frac{\left(T\ \frac{gpd}{ft}\right)(s\ ft)}{1500}$$

$$T = (K)(b_T)$$

$$T = \left(0.1\frac{cm}{sec}\right)\left(\frac{86400\ \sec}{day}\right)\left(\frac{in}{2.54\ cm}\right)\left(\frac{ft}{12\ in}\right)(80ft)\left(\frac{7.48\ gal}{ft^3}\right)$$

$$T = 170{,}000\ gpd/ft$$

$$Q(gpm) = \frac{\left(170{,}000\ \frac{gpd}{ft}\right)(15\,ft)}{1500}$$

$Q = 1700\ gpm$

so that:

Q = 1700 gpm for pumping from a regional pumping well

Method 2. Schicht (1965) reports that "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." Therefore based on this method:

Q = 1000 gpm for pumping from a regional pumping well

Method 3. Schicht also reports the specific capacity from three wells in T2N, R10W (where Area 1 is located) as being 152.5, 188, and 158 gpm/ft, respectively. By averaging these specific capacities (166 gpm/ft), and multiplying by an assumed drawdown of 15 ft, a pumping rate of **2490 gpm** is obtained. As would be expected, use of specific capacities results in a wide range of predicted well pumping rates due to the effects of well construction, well condition, and local hydrogeologic conditions.

Q = 2500 gpm for pumping from a regional pumping well

RESULT: These calculation approaches suggest that a regional pumping well could yield from 1000 to 2500 gpm in the Area 1 location.

Therefore, it was assumed that the total pumping rate of any intensive pumping system would also be in this range, although the flow would be distributed among several wells. Therefore the following conceptual design was developed:

Qtotal = 1500 gpm total flowrate (based on lower-middle range of flowrate estimates to be conservative)

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APPENDIX B
RELATIVE SOURCE LIFETIME OF AREA I UNDER NATURAL ATTENUATION VS. INTENSIVE PUMP AND TREAT
Groundwater Alternative D, Intensive Pumping, Sites G, H, I, and L
Sauget Area 1, Sauget and Cahokia, Illinois

I. SOURCE LIFETIME CALCULATION: SITE I

PROBLEM: What is relative source lifetime of Site I under natural attenuation vs. intensive pump and treat conditions?

ASSUMPTIONS:

1. Source Volume = $(1400ft)(500ft)(95ft)\left(7.48\frac{gal}{ft^3}\right)(0.35)(0.05)(0.01) = 87{,}000\ gals$

   Source Mass = $(87{,}000gal)\left(\frac{3.78L}{gal}\right)\left(\frac{1.25kg}{L}\right) = 410{,}000\ kg$

   Where: width = 1400 ft; length = 500 ft; sat. thickness = 95 ft; porosity = 0.35; assumed residual saturation = 0.05; fraction of source containing residual saturation = 0.01. (see text)

2. Current Mass Removal Rate: 7000 kg/yr (Natural Attenuation) (see text)
   Initial Intensive Pumping Removal Rate: 17,500 kg/yr (Intensive Pump-and-Treat) (see text).

3. Case 1 Natural Attenuation Only – Site I
   Case 2 1 Yr of Intensive Pump-and-Treat + Natural Attenuation – Site I
   Case 3 5 Yrs of Intensive Pump-and-Treat + Natural Attenuation – Site I
   Case 4 10 Yrs of Intensive Pump-and-Treat + Natural Attenuation – Site I
   Case 5 30 Yrs of Intensive Pump-and-Treat + Natural Attenuation – Site I

4. Starting concentration under natural conditions: 20 mg/L (representative of middle and deep units in 1999). Starting concentration under pumping conditions: 5.5 mg/L (due to mass-transfer effects for deep and middle units; factor of 3.6 reduction).

5. Assumed ending concentration: 0.005 mg/L (MCL for several constituents).

MODEL:

$$\frac{C_{(t)}}{C_{(now)}} = e^{-k_s t}$$ (from BIOSCREEN and BIOCHLOR models; see text)

$$k_s = \frac{Mass\ Removal\ Rate\ (kg/yr)}{Mass\ (kg)}$$ (k_s = source decay constant)

$$t(yr) = \frac{-\ln\left(\frac{C_t}{C_{now}}\right)}{k_s}$$

Model Applied to Case 1: Natural Attenuation Only

$$k_s = \frac{7000\frac{kg}{yr}}{410{,}000\ kg} = 0.017\ yr^{-1}$$

$$t(yr) = \frac{-\ln\left(\frac{0.005}{20}\right)}{0.017}$$

RESULT (Case 1): *t = 488 years*

Model Applied to Case 2: Intensive Pump and Treat With 1 Year of Pumping

$$k_s = \frac{17{,}500\frac{kg}{yr}}{410{,}000\ kg} = 0.043\,yr^{-1}$$

$$\frac{C_{1yr}}{C_{now}} = e^{-k_s t}$$

$$\frac{C_{1yr}}{5.5\ mg/L} = e^{-(0.043yr^{-1})(1yr)}$$

$$C_{1\ yr} = 5.27\ mg/L$$

Time to cleanup after 1 yr of pumping is finished, with 3.6-times increase in concentration due to rebound (5.27 mg/L * 3.6 = 19.0 mg/L).

$$\frac{0.005\ mg/L}{19.0\ mg/L} = e^{-(0.017yr^{-1})(t\ yrs)}$$

$$t = \frac{-\ln\left(\frac{0.005}{19.0}\right)}{0.017}$$

$$t = 485\ years$$

RESULT (Case 2): Total time to cleanup (Intensive Pump-and-Treat for 1 yr) = 1 + 485 = **486 years**

Model Applied to Case 3: Intensive Pump and Treat With 5 Years of Pumping

$$k_S = \frac{17{,}500 \frac{kg}{yr}}{410{,}000\ kg} = 0.043\ yr^{-1}$$

$$\frac{C_{5\ yrs}}{C_{now}} = e^{-k_s t}$$

$$\frac{C_{5\ \mathrm{yrs}}}{5.5\ \mathrm{mg/L}} = e^{-(0.043\ \mathrm{yr}^{-1})(5\ \mathrm{yrs})}$$

$$C_{5\ yrs} = 4.44\ mg/L$$

Time to cleanup after 5 yrs of pumping is finished, with 3.6-times increase in concentration due to rebound (4.44 mg/L * 3.6 = 16.0 mg/L).

$$\frac{0.005\ mg/L}{16.0\ mg/L} = e^{-(0.017\ yr^{-1})(t\ yrs)}$$

$$t = \frac{-\ln\left(\frac{0.005}{16.0}\right)}{0.017}$$

$$t = 475\ years$$

RESULT (Case 3): Total time to cleanup (Intensive Pump-and-Treat for 5 yrs) = 5 + 475 = **480 years**

Model Applied to Case 4: Intensive Pump and Treat With 10 Years of Pumping

$$k_s = \frac{17{,}500 \frac{kg}{yr}}{410{,}000\ kg} = 0.043\, yr^{-1}$$

$$\frac{C_{10yrs}}{C_{now}} = e^{-k_s t}$$

$$\frac{C_{10yrs}}{5.5\ mg/L} = e^{-(0.043 yr^{-1})(10 yrs)}$$

$$C_{10\ yrs} = 3.58\ mg/L$$

Time to cleanup after 10 yrs of pumping is finished, with 3.6-times increase in concentration due to rebound (3.58 mg/L * 3.6 = 12.9 mg/L).

$$\frac{0.005\ mg/L}{12.9\ mg/L} = e^{-(0.017 yr^{-1})(t\ yrs)}$$

$$t = \frac{-\ln\left(\frac{0.005}{12.9}\right)}{0.017}$$

$$t = 462\ years$$

RESULT (Case 4): Total time to cleanup (Intensive Pump-and-Treat for 10 yrs) = 10 + 462 = **472 years**

Model Applied to Case 5: Intensive Pump and Treat With 30 Years of Pumping

$$k_s = \frac{17{,}500 \frac{kg}{yr}}{410{,}000\ kg} = 0.043\, yr^{-1}$$

$$\frac{C_{30yrs}}{C_{now}} = e^{-k_s t}$$

$$\frac{C_{30yrs}}{5.5\ mg/L} = e^{-(0.043 yr^{-1})(30 yrs)}$$

$$C_{30\ yrs} = 1.51\ mg/L$$

Time to cleanup after 30 yrs of pumping is finished, with 3.6-times increase in concentration due to rebound (1.51 mg/L * 3.6 = 5.4 mg/L).

$$\frac{0.005\ mg/L}{5.4\ mg/L} = e^{-(0.017 yr^{-1})(t\ yrs)}$$

$$t = \frac{-\ln\left(\frac{0.005}{5.4}\right)}{0.017}$$

$$t = 411\ years$$

RESULT (Case 5): Total time to cleanup (Intensive Pump-and-Treat for 30 yrs) = 30 + 411 = **441 years**

CONCLUSION: Comparison of Cleanup Times – Site I

Case 1	Natural Attenuation Only	488 years
Case 2	1 Yr of Intensive Pump-and-Treat + Natural Attenuation	486 years
Case 3	5 Yrs of Intensive Pump-and-Treat + Natural Attenuation	480 years
Case 4	10 Yrs of Intensive Pump-and-Treat + Natural Attenuation	472 years
Case 5	30 Yrs of Intensive Pump-and-Treat + Natural Attenuation	441 years

II. SOURCE LIFETIME CALCULATION: SITES G/H/L

PROBLEM: What is relative source lifetime of Sites G/H/L under natural attenuation vs. intensive pump and trea conditions?

ASSUMPTIONS:

1. Assume Sites G/H/L together have approximately same dimensions as Site I, with 1% of the starting mass as Site I, because VOC+SVOC concentrations are much lower leaving Sites G/H/L (~ 0.20 mg/L) than Site I (~ 20 mg/L).

2. Assume ratio of source mass at Sites G/H/L and Site I are proportional to ratio of representative concentrations and width parallel to groundwater flow leaving Sites G/H/L and Site I.

$$\text{Source Mass} = \left(\frac{\text{representative conc. Sites G/H/L mg/L}}{\text{representative conc. Site I mg/L}}\right)(\text{Est. Mass Site I kg})$$

$$= \left(\frac{0.20\ \text{mg/L}}{20\ \text{mg/L}}\right)(410{,}000\ \text{kg})$$

$$= 4100\ \text{kgs}$$

3. Current Mass Removal Rate: Assume 1% of Site I mass removal rate based on ratio of representative concentration at Site I (20 mg/L VOC+SVOC) to representative concentration at Sites G/H/L (0.20 mg/L VOC+SVOC).

Natural Source Removal Rate =

$$\left(\frac{\text{representative conc. Sites G/H/L mg/L}}{\text{representative conc. Site I mg/L}}\right)\left(\frac{\text{Width Sites G/H/L}}{\text{Width Site I}}\right)(\text{Est. Mass Removal Rate Site I kg/yr})$$

$$= \left(\frac{0.20\ \text{mg/L}}{20\ \text{mg/L}}\right)\left(\frac{750\ \text{ft}}{1400\ \text{ft}}\right)(7000\ \text{kg/yr})$$

$$= 35\ \text{kg/yr}$$

Use Initial Intensive Pumping Removal Rate of 2.5 times 34 kg/yr (Intensive Pump-and-Treat= 87.5 kg/yr

4. Run analysis for two cases:

   Case 6 Natural Attenuation Only – Site G/H/L

   Case 7 5 Yrs of Intensive Pump-and-Treat + Natural Attenuation – Site G/H/L

5. Starting concentration under natural conditions: 0.20 mg/L (representative of middle and deep units in 1999). Starting concentration under pumping conditions: 0.056 mg/L (due to mass-transfer effects for middle and deep units; factor of 3.6 reduction).

6. Assumed ending concentration: 0.005 mg/L (MCL for several constituents).

MODEL:

$$\frac{C_{(t)}}{C_{(now)}} = e^{-k_s t}$$ (from BIOSCREEN and BIOCHLOR models; see text)

$$k_s = \frac{Mass\ Removal\ Rate\ (kg/yr)}{Mass\ (kg)}$$ (k_s = source decay constant)

$$t(yr) = \frac{-\ln\left(\frac{C_t}{C_{now}}\right)}{k_s}$$

Model Applied to Case 6: Natural Attenuation Only

$$k_s = \frac{35\frac{kg}{yr}}{4100\ kg} = 0.0085\ yr^{-1}$$

$$t(yr) = \frac{-\ln\left(\frac{0.005}{0.2}\right)}{0.0085}$$

RESULT (Case 6): ***t = 434 years***

Model Applied to Case 7: Intensive Pump and Treat With 5 Years of Pumping

$$k_s = \frac{87.5\frac{kg}{yr}}{4100\ kg} = 0.021 yr^{-1}$$

$$\frac{C_{1yr}}{C_{now}} = e^{-k_s t}$$

Starting concentration under pumping conditions: 0.056 mg/L (due to mass-transfer effects for deep and middle units; factor of 3.6 reduction).

$$\frac{C_{5yr}}{0.056\ mg/L} = e^{-(0.021 yr^{-1})(5 yrs)}$$

$$C_{5\ yr} = 0.050\ mg/L$$

Time to cleanup after 5 yrs of pumping is finished, with 3.6-times increase in concentration due to rebound (0.050 mg/L * 3.6 = 0.18 mg/L).

$$\frac{0.005\ mg/L}{0.18\ mg/L} = e^{-\left(0.00085 yr^{-1}\right)(t\ yrs)}$$

$$t = \frac{-\ln\left(\frac{0.005}{0.18}\right)}{0.0085}$$

$$t = 422\ years$$

RESULT (Case 7): Total time to cleanup (Intensive Pump-and-Treat for 5 yrs) = 5 + 422 = **427 years**

CONCLUSION: Comparison of Cleanup Times – Sites G/H/L

Case 6	Natural Attenuation Only – Sites G/H/L	434 years
Case 7	30 Yrs of Intensive Pump-and-Treat + Natural Attenuation – Sites G/H/L	427 years

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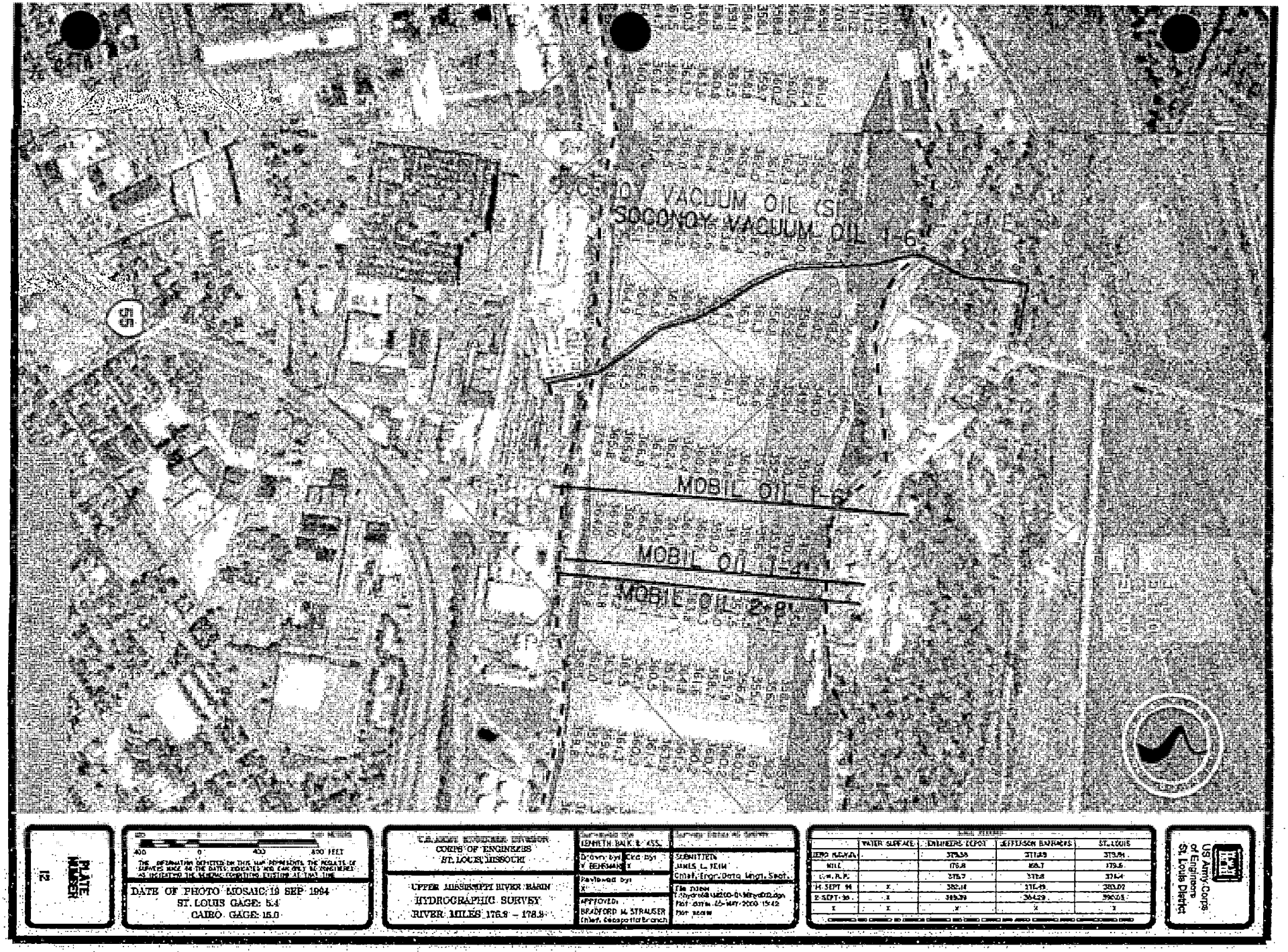
DRAFT
GSI Job No. G-2561
March 31, 2002

GROUNDWATER
SERVICES, INC.

Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

ATTACHMENT 9



Attachment 9
US Corps of Army Engineers
River Map

DRAFT
GSI Job No. G-2561
March 31, 2002

GROUNDWATER
SERVICES, INC.

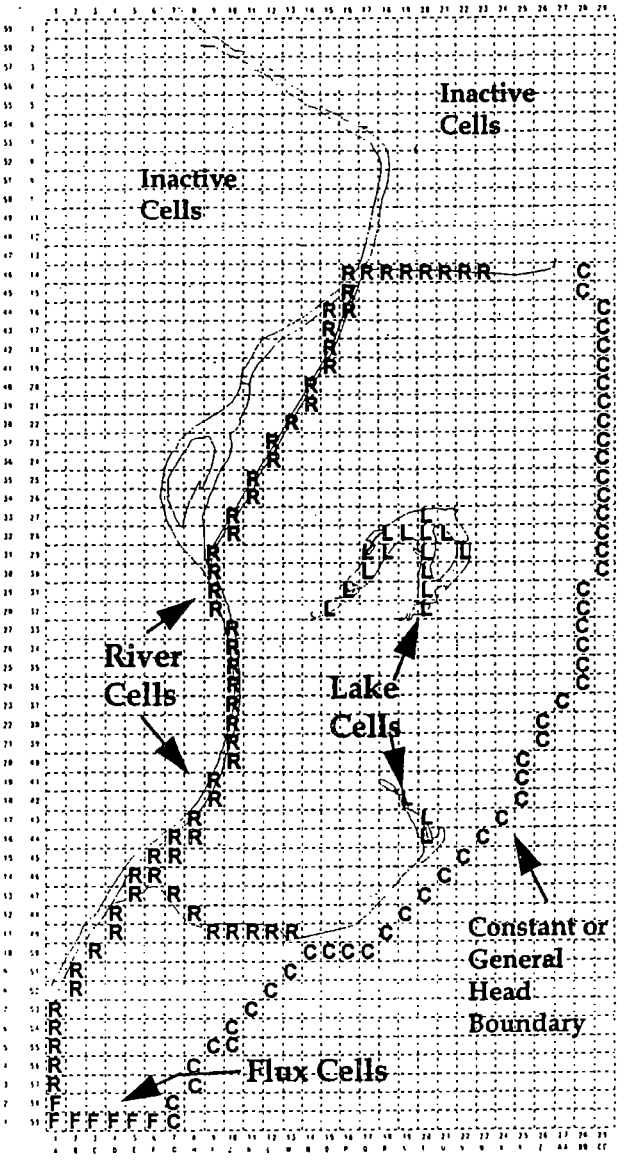
Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

ATTACHMENT 11

AMERICAN BOTTOMS REGIONAL GROUND-WATER FLOW MODEL

Prepared by
Gary R. Clark, P.E.



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AMERICAN BOTTOMS REGIONAL GROUND-WATER FLOW MODEL

by

Gary R. Clark, P.E.

August 25, 1997

Prepared in partial fulfilment
of Section 5 of the Flood Control Act of 1945,
615 ILCS 15/5

Illinois Department of Natural Resources
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ABSTRACT

The American Bottoms is a 210 square mile area of Mississippi River lowlands located in Illinois directly across the Mississippi River from St. Louis, Missouri. The area includes portions of Madison and St. Clair Counties and includes the municipalities of Cahokia, East St. Louis, Granite City, Wood River and Alton, Illinois. In 1974, the Corps of Engineers initiated an extensive study of flooding and drainage problems in this area and in the early 1980s the Corps of Engineers contracted with the Illinois State Water Survey for an analysis of the groundwater hydrology of the American Bottoms aquifer system as well as the development of a mainframe computer based digital groundwater flow model.

The results of the State Water Survey's evaluation of the groundwater hydrology along with an overview of historical water levels and pumpage is summarized. The assumptions and data that were used to develop the State Water Survey PLASM version of a digital groundwater flow model are discussed to document the development of an updated MODFLOW regional groundwater flow model that was developed in this study. The new MODFLOW model covers a 180 square mile area of the American Bottoms. The new model was developed for the years of 1905-1994 and was revised by the use of the PACE method for calculating recharge along with the use of constant head cells as a model boundary to reduce errors in the volumetric budget output. Output of the MODFLOW model compared favorably to the output from the PLASM model developed by the State Water Survey. Model output was compared to data from three long term observation wells located in the Granite City area. Output was also generated to construct groundwater level exceedance probability curves for model cells located in area of high damages due to groundwater flooding.

The procedures used by the Corps of Engineers to estimate damages due to groundwater flooding in the modeled area are documented. These procedures are applied to re-evaluate the economic benefits of different regional and local pumping systems designed to manage and reduce high groundwater elevations and associated damages in the Granite City area. The benefits and costs two example pumping systems are presented.

INTRODUCTION

Background and Purpose of the Study

In 1992, the St. Louis District of the Corps of Engineers released a final report on their evaluation of the feasibility and economics of various measures designed to reduce flood damages in the American Bottoms area which were the result of high groundwater elevations. The Corps' original plan had a favorable benefit-cost ratio of 1.6, but follow-up studies suggested that substantial costs could be required to properly treat and discharge the pumped groundwater. Revised estimates lowered the benefit-cost ratio of the Corp's selected plan to less than 0.80. Due to the lack of economic justification, the Corps terminated their study and active interest in implementing any regional groundwater flood hazard reduction measures.

In 1993, Granite City approached the Director of the Office of Water Resources and requested that the state consider an evaluation of a limited groundwater flood hazard reduction project for the Granite City area along with an innovative means of dealing with the water quality issues associated with discharging the pumped groundwater. The Director of the Office Water Resources requested that the author of this study consider the feasibility of developing a new PC based groundwater flow model for the American Bottoms region that could be used to design and evaluate a groundwater flood hazard reduction project for the Granite City area. Capital funding for this project was requested and finally authorized in 1997.

Previous Studies

A significant study of the hydrogeology of American Bottoms area was published by the Illinois State Water Survey (Schicht, 1965). This report describes the geologic setting and the hydrologic characteristics of the extensive sand and gravel aquifer system. The report includes numerous historical water table maps along with estimates of recharge, hydraulic conductivity and storage coefficients for various regions of the aquifer system. The report also evaluates the significance of the major boundaries to the aquifer system.

In 1976, the Corps published a report entitled "Preliminary Groundwater Analysis." This report was developed to summarize all previous studies of the American Bottoms groundwater system and to define the extent of groundwater related problems as well as to identify possible long term solutions to these problems. This report also laid out a plan of study for a more comprehensive Survey Report for evaluating in detail, measures to reduce groundwater related damages. A number of groundwater resource maps were developed for this report. The geology and soils sections of this study are derived from this report.

A 1984, contract report published by the Illinois State Water Survey for the Corps of Engineers was entitled: "Ground Water Level Analysis by Computer Modeling: American

Bottoms Ground Water Study" (Schicht, R.J., J. D. Ritchey, and L.S. Weiss, 1984). This report is an in-depth investigation of groundwater flow in the American Bottoms area. There were five identified objectives to this study. They were 1) to compile current hydrologic data pertaining to the area, 2) to develop a computer model (PLASM) that could simulate the movement of groundwater, 3) to analyze existing and future groundwater levels in the area, 4) to present alternatives to lower or maintain groundwater levels at specified elevations in a designated area of interest and 5) to provide documentation of the model including a user's guide. This contract report was prepared in five separate documents and was the foundation for the development of the MODFLOW groundwater flow model developed for this current study. Pages 13-37 of this study are derived from this State Water Survey Report.

The St. Louis District of the Corps published a final feasibility report and draft environmental impact statement as final documentation of their analysis of the groundwater relating flooding problems of the American Bottoms area. This report presented in detail the procedures developed by the Corps to evaluate the damages caused by groundwater flooding and the resultant economic benefits of various alternatives selected to reduce these damages. The major portions of the economic analysis sections of this study are derived from the Corps' feasibility report.

Acknowledgments

This report was prepared in-house as one of the ongoing research and studies projects of the Planning and Research Section of the Office of Water Resources. A special note of thanks to all the individuals who provided updated as well as historical data for this groundwater study. Dan Marshall and Ron Dieckman of the St. Louis Corps spent numerous hours of staff time retrieving invaluable data that was required to re-generate the damage estimates and economic analysis used in this study. Andy Buck, Vern Knapp, Ellis Sanderson and Ken Hlinka of the Illinois State Water Survey also provided updated data concerning the hydrology of the American Bottoms aquifer system which was used to extend the modeling period into the 1990's. The assistance of Jamie Dehner who wrote the computer program used to translate river stage data into formatted input files is also greatly appreciated.

DESCRIPTION OF THE STUDY AREA

Corps of Engineers Study

The Corps of Engineers initiated an extensive study of the groundwater problems in the American Bottoms area in 1974. These studies were initiated as part of an overall authority to study water resources problems in the greater St. Louis area and this study became known as the St. Louis Metropolitan Area, Missouri and Illinois, Study ("Metro Study"). Six separate Congressional resolutions provided for this overall study authority. The two that were adopted in September of 1972 applied specifically to the counties in the American Bottoms area.

During the initial scoping for this study, numerous local interests made it clear that rising groundwater levels were causing serious problems with widespread sewer breaks, flooded basements and related structural damages. Due to these concerns, groundwater was added as an eighth study purpose to the Metro Study.

Study Area

The name "American Bottoms" used herein refers to the Illinois side of the Mississippi River flood plain east of St. Louis, Missouri between Alton and Dupo, Illinois. See figure 1. The area covers 175 square miles, is approximately 30 miles long, and has a maximum width of 11 miles from the Mississippi River to the eastern bluff line. Portions of Madison and St. Clair Counties and a small corner of Monroe County are included in the bottoms area. The principal cities are Cahokia, East St. Louis, Granite City, Wood River and Alton. The land use in the area is quite varied. Municipal areas such as East St. Louis, Granite City and Cahokia are highly developed for residential, industrial and commercial uses. Large sections of the rural areas are devoted to agricultural use. The remaining natural areas consist of a few large lakes such as Horseshoe Lake and lakes in Frank Holton State Park as well as some scattered marshes and seasonal wetlands. Only a few scattered remnants of forest remain. The American bottoms is extremely rich in prehistoric resources having had the largest level of prehistoric occupation north of Mexico.

Topography

As any large floodplain area, the American Bottoms is relatively flat. Overall relief varies from 40 to 50 feet. Maximum elevations are about 445 ft msl near the eastern bluffs in the Roxana area and minimum elevations are slightly below 400 ft msl near the river bank and low swales in the southern portion of the area. Since only 10 percent of the land surface is more than 5 feet above flood stage, the area is now protected from Mississippi River flooding by federally constructed and maintained levees.

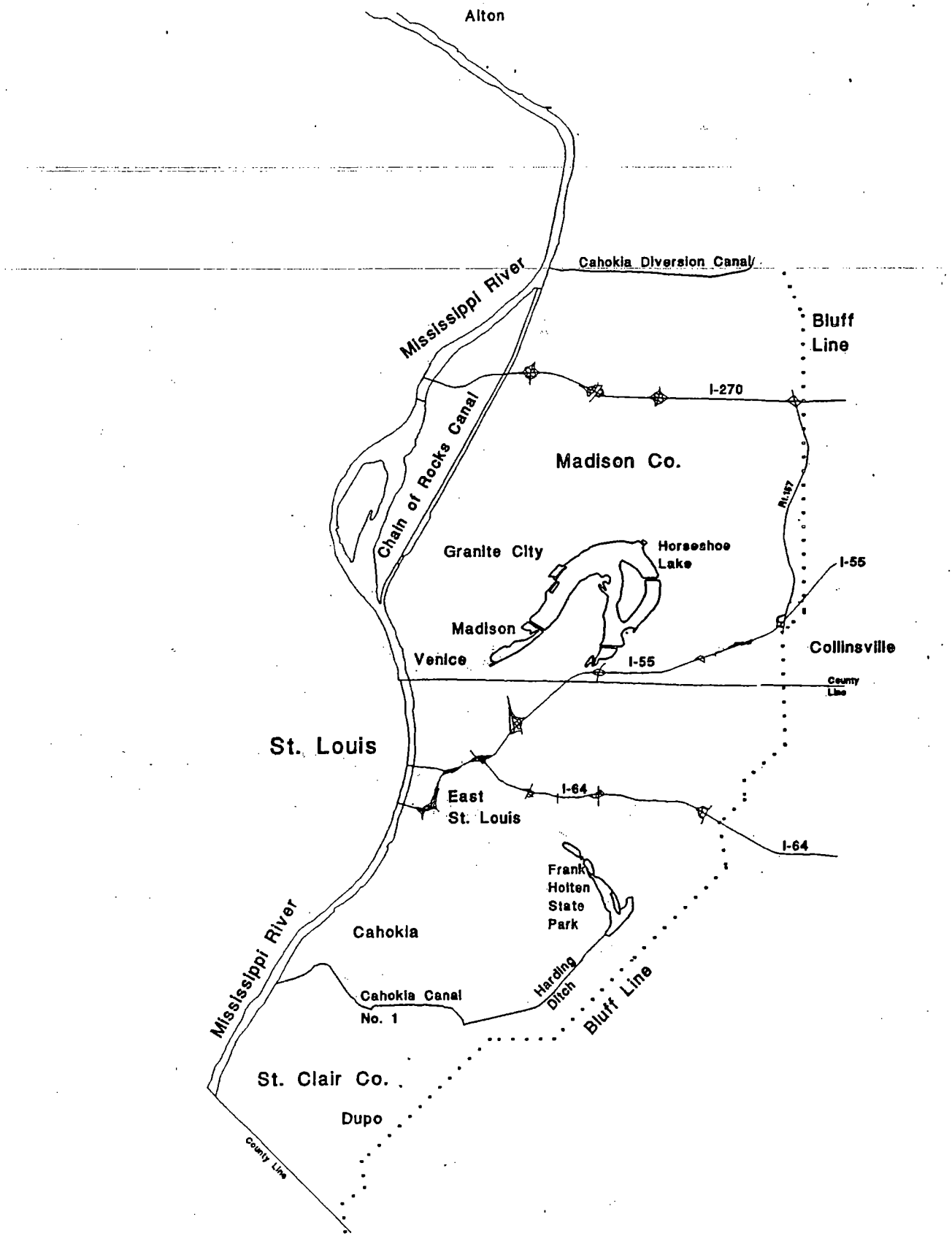


Figure 1. Study area location map.

Surficial Geology

A detailed study of the geology of the American Bottoms was reported by Berstrom and Walker (1956) and summarized by the Corps (1976). The American Bottoms is located over a bedrock valley with bedrock depth averaging about 120 feet below ground level. This valley is filled with sand, gravels, silts, and clays and because of the variations in the bedrock surface and ground elevations, the thickness of the valley fill ranges from a few feet near the bluffs and the Chain of Rocks area of the Mississippi River to more than 170 feet near Wood River. The valley fill is composed both of glacial materials, deposited by meltwaters from the Ice Ages, and recent alluvium, deposited by the river during floods in post-glacial times. Generally, the glacial materials consist of sand and gravel, and are located near the bottom of the valley fill: whereas the alluvium may be gravel, sand, silt and clay and is found in the upper part of the fill. At any specific location the actual composition of the deposits can be very complex and variable. Although levees now protect the area from Mississippi River flooding, interior stream and drainage ditches continue to erode and redeposit surface materials. It should be noted that some of the area having the poorest foundation conditions were actual lake bottoms less than a century ago.

Landforms

Even though the American Bottoms is a relatively flat floodplain area, there are some significant variations in surface topography related to the unique depositional history for each area. The Corps (1976) summarized the work of Yarborough and Chistie (1972) in categorizing the American Bottoms into the seven general landform regions described below. See figure 2.

Terrace Region - The terrace region lies in the northern section of the floodplain above the confluence of the Missouri River. It is the only area where the original glacial deposits called the Henry formation extend to the surface. This area is 10 to 15 feet higher than the adjacent bottom lands and has very sandy, permeable soils. Portions of East Alton, Wood River, and Roxana occupy the terrace region.

Ridge and Swale Region - This region lies adjacent to the river and is associated with the most recent cut and fill action by the river. In the past this was a region of swampy, partially filled watercourses, active sloughs, natural levees and sand bars. Today the old chutes have been filled and islands attached to the mainland. Soils in this region are variable with sandy soils generally occurring in the ridges and soft weak clays located in the swales. Major development in this region includes portions of Cahokia, Madison, and Granite City.

East St. Louis Rise - This region is a topographic high, although lower than the terrace region, consisting in part of an old alluvial fan, or deposit of soil washed onto the floodplain from the bluffs. East St. Louis occupies this region.

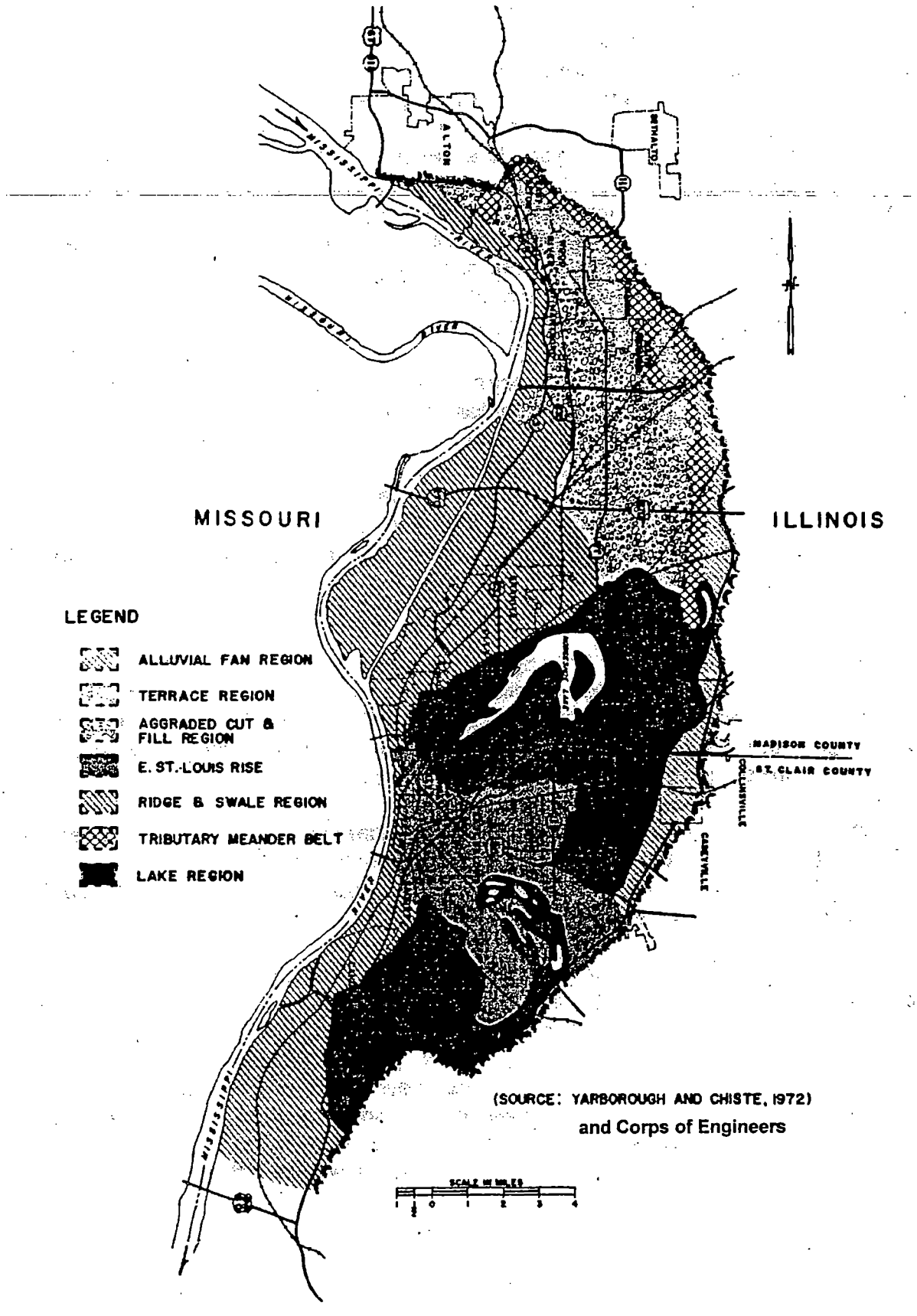


Figure 2. Landform Regions

Lake Region - This is an area of recent oxbow lakes (old river bends subsequently cut off by natural river changes) that were dried up by drainage projects, filling and groundwater pumping, leaving Horseshoe Lake as the only major remaining water body. Soils in these dried up lake areas are frequently poorly drained clays. As areas with the better soils become developed, this area is starting to experience continued development pressures.

Tributary Meander Belt - This region consists of the areas in which Wood River and Cahokia creeks cut through and meandered about the floodplain en route to the Mississippi. During recent development of the American Bottoms these streams were rerouted into major drainage ditches to reduce flooding, leaving this belt as a low area that continues to have surface ponding problems.

Alluvial Fan Area - This is the area that parallels the bluffs and consists of relatively high deposits of loess or silty materials carried down from the bluffs by small streams. The town of Caseyville is located in this region.

Aggraded Cut and Fill Region - This region is the most level in the American Bottoms because it has been covered by clay sediments during flooding of Long Lake and Cahokia Creek. The area suffers from localized ponding, a high water table and clay soils that shrink and swell. This area is primarily agricultural.

Soils

As discussed previously, the types of soils that cover the American Bottoms study area are related to their mode of deposition. The Corps (1987) described the five separate units that comprise the majority of the unconsolidated deposits in the American Bottoms area. These units are described below.

Abandoned Channel - This environment consists of predominately fine-grained sediments introduced into the abandoned channel during periods of flooding. These deposits normally accumulate slowly in thin layers. The predominant soil type found in the abandoned channel is clay.

Backswamp - Sediments within these areas occur as thin layers deposited by the flood waters which periodically covered portions of the flood plain. The soil types found in the backswamp deposits are sand-silt mixtures to clays with silty clays predominating.

Point Bar - Point bar deposits extend as deep as the thalwag of the old river channel. There are two main types of deposits within the point bar: silty and sandy elongate bar deposits laid down during high river stages and silty and clayey deposits in depressions or swales deposited during falling flood stages. Soil types associated with point bar deposits vary from silty sands to clays.

Chutes and Bars - The sediments resulting from the chute and bar deposition forms more of an irregular surface than a point bar and varies from sand and gravel at the base to silty sand to sand ridges or bars and silty clay to clay filled chutes.

Glacial Deposits - In most of the American Bottoms area the modern surficial soils are underlain by a relatively thick deposit of sand and gravels deposited by glacial meltwater streams during the Pleistocene Epoch. Poorly graded, medium to coarse grained sands, comprise the major soil type in the glacial deposits.

Surficial Deposits - Surficial deposits in the area as discussed previously are highly variable. The Corps (1976) combined various soils with similar attributes into three differentiated groups that are depicted in figure 3. The first group which is shown in dark gray consists of soil associations in which most of the soils have seasonally high water tables. This is due to the existence of clay layers near the surface that restrict the downward movement of water to a rate less than surface recharge from rainfall. This condition is referred to as a perched water table. Much of the soils in the lake region and nearly all the soils in the aggraded cut and fill region fall into this category. The second group consists of soil associations in which nearly half the land experiences seasonally high water tables. The ridge and swale region and the East St. Louis Rise typically exhibit these conditions. The third group consist of very sandy soils, well drained soils and is only found in the Terrace Region.

Historical Water Levels, Pumpage and Recharge

Groundwater in the American Bottoms is a dynamic system that is constantly changing in response to variations in rainfall, river levels, and pumpage. Regionally, groundwater levels vary primarily with seasonal and long-term variations in precipitation, pumpage and river stages. When precipitation is above average, water levels rise and conversely if pumpage exceeds average rainfall induced recharge, groundwater levels will decline. The following paragraphs summarize the major trends that have occurred since the early 1900's. Most of this information was summarized by the Corps (1976) from numerous reports developed for the American Bottoms region by the Illinois State Water Survey. See bibliography.

A general discussion of regional water levels was drawn from an analysis of observation well data by Schicht, 1965. For wells remote from major pumping centers, water levels generally receded in the late spring, summer, and early fall when discharge from the groundwater system by evapotranspiration, by groundwater 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 streams. Water levels generally begin to recover in early winter when conditions are favorable for rainfall induced recharge and increases in water levels are most pronounced during the spring months. Water levels therefore are the highest in May and the lowest in December. Water levels in wells generally have a seasonal fluctuation ranging from 1 to 13 feet and average about 4 feet.

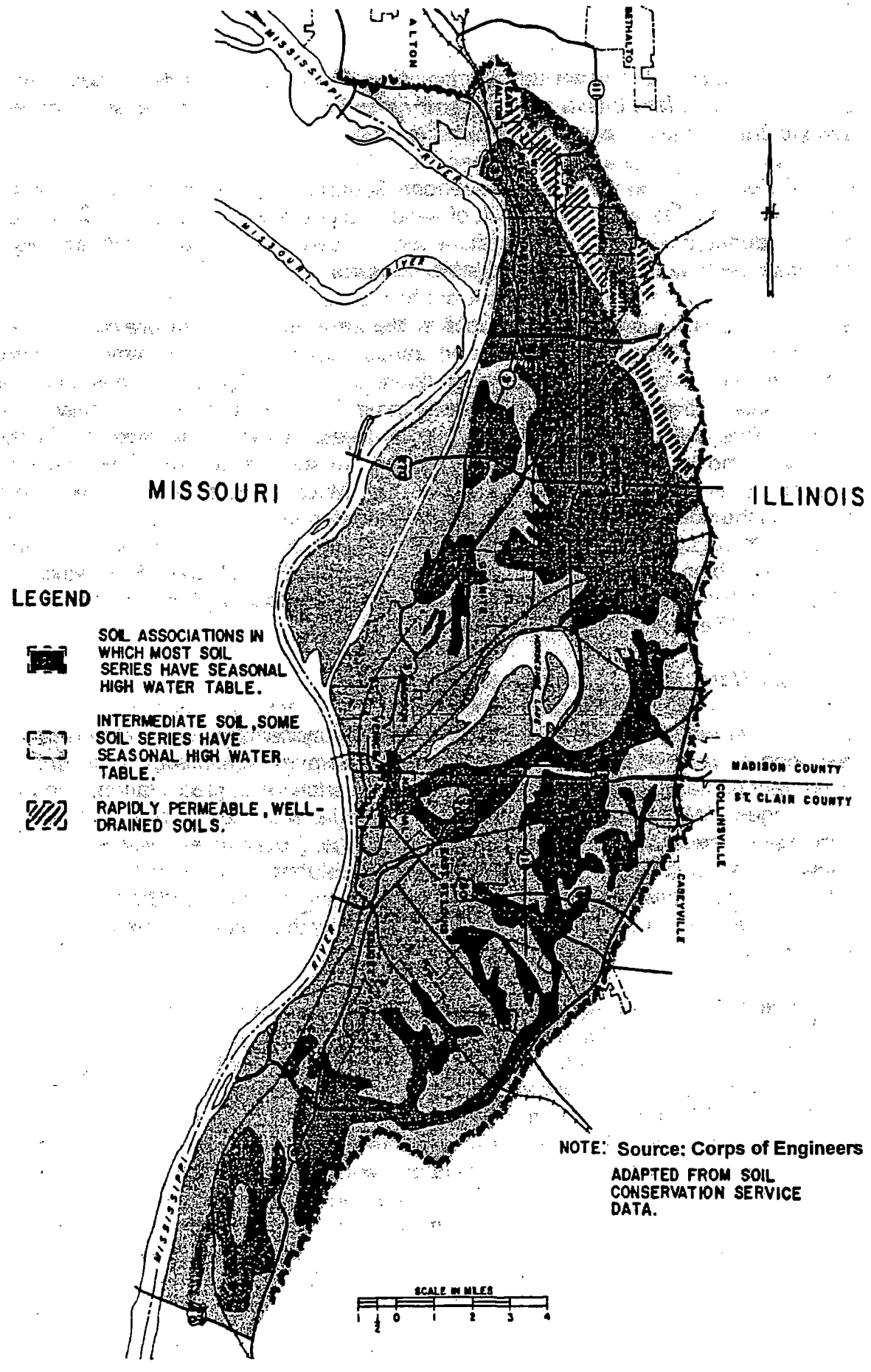


Figure 3. Soil Groups

1900 to 1956

In a 1907 State Geological Survey study, Bowman and Reeds, stated that "Shallow wells indicate that the water level occurs normally from a few inches to a few feet below the surface..." This statement tends to characterize the condition of the groundwater prior to modern development. With development came the construction of numerous levees and drainage ditches, initially in the area of East St. Louis. In 1953, Bruin and Smith estimated that the natural lake area had been reduced by more than 40 percent between 1907 and 1950 and that more than 40 miles of improved drainage ditches had been constructed during that time period. They further estimated that these developments caused a lowering of the groundwater levels by 2 to 12 feet. During the period of 1900 to 1956, groundwater pumpage, mostly industrial, increased steadily from 2.1 million gallons per day (mgd) to 111.0 mgd. As a result of this increase in pumping, water levels declined about 50 feet in the Monsanto area, 40 feet in the Wood River area, and 60 feet in the Granite City area. Due to alarming water level recessions and concern with depleted water supplies, the Illinois State Water Survey accelerated its program of groundwater investigations in 1941. A prolonged drought occurred from 1952 to 1956 when rainfall at Edwardsville only averaged 34.3 inches per year, or about 6.5 inches below normal. This drought further contributed to low groundwater levels, causing the lowest recorded levels to occur in 1956.

1957 to 1961

Groundwater levels were so low in 1956 that the Granite City Steel Company abandoned its wells in 1957 and began obtaining its water from the Mississippi River (Schicht and Jones, 1962). As a result of this and other reductions in pumpage, withdrawals in the area of Granite City dropped from a peak of 31.6 mgd in 1954 to 7.6 mgd in 1958. From 1957 to 1961, groundwater levels in this area rose 50 feet. Overall pumpage in the American Bottoms declined from 111.0 mgd in 1956 to 93.0 mgd in 1960. Outside the area of Granite City, the rise in groundwater levels varied from 0 to 10 feet during this time period.

1962 to 1966

During this period, pumpage increased erratically from 99.4 to 108.1 mgd (Reitz, 1968). Pumpage was greatest in 1964 when 110.2 mgd was withdrawn. Pumpage in all major centers except the Monsanto area showed an increase. Over most of the American Bottoms, groundwater levels declined from 0 to 5 feet.

For water levels measured during the year of 1962, the Illinois State Water Survey (Schicht, 1965) calculated a mass balance for the aquifer system that estimated recharge from precipitation at 65 mgd, subsurface flow from the bluffs at 12.8 mgd, induced infiltration from the river as 48.2 mgd, for a total recharge value of 126.0 mgd. The difference between the 126.0 mgd recharge and the estimated pumpage of 96.8 mgd amounts to groundwater flow to the river.

1967 to 1971

During this period pumpage steadily declined from 108.1 mgd in 1966 to 79.5 mgd in 1971 (Baker, 1972). The decrease in pumpage was attributed to the closing of two large groundwater using industries and to conservation measures being introduced by almost all of the remaining industries. Pumpage in four of the five major centers declined with the Monsanto area showing the largest decline. High Mississippi River stages occurred during this time period while precipitation was normal. The rise in groundwater levels over the period was 25 feet in the Monsanto area, 5 feet in the National City area and 10 feet in the Granite City area. In remote areas groundwater levels again rose from 0 to 5 feet.

1972 to 1976

Above average precipitation and record long duration high river stages which included the 1973 flood, existed along with the continuing decline in pumping rates. The result was record high groundwater levels being measured in the summer of 1973. See figure 12.

1977 to 1990

Pumping steadily declined to 62.1 mgd in 1977. There was a slight increase to 68.2 mgd in 1978 followed by continuing declines until withdrawals were down to 45 mgd in 1981. A survey of groundwater use in 1981 estimated that industry accounted for 30.1 mgd in use (67%), municipalities used 9.5 mgd (21.5%) and the Illinois Department of Transportation withdrew 5.1 mgd or (11.4%). The remaining 1% of use was considered for electrical power generation. In 1990, the U.S.G.S. estimated total pumping at 20.2 mgd with industry using 10.4 mgd and municipalities using 7.7 mgd.

Groundwater Quality

Numerous water quality studies were conducted for the Corps of Engineers during their American Bottoms Groundwater study. A study conducted by the U.S. Geological Survey in 1982, Voelker, 1985, concluded from the analyses of samples from 63 wells that most constituent concentrations do not exceed State of Illinois water quality standards. The waters are primarily of the calcium-magnesium-bicarbonate type with some calcium-sulfate type water. Iron concentrations ranged from less than 3 to 82,000 micrograms per liter, manganese from 5 to 5,300 micrograms per liter and dissolved solids from 140 to 3,000 milligrams per liter. These three constituent concentrations exceeded Illinois' public water supply, effluent, and general water-quality standards in 79, 92, and 67 percent of the samples, respectively. Concentrations of nitrate + nitrate nitrogen, fluoride, mercury, zinc, lead and sulfate also exceeded Illinois water quality standards in a few samples from individual wells. With the exception of one sample, containing 0.01 micrograms per liter of Dieldren, concentrations of organic pesticides, polychlorinated biphenyls and polychlorinated naphthalenes were below analytical detection limits in the 15 samples analyzed for these constituents.

ANALYSIS OF WATER LEVELS IN MONITORING WELLS

The State Water Survey conducted a detailed analysis of existing water level data as a part of their study of the aquifer system for the Corps of Engineers. The following paragraphs give an overview of their analysis.

Water levels in wells in the American Bottoms area have been measured periodically since the 1940's. In the 1950's a network of observation wells was established with the main objective of providing data on long-term trends in groundwater levels. The location of the wells in the network are shown in figure 4.

For their analysis, the State Water Survey selected 10 key observation wells to show the apparent effects of pumpage, river stage, and climatic conditions on groundwater levels. Hydrographs from these wells show that water levels in the American Bottoms area generally recede in the late spring, summer, and early fall when combinations of discharge from the groundwater system by evapotranspiration, by groundwater runoff into streams, and by pumping from wells is greater than recharge from precipitation and induced infiltration of surface water from the Mississippi River and other streams. Water levels generally begin to recover in the early winter when the 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 groundwater reservoir receives most of its annual recharge. Maximum and minimum annual water levels are recorded at different times of the year. Water levels are frequently highest in May and lowest in December, depending primarily upon climatic conditions, pumping and the stage of the Mississippi River.

Water levels in three wells, denoted MAD 3N8W-31.2a, MAD 3N9W-14.2c, and STC 2N9W-26.7e, reflect changes in water levels due mainly to climatic conditions.

Water levels in Mad 3N8W-31.2a have been measured since the summer of 1941 and monthly measurements have been taken since 1953. The well is located about 1/2 mile from the bluffs and near the Collinsville Municipal Well field. Water levels are effected by climatic conditions and nearby pumpage in the Caseyville area and mainly the Collinsville well field. Since pumping has only increased gradually the water levels reflect primarily long term climatic trends. The effects of the drought conditions on water levels are clearly shown during 1952-1956 1962-1966, 1976-1977, 1980 and 1988-1990.

Water levels in wells MAD 3N9W-14.2c and STC 2N9W-26.7e have been measured since 1956 and 1952 respectively. Both of these wells show the impacts of the drought conditions of 1952-1956 1962-1966, 1976-1977, 1980 and 1988-1990.

Water levels in three observation wells, wells MAD 5N9W-29.4f, MAD 3N10W-12.4f, and STC 2N10W-33.2f, reflect changes in water levels due mainly to changes in river stage.

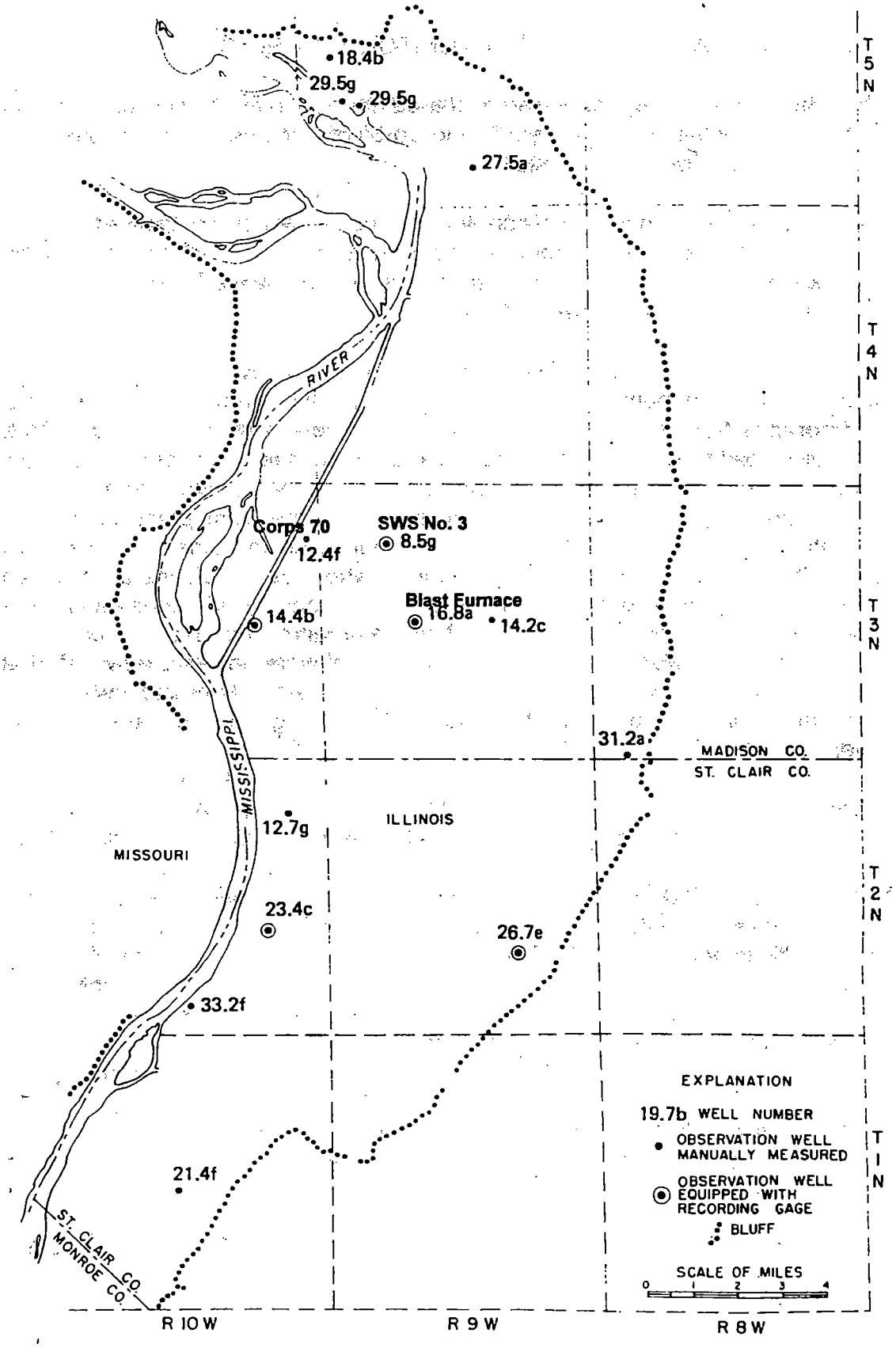


Figure 4. Location of State Water Survey Observation Wells

Observation well MAD 5N9W-29.4f is a relief well that is located 300 yards from the Mississippi River. It has been equipped with a automatic water level recorder since 1956. This well clearly reflects changes in stage of the Mississippi River even though the well is located near a large pumping center. Annual water level fluctuations have ranged from a low of 6 feet per year in 1968 to a high of 19 feet in 1960. Water levels were above the land surface during the flood of 1993. The lowest water level on record was measured early in 1957.

Observation well MAD 3N10W-12.4f (Corps of Engineers relief well no. 70) is a levee relief well located approximately 100 yards from the Chain of Rocks Canal. Water levels have been taken monthly since 1953. The lowest annual water level fluctuations was recorded in 1968 at 2 1/4 feet with the highest recorded in 1960 at about 13 feet which are the corresponding years for MAD 5N9W-29.4f. Groundwater levels as a result of high river stages were above land surface in 1960, 1961, 1969, 1970, 1973, 1974, 1975, 1976, 1978, 1981-1987, 1990-1994, and 1996. The lowest water level was recorded in 1957. This observation well was used for verification of the updated MODFLOW model developed in this current study. See figure 5 and figure 25.

Water levels were measured in observation well STC 2N10W-33.2f from 1950 to the present. The well is located approximately 100 yards from the river. The maximum annual recorded fluctuation of 30 feet occurred in 1951 and the minimum of 18 feet occurred in 1968. The highest recorded level, less than 5 feet below land surface, occurred in 1973. The lowest, 38 feet below land surface, occurred in 1964 and 1981.

A comparison with river stages measured near Jefferson Barracks shows the relationship between river stages and groundwater levels in an area remote from a pumping center. During periods of high river stages, groundwater levels are generally at lower elevations than the river. The opposite is generally true during periods of low river stages.

Observation well MAD 3N9W-8.5g is located about 2 1/2 miles northeast of the Granite City Pumping Center. The well was constructed for the State of Illinois for the purpose of observing water levels and has been equipped with an automatic water level recorder since 1952. A new well, SWS No. 3 was drilled in 1979 to replace the old SWS No. 3. The wells were measured concurrently for several months to detect any differences in water level measurements. The hydrograph for this well is shown in figure 6. Water levels in SWS No. 3 (MAD 3N9W-8.5g) are effected mainly by climatic conditions and pumpage at Granite City. The lowest recorded water levels occurred in the late winter of 1957 after the drought of 1952-1956 and during the peak period of pumping at Granite City. Groundwater levels recovered after 1957 as recharge from precipitation increased and pumpage at Granite City declined. The highest groundwater levels were recorded in 1973, 1974, 1975, 1983-1986, and 1993 due to above normal recharge conditions. This observation well was used for verification of the updated MODFLOW model developed in this current study. See figure 23.

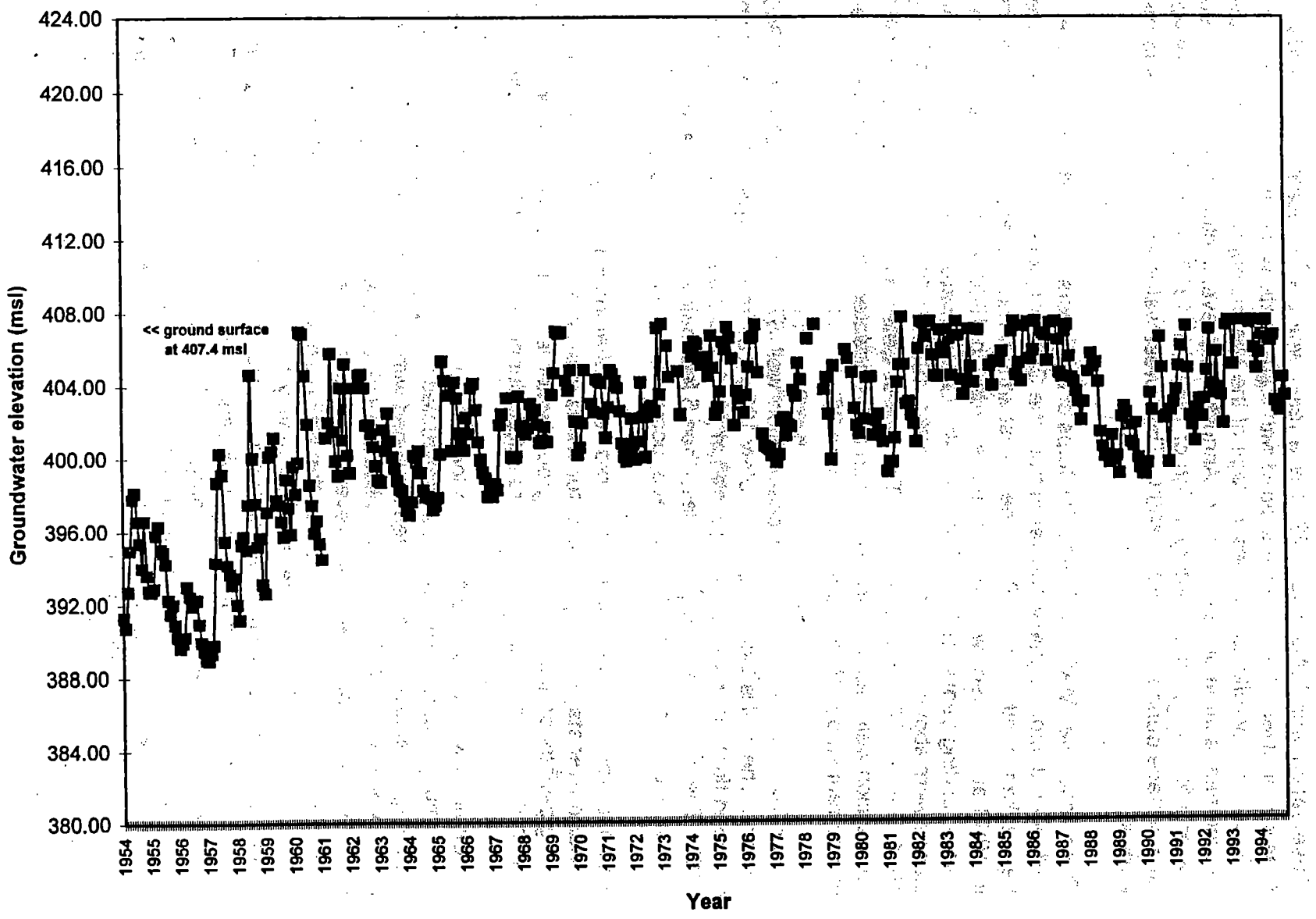


Figure 5. Hydrograph for State Water Surv ıservation well - Corps of Engineers No. 70.

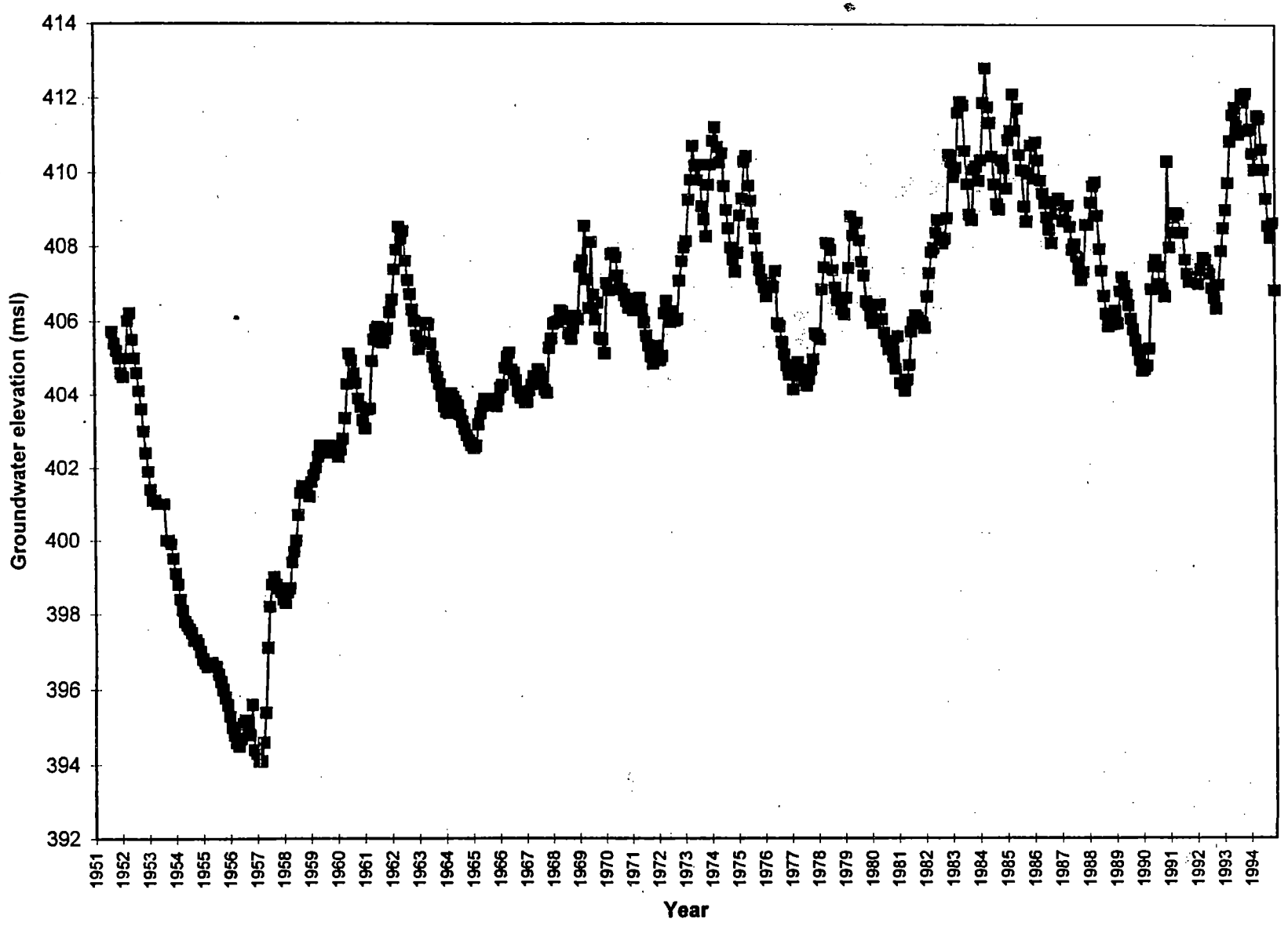


Figure 6. Hydrograph for State Water Survey observation well - No. 3.

Potentiometric Surface Maps

At least 14 potentiometric surface maps are available from reports of the State Water Survey. These maps were reviewed as part of the Survey's study of the American Bottoms Groundwater Model for the Corps of Engineers. The summary of this analysis is covered in the following paragraphs.

The fourteen maps that were available are listed below.

Date of Map	Reference
circa 1900	Schicht and Jones (1962)
November, 1951	Bruin and Smith (1953)
December, 1956	Schicht and Jones (1962)
June, 1961	Schicht and Jones (1962)
November, 1961	Schicht and Jones (1962)
June, 1962	Schicht (1964)
November, 1966	Reitz (1968)
November, 1971	Baker (1972)
June, 1973	Schicht (1974)
September, 1973	Schicht (1974)
June, 1974	Water Survey Files
September, 1974	Water Survey Files
November, 1977	Emmons (1977)
November, 1980	Richards and Sanderson

Schicht and Jones (1962) estimated the elevations of the potentiometric surface for the time period around 1900. This map was based on interpretation of the early drainage system and data in Bowman and Reeds (1907). This map, figure 7 shows that prior to settlement of the American Bottoms, the water table was very near the surface and shallow lakes, ponds, swamps and poorly drained areas where widespread throughout the area. Flood waters from the Mississippi River, Wood River, Cahokia Creek, Canteen Creek, Schoenberger Creek, and Prairie Du Pont Creek frequently inundated large areas of the lowlands. The general direction of groundwater movement was to the west and south towards the Mississippi River. Figure 7 shows a potentiometric surface that slopes from an elevation of 420 feet msl near the bluffs to about 400 feet msl near the Mississippi River. The average slope of the potentiometric surface was about 3 feet per mile with a range of 6 feet per mile in the Alton area to 1 foot per mile in the Dupo area. The slope of the potentiometric surface was greatest near the bluffs.

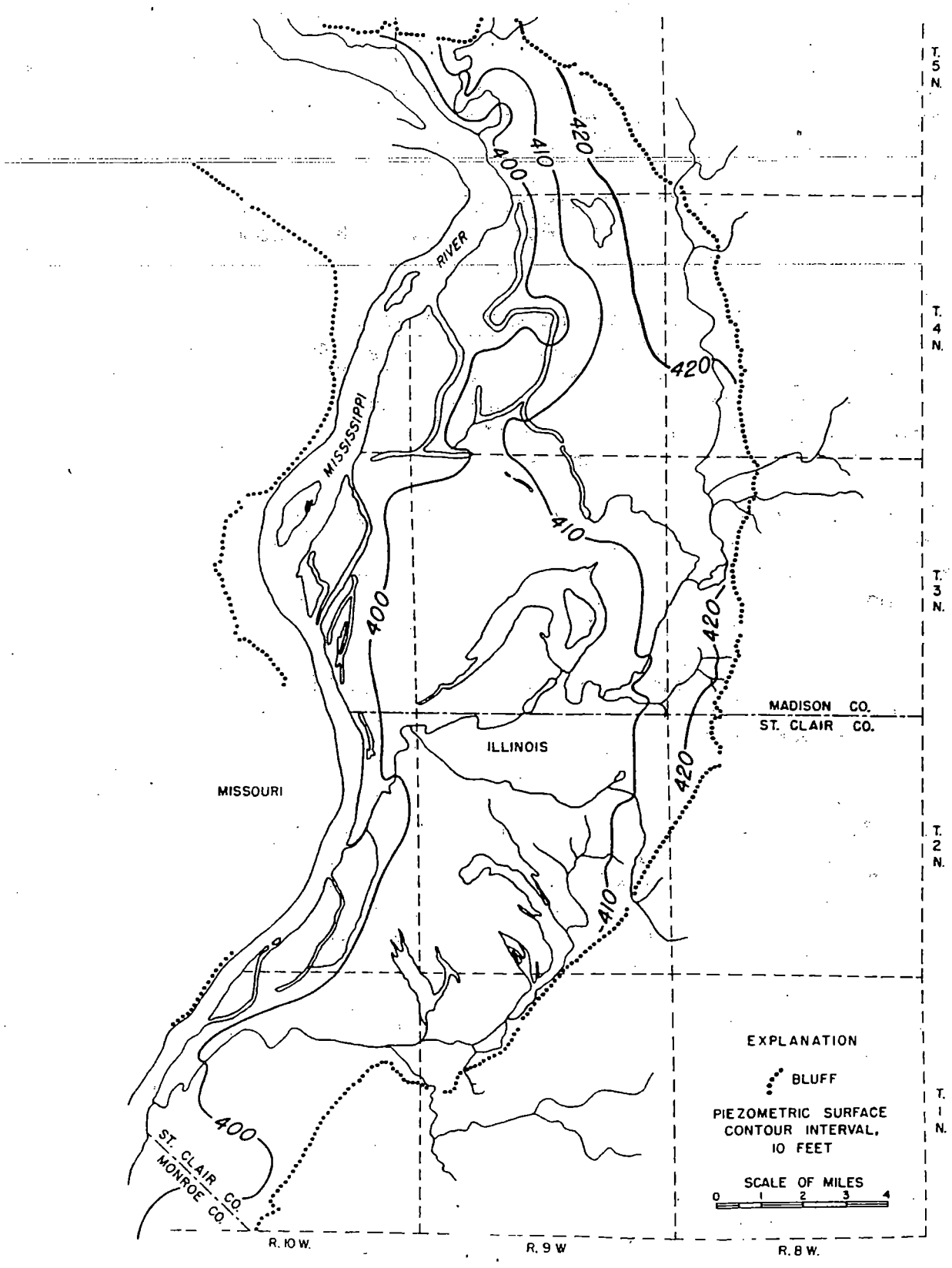


Figure 7. Drainage system and estimated elevation of potentiometric surface, circa 1900, (from Schicht, et al., 1984).

Development of the American Bottoms area led to the construction of levees and drainage ditches and the resultant change in water levels. Bruin and Smith (1953) estimated that the natural lake area was reduced by more than 40% between 1907 and 1950 and that over 40 miles of improved drainage ditches had been constructed during the same period. They further estimated that these developments caused lowering of groundwater levels by 2 to 12 feet. Also, the establishment of industrial centers and the subsequent use of large quantities of groundwater by industries and municipalities had lowered water levels in the Alton, Wood River, Granite City, National City, East St. Louis and Monsanto areas. Pumping impacts were also evident in Poag, Caseyville, Glen Carbon, Troy, and Fairmont City areas.

From 1952 to 1956 water levels declined appreciably in the American Bottoms area as the result of drought conditions, low Mississippi River stages and record high groundwater withdrawals. Figure 8 shows the potentiometric surface in December of 1956, when water levels were at record low stages at many locations. This figure clearly shows the cones of depression which developed as a result of heavy pumping. In 1956 the deepest cone of depression was in the Granite City area.

The potentiometric surface map for November 1961, see figure 9, is similar in many respects to the December 1956 map. A significant difference is that the cone of depression in the Granite City area was reduced by more than 50 feet due to reductions in pumpage. Other differences are that groundwater levels were generally lower in the vicinity of streams and lakes in 1956 than they were in 1961.

The features of the November 1961 and November 1966 maps, see figure 10, were generally the same except in the vicinity of the Monsanto area. In 1966 a reduction in pumpage caused water levels away from the river to recover.

A comparison of the November 1966 and November 1971 maps, see figure 11, indicate major changes in the Wood River, Alton, and Monsanto areas. In November 1966, one clearly defined cone of depression was located in the Wood River area, where water levels were at 385 feet. By 1971, a reduction in pumpage by one industry and an increase in pumpage by another industry caused water levels to recover in one area and decline in another. Water levels in the Wood River area declined to an elevation of 375 feet. In the Alton area, a shift in the center of pumpage moved the cone of depression a few miles to the northwest. In the Monsanto area, two cones of depression are evident in 1971 as a result of changing patterns in groundwater withdrawals.

There are significant changes between the November 1971 and the June 1973 maps, see figure 12. Groundwater levels in June 1973 were at record high stages due to the prolonged period of high river stages during the 1973 floods. From November 1971 to June 1973, groundwater levels rose 15 or more feet along the Mississippi River and Chain of Rocks Canal. Rises from 10 to 15 feet extended 1 to 3 miles from the river and canal. Much of the area experienced rises from 5 to 10 feet. In the Wood River area a major shift

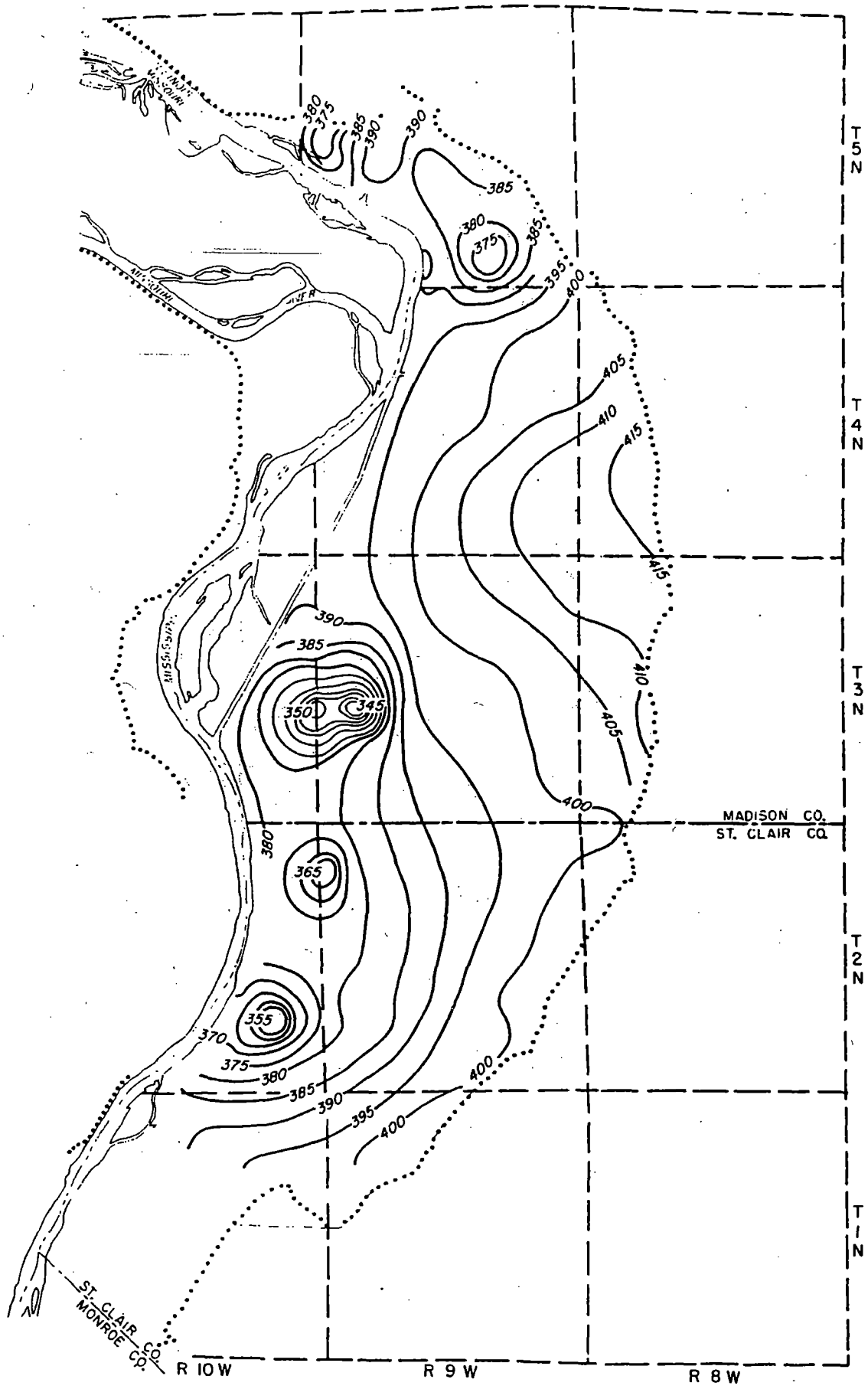


Figure 8. Approximate elevation of potentiometric surface, December 1956, (from Schicht et al., 1984).

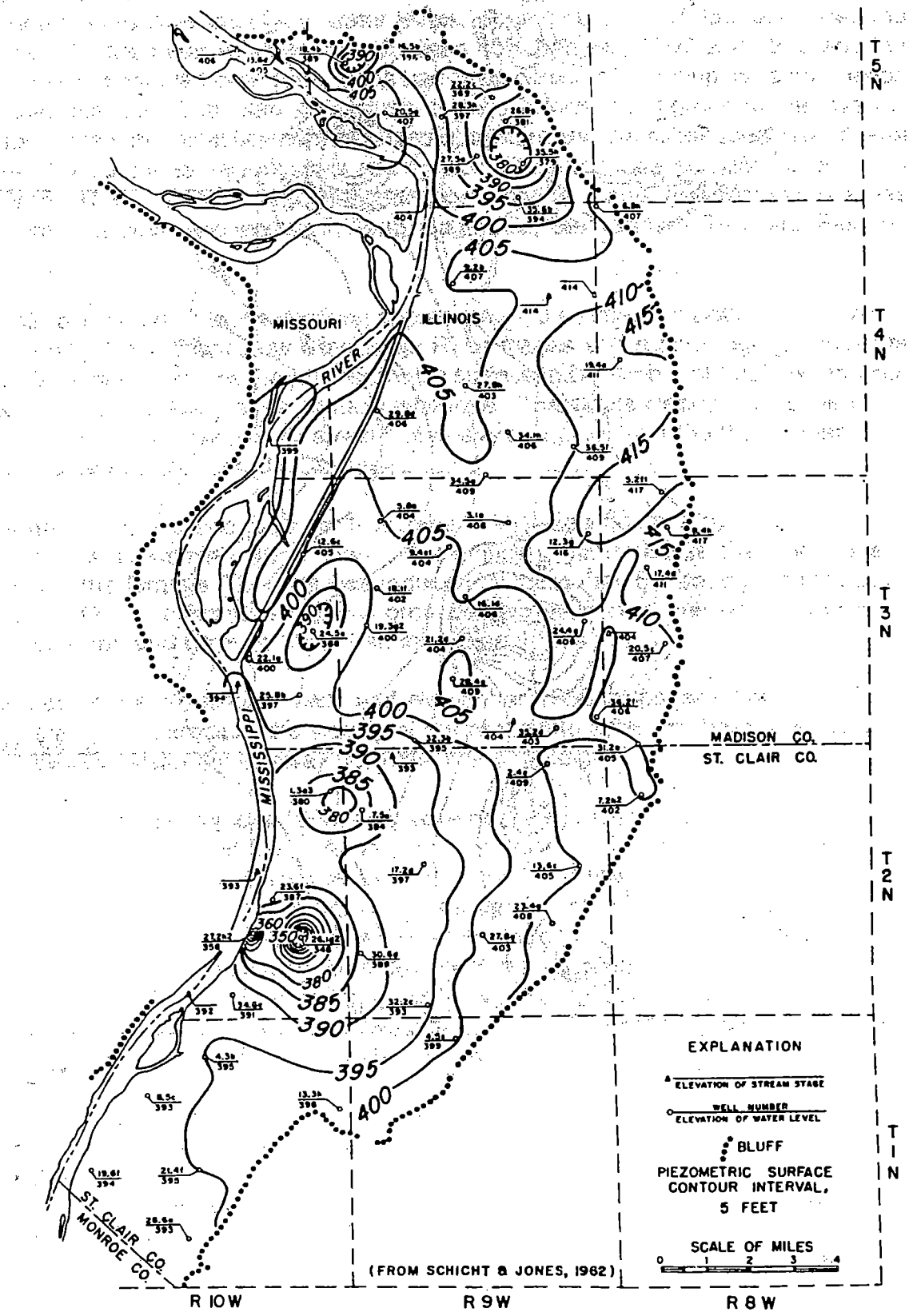


Figure 9. Approximate elevation of potentiometric surface, November 1961, (from Schicht et al., 1984).

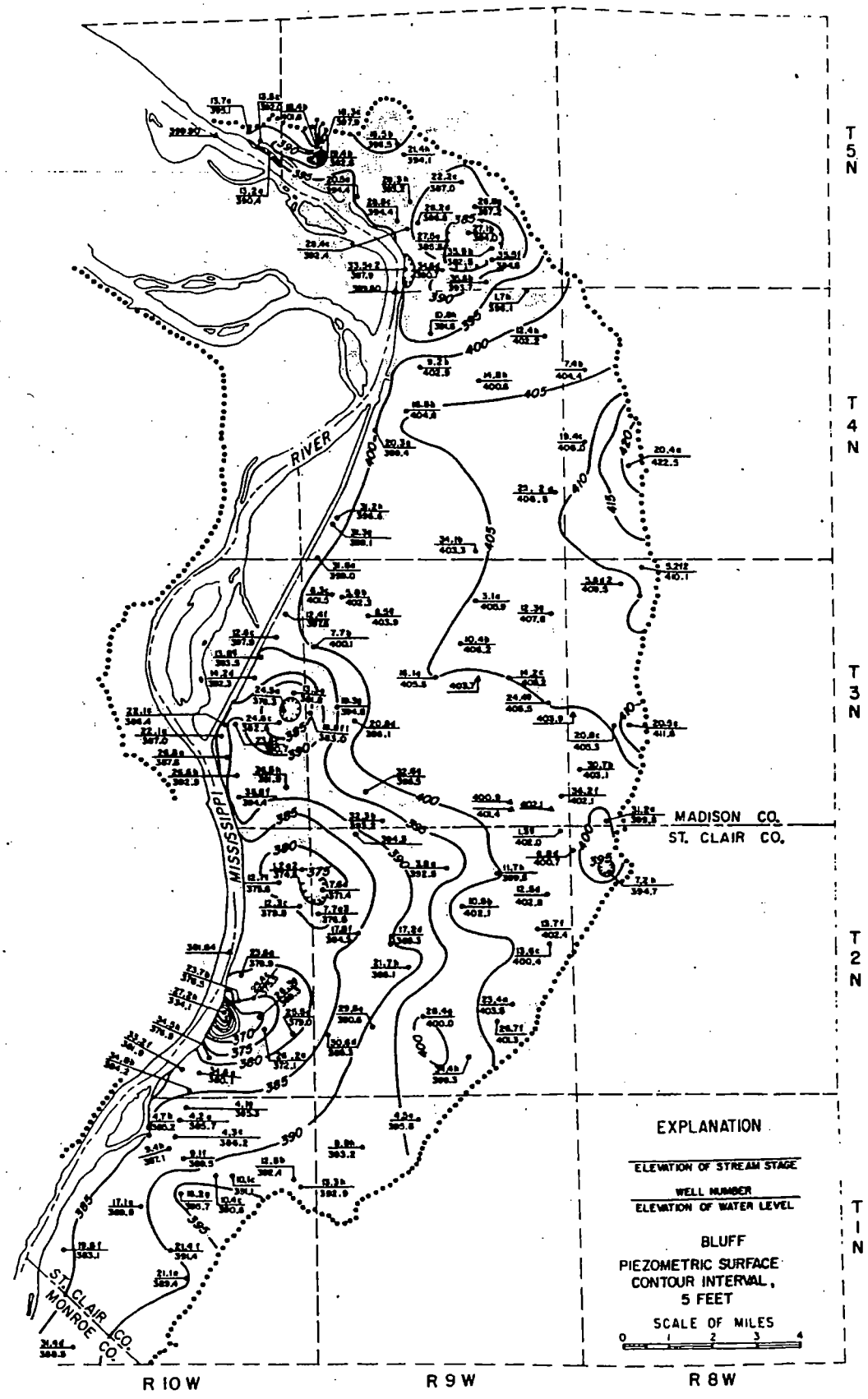


Figure 10. Approximate elevation of potentiometric surface, November 1966, (from Schicht et al., 1984).

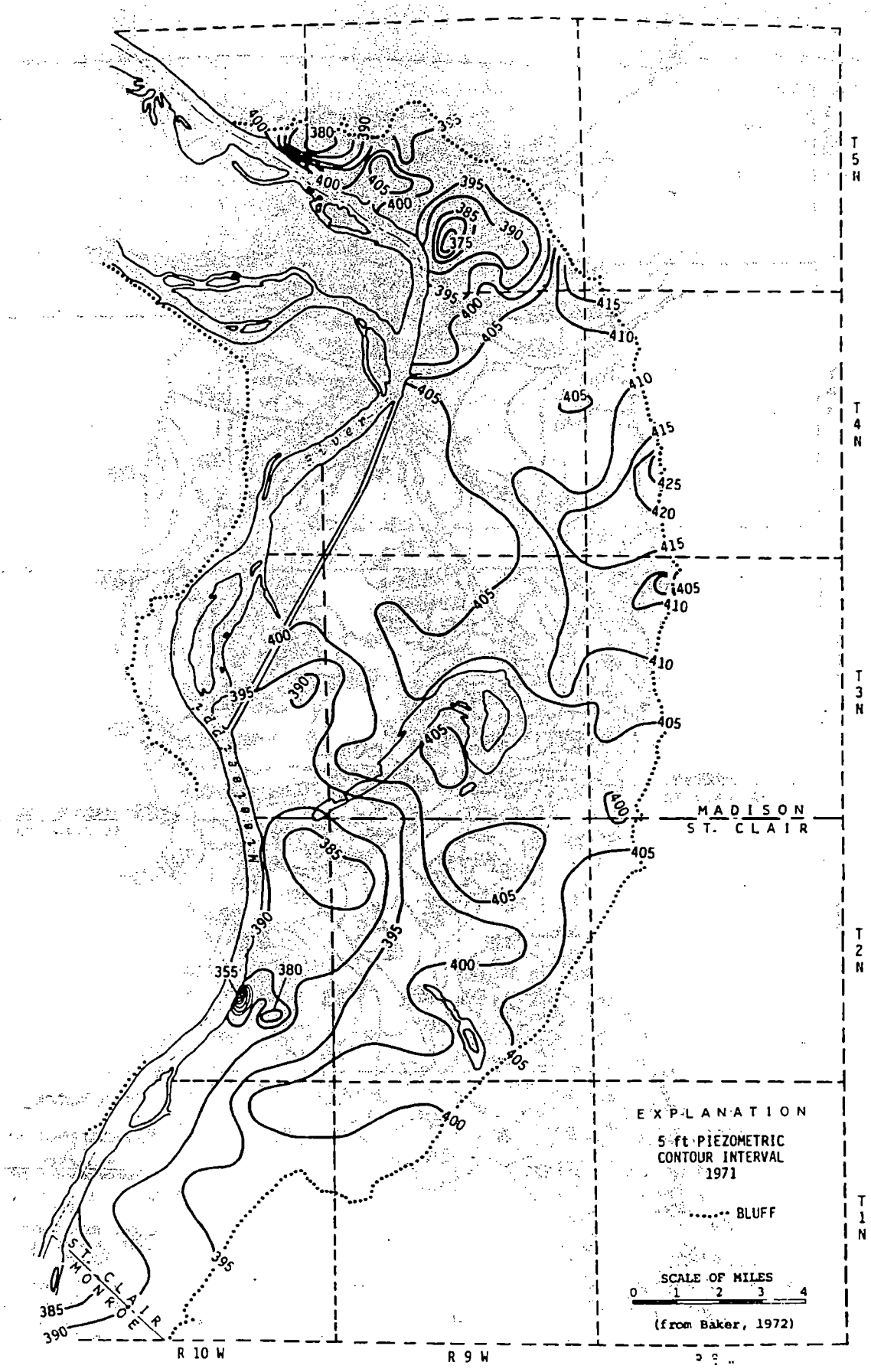


Figure 11. Approximate elevation of potentiometric surface, November 1971, (from Schicht et al., 1984).

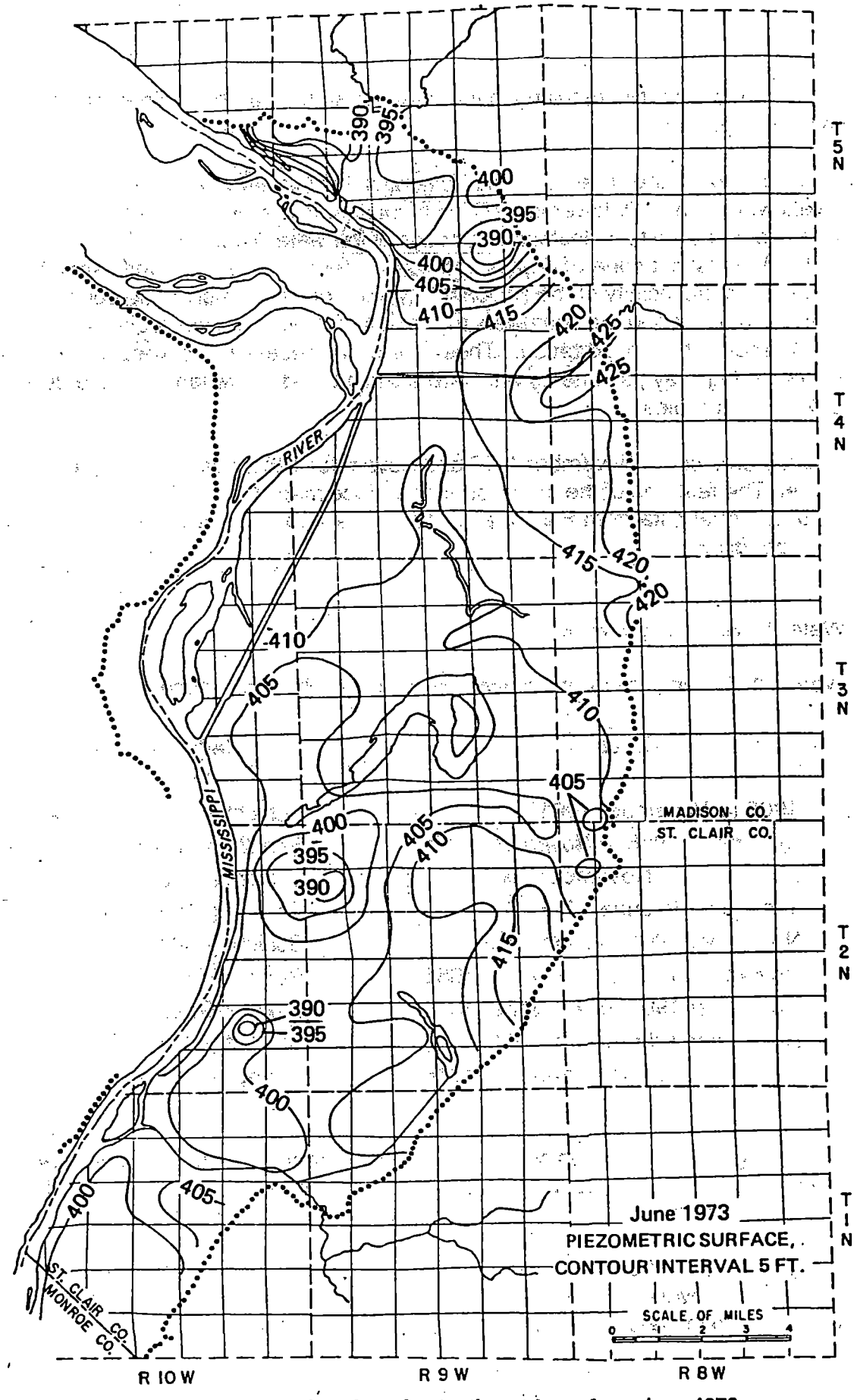


Figure 12. Approximate elevation of potentiometric surface, June 1973, (from Schicht et al., 1984).

in pumpage occurred resulting in the re-establishment of the main cone of depression 2 miles to the east.

A comparison of the June 1973 and November 1977 maps, see figure 13, shows that water levels were generally lower in 1977 as a result of recharge and river levels returning to a more normal condition. Instead of a trough of low water levels in the vicinity of Granite City in 1973, a distinct cone of depression was evident in 1977. A significant change should be noted in the vicinity of the National City cone of depression. The cone has expanded to the southeast and the southwest as a result of dewatering wells operated by the Illinois Department of Transportation. These wells are necessary to keep water levels below interstate highway pavements which were constructed below land surface during a period of low water tables.

The features of the November 1977 and November 1980, see figure 14, are generally the same. The features of the potentiometric surface maps for 1985, 1990, and 1995 (based on provisional data from Hlinka, et al, 1997) are also generally the same and are shown in Appendix F.

Water Level Change Maps

Water level change maps have been developed by the State Water Survey as listed below.

Dates	Reference
1900 to November 1961	Schicht and Jones (1962)
December 1956 to November 1961	Schicht and Jones (1962)
June 1961 to November 1961	Schicht and Jones (1962)
June 1961 to June 1962	Schicht (1964)
November 1961 to June 1962	Schicht (1964)
November 1961 to November 1966	Reitz (1968)
November 1966 to November 1971	Baker (1972)
December 1956 to June 1973	Schicht (1974)
November 1971 to November 1976	Emmons (1978)

The change maps that depicted key seasonal, annual, 5 year, and long-term water level changes were described by the State Water Survey in their report to the Corps of Engineers and their findings are summarized in the following paragraphs.

The November 1961 to June 1962 change map is typical of seasonal changes in groundwater levels. See figure 15. Water levels are frequently highest in the late spring and lowest in the late fall or early winter depending on climatic conditions, pumping and the

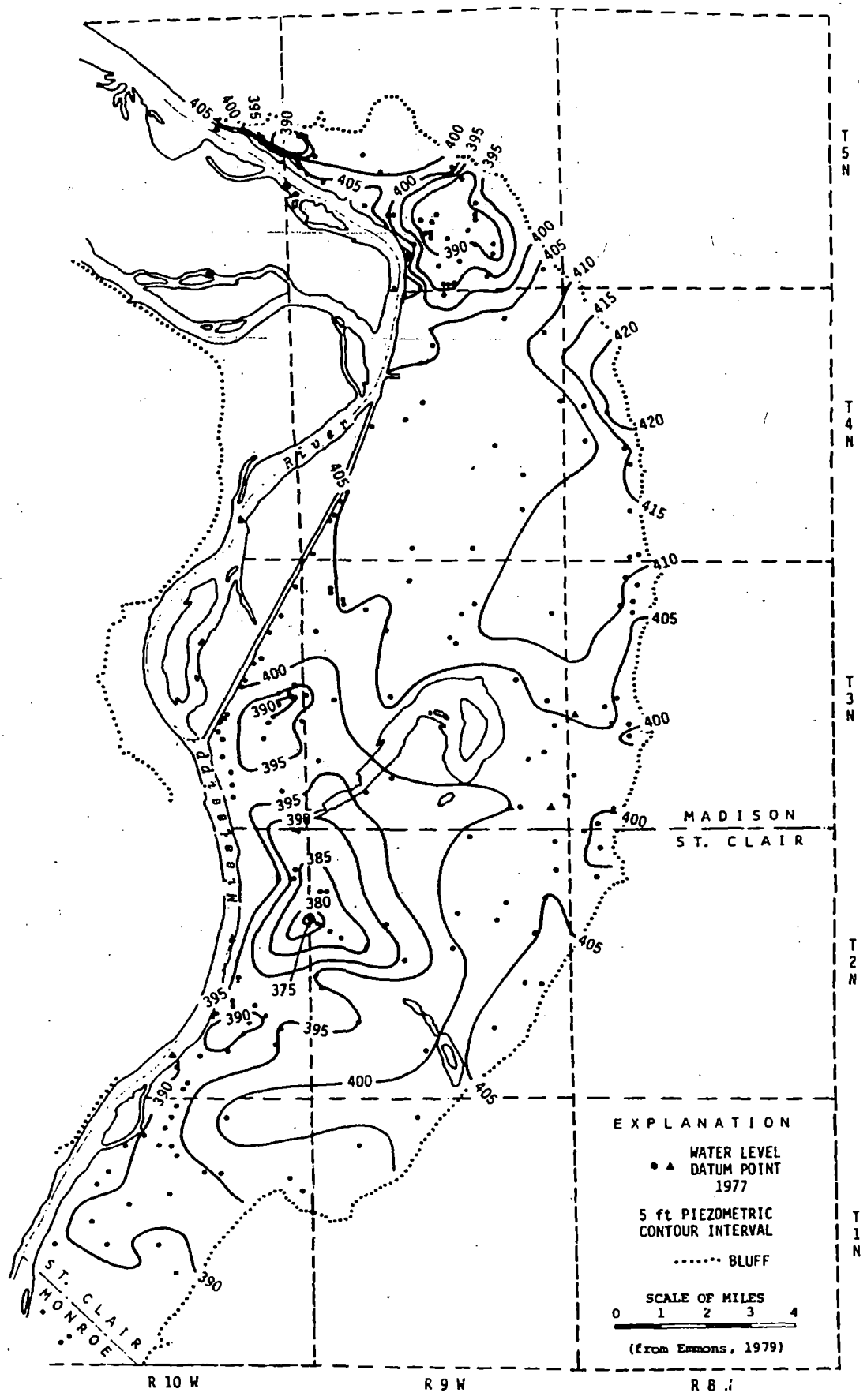


Figure 13. Approximate elevation of potentiometric surface, November 1977, (from Schicht et al., 1984).

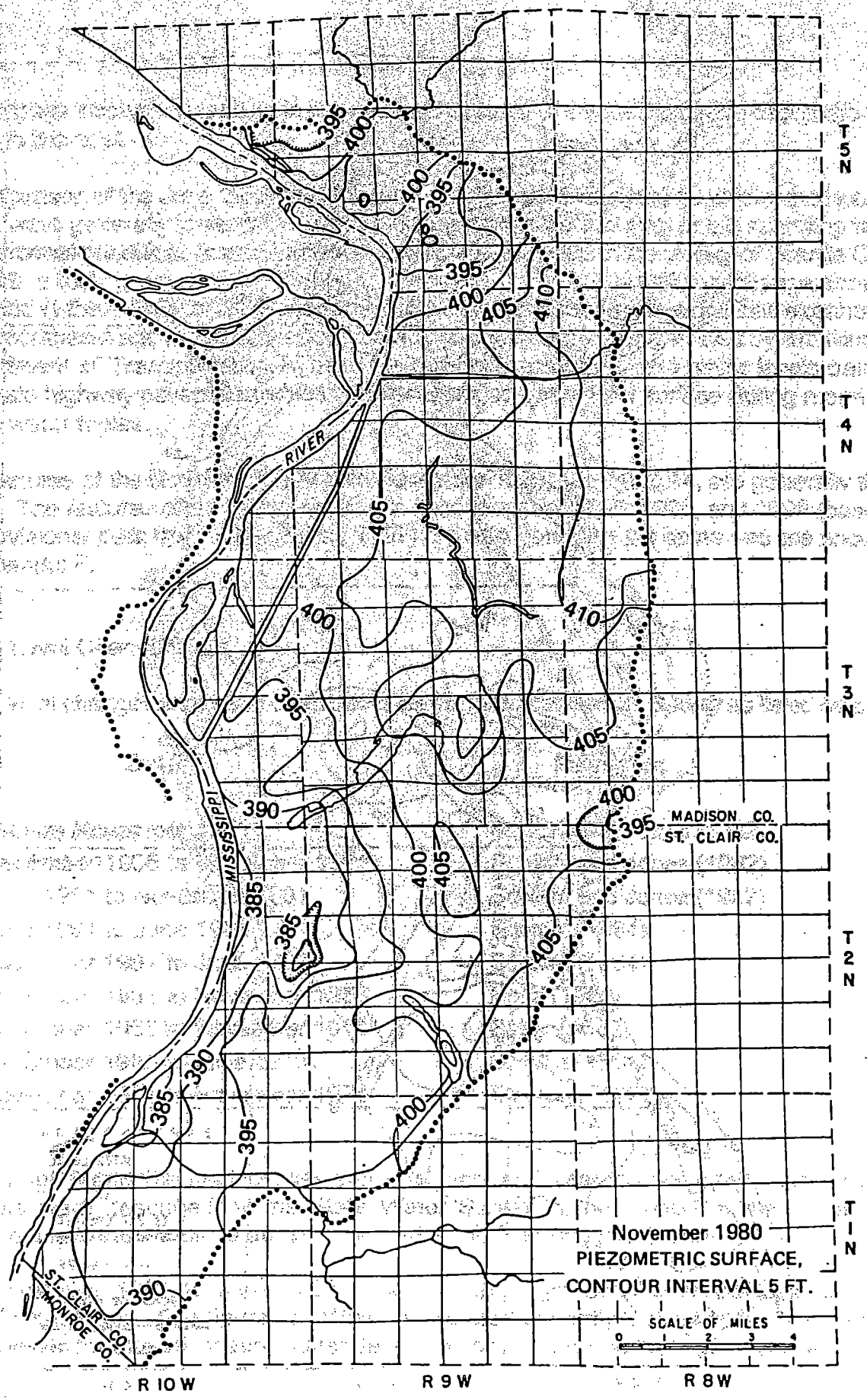


Figure 14. Approximate elevation of potentiometric surface, November 1980, (from Schicht et al., 1984).

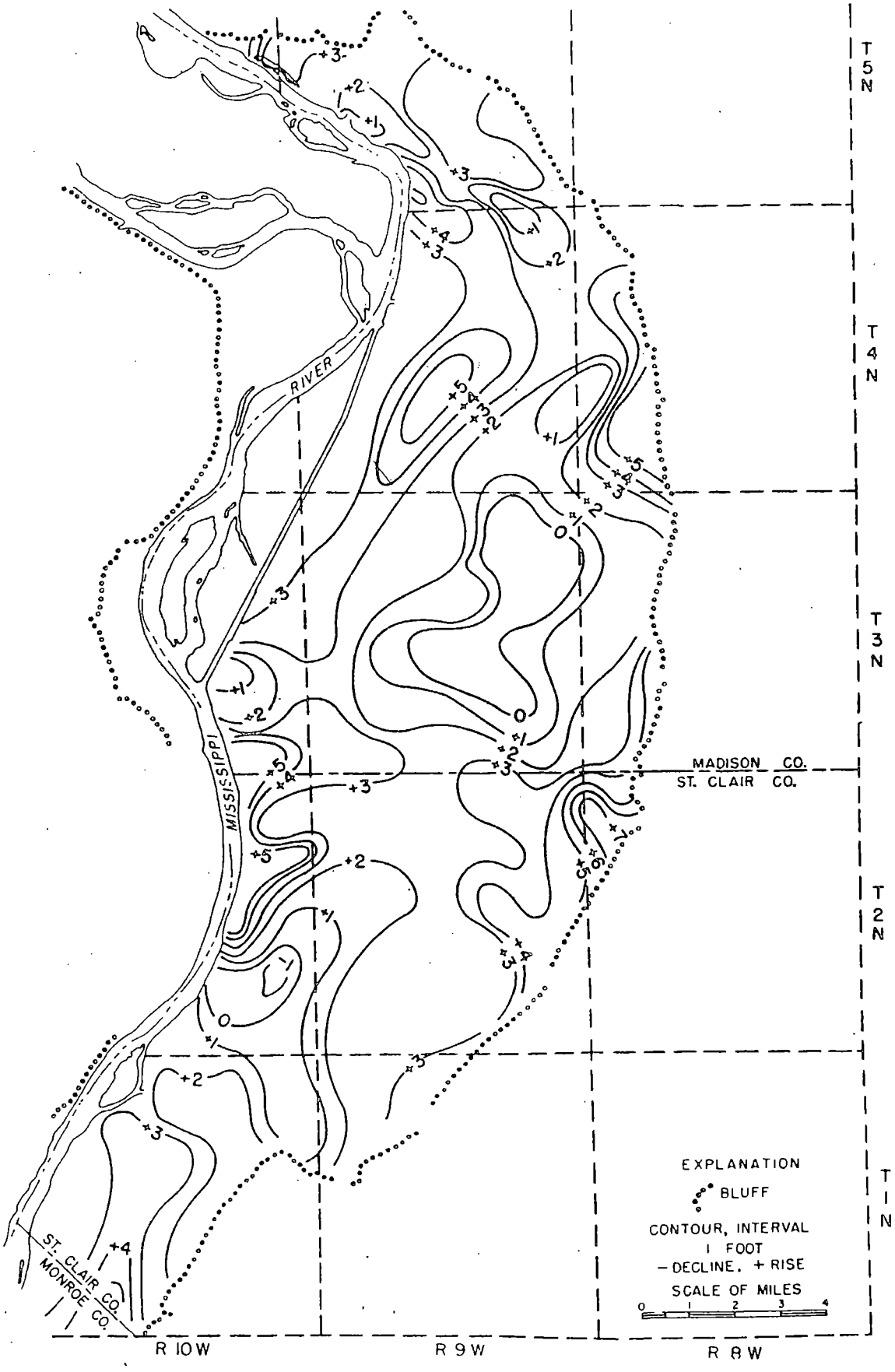


Figure 15. Estimated change in water levels, November 1961 to June 1962, (from Schicht et al., 1984).

stage of the Mississippi River. Groundwater levels rose appreciably because Mississippi River stages were higher in June 1962 than in November 1961. Groundwater levels rose appreciably along the bluffs, the rise exceeding 7 feet in places. Groundwater level rises along the Mississippi River exceeded 5 feet east of Wood River and east of National City while groundwater levels also exceeded 5 feet at the northern edge 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.

The June 1961 to June 1962 water level change map is typical of annual changes in the water table. See figure 16. Water levels rose along the river due to higher river stages while water levels declined inland near Monsanto as a result of heavy pumping. Water levels declined less that a foot in the Horseshoe Lake area and in places along the bluffs. Water levels rose in excess of 5 feet along the Mississippi River in the Alton area and Wood River area along the Chain of Rocks Canal.

The water level change map for the period 1900 to November 1961 illustrates changes in water levels over a long period mainly due to effects of groundwater withdrawals. See figure 17. The greatest declines occurred in the major pumping centers and were as follows: 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 the Chain of Rocks Canal upstream of the locks were the surface water was raised for navigation purposes. In areas remote from major pumping centers and the Mississippi River, water levels declined an average of about 5 feet. Water levels did not change appreciably in the area around Horseshoe Lake.

The potentiometric surface map for December 1956 (near record low groundwater stages) was compared with the June 1973 potentiometric surface map (record high groundwater stages). Water level changes are shown in figure 18. The greatest changes occurred in the Granite City area where groundwater levels recovered more than 55 feet due to the reduction in pumpage after 1957 and high river stages and favorable conditions for recharge during 1973. Groundwater levels recovered more than 45 feet in the vicinity of Monsanto, also due to a reduction in pumpage after 1970 and high river stages and favorable conditions for recharge during 1973.

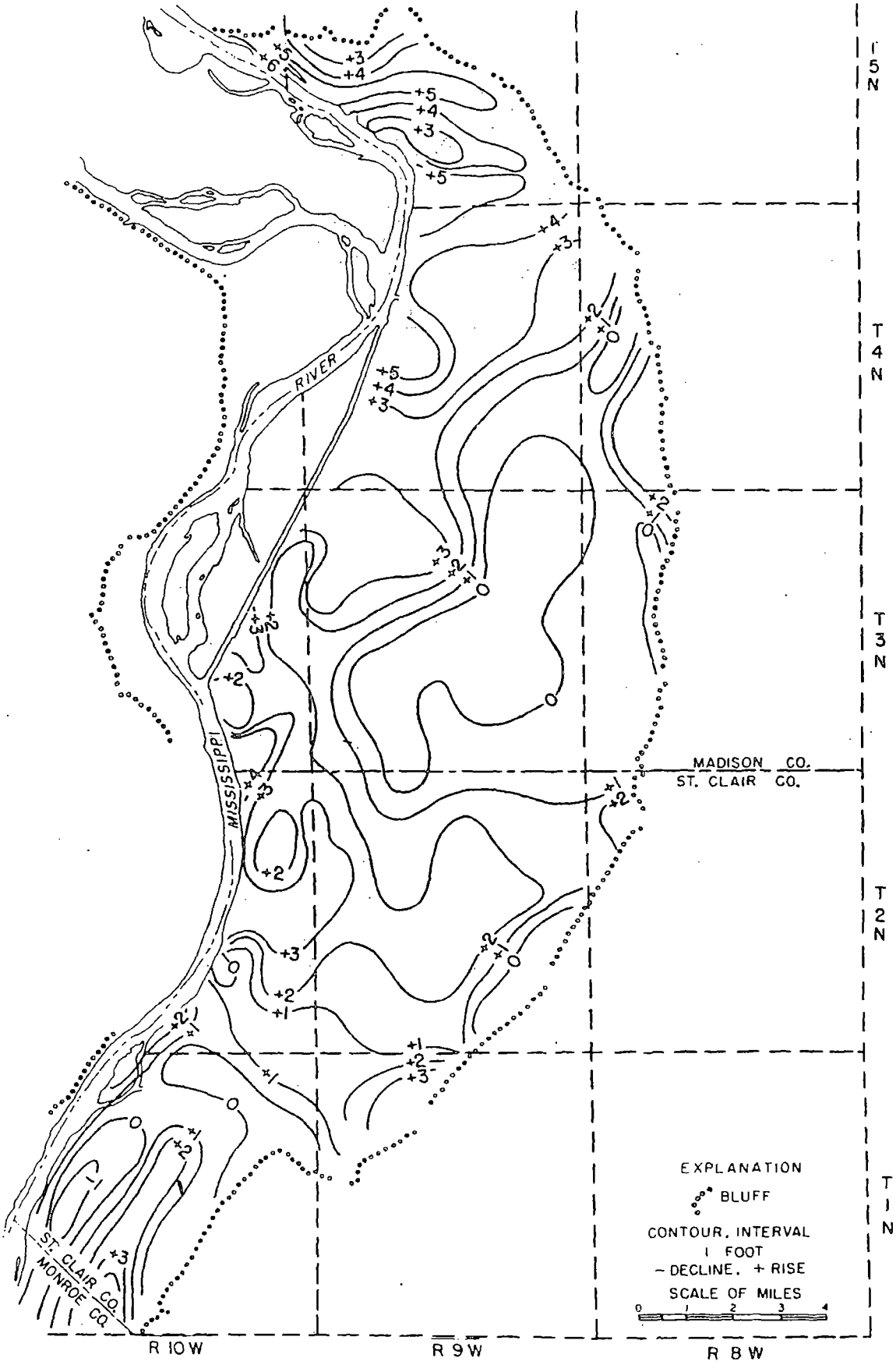


Figure 16. Estimated change in water levels, June 1961 to June 1962, (from Schicht et al., 1984).

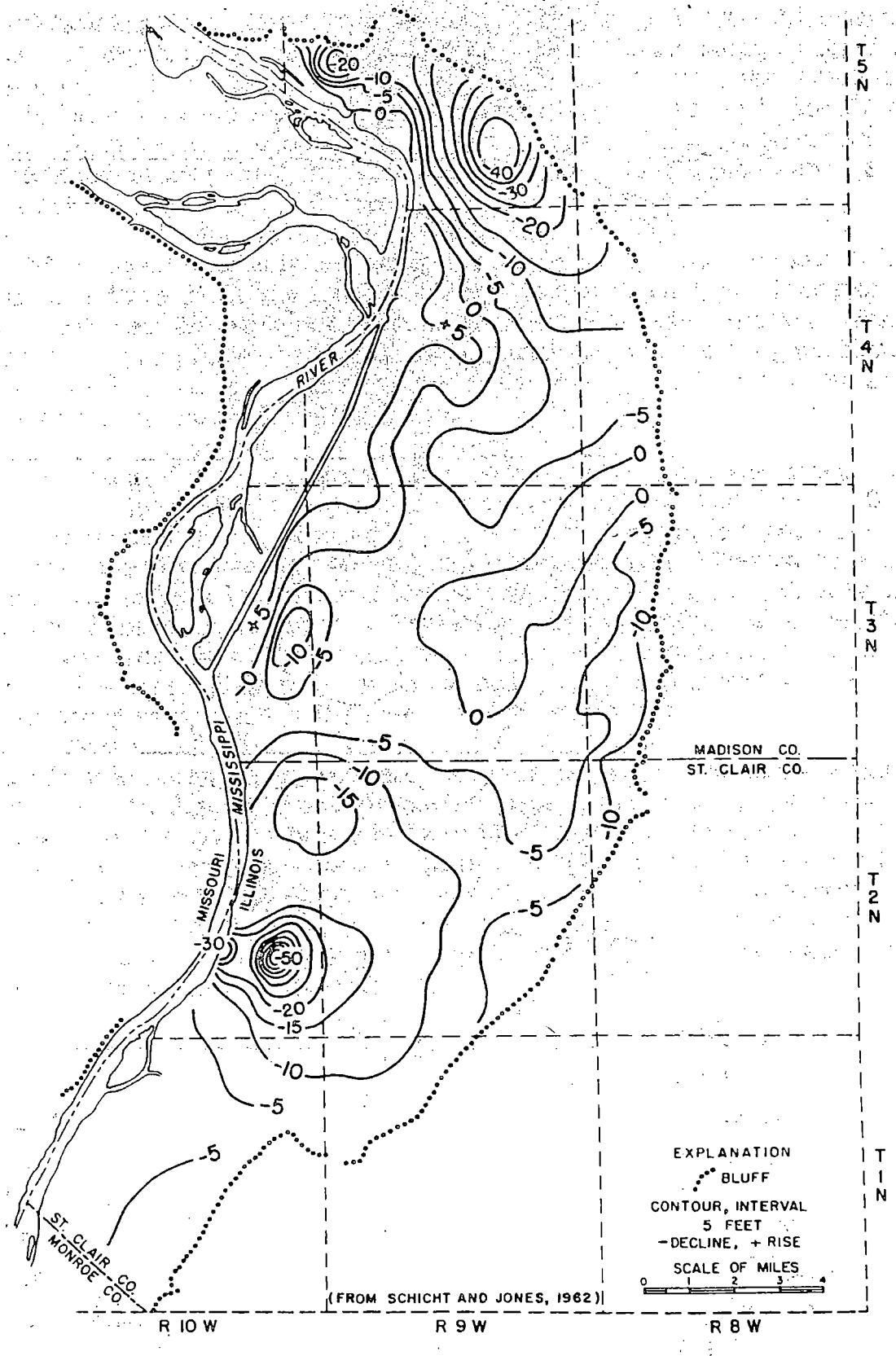


Figure 17. Estimated change in water levels, 1900 to November 1961, (from Schicht et al., 1984).

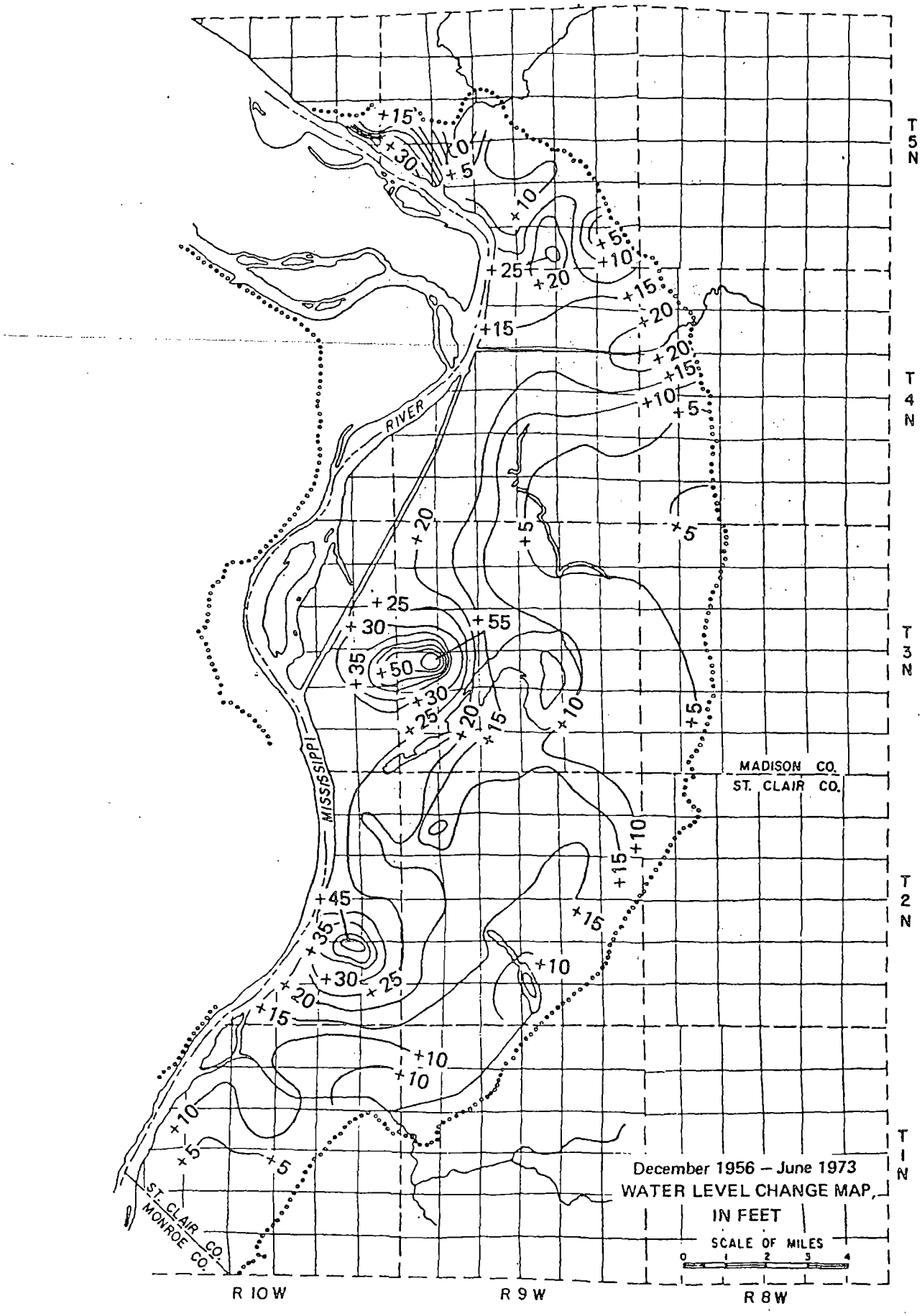


Figure 18. Estimated change in water levels, December 1956 to June 1973, (from Schicht et al., 1984).

STATE WATER SURVEY GROUNDWATER FLOW MODEL

The St. Louis District Corps of Engineers contracted with the Illinois State Water Survey (ISWS) in the early 1980's for the development of a mainframe computer based digital groundwater flow model for the American Bottoms area. The five main objectives that were included in the ISWS study were 1) to compile current hydrologic data pertaining to the area, 2) to develop a computer model that could simulate the movement of groundwater, 3) to analyze existing and future groundwater levels in the area, 4) to present and evaluate alternatives to lower or maintain groundwater levels at specified elevations in a designated area of interest and 5) to provide documentation of the model including a detailed user's guide. The ISWS prepared a separate report for each study objective.

Sources of inflow for the ISWS model included water entering the system as surface water (the Mississippi River); leakage from lakes, streams and canals; precipitation; and inflow through model boundaries (the bluff). Outflow of groundwater from the system included: leakage to lakes, streams and canals; evapotranspiration; pumpage; and outflow across model boundaries.

Model Assumptions

The assumptions that define the mathematical capabilities of the digital flow model are: the aquifer is homogenous and isotropic in the vertical direction; recharge to the water table occurs instantaneously; and vertical flow components are negligible when compared with horizontal flow components.

For the assumption that the aquifer was homogenous and isotropic in the vertical direction, the ISWS observed that wells logs show that the valley fill consists of coarse and fine grained materials. Although there was some disparity of grain sizes present, materials were not divided into separate unrelated zones of high and low conductivity. The assumption that the aquifer was homogenous and isotropic in the vertical direction allows consideration of the flow system in two directions rather than three. The ISWS noted that several past models developed for this system were done successfully in two dimensions by using this assumption.

For the assumption that recharge to the water table occurs instantaneously, the ISWS noted that the water table is generally at depths of less than 20 feet from the land surface. Records from observation wells and rain gages show that arise in the water table occurs within a few days after a precipitation event. This is a short length of time compared to the length of a time step used in the model which was one month. This assumption allowed the ISWS to assume precipitation instantaneously affected recharge and that recharge occurs during the month in which it was recorded.

Vertical flow components were considered as negligible when compared with horizontal flow components. Generally vertical flow in a water table aquifer is not significant under typical hydraulic gradients. This assumption is often referred to as the Dupuit-Forchheimer

Theory. The hydraulic gradient between the bluff and the Mississippi River is approximately .0068 ft/ft or 3.6 feet per mile. Vertical flow occurs in the vicinity of pumpage under water table conditions. It also occurs along the Mississippi River due to discharge of regional flow. The quantity of vertical flow, however is small compared to the quantity of horizontal flow.

Assumptions made by the ISWS to describe hydrologic conditions as they exist in the American Bottoms were: leakage from underlying indurated rocks is negligible; groundwater flow from west of the Mississippi and Missouri Rivers has no direct impact on water levels in the valley aquifer; the Cahokia Diversion Canal and the Prairie Du Pont Diversion Canal Floodway are hydraulically connected to the aquifer, all others are not; the Mississippi River stage affects groundwater levels near the river; Horseshoe Lake and Frank Holton Lake are hydraulically connected to the aquifer; and precipitation affects groundwater levels.

Leakage from the underlying indurated rocks was assumed to be negligible. Rocks that lie under the valley fill are predominantly carbonate rocks that are not able to transmit water because they are massive. In the southern part, these rocks may be able to transmit limited quantities of water because the rocks are fractured. There is an insufficient pressure head difference to cause significant flow between the valley fill and the indurated carbonate rocks. This assumption allows treating the bottom of the valley fill as a zero flow boundary.

Impacts to the shallow groundwater located west of the Mississippi and Missouri Rivers does not affect water levels to the east of the rivers. The rivers are lines of discharge and recharge for shallow groundwater. The rivers are a boundary to flow because they are lines of either higher of lower head to the surrounding area. Groundwater flows from high head to low head, and therefore either discharges to the river or is recharged by the river, thus not impacting water levels on the opposite side. This allows the model to be terminated at a river boundary.

The Cahokia diversion canal and the Prairie DuPont Floodway are assumed to be hydraulically connected to the aquifer and all other canals are not. Flow directions indicated on piezometric maps show that the canals are hydraulically connected to the aquifer while other smaller canals such as Harding Ditch and Cahokia Canal provide surface water drainage and do not appear to impact water levels in the aquifer.

The Mississippi River stage clearly impacts groundwater levels near the river. A comparison of water levels in a well near the Mississippi River Gage at Jefferson Barracks shows a close correlation that supports this assumption.

Horseshoe Lake and Frank Holten Lake have an impact on groundwater levels in the American Bottoms. Horseshoe Lake was determined as a source of Groundwater recharge during period of heavy pumpage in the Granite City area as was Frank Holten Lake for nearby wells.

Precipitation in the American Bottoms area closely affects water levels in wells. A comparison by the ISWS of monthly total precipitation as measured at the St. Louis Airport shows a close correlation with monthly average groundwater level fluctuations.

Development of the State Water Survey Model

The groundwater flow model developed by the Illinois State Water Survey is now commonly known as the PLASM model. This mathematical model uses a partial differential equation for nonsteady-state, two-dimensional (horizontal) flow in a nonhomogeneous and isotropic aquifer. This model is described in ISWS Bulletin 55, Pricket and Lonnquist (1971).

In the development of the PLASM model, the State Water Survey made a number of modifications to the basic model described in Bulletin 55 which included changes to the calculation of recharge at river nodes, calculation of recharge and evapotranspiration at land nodes, utilization of annual pumpage data and manipulation of boundary conditions. These changes were made since earlier versions of PLASM treated a number of these input variables as constants throughout a simulation. Since the development of this input data is important in understanding this model and how the data was regenerated for the updated MODFLOW model, these modifications are described in detail in the following paragraphs.

The calculation of the river stage at each cell was performed by in a subroutine which used as input data, the river mile mark of each cell and the monthly average Mississippi River stages at the St. Louis gage (mile mark 179.6). The subroutine used this input along with a slope profile matrix to relate by interpolation, the stage at the St. Louis gage to other mile marks and cell locations. See table 1. A further modification to river cell recharge was performed in an additional subroutine which accounted for the variability to the hydraulic conductivity of the riverbed based on temperature. The subroutine calculated a temperature adjusted recharge factor based on a ratio of monthly average river temperature and the average annual river temperature (58.33° F).

Mississippi River Slope Profiles, Mile 164 to 206
(St. Louis Gage at mile **179.6**, first line in table is River Miles)

206.0	**202.70**	**194.00**	**190.40**	**190.30**	**184.30**	**179.60**	**176.80**	**172.60**	**169.30**	**164.0**
443.9	443.40	441.80	439.20	439.10	434.20	431.90	428.80	425.80	423.80	420.6
438.9	438.60	437.50	434.60	434.50	431.30	428.40	426.40	423.80	422.30	419.2
435.2	434.84	433.67	431.10	431.00	427.74	424.94	422.92	420.06	418.47	415.3
429.7	429.47	428.20	425.90	425.70	422.65	419.94	417.96	414.72	412.99	409.7
424.9	424.38	422.92	420.20	420.00	417.15	414.94	412.90	410.40	408.44	405.5
421.4	420.59	418.32	415.30	415.00	412.19	409.91	408.72	406.55	405.10	402.0
418.0	415.52	413.90	410.00	409.60	406.88	404.94	403.64	402.00	400.14	397.1
418.0	411.04	409.40	405.45	404.87	401.59	399.94	399.07	397.22	396.18	393.0

418.0	407.00	406.00	402.00	400.63	396.34	394.94	394.15	392.21	390.81	387.9
418.0	403.90	403.13	399.10	396.52	391.33	389.94	389.17	387.42	386.12	383.2
418.0	400.90	400.30	398.50	393.54	386.55	384.94	384.31	382.60	381.50	378.5
418.0	399.80	399.30	397.50	390.20	381.91	379.94	379.22	377.80	376.57	373.5
418.0	398.70	398.20	397.30	388.85	380.28	377.94	377.22	375.90	374.70	372.0
418.0	398.50	398.00	397.20	388.05	378.78	376.44	375.74	374.53	373.09	370.2
418.0	398.20	397.70	397.00	387.25	377.29	374.94	374.27	373.16	371.49	368.6

Table 1. Revised Slope Profile Matrix

A subroutine was also developed to aid in the calculation of recharge and evapotranspiration at land nodes. The recharge factor is modified by the ratio of recorded monthly total precipitation at the St. Louis Airport, the thirty year average monthly total precipitation and by a monthly multiplier. The monthly multiplier converted the value of recharge which is input as an average value for the entire year to a monthly value. The sum of all the multipliers is 12. The evapotranspiration factor which was input as an average value for an entire year was also modified by a factor to convert it to a monthly value. See table 2.

Model Variable	Months Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
AMRT	35	35	42	55	65	75	81	82	76	64	51	39
AMPT	2.2	2.3	3.3	3.7	4.1	4.0	3.3	2.9	3.2	2.6	2.6	2.2
WR	0.7	1.0	3.0	2.5	2.0	1.0	0.5	0.3	0.1	0.2	0.3	0.4
ETR	.24	.34	.68	1.1	1.6	1.8	2.0	1.6	1.2	.79	0.4	.24

AMRT = average monthly river temperature
AMPT = average monthly precipitation
WR = monthly recharge factor, (sum = 12)
ETR = monthly evapotranspiration factor, (sum = 12)

Table 2. Model Variable Values for Recharge and River Stage.

The State Water Survey also manipulated the boundary conditions to accommodate some of the extreme climate conditions that occurred during the periods simulated. Conditions that were automatically altered in the model include the water level in Horseshoe Lake, the water levels along the bluff and storage coefficients in areas near the bluff.

A complete set of the input data used by the State Water Survey for aquifer properties and boundary conditions is listed in Appendix A and B.

MODFLOW GROUNDWATER MODEL

The first objective of this current modeling effort was to translate all input data developed for the State Water Survey PLASM model into a form useful for input into the MODFLOW groundwater model and then develop a working transient model comparable to the State Water Survey model. The second objective of the modeling effort was to apply the MODFLOW model to the groundwater management flood control project developed for the Corps of Engineers by the State Water Survey.

The groundwater modeling effort in this study utilized MODFLOW7[386], a software package supplied by Geraghty & Miller, Inc. This computer program is an enhanced version of the MODFLOW finite difference, modular groundwater model developed by McDonald and Harbaugh (1988) for the U.S. Geological Survey. MODFLOW is perhaps the most popular groundwater flow model currently used by government agencies and consulting firms. MODFLOW analyzes groundwater flow by solving a partial differential equation which describes the three dimensional movement of groundwater of constant density through porous material. This equation in combination with the specification of boundary and initial conditions represents a complete mathematical expression of a groundwater flow system. MODFLOW uses the finite difference numerical method to obtain an approximate solution to this equation. Hydrogeologic layers can be simulated as confined, unconfined, or a combination of both. External stresses such as wells, recharge, evapotranspiration, drains and streams can also be simulated. Boundary conditions include specified head, specified flux, and head dependent flux. Three iterative solution techniques, the Strongly Implicit Procedure, Slice Successive Over Relaxation, and Preconditioned Conjugate Gradient are available within MODFLOW7[386] to solve the finite difference equations.

Model Grid and Boundaries

Finite difference modeling is based upon solving mathematical equations at nodes on a pre-defined grid. The grid used for this study is shown as figure 19. This grid covers an area that is 14.5 miles wide and 29.5 miles long, divided into 59 rows and 28 columns of equally spaced ½ mile by ½ half mile grids. The modeled area covers 180.5 square miles and includes 722 active grid cells.

The general flow direction in the modeled area is from east to west when not influenced by nearby streams, rivers and drainage ditches. The modeled area is bounded on the east by general head boundary cells representing input to the model at the toe of the bluffs. At the west edge the model is bounded by river cells representing the Mississippi River. River cells representing the Cahokia Diversion Channel also bound the model at the north edge of active cells. Constant flux cells which represent outflow from the model are located at the southern most edge of active cells in the grid. The model includes 64 total rivers cells, 18 cells representing lakes within the model grid, 49 general head cells and 7 constant flux cells.

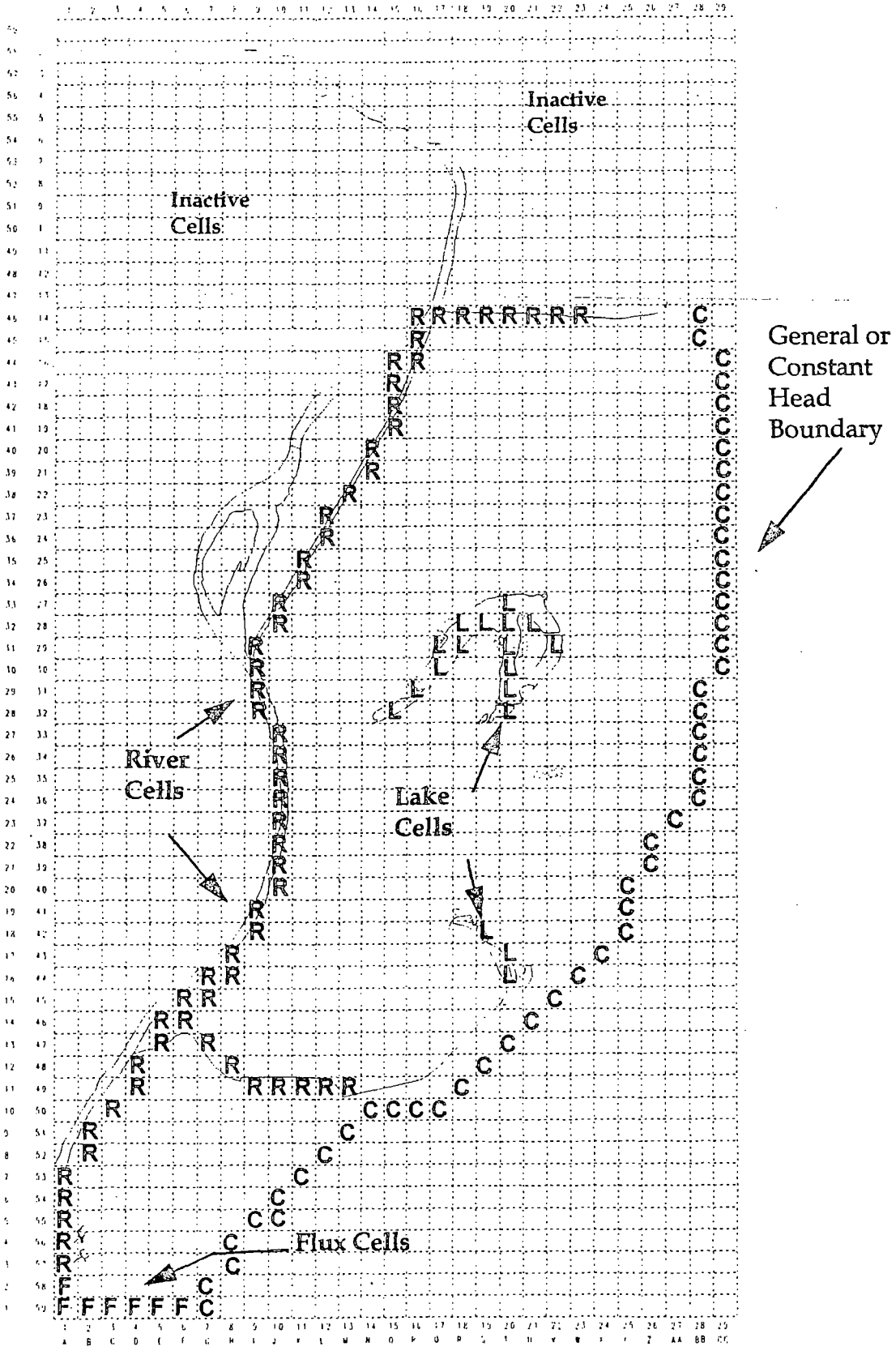


Figure 19. Model grid and Boundaries.

Model Development

The basic MODFLOW model requires input data for each active grid cell in the model grid for the following principle aquifer properties:

- hydraulic conductivity
- storage coefficient
- aquifer bottom elevation
- aquifer top elevation
- recharge
- evapotranspiration

A completed working model requires further input for the following items:

- boundary condition factors for river and general head cells where appropriate
- well pumpage information

A complete set of the input data used by the State Water Survey for the development of the PLASM model is listed in Appendix A and B. A complete set of translated input data for the MODFLOW model is listed in Appendix C and D.

Description of Model Input Data

Hydraulic Conductivity - Translated hydraulic conductivity values varied from 504 ft/day down to 1.3 ft/day. This range of values was loaded into 50 zones in the MODELCAD preprocessing program using formatted data files.

Storage Coefficient - Two values for storage coefficients were input into the model. For cells representing an active part of the aquifer, values were set at 0.20 which correspond to the value recommended by Schicht, 1965. For the general head boundary cells, values were set at 1.9E+12.

Aquifer bottom elevation - Elevations for the bottom of all grid cells ranged from 280 to 400 msl. These data were input into MODELCAD using eight zones.

Aquifer top elevation - Elevations for the top of all grid cells ranged from 369 to 600 msl. These data were input into MODELCAD using 50 zones.

Recharge - The subroutine developed by the State Water Survey to calculate recharge was translated into a spreadsheet format along with input data for monthly precipitation values from the St. Louis Airport gage. Precipitation data covered the period of 1905 through 1994. This spreadsheet generated monthly recharge rate modifiers which were used as input to the MODELCAD generated recharge files. The MODELCAD cell specific recharge file was generated from regional recharge rate values modified from the State Water Survey input data. Ten recharge zones were identified from the original input data. The rate used for the majority of the modeled area was an annual recharge rate of 7.8 inches per year. Along the bluffs recharge rates were generally set at 5.2 inches per year.

Evapotranspiration - The State Water Survey input two regional values for average annual evapotranspiration. These values equated to 4.2 and 5.6 inches per year. These values were then adjusted by the monthly multipliers developed in a subroutine by the Survey to generate monthly average values for evapotranspiration.

River Cell Boundary Conditions - The subroutine developed by the State Water Survey to select monthly river stages from gage data as well as modify bed conductance values based on monthly average river temperatures was re-developed into a computer program written in the C++ program language. This program was designed to generate river module formatted input files which could be used directly by MODFLOW. Horseshoe Lake and the two lakes at Frank Holton State Park as well as cells representing the Cahokia Diversion Canal and Blue Waters/Prairie Dupont floodway system were also selected as river cells to represent these interior boundaries.

Constant Flux Boundary - Seven cells at the southern most boundary of the model were specified by the State Water Survey as constant flux cells. The flux cells were specified in MODFLOW as a series of discharge wells with input values ranging from 1872 to 6283 cubic feet per day of outflow.

General Head Boundary - Cells located along the bluffs at the eastern edge of the model were specified as a general head boundary. In MODFLOW general head boundary conductance values are usually set at a large value to simulate low resistance e.g. 1.0E+9, although as noted by Anderson and Woessner, 1992, large water balance errors can occur and appropriate adjustments must be made. Numerous steady state and transient model runs were made until a conductance value of 1.0E+7 was selected as generating the most acceptable error values in the model's water balance between inflow and outflow. See section on constant head model runs.

Well Pumpage - The State Water Survey did a detailed study of well pumpage from the aquifer system for the time period of 1940 to 1981. These values were input into the model for the transient simulations along with forecasted pumpage data generated by the State Water Survey for the time period up to the year 2000. Pumpage from 1981 through all subsequent years was set by the State Water Survey and the Corps as equal to the year 2000 pumpage which represents a somewhat reduced level of pumping at a regional scale.

Model Output

Steady State Model Simulation

A steady state model was developed which used input values for average recharge and river stages. Output from this model which depicts contours of the potentiometric surface is shown in figure 20. There was no pumpage from the system for this model.

The results of the volumetric budget for the MODFLOW run of the steady state model showed that 61.88 million gallons per day (mgd) was received as recharge into the modeled area while 51.3 mgd left the model through river leakage. River leakage below the Chain of Rocks lock was 24.8 mgd and above the Chain of Rocks Lock, river leakage from the system was 10.1 mgd. Leakage into Harding Ditch in the southern area of the model was 3.1 mgd and 3.9 mgd of leakage left the system at the northern boundary which is the Cahokia Diversion Canal. Leakage from the system into Horseshoe Lake amounted to 9.3 mgd. There was an interaction of 11.2 mgd along the constant head cells representing the eastern boundary of the model at the toe of the buff line. This interaction indicates that flow can enter or leave the modeled area at the bluff line depending on the configuration of the potentiometric surface and resultant gradients.

Transient Simulations

Model runs for the transient simulations covered a period of 90 years (1905-1994). The State Water Survey in the development of their PLASM model used the low water period of December 1956 and the high water period of June 1973 for model calibration and verification. Output from the revised MODFLOW model representing contours of the potentiometric surface for these same stress period are shown in figure 21 and figure 22.

Hydrographs showing simulated and observed water levels for three State Water Survey observation wells located in the Granite City area are depicted in figures 23, 24, and 25. The differences between observed and simulated water levels agree favorably with the analysis of similar output documented in the State Water Survey model study.

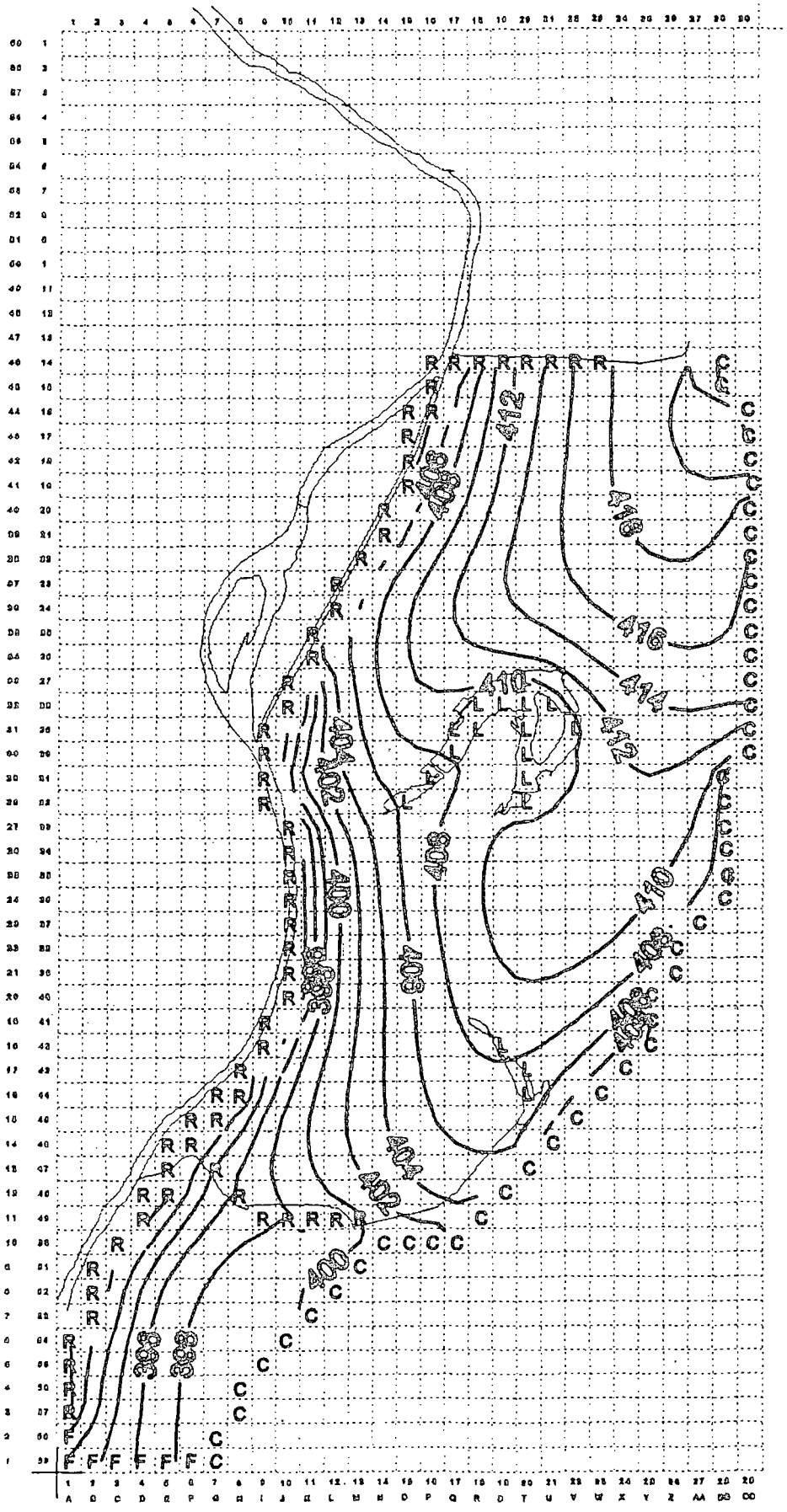


Figure 20. Steady State Model Output.

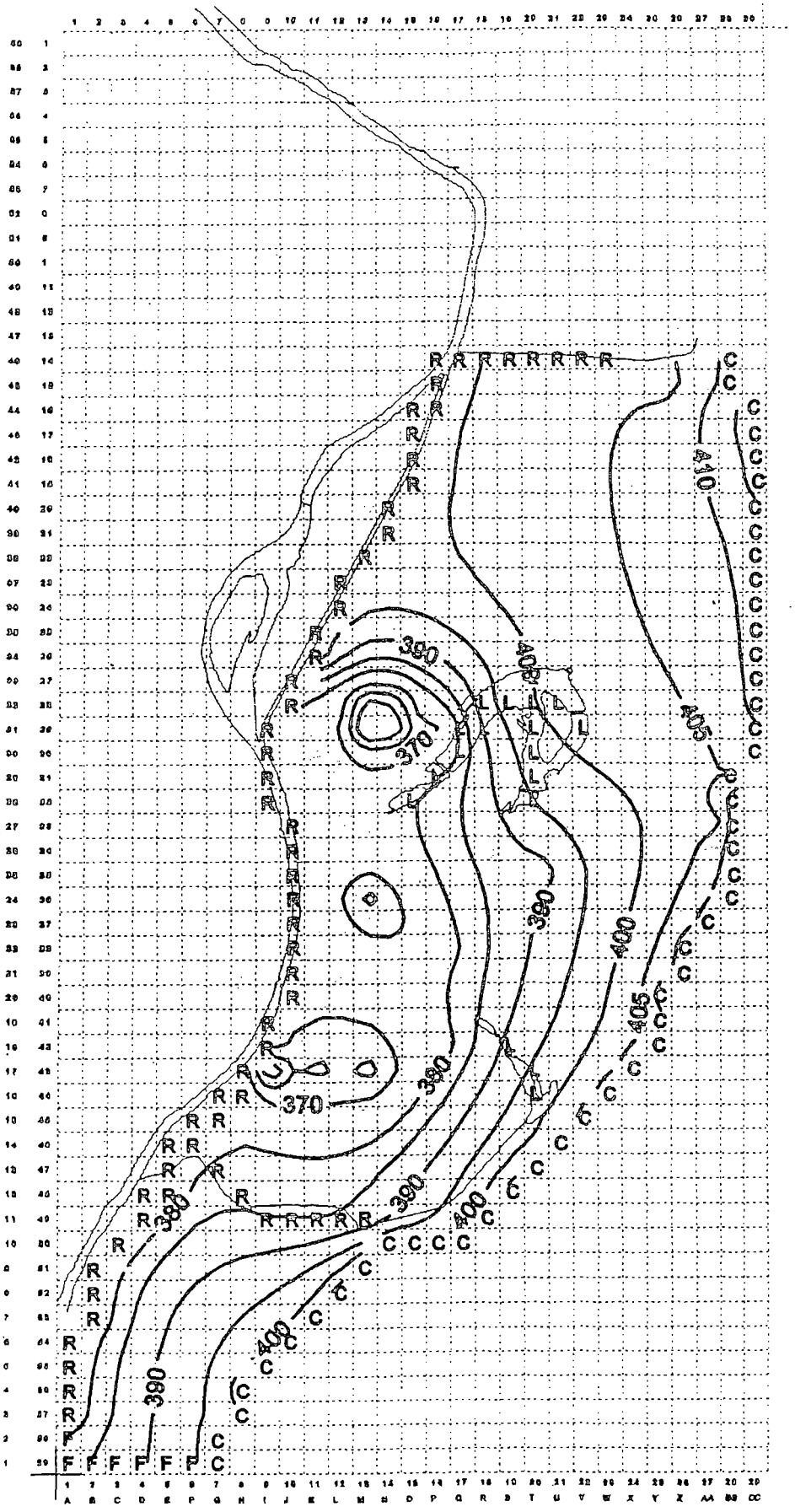


Figure 21. Transient Simulation Model Output for December, 1956.

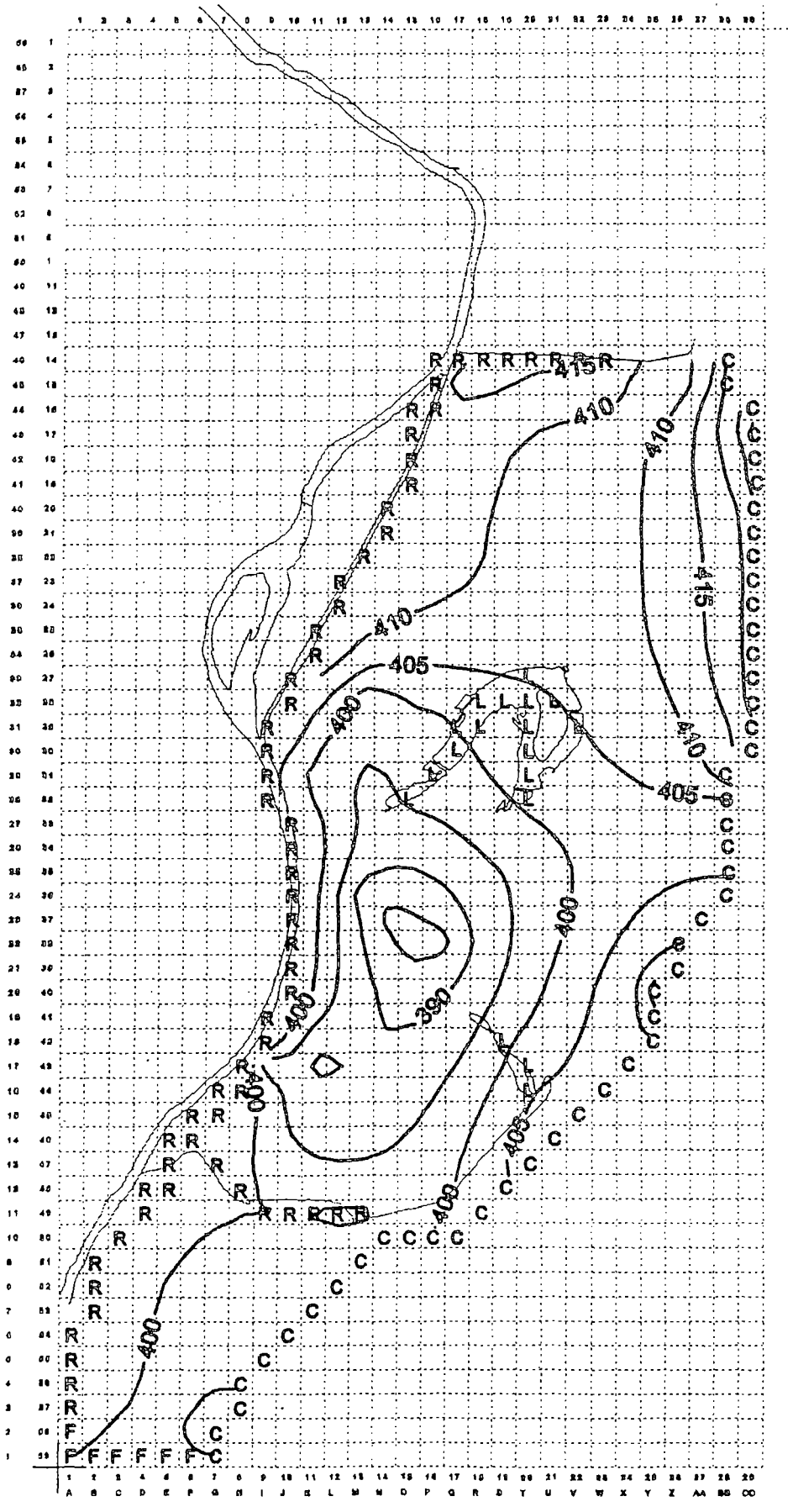


Figure 22. Transient Simulation Model Output for June, 1973.

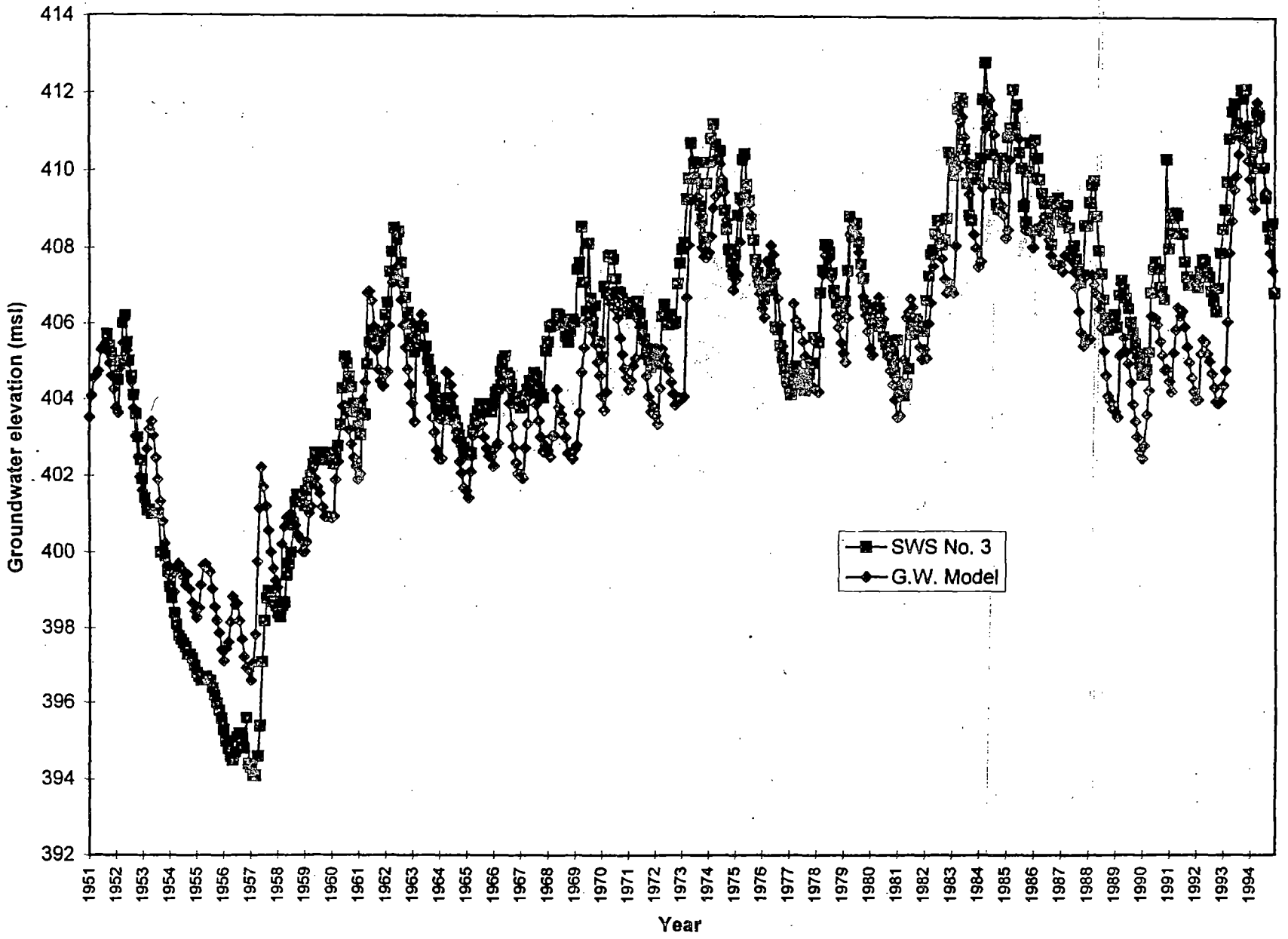


Figure 23. Hydrograph for State Water Survey ervation well No. 3 and model simulation.

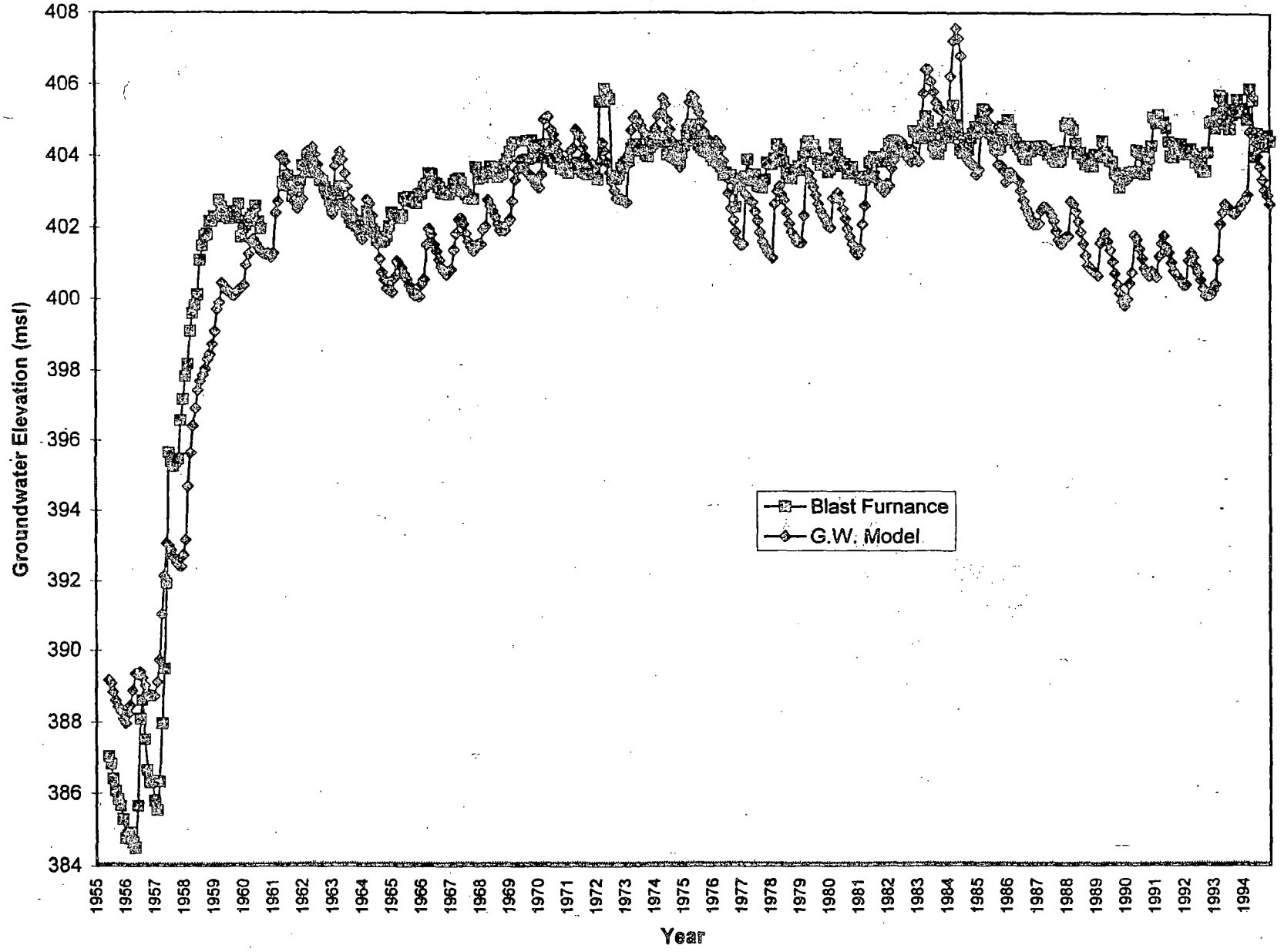


Figure 24. Hydrograph for State Water Survey observation well - Granite City Blast Furnace and model simulation.

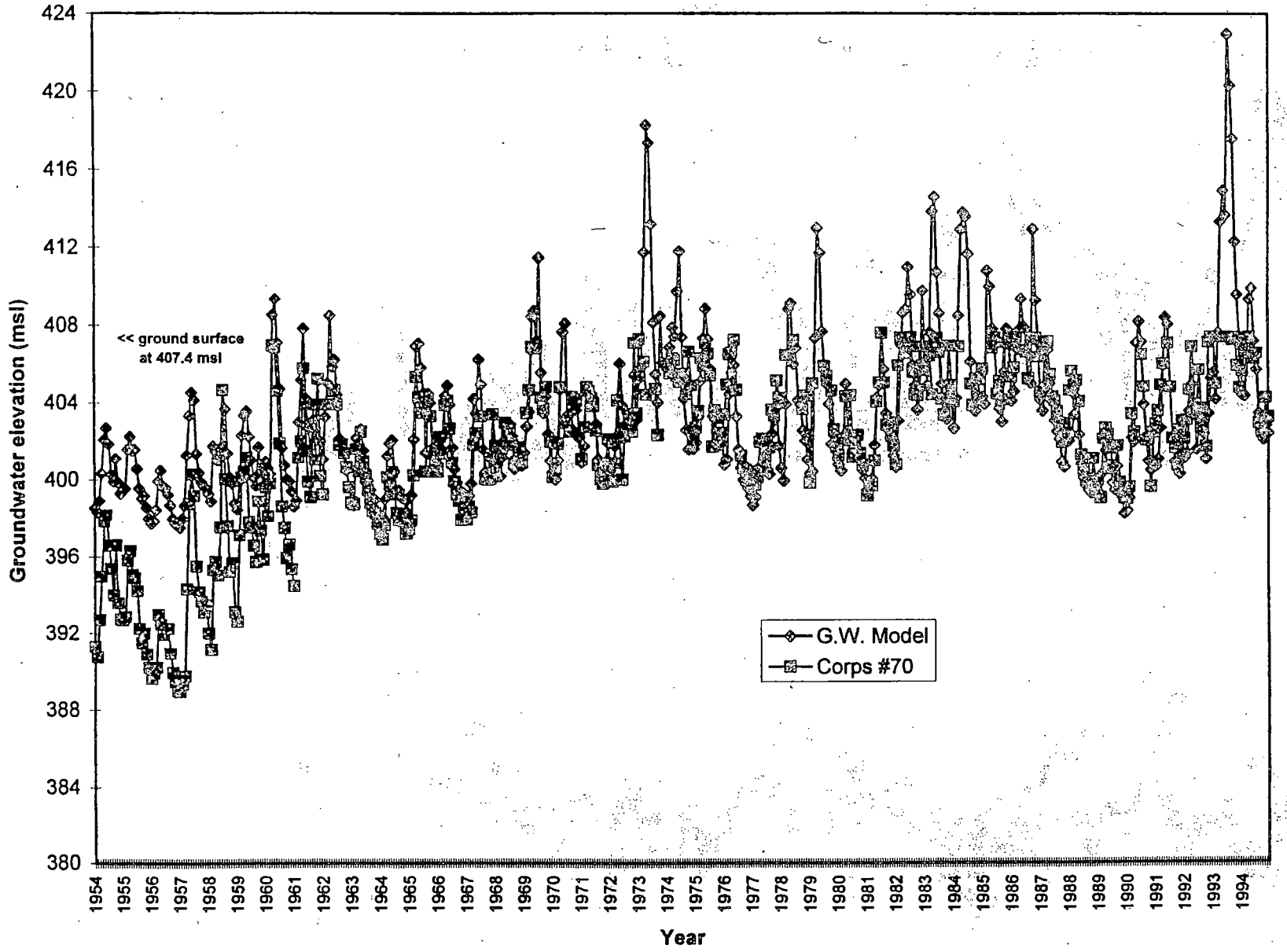


Figure 25. Hydrograph for State Water Survey obser[illegible] well - Corps of Engineers No. 70 and model simulation.

Table 3 shown below lists values in million gallons per day for the MODFLOW volumetric budget output for the stress periods ending at the simulation periods of December 1956 and June 1973. Note that positive values indicate flow into the modeled system while negative values indicate flow leaving the system as leakage into cells representing rivers, lakes and general head (or constant head) boundaries.

December 1956			June 1973	
Cell /Boundary	**Model inflow or outflow (mgd)**		**Cell /Boundary**	**Model inflow or outflow (mgd)**
Constant Head	+18.52 (1 neg. cell)		Constant Head	+8.93 mgd (5 neg. cells)
Miss. River (lowest 17 cells)	-12.9		Miss. River (lowest 17 cells)	-2.15
Miss. River (middle 12 cells)	+9.57		Miss. River (middle 12 cells)	+13.52
Miss. River (upper 5 cells)	-0.10 (3 neg., 2 pos.)		Miss. River (upper 5 cells)	+2.10 (3 neg, 2 pos.)
Chain of Rocks	+3.83 upper 4 cells -2.70 lower 10 cells		Chain of Rocks	+0.22 (mixed, 9 neg. and 6 pos.)
Cahokia Diversion Canal North	-1.99		Cahokia Diversion Canal North	-1.68
Cahokia Canal-Harding Ditch South	-3.60		Cahokia Canal-Harding Ditch South	+2.87
Horseshoe Lake	+7.22		Horseshoe Lake	+0.36 (8 neg. and 8 pos. cells)
Frank Holten Park Lake	+0.90		Frank Holten Park Lake	+0.09
Note: Positive values indicate flow into modeled system, negative values represent flow leaving the modeled system as leakage.				

Table 3. Volumetric budget values for transient model stress periods.

Variations to MODFLOW Model Configuration

Pace Recharge Model

An extensive evaluation of recharge rates for an unconfined aquifer system area was conducted by Clark (1994). In the development of a regional model for the Mason County area of Illinois it was realized by Clark that recharge parameters would have a significant impact on model accuracy and transient simulations. For the development of rainfall event specific recharge estimates, Clark in his 1994 study used the Pace-GC model provided by Vern Knapp of the State Water Survey.

The Pace-GC soil moisture model is a submodel of the PACE watershed model (Durgunoglu et al., 1987) developed through the Precipitation Augmentation for Crops Experiment (PACE) project which was initiated in the mid 1980's by the Illinois State Water Survey. The PACE model is a quasi-distributed-parameter model which was designed and constructed to simulate the movement of water through the hydrologic system for the purpose of analyzing the effects of changes in precipitation. The overall PACE model is modular in construction with major components for soil moisture, groundwater, and surface water. The PACE-GC model is a subset of this modular structure which computes a water balance between rainfall and recharge into shallow groundwater based daily rainfall and temperature values along with crop type and soil characteristics. The advantage of using the PACE model is that recharge estimates are calculated directly without the need to estimate evapotranspiration or deal with other seasonally factors.

The American Bottoms MODFLOW model was run for the time period of 1950 through 1980 to compare the PACE recharge method with the method used by the State Water Survey in the development of the PLASM model. Model outputs were plotted along with the hydrographs from observation wells in the Granite City area. Model results based on the PACE recharge model appeared to reasonably replicate the water table fluctuations recorded in the observation wells although there was more variability in the curves from the PACE based recharge model. The correlation coefficient for the PACE model simulations in comparison to the observation well data was 0.855 while the correlation coefficient for the State Water Survey based recharge model was 0.952. The relative standard deviations were 2.10 and 1.11. The State Water Survey based recharge method did give more realistic model results and therefore this method was used for all final transient model applications. The PACE based results although, are not unreasonable and further calibration based on improved selection of soil types and other model parameters could improve the PACE based MODFLOW output.

Constant Head Cells

As described previously, the State Water Survey used variable head cells to represent the eastern boundary of their PLASM model. The use of variable or general head boundary cells in MODFLOW can cause substantial errors to occur in the volumetric budget output. This problem was noted by Anderson and Woessner, 1992, in their text on applied

groundwater modeling. Substantial errors were also noted in the volumetric output of this MODFLOW model and conductance values of the general head boundary cells were adjusted to minimize the differences between inflows and outflows. As an additional check on the models output and validity, a transient model was developed which used constant head cells as the eastern boundary. With this version of the MODFLOW model, differences in volumetric totals for input and output were virtually non-existent and there were no distinguishable differences in head elevations for the time periods used for initial model calibration. Therefore, it appears that it is not necessary to use variable head cells at the eastern boundary of the model unless there is specific interest and application of the model at the toe of the bluff line.

Water Level Exceedance Probability Curves

Stage frequency graphs are commonly used by engineers to evaluate surface water gaging records. Stage frequency information is also used by engineers to evaluate the impacts, benefits and project economics of flood control project alternatives. At the direction of the Corps of Engineers, the State Water Survey set up their groundwater model to generate output which could be used to complete the economic analysis of various with and without project alternatives to control groundwater flooding in the American Bottoms area.

Stage frequency or exceedance probability curves are a graphic presentation of a simple statistical analysis. They were constructed to evaluate the probability that a particular groundwater level would be equaled or exceeded during a given period of time. The plotting position or frequency table is generated by using a formula presented by Weibull, 1939. This formula is:

$$p = [m/(n+1)] \times 100$$

where: p = exceedance probability, in percent
n = number of years of record
m = rank of event in order of magnitude, the largest event having m = 1.

Probability exceedance curves for grid cells corresponding to the three State Water Survey observation wells located in the Granite City area are shown in the figures 26, 27 and 28. Tables of data used to generate these curves are included in Appendix E.

The with and without project evaluation of groundwater flood control alternatives requires model generated exceedance probability curves for each impacted cell in the project area. Therefore, the generation of these curves or tables is the primary model output used in the economic and feasibility stage of plan formulation that was developed by the Corps and is documented in the following section.

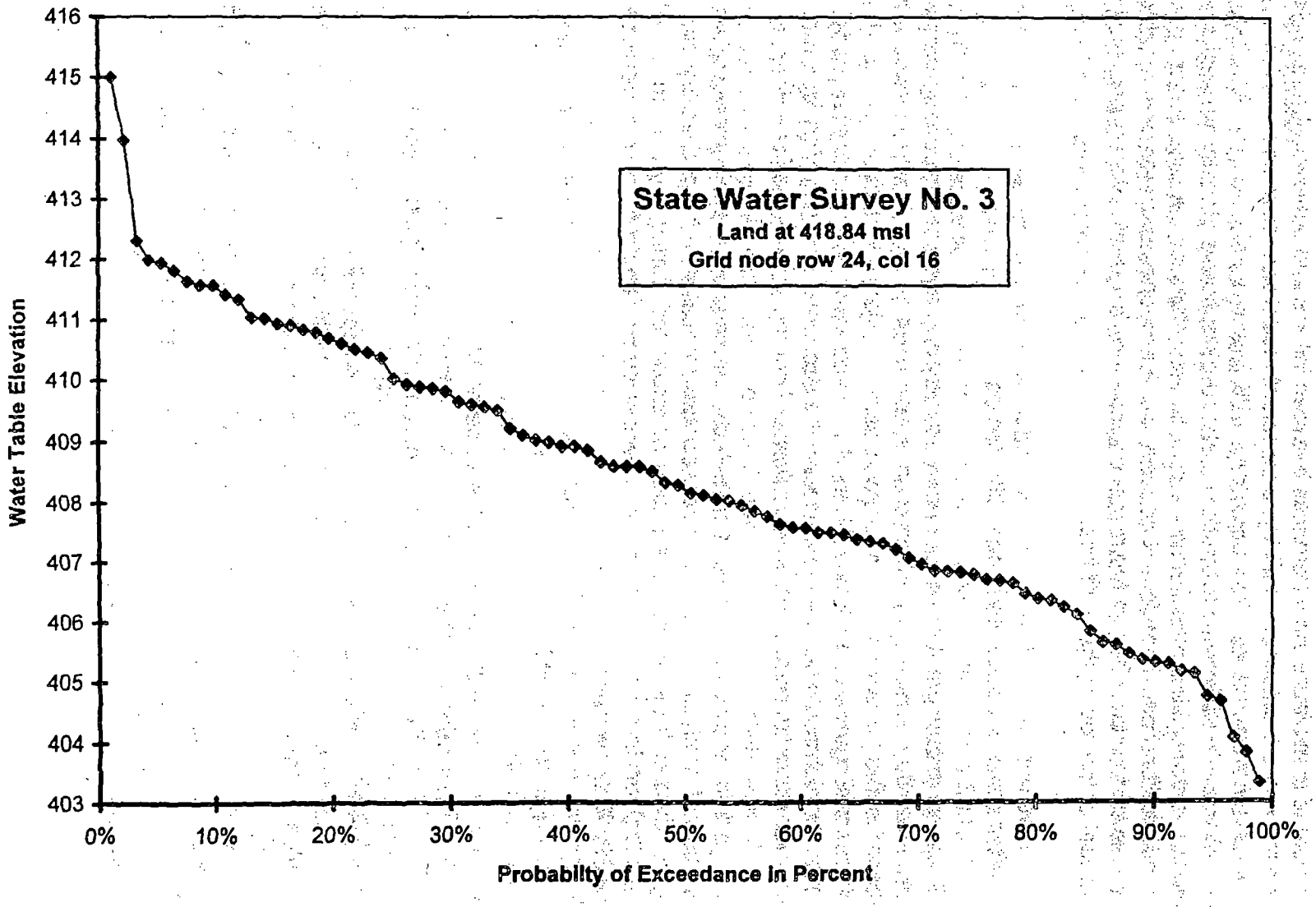


Figure 26. Water level exceedance probability cur r State Water Survey observation well - No. 3.

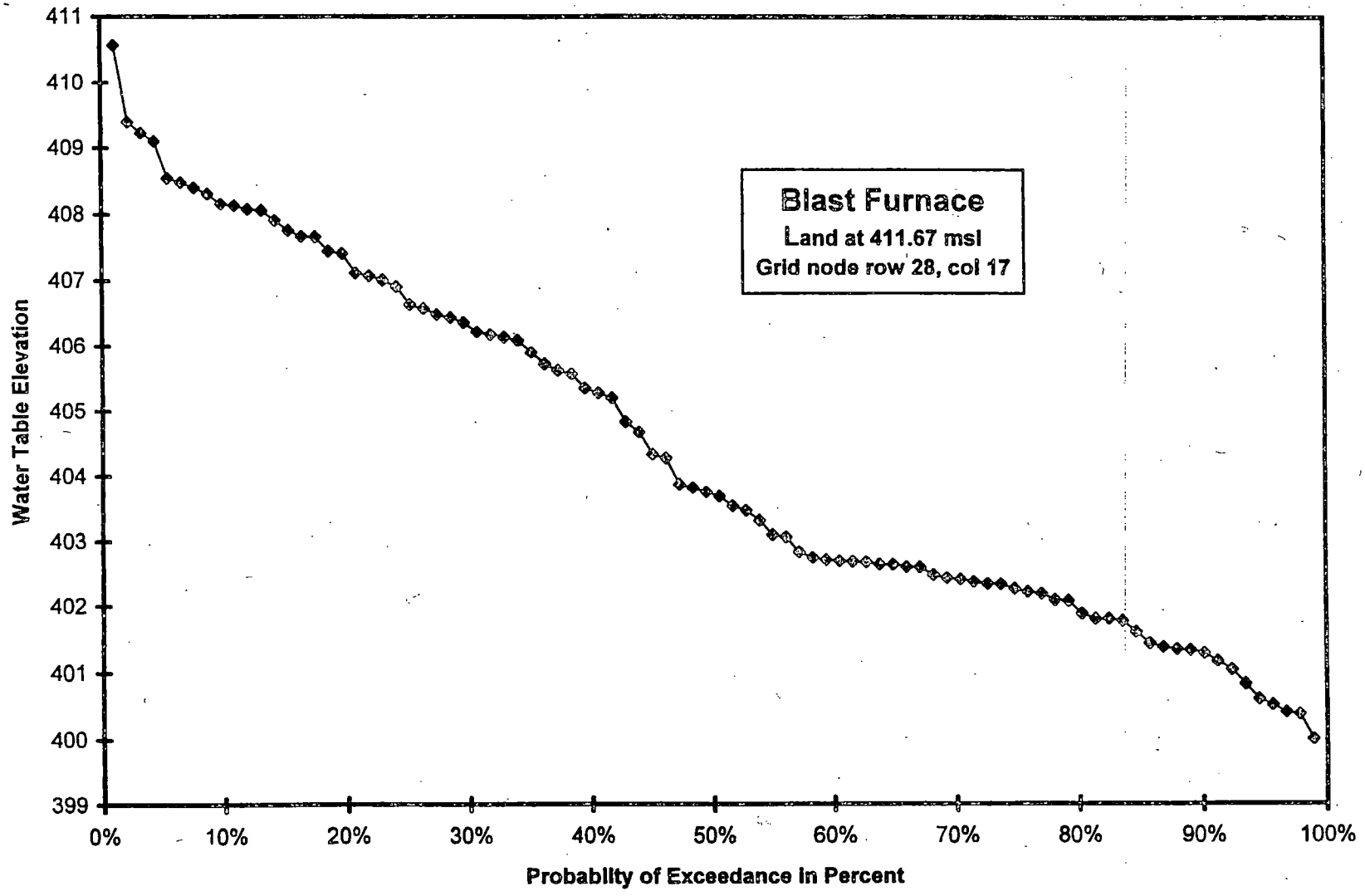
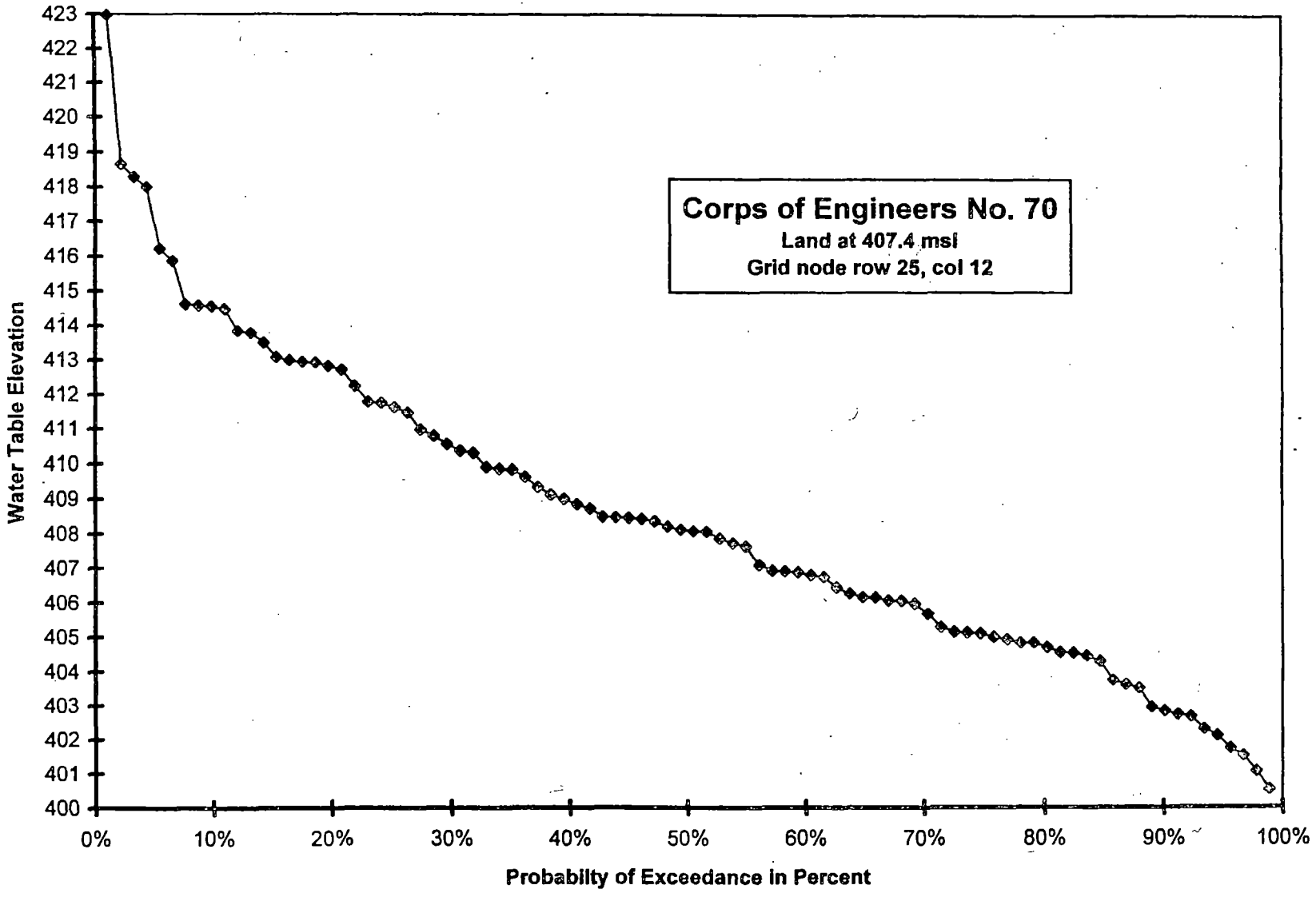


Figure 27. Water level exceedance probability curve for State Water Survey observation well - Granite City Blast Furnace.



gure 28. Water level exceedance probability curve for Sta ter Survey observation well - Corps of Engineers No. 70.

CORPS OF ENGINEERS PLAN FORMULATION AND ECONOMIC ANALYSIS

The Corps of Engineers identified two major problem areas very early on in their Metro Study effort. One was the reports of a large number of sanitary and combined sewer breaks caused by high groundwater levels. The other was structural damage to basements of many homes, again blamed on high groundwater levels. A third, and eventual major damage and benefit category, that was identified later was the infiltration of groundwater into the sanitary sewer system causing unnecessary waste water treatment costs.

The Corps of Engineers conducted a major data collection effort during their American Bottoms study effort to help quantify the extent of the damages associated with the high groundwater levels. The data collected by the Corps was referenced to the same grid system that was used for the development of the State Water Survey groundwater model. The following is a discussion of the extensive data collection effort conducted by the Corps during the early 1980's.

Sewer Inventory

The Corps conducted a complete inventory of all sanitary, storm, and combined sewer lines in the study area. Each sewer line was located by grid coordinates and the following information was documented for each line: location (street), type (lateral or trunk), size (inches), length (feet), depth of invert (feet), type of material, type of joint, age, condition, dry weather capacity, wet weather capacity, and outfall location.

Municipalities, planning agencies, township governments and their consultants were contacted to obtain all available data on these sewer systems. The data was limited to all sanitary sewers eight inches or greater in diameter, all storm and combined sewers 12 inches or greater in diameter, and all lift stations and waste treatment plants.

Other underground utilities were also considered by the Corps. Gas, water, electric and telephone utility companies within the study area were contacted to determine whether significant groundwater problems currently affect or had affected their systems in the past. It was generally found that the utility companies contacted indicated that they did not have any significant problems due to high groundwater levels. The average depth of the utilities within the study area is within the range of three to five feet from ground surface. Most electric lines are above ground. The natural gas lines are high pressure lines with welded steel connections although at one time when low pressure lines were used they reported problems with water seeping into lines and causing problems in homes with the efficiency of gas appliances. Telephone and water supply companies did not report any problems due to high groundwater levels.

To determine that damages caused to sewer systems in the study area, the Corps conducted an extensive search to identify and analyze data regarding the occurrence of sewer line failures, the associated repair costs, and the relationships of sewer failures to

fluctuations in groundwater levels. This effort revealed that in the period from 1969 to 1985 there were 441 distinct and documented sewer failures in the American Bottoms study area, or approximately 26 failures annually. The 441 sewer failures were matched by the Corps to the corresponding sewer line segments in the study area sewer inventory. The geographic distribution of the identified failures for the Granite City area is displayed in figure 29.

The sewer failure data was obtained from interviews with state and local agencies within the study area. Plans and specifications for sewer rehab projects in the area were reviewed and local engineering firms responsible for the operation and maintenance of these sewer systems were consulted. Flood damage Survey Reports prepared for the Federal Disaster Assistance Administration (now FEMA) were also reviewed and information obtained on actual breaks, location of break, date of break, depth of pipe, depth of groundwater, extent of damage, and repair cost estimates. The cost estimates consisted of a temporary remedial repair cost, such as by pass pumping, as well as an estimated cost of actual repair. Most of these estimates included the cost for dewatering which was considered a strong indication that ground water was in the vicinity of the sewers when they failed. Granite City was cited for maintaining thorough records on the sewer failures and groundwater conditions in their community.

The Corps found that the failure of a section of sewer line can be the result of a number of factors that can often act in combination. Some of these include inadequate design or changes in design assumptions after construction, inappropriate materials, faulty construction practices and inundation by groundwater. The Corps could not find a strong statistical correlation between breaks and any of these factors but did discuss extensively some of the conditions and construction methods common to many of the problem sewer lines that most likely related to cause of many of the breaks.

Many of the sewer failures have occurred in lines constructed during the mid-fifties. This period was previously discussed as one of high groundwater withdrawals by industry and low precipitation. Since groundwater levels were well below pipe invert elevations at this time, it was likely that sewer lines were not designed or constructed of the proper materials to withstand the additional stresses caused by submergence of groundwater. As groundwater rises above the height of flow in the sewer line, the pipe is subject to buoyant forces which will attempt to float the pipe line. If the line is not constructed for this condition the sewer will move and break the seal at the joints or the line itself. The pipes materials commonly used in the project area are susceptible to shearing and beam breakage when insufficiently supported. The predominate pipe materials found were concrete pipe, reinforced concrete pipe and vitrified clay pipe. The joints were constructed of cement mortar, hot poured, or cold-installed bituminous material which are the types of materials were it is hard to obtain a tight and long lasting joint. Also due to their rigidity when set, these materials tend to break away from the bell and spigot of the pipe if subjected to even slight deflective forces. Also, the lines in the American Bottoms were laid in short lengths which increases the number of joints and probability for infiltration.

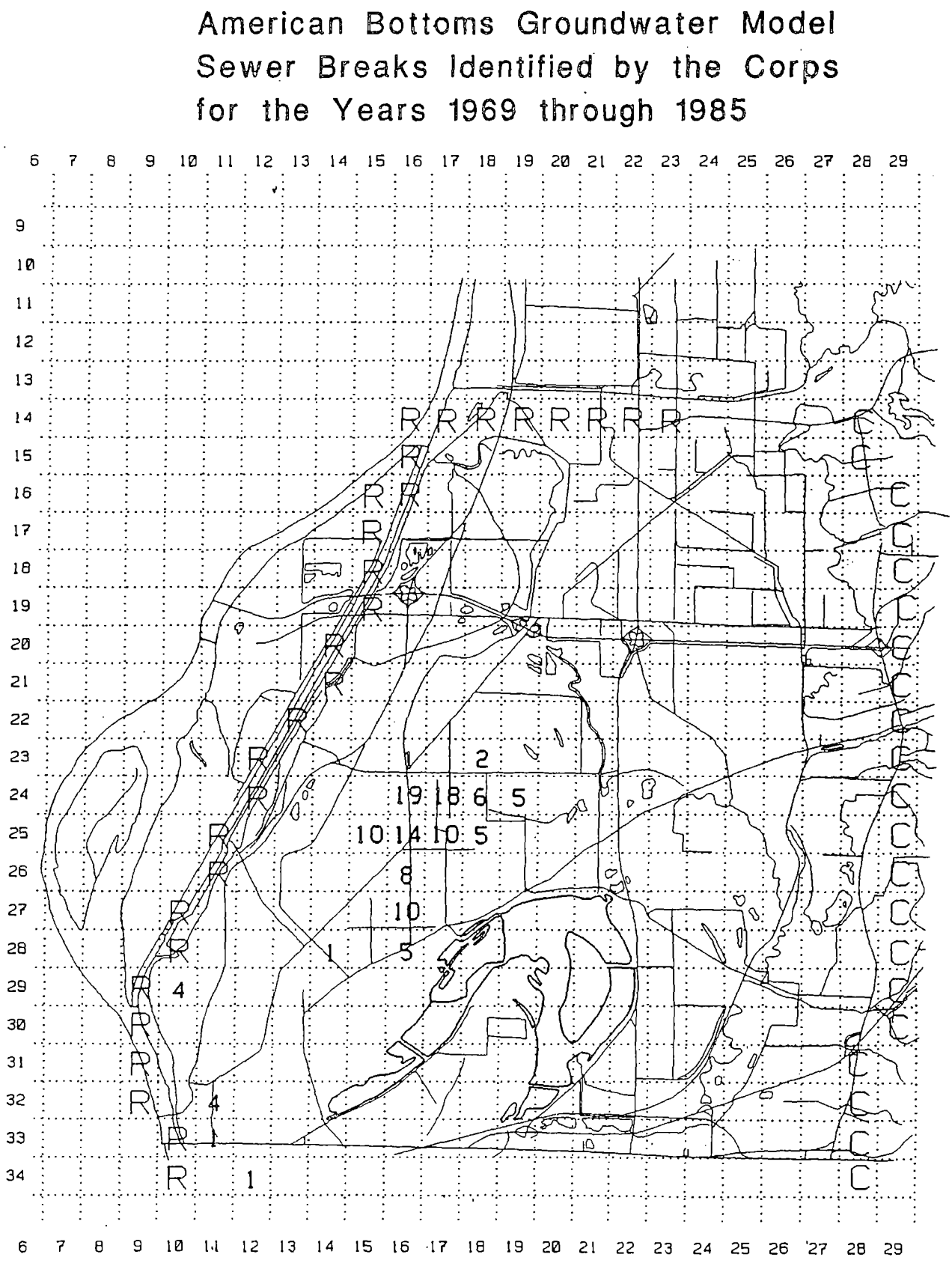


Figure 29. Sewer breaks in the Granite City area.

A route for infiltration of ground water as well as soil particles is created if deflections in the lines break the joint seals. The soils which comprise much of the trench wall material is mainly very fine silty sands and sandy silts which are very susceptible to erosion and transportation by flowing water. When the hydrostatic pressure in the aquifer is greater than that in the pipe, groundwater will naturally flow into the sewer and carry erodible soils materials with it. The result is voids in the foundation of the sewer and also loose bedding material can compact when inundated and then drained which will also cause the pipe to lose support due to further creation of voids. With time, the continual removal of foundation material will so undermine the sewer pipe that a large scale failure occurs that is often reflected at the surface by ground subsidence resulting in damaged streets, sidewalks and nearby structures.

In some cases the infiltrated material was collected in the sewer line and effectively reducing or even stopping flow. Fine grained material which is not deposited in the lines is transported through pump stations and treatment facilities resulting in increased costs for operation and maintenance.

The Corps concluded that groundwater was the major suspect for the sewer failures, infiltration, and subsidence damages experienced in the American Bottoms. Sewer problems and infiltration would have occurred in the absence of the groundwater condition due to the pipe and joint material used, the age of the pipe and the surrounding soil material in the project area. However, the Corps concluded that the magnitude, the intensity and the frequency of these problems were all increased by groundwater facilitating and/or expediting these conditions. In some sewer failures, groundwater was therefore either solely responsible or the major contributor to eventual failure.

The Corps also looked at sewer infiltration and exfiltration. Groundwater levels above the flow line of a sewer may infiltrate the system through defective joints, broken or cracked pipes, and/or poor connections at manholes or pipe laterals. This infiltrating groundwater causes increased volumes of water to be treated at area waste water treatment facilities at additional and unnecessary costs. There were thirteen major waste treatment plants operating in the American Bottoms study area in 1986. Research of existing flow infiltration studies on existing plants, in addition to personal contacts were utilized to determine infiltration rates and associated dollar costs connected with this category of damage. Infiltration rates between 7.3 and 12.2 mgd of groundwater was being treated at the existing sanitary treatment plants.

Infiltration of water generally results from the advanced age of the sewer, soil conditions, poor construction materials and/or poor methods of construction. The amount of water that enters the systems is dependent upon the length of sewers within the area served as well as the condition of soil, topography surrounding the pipes, quality of the materials used and the quality of the construction. In addition the number of building connections, the type of maintenance program exercised and the relative elevation of the groundwater to the sewer all influence the quantity of infiltration that enters the system. Excessive amounts of infiltration consumes sewer and plant capacity and infiltration also transports silts and sand

into the sewers and pump stations. This material can accumulate over time and cause line blockages and increased wear on pumps and related facilities which naturally cause an increase in operation and maintenance costs.

The Corps used previous engineering studies in the American Bottoms area to determine infiltration rates in gallons per day per inch mile (gpdim). Various methods and calculations were used to determine these infiltration rates. The sewage treatment plant 24 hour sewage flows into the plant were examined and adjusted to reflect the flows that did not reach the plant due to known by-passes and overflows. Electric power consumption records were examined to determine the quantity of flows being pumped from the lift stations within the collection system since the stations were not equipped with flow meters. Sanitary flows were estimated by assuming a percentage of the metered water used by consumers within a given time period will return as waste into the sewer system. Estimates for major industries and major water consumers were developed from contacts with each establishment. Where infiltration rates were not previously calculated, estimates were determined after consulting with the city engineers, waste water treatment plant superintendents and relations to infiltration rates from adjacent areas.

Infiltration was then estimated by subtracting the 24 hour sanitary flow from the 24 hour waste water flow. The amount of infiltration does vary throughout the year depending upon the variations in ground water levels above the sewer flow line. Due to these variations in groundwater levels, minimum and maximum rates of infiltration were determined. It was assumed from reviewing available waste water flow information that the maximum infiltration occurs from February to July and the minimum from August to January.

Inch miles of pipe is equal to the diameter in inches of a pipe times its length in miles. The summation of the diameter times their respective lengths equals the inch-miles of pipe in the collection system. This summation of inch-miles of pipe was divided into the total amount of infiltration attributed to the corresponding collection system to obtain a unit rate in gpdim for the collection system. The infiltration quantity was then allocated to each system grid based on the infiltration rate of the corresponding treatment plant and the inch-miles of pipe submerged within each grid.

The Corps also considered exfiltration which is the outward flow of water from the sewer pipe through defective joints, broken or cracked pipes and poor connections at manholes or pipe laterals.

The conditions that contribute to infiltration do not necessarily contribute to exfiltration. Most sewers are designed to flow partially full: therefore any cracks or defects that are within top portion of the pipe are where no exfiltration should occur during normal flow conditions. Also, the hydrostatic pressure that causes infiltration to occur on the weak or defective joints is nonexistent once the groundwater is below the flowline of the sewer. The same cracks and defective joints that allow infiltration to occur, in many cases, will be sealed with debris settling out, thus eliminating or minimizing the avenue for exfiltration.

Exfiltration has probably occurred in the past due to historical fluctuations of groundwater levels. Any dewatering project is not expected to cause an acceleration in exfiltration compared to what has naturally occurred.

Corps Benefit Categories

The Corps identified three National Economic Development (NED) benefit categories for the evaluation of potential project related benefits in the American Bottoms study area. NED benefits are the increases in the value of the nation's goods and services as a result of positive project impacts. The NED categories in the study area were: 1) groundwater related damages incurred by structures with basements; 2) repair costs of structural failures in sewer lines in the study area caused by fluctuations in groundwater levels; and 3) infiltration of groundwater into the sewer systems necessitating increased waste water treatment costs. The Corps felt there were other potential NED benefits categories in the study area but choose to identify these three as the main areas of potential benefit.

Existing Damages Identification

Structures with Basements

Inventory of Structures - The identification of those residential structures with a groundwater problem was accomplished by the Corps through the use of two questionnaires. (Note that virtually no commercial structures in the study area have basements.) The first questionnaire was a business reply postcard mailing which was made in two phases, the first going to a single zipcode in Madison county (Granite City) and the second phase to all remaining zipcodes within the study area recommended for further study. There were 17,000 cards sent to the Granite City area and 30,000 mailed to the remaining zipcode areas. Key questions asked were whether the residence had problems with water in the basement and if so when; was there water seeping in the basement now; and were there cracks in the basement floor or walls. The number of completed and returned post cards was 2,701 and this was the assumed number of all structures potentially damaged by high groundwater levels.

The second questionnaire was more detailed and was sent to those respondents of the first questionnaire who had indicated that their residence had a potential groundwater problem. This questionnaire asked for more specific information regarding the occurrence of water problems in the basement; the relationship of water problems with rainfall periods; type of foundation; type of basement - finished or not; detail as to any actual structural damages incurred; and measures taken along with money spent to prevent reoccurrence. There were 1,343 respondents to this questionnaire

The Corps used the State Water Survey output to screen the 1,343 respondents to eliminate those which never had groundwater within 8.6 feet of the first floor elevation. The first floor elevations were determined by a field survey of all the respondents to the first questionnaire. The figure of 8.6 feet below the first floor elevation was chosen to represent

the typical depth of a basement and foundation below first floor levels. Those structures which the model predicted had groundwater within 8.6 feet of the first floor were assumed to be affected by groundwater. A total of 774 existing residential structures in the study area were found by this comparison to have potential groundwater problems. The geographic distribution of the screened respondents in the Granite City area is displayed in figure 30.

To estimate the economic losses occurring in the American Bottoms study area to residential structures with basements, the Corps used historic real estate sales data concerning actual market transactions to measure the impacts on housing values of groundwater levels reaching the basement of a residential structure. The difference in market values resulting from groundwater related problems in similar structures should be a reasonable estimate of the capitalized annual economic losses over the life of the structure. A sample of all transactions occurring in the period from 1979 through 1984 was collected for any residence responding to the second questionnaire and identified by the State Water Survey model as a structure with potential groundwater related damages. The data was compiled from summary of sales information contained in the Granite City Realtors Multiple Listing Service. Data included date of sale, location, purchase price, lot size, number of rooms, number of bedrooms, size of garage or carport if present, size and condition of basement, and other pertinent data that could affect the purchase price of the structure. Also, integrated into this data set was the estimate of the relative frequency by the State Water Survey model for groundwater reaching the structure's basement elevation. The data set consisted of 42 observations of real estate transactions in the Granite City area. The purchase price information was updated to October 1984 price levels and later economic loses were updated to 1997 price levels.

The Corps used the data to conduct a "principal components" analysis that found that one factor could be used to explain 65 percent of the correlation of the original variables. The factor appeared to be some weighted measure of the usable space of the residence.

The value of this factor was then computed for each residence in the sample of transactions and used in a multiple regression model to explain the observed adjusted purchase price (market value) in each transaction. The historic frequency, as given by the State Water Survey model, of groundwater reaching the estimated level of the residence's basement was also included as an explanatory variable in the regression equation. The computed regression equation is:

$$ADJPRICE = (\$19973.37 * FACTOR1) - (\$16376.42 * FREQ)$$

where: ADJPRICE = the purchase price in October 1984 prices
FACTOR1 = the principle component factor
FREQ = the estimated frequency of groundwater reaching a level within 8.6 feet of the first floor elevation.

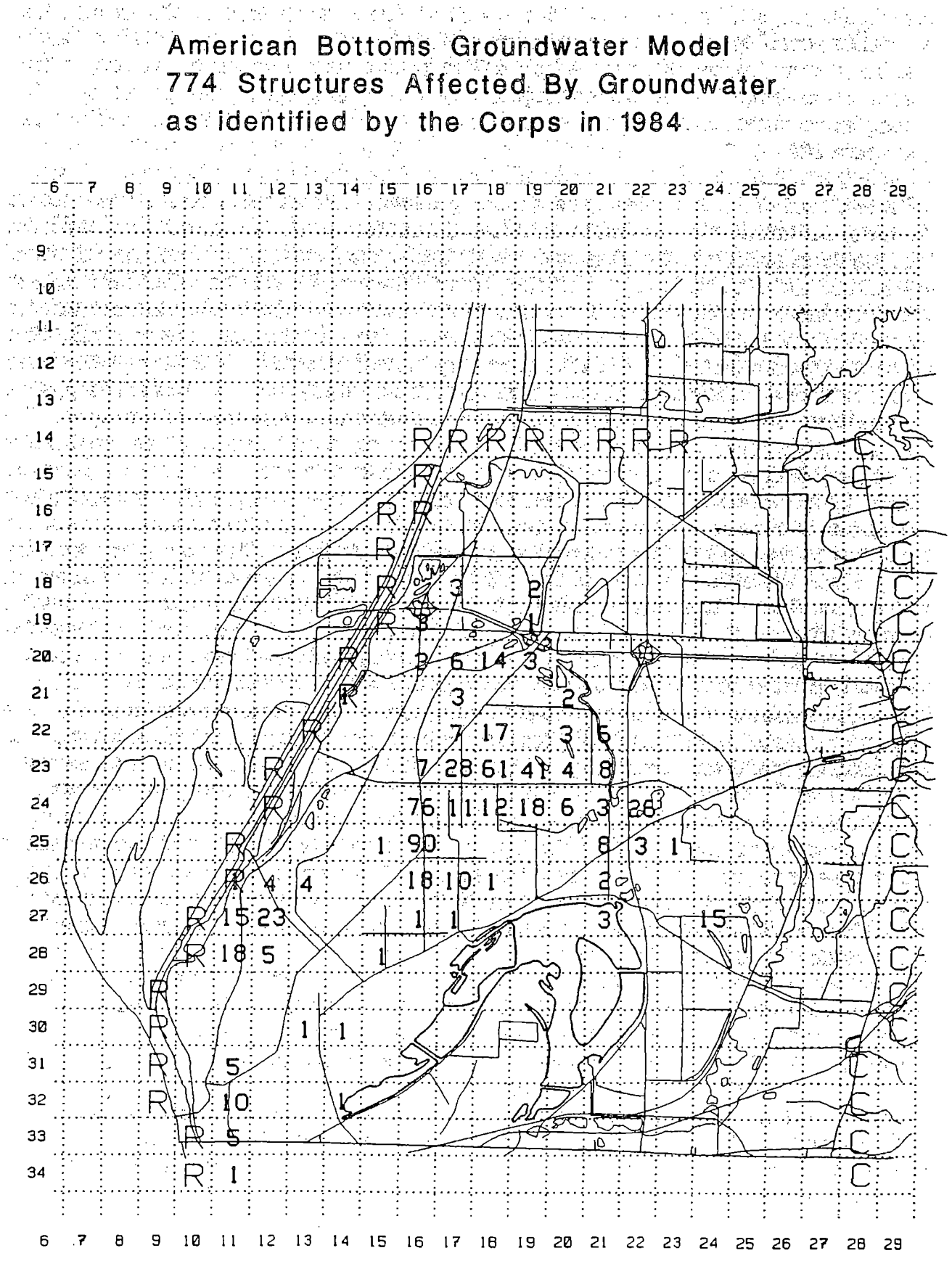


Figure 30. Distribution of structures affected by groundwater.

This equation estimates the dollar impact of varying frequencies of having groundwater levels reach the estimated basement elevations of residences in the American Bottoms study area. The regression equation was specifically designed to separate the impacts of groundwater from the other components of housing values. The results of the regression equation were used to estimate the existing economic losses in the study area due to groundwater flooding. The coefficient of the groundwater frequency yields the loss in value of a structure related to groundwater problems. For example, a residence which has never experienced groundwater at a level equal to the estimated basement elevation (FREQ=0) would incur no loss of value. A residence which always had groundwater at a level at least equal to the estimated basement elevation (FREQ=1) would incur a loss in value equal to $16,400. Assuming that the estimated loss in value represents the capitalized average annual damages related to given groundwater frequencies, average annual groundwater damages may then be computed as a function of groundwater frequencies.

Using an estimated life of 50 years for a residential structure and a real interest rate of 5.34 percent (Social Rate of Discount, Office of the Chief of Engineers, June, 1983, TABLE 5.4, page 32) for household sector of the economy yields:

AVERAGE ANNUAL DAMAGES = (FREQ * $944.58)

where the FREQ represents the State Water Survey model estimate for the frequency of groundwater reaching the structures basement. The figure of $944.58 represents the average annual equivalent in 1984 dollars for a period of 50 years of a present value of $16,376.42 at 5.34 percent annual real interest.

Computing this figure in 1997 dollars for the 774 residences in the study area and summing, yields an estimate of $223,271 for the average annual existing groundwater related structural damages.

Sewer Failures

The data collected by the Corps, and described above, concerning sewer failures was used through a statistical analysis to determine breaking probabilities. The 441 sewer failures that were identified were matched with their respective average invert elevation, to the State Water Survey estimate for the frequency of groundwater reaching the invert elevation. The Corps used the data collected in an attempt to relate meaningful variables to sewer line failures. All of the statistical models that they developed were unsatisfactory in their relationship to actual historic sewer line failure patterns. While the models did accurately and significantly predict overall sewer failure rates, the geographic locations of the failure, which is important to direct project benefit calculations, were poorly predicted. Some undetermined factors appear to make some sewers more likely to fail than others. Three potential factors could be site specific geology, site specific soil conditions, and specific construction techniques. The frequency of groundwater overtopping the sewer

invert elevation does, however, impact on overall failure rates. With these results the Corps based their benefit calculations on the following assumptions:

1) Any failure in a sewer line predicted by the State Water Survey model to never have had groundwater above the invert elevation is not groundwater related (there were 50 over the 17 year period), and

2) All other sewer failures are groundwater related (there were 391 failures of this kind in the 17 year period or approximately 23 annually).

The Corps also collected data on the actual costs of repairing the failures identified which included permanent repair costs as well as emergency costs. Cost were available for 219 of the 391 groundwater related failures identified from 1969 through 1985 in the study area. A multiple regression was performed to analyze the data so that costs could be estimated for the repair of failures where cost information was not available. The regression developed is as follows:

cost = ($1797.23 * diam) + ($2035.49 * depth)

With this equation the Corps calculated that the total repair costs for the 391 failures over the 17 year period of record was $1,096,000 annually. The Corps added to these costs the emergency costs to bypass the sewer failure before repairs were completed which were estimated from available data to average $4,422 per failure or $101,800 annually in 1984 dollars. These cost generally consisted of the setup, operation and maintenance of a bypass pump until permanent repairs were accomplished.

Summing the permanent and emergency repair costs gives an expected average annual failure repair cost for existing conditions of $900,076 in 1997 dollars. This figure represents the economic losses occurring as a result of groundwater related sewer failures in the Granite City area.

Sewer Infiltration

Damages caused by sewer infiltration were described previously as the increased volumes of water that has to be treated in the study area at wastewater treatment facilities and the resulting increased total treatment costs. The Corps identified six major wastewater treatment plants that were operating in the study area in 1986. The total volume of water treated at each plant varies throughout the year as a function of rainfall, infiltration, system usage, and other variables. The estimated volume of groundwater infiltration treated at each facility as well as the total inch-miles of sewer collection systems serviced is displayed in the table below. The volume treated is displayed for wet and dry periods (consisting of 3 month averages each) emphasizing the very large differences in volumes treated as a result of greater infiltration amounts during relatively wet periods.

Treatment Plant	Wet	Dry	Inch-Miles
	(millions of gallons per day)		
Cahokia	1.69	1.08	814
Landstone	3.32	0.00	510
East St. Louis	3.47	3.22	3,934
Granite City	2.52	2.04	1,356
Dupo	0.18	0.14	97
Collinsville	1.05	0.84	65

The Corps contacted the individual treatment facilities to determine treatment costs per one thousand gallons per day. The total annual costs for treatment at each major plant is included in the table below. Figures are updated to 1986 price levels.

Treatment Plant	Volume (million of gallons per year)	Unit Costs ($ per thousand gallons)	Total Cost
Cahokia	505.525	0.64	$323,536
Collinsville	344.925	0.19	65,536
Dupu	58.400	1.65	96,360
East St. Louis	1220.925	0.64	781,392
Granite City	832.200	0.44	366,168
Landstone	605.900	0.19	115,121
Total	3567.875	0.49	1,748,116

The total average annual damages in 1997 dollars for sewer infiltration in the Granite City area were calculated at $647,596.

Summary of Existing Damages in the Granite City Area

The summary of estimated average annual groundwater damages as calculated through the use of the Corps methodology is listed below as adjusted to 1997 price levels. The distribution of average annual damages as calculated with the revised model for the Granite City area in 1984 dollars is shown in figure 31.

Damage calculations are presented in some graphics in 1984 dollars to allow for a comparison of the results of the revised model and economic analysis with the results of the initial Corps study. A factor of 1.4 to 1.5 can be used in general to update the 1984 values to 1997 values. A factor of 1.4 was derived from a review of the construction cost index and the building cost index published in the Engineering News Record. A factor of 1.5 was derived from changes over this time period in the consumer price index.

Damage Category	Ave. Annual - 1997 Price Level
Sewer Line failures	$900,076
Sewer Infiltration	$647,596
Structures with Basements	$223,271
Total	$1,770,943

Benefit Computation Methodology

The Corps assumed that all potential projects plans would have a 100 year economic life. Also, all NED project impacts were initially expressed at October 1984 price levels using a discount rate of 8-7/8 percent to annualize both NED benefits and costs. (A discount rate of 7-5/8 percent was used for the 1997 calculations of benefits and costs over a 50 year project life.) The without project condition was taken to be that there would be little change in the economy of the American Bottoms study area. Groundwater withdrawal rates will increase only slightly through the future of the project with a slight reduction in future without project damages. These somewhat reduced damages become the basis for the potential benefits for each project alternative. The specific methodologies employed to estimate project impacts in each of the three quantified NED benefit categories are described below.

Structures with Basements

The following assumptions were employed by the Corps and in this study to estimate the NED benefits of project alternatives for residential structures.

1) Future without project damages for a structure are equal to the projected without project future frequency groundwater reaching the basement elevation multiplied by the $944.58 figure calculated above.

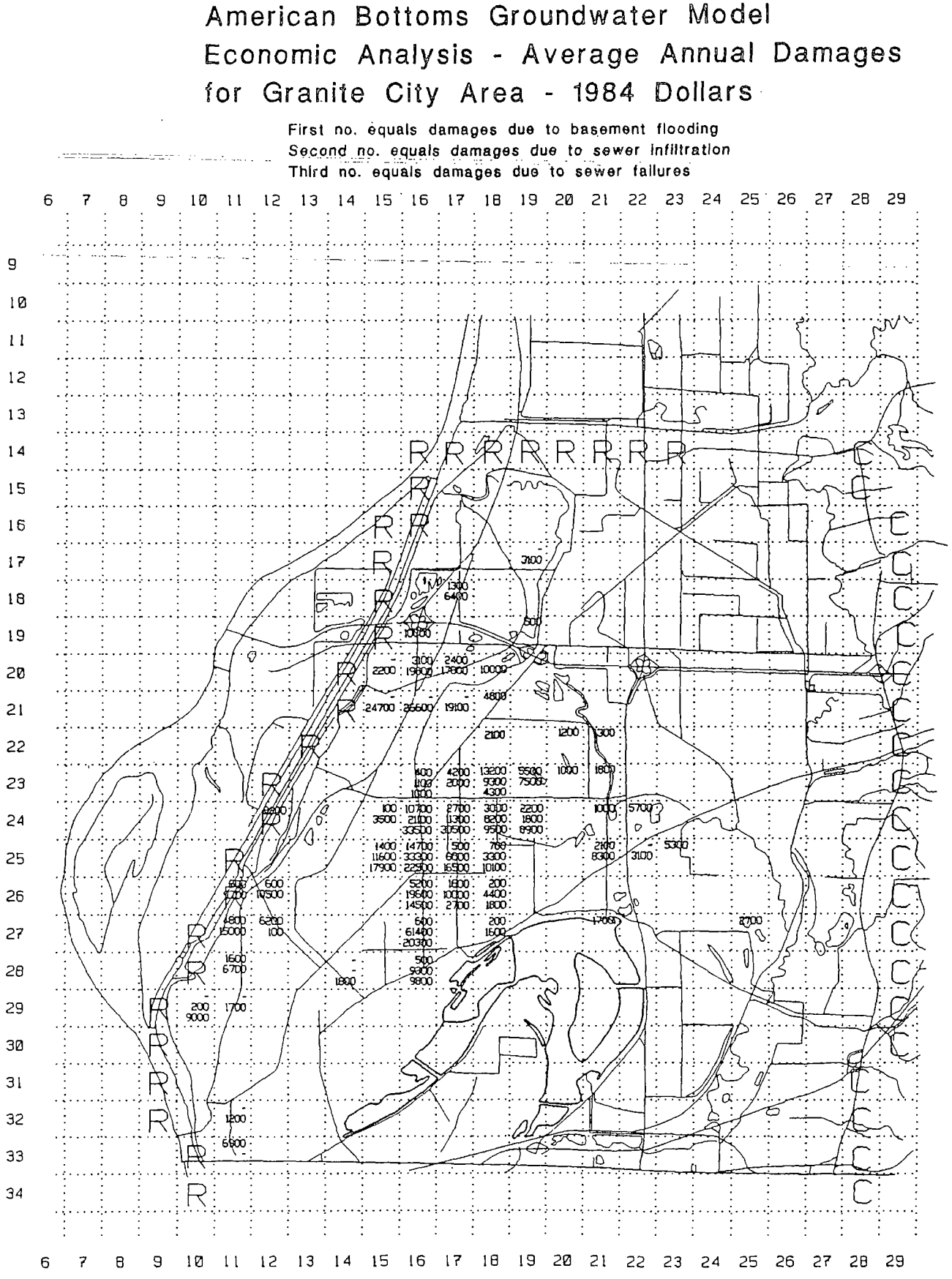


Figure 31. Distribution of average annual damages in the Granite City area.

2) Future with project damages for residential structures are equal to the with project estimate of the frequency of groundwater reaching the basement elevation multiplied by $944.58. (A factor of $1766 in 1997 dollars.)

3) The average annual NED benefits of the project are the difference between the without project average annual total future losses and the total with project average annual groundwater related losses.

Sewer Failures

The following assumptions were employed by the corps and in this study to estimate the NED benefits of project alternatives for sewer failures.

1) Sewer failures appear from the historical record to be centered in specific sewer lines and geographical areas. The without project condition was assumed by the Corps to be that the same sewer lines that have failed in the past will continue to fail at the same rates into the future, proportionately adjusted to reflect changes in groundwater overtopping the invert elevation. A sewer line that has not failed in the past was assumed to not fail in the future under the without project condition.

2) If a sewer line has failed in the past, the expected number of annual breaks in that sewer line were reduced proportionately by the relative decrease in the frequency of the groundwater overtopping the invert elevation in the with project future compared to the without project future.

3) Failure in sewer lines not currently groundwater related as measured by the groundwater model were projected to continue to occur at the same rates with or without a project.

4) Annual NED benefits for a project alternative were the decrease in annual expected sewer failure repair costs with the project from the without project annual expected sewer repair costs.

Sewer Infiltration

The following assumptions were employed by the Corps and in this study to estimate the NED benefits of project alternatives.

1) The without project future condition was assumed to be that existing annual infiltration rates will continue with a proportional adjustment for a decrease in expected annual inch-miles of sewer lines below groundwater levels. Infiltration rates per inch-mile of overtopped sewer line will remain constant in the future, however, the expected inch-miles of sewer lines overtopped will decrease as a result of the slightly increased without project future groundwater withdrawal rates.

2) The with project future will reduce expected annual groundwater infiltration levels as a result of decreasing the expected amount of inch-miles of sewer line below groundwater levels. The reduction in infiltration levels will be proportional to the decrease in expected inch-miles of pipe under groundwater levels.

3) Unit costs for treatment of groundwater infiltration will not change over the life of the project.

4) The annual NED benefits of the alternatives resulting from the reduction in infiltration levels are the annual expected without project wastewater treatment costs less the annual expected treatment costs with an alternative in place.

CORPS OF ENGINEERS PROJECT PLAN DEVELOPMENT

The Corps of Engineers used the State Water Survey model to evaluate the effectiveness and economic impacts of at least six pumping systems with various distributions of pumps and pump capacities. Total pumping capacities ranged from 17.24 to 103.5 mgd. The selected region-wide NED plan had a designed capacity of 41.25 mgd and the distribution of the 17 pumps in the Granite City area is shown in figure 32. The distribution of average annual economic benefits in the Granite City area for this plan is shown in figure 33. These benefits are based on output from the updated MODFLOW model.

ECONOMIC EVALUATION OF A ONE MGD PUMPING SYSTEM

An economic analysis was also developed using the MODFLOW model for a one mgd pumping system located in Granite City at model cell location (row-25, column-16). This location is centered in the area of highest average annual damages in Granite City. The distribution of average annual benefits for the three categories of damages is shown in figure 34. Average annual benefits are as follows:

$11,608 for structures
$44,056 for sewer infiltration
$38,261 for sewer breakage

Total annual benefits equal $93,925. The estimated annual operating and replacement cost of a one mgd system is $16,065 based on previous Corps of Engineers estimates updated to 1997. With these annual values and the criteria that any proposed project must meet a favorable benefit / cost ratio, the maximum first costs for project construction could be no greater than $995,200 based on a current project discount rate of 7-5/8 percent applied over a fifty year project life.

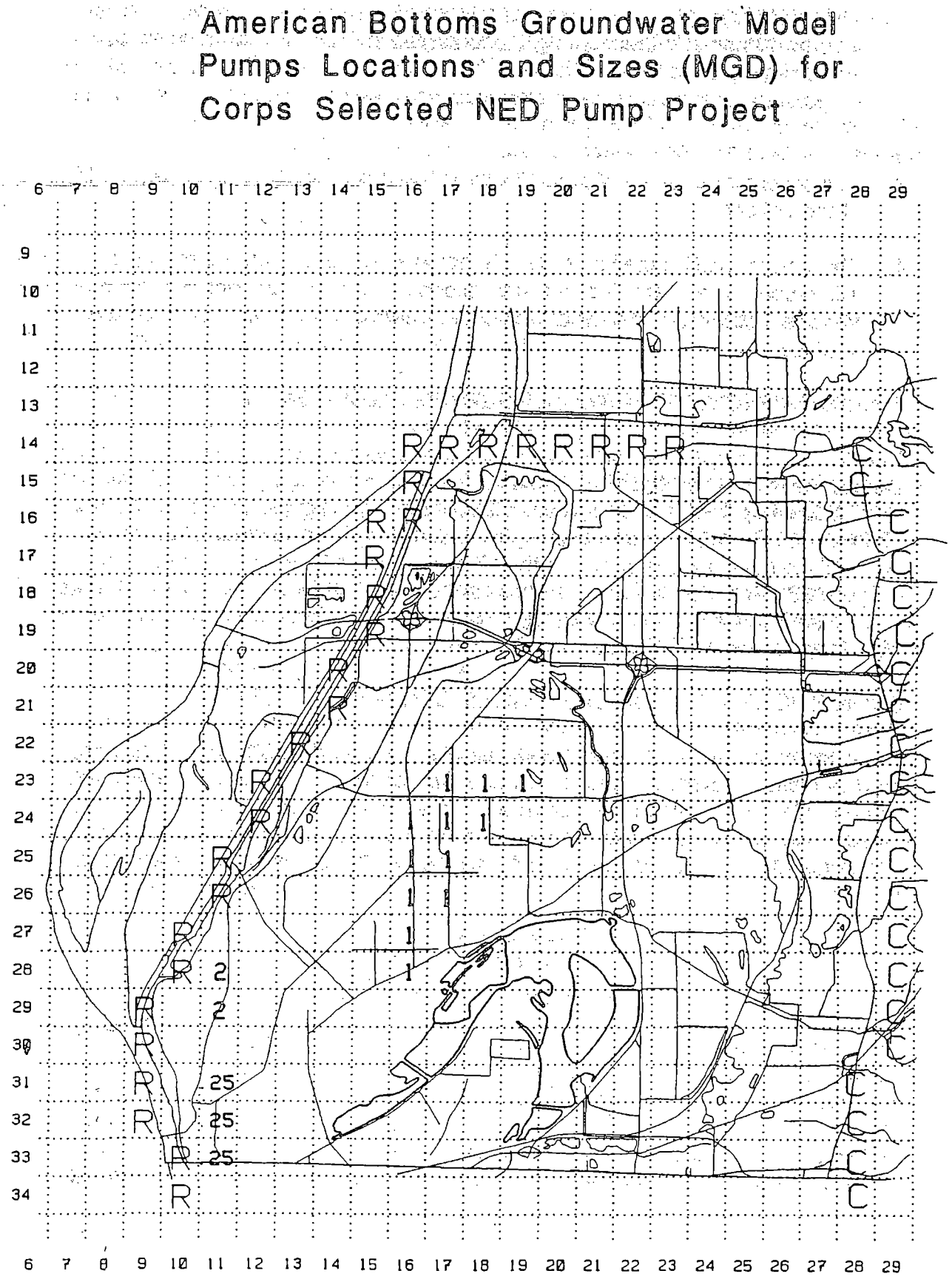


Figure 32. Pump size and location for Corps of Engineers selected NED plan..

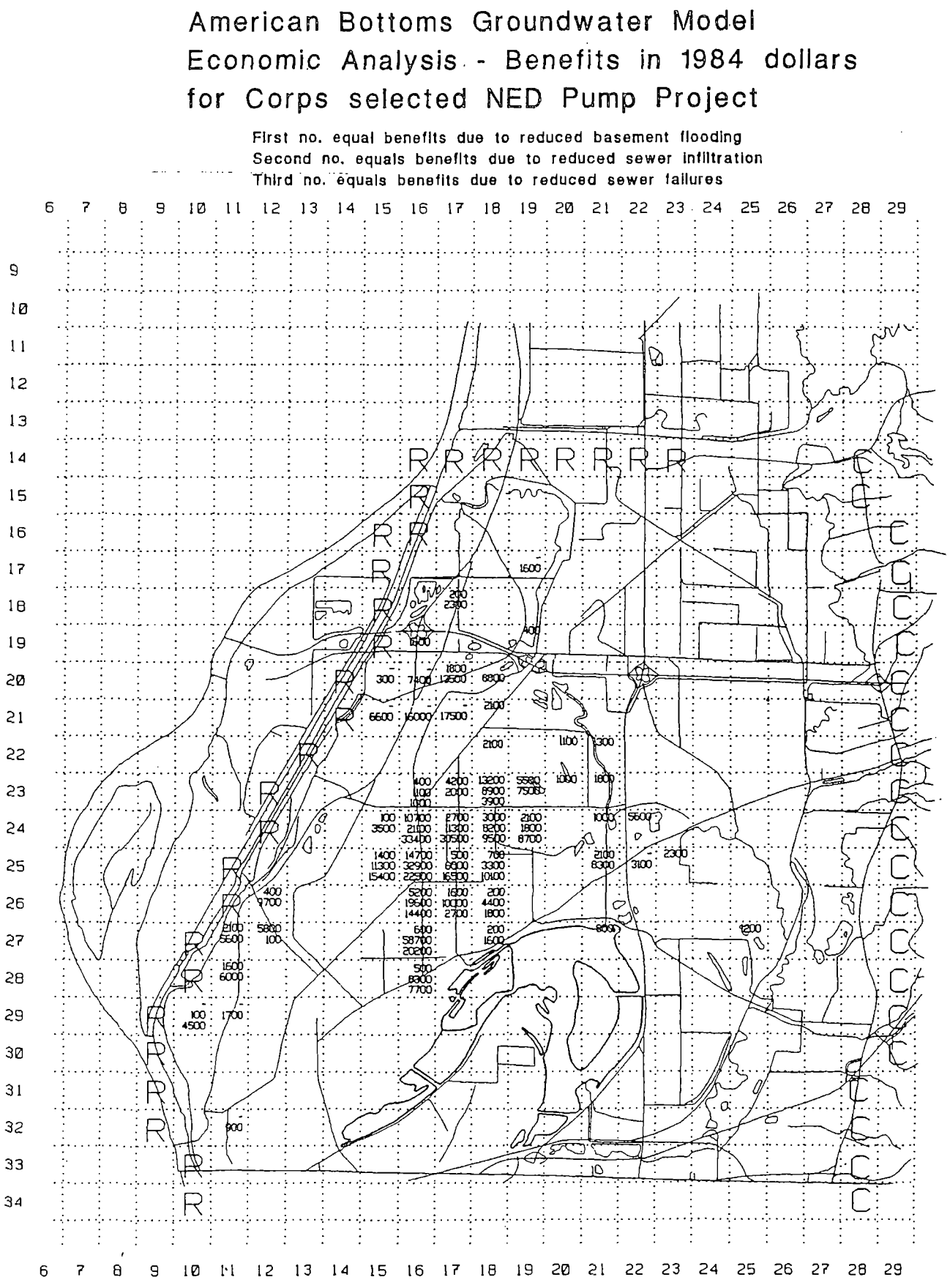


Figure 33. Distribution of average annual benefits for Corps of Engineers selected NED plan.

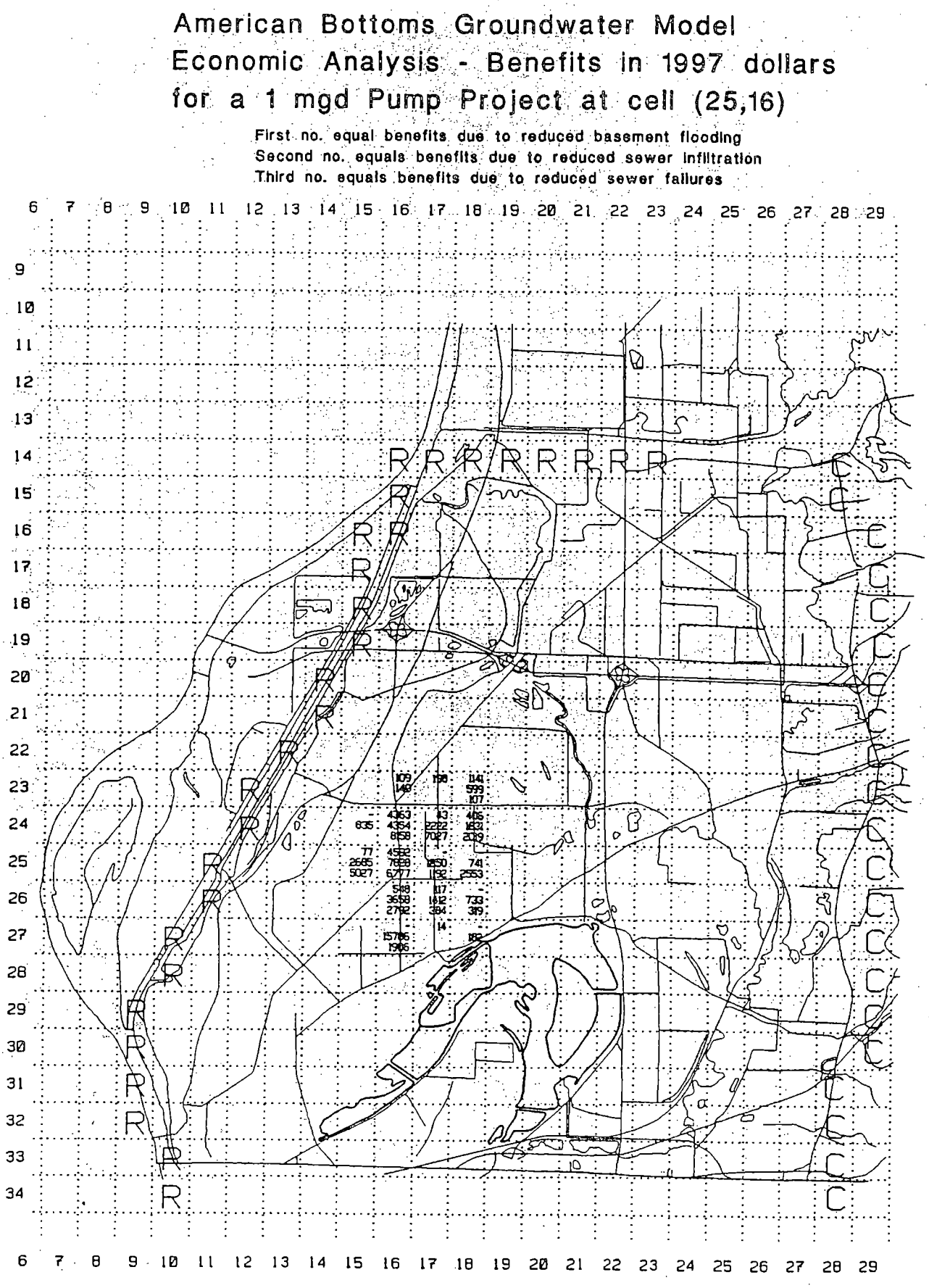


Figure 34. Distribution of average annual benefits for a one mgd pumping system at cell location (25,16).

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APPENDIX A

PLASM Groundwater Model Input Data.

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
A	1	22.E+04	2.6E+06	390	0	6.70E+03	400	395	1.0E+07	400	2700	320	0.0	-4.7E+04
A	2	22.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	2700	320	0.0	-4.7E+04
A	3	16.E+04	2.6E+06	395	0	1.06E+07	395	369	1.0E+07	369	3296	320	169.6	-5.7E+04
A	4	11.E+04	2.6E+06	395	0	1.06E+07	395	370	1.0E+07	370	2170	320	170.1	-3.7E+04
A	5	11.E+04	2.6E+06	395	0	1.06E+07	395	370	1.0E+07	370	2170	320	170.6	
A	6	11.E+04	2.6E+06	395	0	1.06E+07	395	370	1.0E+07	370	2170	320	170.9	
A	7	00.E+00	1.0E+20	420	0	0.00E+00	430	420	1.0E+20	430	0	300	0.0	
A	8	00.E+00	1.0E+20	420	0	0.00E+00	430	420	1.0E+20	430	0	300	0.0	
A	9	00.E+00	1.0E+20	420	0	0.00E+00	430	420	1.0E+20	430	0	300	0.0	
B	1	25.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	2050	280	0.0	-1.5E+04
B	2	22.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	2700	320	0.0	
B	3	22.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	2700	320	0.0	
B	4	22.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	2700	320	0.0	
B	5	22.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	2700	320	0.0	
B	6	22.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	2700	320	0.0	
B	7	11.E+04	2.6E+06	395	0	1.06E+07	395	370	1.0E+07	370	2170	320	171.7	
B	8	11.E+04	2.6E+06	395	0	1.06E+07	395	370	1.0E+07	370	2170	320	172.0	
B	9	11.E+04	2.6E+06	395	0	1.06E+07	395	370	1.0E+07	370	2170	320	172.3	
B	10	00.E+00	1.0E+20	420	0	0.00E+00	430	420	1.0E+20	430	0	300	0.0	
C	1	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	-1.5E+04
C	2	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
C	3	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
C	4	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
C	5	25.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	1637	320	0.0	
C	6	13.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	1637	320	0.0	
C	7	13.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	1637	320	0.0	
C	8	13.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	1637	320	0.0	
C	9	13.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	1637	320	0.0	
C	10	13.E+04	2.6E+06	395	0	1.06E+07	395	371	1.0E+07	371	2142	320	172.9	
C	11	11.E+04	1.0E+20	420	0	0.00E+00	430	420	1.0E+20	430	0	300	0.0	
C	12	00.E+00	1.0E+20	420	0	0.00E+00	430	420	1.0E+20	430	0	300	0.0	
D	1	00.E+00	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	-1.5E+04
D	2	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
D	3	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
D	4	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
D	5	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
D	6	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
D	7	13.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	1637	320	0.0	
D	8	13.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	1637	320	0.0	
D	9	13.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	1637	320	0.0	
D	10	13.E+04	2.6E+06	390	0	6.70E+03	410	385	1.0E+07	410	1637	320	0.0	
D	11	11.E+04	2.6E+06	395	0	1.06E+07	405	372	1.0E+07	372	2115	320	173.9	
D	12	11.E+04	2.6E+06	395	0	1.06E+07	395	372	1.0E+07	372	2115	320	173.9	
E	1	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	-1.5E+04
E	2	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
E	3	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
E	4	33.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2758	280	0.0	
E	5	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
E	6	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
E	7	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
E	8	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	
E	9	13.E+04	2.6E+06	390	50000	1.00E+04	405	385	1.0E+07	405	1637	320	0.0	
E	10	13.E+04	2.6E+06	390	50000	1.00E+04	405	385	1.0E+07	405	1637	320	0.0	
E	11	13.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	1637	320	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
E	12	11.E+04	2.6E+06	395	0	1.06E+07	407	372	1.0E+07	372	2115	320	174.5	
E	13	11.E+04	2.6E+06	395	0	1.06E+07	407	372	1.0E+07	372	2115	320	174.8	
E	14	11.E+04	2.6E+06	395	0	1.06E+07	395	372	1.0E+07	372	2115	320	175.1	
E	15	00.E+00	1.0E+20	420	0	0.00E+00	430	420	1.0E+20	430	0	300	0.0	
F	1	23.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	2235	300	0.0	-1.4E+04
F	2	23.E+04	2.6E+06	390	50000	1.00E+04	405	390	1.0E+07	405	2235	300	0.0	
F	3	23.E+04	2.6E+06	390	50000	1.00E+04	405	390	1.0E+07	405	2235	300	0.0	
F	4	33.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2758	280	0.0	
F	5	33.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2758	280	0.0	
F	6	33.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2758	280	0.0	
F	7	33.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2758	280	0.0	
F	8	33.E+04	2.6E+06	390	50000	1.00E+04	405	385	1.0E+07	405	2758	280	0.0	
F	9	25.E+04	2.6E+06	390	50000	1.00E+04	405	385	1.0E+07	405	2050	280	0.0	
F	10	22.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	2585	320	0.0	
F	11	22.E+04	2.6E+06	395	0	6.70E+03	405	390	1.0E+07	405	2585	320	0.0	
F	12	13.E+04	2.6E+06	390	0	6.70E+03	407	390	1.0E+07	407	1637	320	0.0	
F	13	13.E+04	2.6E+06	390	0	6.70E+03	408	390	1.0E+07	408	1637	320	0.0	
F	14	87.0E+3	2.6E+06	395	0	1.06E+07	409	372	1.0E+07	372	1681	320	175.4	
F	15	87.0E+3	2.6E+06	395	0	1.06E+07	395	372	1.0E+07	372	1681	320	175.4	
G	1	22.E+02	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	370	0.0	
G	2	22.E+02	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	370	0.0	
G	3	15.E+04	2.6E+06	395	50000	1.00E+04	425	390	1.0E+07	425	1428	300	0.0	
G	4	32.E+04	2.6E+06	395	50000	1.00E+04	415	390	1.0E+07	415	3045	300	0.0	
G	5	32.E+04	2.6E+06	390	50000	1.00E+04	418	390	1.0E+07	418	3045	300	0.0	
G	6	32.E+04	2.6E+06	390	50000	1.00E+04	418	390	1.0E+07	418	3045	300	0.0	
G	7	32.E+04	2.6E+06	390	50000	1.00E+04	410	385	1.0E+07	410	3160	300	0.0	
G	8	32.E+04	2.6E+06	390	50000	1.00E+04	408	385	1.0E+07	408	3160	300	0.0	
G	9	33.E+04	2.6E+06	390	50000	1.00E+04	408	385	1.0E+07	408	2758	280	0.0	
G	10	25.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	1998	280	0.0	
G	11	22.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	2585	320	0.0	
G	12	22.E+04	2.6E+06	395	50000	1.00E+04	410	390	1.0E+07	410	2585	320	0.0	
G	13	13.E+04	2.6E+06	390	0	6.70E+04	408	372	1.0E+07	372	1637	320	175.3	
G	14	22.E+04	2.6E+06	395	0	6.70E+03	407	390	1.0E+07	407	2585	320	0.0	
G	15	87.0E+3	2.6E+06	395	0	1.06E+07	415	372	1.0E+07	372	1681	320	176.1	
G	16	87.0E+3	2.6E+06	395	0	1.06E+07	395	372	1.0E+07	372	1681	320	176.4	
G	17	00.E+00	1.0E+20	430	0	0.00E+00	430	420	1.0E+20	430	0	320	0.0	
H	3	22.E+02	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	380	0.0	
H	4	22.E+02	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	380	0.0	
H	5	32.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	3045	300	0.0	
H	6	32.E+04	2.6E+06	395	50000	1.00E+04	415	395	1.0E+07	415	2940	300	0.0	
H	7	32.E+04	2.6E+06	390	50000	1.00E+04	410	395	1.0E+07	410	2940	300	0.0	
H	8	32.E+04	2.6E+06	390	50000	1.00E+04	408	395	1.0E+07	408	2940	300	0.0	
H	9	32.E+04	2.6E+06	390	50000	1.00E+04	408	390	1.0E+07	408	3045	300	0.0	
H	10	25.E+04	2.6E+06	395	50000	1.00E+04	416	390	1.0E+07	416	1998	280	0.0	
H	11	25.E+04	2.6E+06	395	50000	1.00E+04	410	390	1.0E+07	410	1998	280	0.0	
H	12	22.E+04	2.6E+06	400	0	1.00E+04	404	372	1.0E+07	372	2585	320	175.1	
H	13	13.E+04	2.6E+06	395	0	6.70E+03	406	390	1.0E+07	405	1585	320	0.0	
H	14	12.E+04	2.6E+06	395	0	6.70E+03	406	390	1.0E+07	405	1420	320	0.0	
H	15	12.E+04	2.6E+06	395	0	6.70E+03	408	390	1.0E+07	405	1420	320	0.0	
H	16	87.0E+3	2.6E+06	395	0	1.06E+07	409	372	1.0E+07	372	1681	320	176.9	
H	17	87.0E+3	2.6E+06	395	0	1.06E+07	409	372	1.0E+07	372	1681	320	177.4	
I	5	22.E+02	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	380	0.0	
I	6	30.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	3585	320	0.0	
I	7	30.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	3585	320	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
I	8	30.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	3585	320	0.0	
I	9	30.E+04	2.6E+06	395	50000	1.00E+04	415	390	1.0E+07	415	3585	320	0.0	
I	10	23.E+04	2.6E+06	395	50000	1.00E+04	415	390	1.0E+07	415	2235	300	0.0	
I	11	25.E+04	2.6E+06	400	0	1.00E+05	407	372	1.0E+07	372	1998	280	175.4	
I	12	25.E+04	2.6E+06	395	50000	1.00E+04	402	390	1.0E+07	402	1998	280	0.0	
I	13	25.E+04	2.6E+06	395	50000	1.00E+04	408	390	1.0E+07	408	1998	280	0.0	
I	14	25.E+04	2.6E+06	395	50000	1.00E+04	407	390	1.0E+07	407	1998	280	0.0	
I	15	15.E+04	2.6E+06	395	0	6.70E+03	405	390	1.0E+07	405	1206	280	0.0	
I	16	15.E+04	2.6E+06	395	0	6.70E+03	409	390	1.0E+07	409	1206	280	0.0	
I	17	15.E+04	2.6E+06	395	0	6.70E+03	418	390	1.0E+07	418	1206	280	0.0	
I	18	87.0E+3	2.6E+06	395	0	5.30E+06	395	372	1.0E+07	372	1681	320	177.2	
I	19	87.0E+3	2.6E+06	395	0	5.30E+06	395	372	1.0E+07	372	1681	320	177.5	
I	28	04.E+02	2.6E+06	395	0	1.06E+07	395	375	1.0E+07	375	10	340	183.5	
I	29	04.E+02	2.6E+06	395	0	1.06E+07	395	375	1.0E+07	375	10	340	183.8	
I	30	91.0E+3	2.6E+06	395	0	1.06E+07	395	376	1.0E+07	376	2525	340	184.4	
I	31	91.0E+3	2.6E+06	395	0	1.06E+07	395	376	1.0E+07	376	2525	340	184.7	
I	34	22.0E+3	2.6E+06	400	0	6.70E+03	405	390	1.0E+07	388	458	340	187.7	
I	35	22.0E+3	2.6E+06	400	0	6.70E+03	405	390	1.0E+07	388	458	340	187.9	
I	36	22.0E+3	2.6E+06	400	0	6.70E+03	405	390	1.0E+07	388	458	340	188.3	
I	37	00.E+00	1.0E+20	430	0	0.00E+00	430	420	1.0E+20	430	0	320	0.0	
I	44	00.E+00	1.0E+20	430	0	0.00E+00	430	420	1.0E+20	430	0	320	0.0	
I	45	00.E+00	1.0E+20	430	0	0.00E+00	430	420	1.0E+20	430	0	320	0.0	
I	46	00.E+00	1.0E+20	430	0	0.00E+00	430	420	1.0E+20	430	0	320	0.0	
J	6	00.E+00	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	380	0.0	
J	7	22.E+04	2.6E+06	395	50000	1.00E+04	415	390	1.0E+07	415	2585	320	0.0	
J	8	22.E+04	2.6E+06	395	50000	1.00E+04	415	390	1.0E+07	415	2585	320	0.0	
J	9	22.E+04	2.6E+06	395	50000	1.00E+04	415	390	1.0E+07	415	2585	320	0.0	
J	10	22.E+04	2.6E+06	395	50000	1.00E+04	415	390	1.0E+07	415	2585	320	0.0	
J	11	23.E+04	2.6E+06	405	0	1.00E+05	404	372	1.0E+07	372	2235	300	176.1	
J	12	25.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	1998	280	0.0	
J	13	25.E+04	2.6E+06	395	50000	1.00E+04	409	390	1.0E+07	409	1998	280	0.0	
J	14	25.E+04	2.6E+06	395	50000	1.00E+04	411	390	1.0E+07	411	1998	280	0.0	
J	15	25.E+04	2.6E+06	395	50000	1.00E+04	407	390	1.0E+07	407	1998	280	0.0	
J	16	24.E+04	2.6E+06	395	50000	1.00E+04	410	390	1.0E+07	410	1886	280	0.0	
J	17	24.E+04	2.6E+06	395	50000	1.00E+04	410	390	1.0E+07	410	1886	280	0.0	
J	18	24.E+04	2.6E+06	395	0	6.70E+03	414	390	1.0E+07	414	1886	280	0.0	
J	19	24.E+04	2.6E+06	395	0	6.70E+03	408	390	1.0E+07	408	1886	280	0.0	
J	20	82.0E+3	2.6E+06	395	0	1.06E+07	414	373	1.0E+07	373	2471	340	178.7	
J	21	82.0E+3	2.6E+06	395	0	1.06E+07	400	373	1.0E+07	373	2471	340	179.1	
J	22	82.0E+3	2.6E+06	395	0	1.06E+07	395	373	1.0E+07	373	2471	340	179.5	
J	23	82.0E+3	2.6E+06	395	0	1.06E+07	395	373	1.0E+07	373	2471	340	180.0	
J	24	82.0E+3	2.6E+06	395	0	1.06E+07	395	373	1.0E+07	373	2471	340	180.4	
J	25	62.0E+3	2.6E+06	395	0	1.06E+07	395	373	1.0E+07	373	2471	340	180.9	
J	26	86.0E+3	2.6E+06	395	0	1.06E+07	395	374	1.0E+07	374	2525	340	181.5	
J	27	57.0E+3	2.6E+06	395	0	1.06E+07	395	374	1.0E+07	374	1672	340	182.2	
J	28	12.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	1383	320	0.0	
J	29	12.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	1383	320	0.0	
J	30	12.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	1764	340	0.0	
J	31	12.E+04	2.6E+06	400	50000	1.00E+04	413	395	1.0E+07	413	1764	340	0.0	
J	32	92.0E+3	2.6E+06	395	0	5.30E+06	413	377	1.0E+07	377	2477	340	185.5	
J	33	92.0E+3	2.6E+06	395	0	5.30E+06	395	377	1.0E+07	377	2477	340	185.8	
J	34	13.E+04	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	1696	340	0.0	
J	35	42.0E+3	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	563	340	0.0	
J	36	97.0E+3	2.6E+06	400	0	6.70E+03	415	390	1.0E+07	415	655	340	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
J	37	97.0E+3	2.6E+06	400	0	6.70E+03	415	390	1.0E+07	415	655	340	0.0	
J	38	00.E+00	1.0E+20	430	0	0.00E+00	430	420	1.0E+20	430	0	320	0.0	
J	39	00.E+00	1.0E+20	430	0	0.00E+00	430	420	1.0E+20	430	0	320	0.0	
J	40	00.E+00	1.0E+20	430	0	0.00E+00	430	420	1.0E+20	430	0	320	0.0	
J	43	00.E+00	1.0E+20	430	0	0.00E+00	430	420	1.0E+20	430	0	320	0.0	
K	7	22.E+02	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	380	0.0	
K	8	13.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	1585	320	0.0	
K	9	13.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	1585	320	0.0	
K	10	13.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	1585	320	0.0	
K	11	23.E+04	2.6E+06	405	0	1.00E+05	403	372	1.0E+07	372	2235	300	177.2	
K	12	25.E+04	2.6E+06	395	50000	1.00E+04	407	390	1.0E+07	407	1998	280	0.0	
K	13	25.E+04	2.6E+06	395	50000	1.00E+04	409	390	1.0E+07	409	1998	280	0.0	
K	14	25.E+04	2.6E+06	395	50000	1.00E+04	411	390	1.0E+07	411	1998	280	0.0	
K	15	33.E+04	2.6E+06	395	50000	1.00E+04	407	390	1.0E+07	407	2678	280	0.0	
K	16	33.E+04	2.6E+06	395	50000	1.00E+04	408	390	1.0E+07	408	2678	280	0.0	
K	17	32.E+04	2.6E+06	395	50000	1.00E+04	411	390	1.0E+07	411	2566	280	0.0	
K	18	32.E+04	2.6E+06	394	50000	1.00E+04	409	389	1.0E+07	409	2580	280	0.0	
K	19	24.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	1886	280	0.0	
K	20	24.E+04	2.6E+06	395	50000	1.00E+04	416	390	1.0E+07	418	1886	280	0.0	
K	21	24.E+04	2.6E+06	395	50000	1.00E+04	416	390	1.0E+07	416	2491	320	0.0	
K	22	24.E+04	2.6E+06	395	50000	1.00E+04	417	390	1.0E+07	417	2491	320	0.0	
K	23	24.E+04	2.6E+06	395	50000	1.00E+04	415	390	1.0E+07	415	2491	320	0.0	
K	24	24.E+04	2.6E+06	395	50000	1.00E+04	420	390	1.0E+07	420	2525	320	0.0	
K	25	24.E+04	2.6E+06	395	50000	1.00E+04	423	390	1.0E+07	423	2525	320	0.0	
K	26	21.E+04	2.6E+06	400	50000	1.00E+04	419	395	1.0E+07	419	2327	320	0.0	
K	27	12.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	1383	320	0.0	
K	28	12.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	1383	320	0.0	
K	29	12.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	1383	320	0.0	
K	30	14.E+04	2.6E+06	400	50000	1.00E+04	413	395	1.0E+07	413	1538	320	0.0	
K	31	14.E+04	2.6E+06	400	50000	1.00E+04	411	395	1.0E+07	411	1538	320	0.0	
K	32	14.E+04	2.6E+06	400	50000	1.00E+04	413	395	1.0E+07	413	1538	320	0.0	
K	33	14.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	1538	320	0.0	
K	34	96.0E+3	2.6E+06	395	0	3.57E+07	420	380	1.0E+07	378	2525	340	194.0	
K	35	10.E+04	2.6E+06	405	0	3.57E+07	405	390	1.0E+07	380	2525	340	194.5	
K	36	42.0E+3	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	563	340	0.0	
K	37	42.0E+3	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	563	340	0.0	
K	38	00.E+00	1.0E+20	415	0	0.00E+00	415	410	1.0E+20	415	0	320	0.0	
K	41	00.E+00	1.0E+20	430	0	0.00E+00	430	420	1.0E+20	430	0	320	0.0	
K	42	00.E+00	1.0E+20	430	0	0.00E+00	430	420	1.0E+20	430	0	320	0.0	
K	46	31.0E+3	2.6E+06	410	0	6.70E+03	420	405	1.0E+07	420	0	360	0.0	
L	8	17.E+02	1.0E+20	400	0	0.00E+00	550	540	1.0E+20	550	10	380	0.0	
L	9	13.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	1585	320	0.0	
L	10	13.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	1585	320	0.0	
L	11	15.E+04	2.6E+06	410	0	1.00E+05	406	373	1.0E+07	373	1426	300	178.2	
L	12	25.E+04	2.6E+06	395	50000	1.00E+04	411	390	1.0E+07	411	1998	280	0.0	
L	13	25.E+04	2.6E+06	395	50000	1.00E+04	410	390	1.0E+07	410	1996	280	0.0	
L	14	25.E+04	2.6E+06	395	50000	1.00E+04	410	390	1.0E+07	410	1998	280	0.0	
L	15	33.E+04	2.6E+06	395	50000	1.00E+04	411	390	1.0E+07	411	2678	280	0.0	
L	16	34.E+04	2.6E+06	400	50000	1.00E+04	409	395	1.0E+07	409	2603	280	0.0	
L	17	34.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	2603	280	0.0	
L	18	34.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	2603	280	0.0	
L	19	34.E+04	2.6E+06	400	50000	1.00E+04	409	395	1.0E+07	409	2603	280	0.0	
L	20	34.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	2603	280	0.0	
L	21	31.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	3427	320	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
L	22	31.E+04	2.6E+06	400	50000	1.00E+04	416	395	1.0E+07	416	3427	320	0.0	
L	23	31.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	3427	320	0.0	
L	24	27.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	3020	320	0.0	
L	25	27.E+04	2.6E+06	400	50000	1.00E+04	409	395	1.0E+07	409	3020	320	0.0	
L	26	27.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	3020	320	0.0	
L	27	27.E+04	2.6E+06	400	50000	1.00E+04	416	395	1.0E+07	416	3020	320	0.0	
L	28	12.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	1383	320	0.0	
L	29	12.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	1383	320	0.0	
L	30	21.E+04	2.6E+06	405	50000	1.00E+04	414	400	1.0E+07	414	2244	320	0.0	
L	31	23.E+04	2.6E+06	405	50000	1.00E+04	416	400	1.0E+07	418	2392	320	0.0	
L	32	23.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	2392	320	0.0	
L	33	23.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	2392	320	0.0	
L	34	23.E+04	2.6E+06	405	50000	1.00E+04	414	400	1.0E+07	414	2392	320	0.0	
L	35	23.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	2392	320	0.0	
L	36	12.E+04	2.6E+06	405	0	3.57E+07	418	382	1.0E+07	382	1895	320	194.0	
L	37	12.E+04	2.6E+06	405	0	3.57E+07	405	382	1.0E+07	382	1895	320	194.0	
L	38	42.0E+3	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	563	340	0.0	
L	39	42.0E+3	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	563	340	0.0	
L	40	42.0E+3	2.6E+06	405	0	8.70E+03	415	400	1.0E+07	415	563	340	0.0	
L	41	42.0E+3	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	563	360	0.0	
L	42	20.0E+3	2.6E+06	405	0	6.70E+03	405	386	1.0E+07	386	445	340	192.1	
L	43	31.0E+3	2.6E+06	410	0	6.70E+03	420	405	1.0E+07	420	516	360	0.0	
L	44	16.0E+3	2.6E+06	410	0	6.70E+03	420	405	1.0E+07	420	400	380	0.0	
L	45	31.0E+3	2.6E+06	410	0	6.70E+03	420	405	1.0E+07	420	516	360	0.0	
L	46	12.E+04	2.6E+06	410	0	6.70E+03	420	405	1.0E+07	420	0	360	0.0	
M	9	00.E+00	1.0E+20	400	0	0.00E+00	550	540	1.0E+20	550	0.001	380	0.0	
M	10	00.E+00	2.6E+06	400	67000	6.70E+03	450	430	1.0E+07	450	10	360	0.0	
M	11	15.E+04	2.6E+06	410	0	1.00E+05	409	373	1.0E+07	373	1426	300	178.7	
M	12	23.E+04	2.6E+06	395	50000	1.00E+04	410	390	1.0E+07	410	2235	300	0.0	
M	13	25.E+04	2.6E+06	395	50000	1.00E+04	410	390	1.0E+07	410	1998	280	0.0	
M	14	25.E+04	2.6E+06	395	50000	1.00E+04	408	390	1.0E+07	408	1998	280	0.0	
M	15	34.E+04	2.6E+06	400	50000	1.00E+04	408	395	1.0E+07	408	2603	280	0.0	
M	16	34.E+04	2.6E+06	400	50000	1.00E+04	409	395	1.0E+07	409	2603	280	0.0	
M	17	34.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	2603	280	0.0	
M	18	34.E+04	2.6E+06	400	50000	1.00E+04	411	395	1.0E+07	411	2603	280	0.0	
M	19	25.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	1950	280	0.0	
M	20	34.E+04	2.6E+06	400	50000	1.00E+04	413	395	1.0E+07	413	2603	280	0.0	
M	21	34.E+04	2.6E+06	400	50000	1.00E+04	415	395	1.0E+07	415	2603	280	0.0	
M	22	34.E+04	2.6E+06	400	50000	1.00E+04	417	395	1.0E+07	417	2603	280	0.0	
M	23	31.E+04	2.6E+06	400	50000	1.00E+04	418	395	1.0E+07	418	3427	320	0.0	
M	24	27.E+04	2.6E+06	400	50000	1.00E+04	417	395	1.0E+07	417	3020	320	0.0	
M	25	27.E+04	2.6E+06	400	50000	1.00E+04	407	395	1.0E+07	407	3020	320	0.0	
M	26	27.E+04	2.6E+06	400	50000	1.00E+04	408	395	1.0E+07	408	3020	320	0.0	
M	27	27.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	3020	320	0.0	
M	28	21.E+04	2.6E+06	400	50000	1.00E+04	411	395	1.0E+07	411	2327	320	0.0	
M	29	21.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	2327	320	0.0	
M	30	23.E+04	2.6E+06	405	50000	1.00E+04	413	400	1.0E+07	413	2392	320	0.0	
M	31	23.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	2392	320	0.0	
M	32	23.E+04	2.6E+06	405	50000	1.00E+04	422	400	1.0E+07	422	2392	320	0.0	
M	33	23.E+04	2.6E+06	405	50000	1.00E+04	420	400	1.0E+07	420	2392	320	0.0	
M	34	23.E+04	2.6E+06	405	50000	6.00E+03	420	400	1.0E+07	420	2392	320	0.0	
M	35	22.E+04	2.6E+06	405	50000	6.00E+03	420	390	1.0E+07	420	2585	320	0.0	
M	36	22.E+04	2.6E+06	405	50000	6.00E+03	424	390	1.0E+07	424	2585	320	0.0	
M	37	22.E+04	2.6E+06	405	50000	6.00E+03	421	390	1.0E+07	421	2585	320	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
M	38	12.E+04	2.6E+06	405	0	3.57E+07	405	384	1.0E+07	384	1859	320	194.0	
M	39	14.E+04	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	1497	320	0.0	
M	40	13.E+04	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	1696	340	0.0	
M	41	42.0E+3	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	563	340	0.0	
M	42	42.0E+3	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	563	340	0.0	
M	43	20.0E+3	2.6E+06	405	0	7.14E+07	405	386	1.0E+07	386	445	340	192.1	
M	44	12.E+04	2.6E+06	410	0	6.70E+03	420	405	1.0E+07	420	1933	360	0.0	
M	45	12.E+04	2.6E+06	410	0	6.70E+03	420	405	1.0E+07	420	1933	360	0.0	
M	46	12.E+04	2.6E+06	410	0	6.70E+03	420	405	1.0E+07	420	0	360	0.0	
N	10	00.E+00	1.0E+20	400	0	0.00E+00	475	465	1.0E+20	475	0.001	380	0.0	
N	11	15.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	1426	300	0.0	
N	12	23.E+04	2.6E+06	395	50000	1.00E+04	410	390	1.0E+07	410	2235	300	0.0	
N	13	23.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	2235	300	0.0	
N	14	24.E+04	2.6E+06	400	50000	1.00E+04	405	395	1.0E+07	405	2168	300	0.0	
N	15	24.E+04	2.6E+06	400	50000	1.00E+04	407	395	1.0E+07	407	2168	300	0.0	
N	16	25.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	1950	280	0.0	
N	17	25.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	1950	280	0.0	
N	18	25.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	1950	280	0.0	
N	19	25.E+04	2.6E+06	400	50000	1.00E+04	415	395	1.0E+07	415	1950	280	0.0	
N	20	25.E+04	2.6E+06	400	50000	1.00E+04	416	395	1.0E+07	416	1950	280	0.0	
N	21	25.E+04	2.6E+06	400	50000	1.00E+04	417	395	1.0E+07	417	1950	280	0.0	
N	22	25.E+04	2.6E+06	400	50000	1.00E+04	420	395	1.0E+07	420	1950	280	0.0	
N	23	22.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	2483	320	0.0	
N	24	22.E+04	2.6E+06	400	50000	1.00E+04	416	395	1.0E+07	416	2483	320	0.0	
N	25	21.E+04	2.6E+06	400	50000	1.00E+04	408	395	1.0E+07	408	2327	320	0.0	
N	26	21.E+04	2.6E+06	400	50000	1.00E+04	408	395	1.0E+07	408	2327	320	0.0	
N	27	21.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	2327	320	0.0	
N	28	21.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	2327	320	0.0	
N	29	21.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	2327	320	0.0	
N	30	23.E+04	2.6E+06	405	50000	1.00E+04	413	400	1.0E+07	413	2392	320	0.0	
N	31	23.E+04	2.6E+06	410	0	2.00E+04	412	400	1.0E+07	412	2310	320	0.0	
N	32	23.E+04	2.6E+06	410	0	2.00E+04	422	415	1.0E+07	422	2310	320	0.0	
N	33	23.E+04	2.6E+06	410	0	2.00E+04	420	415	1.0E+07	420	2310	320	0.0	
N	34	23.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	2392	320	0.0	
N	35	23.E+04	2.6E+06	405	50000	1.00E+04	422	400	1.0E+07	422	2392	320	0.0	
N	36	23.E+04	2.6E+06	405	50000	1.00E+04	423	400	1.0E+07	423	2392	320	0.0	
N	37	23.E+04	2.6E+06	405	50000	1.00E+04	424	400	1.0E+07	424	2392	320	0.0	
N	38	23.E+04	2.6E+06	405	50000	1.00E+04	423	400	1.0E+07	423	2392	320	0.0	
N	39	21.E+04	2.6E+06	405	0	3.57E+07	405	390	1.0E+07	388	3044	320	194.0	
N	40	26.E+04	2.6E+06	405	0	3.57E+07	405	388	1.0E+07	388	3762	320	194.0	
N	41	14.E+04	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	1497	320	0.0	
N	42	13.E+04	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	1696	340	0.0	
N	43	13.E+04	2.6E+06	405	0	6.70E+03	415	400	1.0E+07	415	1696	340	0.0	
N	44	11.E+04	2.6E+06	405	0	7.14E+07	405	388	1.0E+07	388	2229	340	192.8	
N	45	13.E+04	2.6E+06	410	0	6.70E+03	420	405	1.0E+07	420	1637	340	0.0	
N	46	13.E+04	2.6E+06	410	0	6.70E+03	420	405	1.0E+07	420	0	340	0.0	
O	10	00.E+00	1.0E+20	400	0	0.00E+00	475	465	1.0E+20	475	0.001	380	0.0	
O	11	14.E+04	2.6E+06	400	50000	1.00E+04	409	395	1.0E+07	409	1268	300	0.0	
O	12	22.E+04	2.6E+06	395	50000	1.00E+04	406	390	1.0E+07	406	2102	300	0.0	
O	13	22.E+04	2.6E+06	400	50000	1.00E+04	404	395	1.0E+07	404	2040	300	0.0	
O	14	22.E+04	2.6E+06	400	50000	1.00E+04	407	395	1.0E+07	407	2040	300	0.0	
O	15	22.E+04	2.6E+06	400	50000	1.00E+04	406	395	1.0E+07	406	2040	300	0.0	
O	16	22.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	2040	300	0.0	
O	17	22.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	410	2040	300	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
O	18	24.E+04	2.6E+06	400	50000	1.00E+04	418	395	1.0E+07	418	1842	280	0.0	
O	19	25.E+04	2.6E+06	400	50000	1.00E+04	413	395	1.0E+07	413	1950	280	0.0	
O	20	25.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	1950	280	0.0	
O	21	26.E+04	2.6E+06	405	50000	1.00E+04	416	400	1.0E+07	416	1950	280	0.0	
O	22	26.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	1950	280	0.0	
O	23	26.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	1950	280	0.0	
O	24	24.E+04	2.6E+06	400	50000	1.00E+04	403	395	1.0E+07	403	2168	300	0.0	
O	25	24.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	2168	300	0.0	
O	26	22.E+04	2.6E+06	400	50000	1.00E+04	409	395	1.0E+07	409	2040	300	0.0	
O	27	22.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	2040	300	0.0	
O	28	22.E+04	2.6E+06	400	0	1.20E+05	408	398	1.0E+07	398	2198	300	0.0	
O	29	22.E+04	2.6E+06	400	50000	1.00E+04	408	395	1.0E+07	408	2040	300	0.0	
O	30	23.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	1984	300	0.0	
O	31	23.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	2392	320	0.0	
O	32	23.E+04	2.6E+06	410	0	2.00E+04	419	415	1.0E+07	419	2310	320	0.0	
O	33	32.E+04	2.6E+06	410	0	2.00E+04	421	415	1.0E+07	421	3160	320	0.0	
O	34	32.E+04	2.6E+06	410	0	2.00E+04	419	415	1.0E+07	419	3160	320	0.0	
O	35	32.E+04	2.6E+06	410	0	2.00E+04	420	415	1.0E+07	420	3160	320	0.0	
O	36	32.E+04	2.6E+06	410	0	2.00E+04	420	415	1.0E+07	420	3160	320	0.0	
O	37	32.E+04	2.6E+06	410	0	2.00E+04	424	415	1.0E+07	424	3160	320	0.0	
O	38	31.E+04	2.6E+06	405	50000	1.00E+04	423	400	1.0E+07	423	3286	320	0.0	
O	39	31.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	3286	320	0.0	
O	40	31.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	3286	320	0.0	
O	41	26.E+04	2.6E+06	405	0	3.57E+07	415	388	1.0E+07	388	3762	320	194.0	
O	42	26.E+04	2.6E+06	405	0	3.57E+07	405	388	1.0E+07	388	3762	320	194.0	
O	43	12.E+04	2.6E+06	405	0	7.14E+07	405	388	1.0E+07	388	1794	320	192.8	
O	44	12.E+04	2.6E+06	405	0	7.14E+07	405	388	1.0E+07	388	1794	320	193.5	
O	45	15.E+04	2.6E+06	410	0	6.70E+03	420	405	1.0E+07	420	1460	320	0.0	
O	46	15.E+04	2.6E+06	410	0	6.70E+03	420	405	1.0E+07	420	0	320	0.0	
P	10	00.E+00	1.0E+20	400	0	0.00E+00	475	465	1.0E+20	475	0.001	380	0.0	
P	11	12.E+04	2.6E+06	400	50000	1.00E+04	413	395	1.0E+07	413	1383	320	0.0	
P	12	22.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	2040	300	0.0	
P	13	22.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	2040	300	0.0	
P	14	22.E+04	2.6E+06	400	50000	1.00E+04	411	395	1.0E+07	411	2040	300	0.0	
P	15	22.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	2040	300	0.0	
P	16	22.E+04	2.6E+06	400	50000	1.00E+04	415	395	1.0E+07	415	2040	300	0.0	
P	17	22.E+04	2.6E+06	400	50000	1.00E+04	417	395	1.0E+07	417	2040	300	0.0	
P	18	24.E+04	2.6E+06	405	50000	1.00E+04	421	400	1.0E+07	421	1801	280	0.0	
P	19	26.E+04	2.6E+06	405	50000	1.00E+04	423	400	1.0E+07	423	1905	280	0.0	
P	20	26.E+04	2.6E+06	405	50000	1.00E+04	416	400	1.0E+07	416	1905	280	0.0	
P	21	26.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	1905	280	0.0	
P	22	26.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	1905	280	0.0	
P	23	25.E+04	2.6E+06	400	50000	1.00E+04	418	395	1.0E+07	416	1950	280	0.0	
P	24	25.E+04	2.6E+06	400	50000	1.00E+04	404	395	1.0E+07	404	1950	280	0.0	
P	25	24.E+04	2.6E+06	400	50000	1.00E+04	409	395	1.0E+07	409	2168	300	0.0	
P	26	24.E+04	2.6E+06	400	50000	1.00E+04	416	395	1.0E+07	416	2168	300	0.0	
P	27	22.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	2040	300	0.0	
P	28	22.E+04	2.6E+06	400	50000	1.00E+04	411	395	1.0E+07	411	2040	300	0.0	
P	29	22.E+04	2.6E+06	400	0	1.20E+05	406	398	1.0E+07	398	2198	300	0.0	
P	30	22.E+04	2.6E+06	400	50000	1.00E+04	408	395	1.0E+07	408	2040	300	0.0	
P	31	23.E+04	2.6E+06	405	50000	1.00E+04	412	400	1.0E+07	412	1984	300	0.0	
P	32	24.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	2106	300	0.0	
P	33	33.E+04	2.6E+06	410	0	2.00E+04	420	415	1.0E+07	420	2758	300	0.0	
P	34	33.E+04	2.6E+06	410	0	2.00E+04	418	415	1.0E+07	418	2758	300	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
P	35	33.E+04	2.6E+06	410	0	2.00E+04	419	415	1.0E+07	419	2758	300	0.0	
P	36	33.E+04	2.6E+06	410	0	2.00E+04	420	415	1.0E+07	420	2758	300	0.0	
P	37	31.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	3286	320	0.0	
P	38	32.E+04	2.6E+06	410	0	2.00E+04	421	415	1.0E+07	421	3160	320	0.0	
P	39	31.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	3286	320	0.0	
P	40	31.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	3286	320	0.0	
P	41	31.E+04	2.6E+06	405	50000	1.00E+04	413	400	1.0E+07	413	3286	320	0.0	
P	42	31.E+04	2.6E+06	405	50000	1.00E+04	409	400	1.0E+07	409	3286	320	0.0	
P	43	31.E+04	2.6E+06	405	50000	1.00E+04	413	400	1.0E+07	413	3286	320	0.0	
P	44	26.E+04	2.6E+06	405	0	3.57E+07	414	388	1.0E+07	388	3762	320	194.0	
P	45	21.E+04	2.6E+06	405	0	3.57E+07	405	388	1.0E+07	388	3044	320	194.0	
P	46	12.E+04	2.6E+06	405	0	7.14E+07	405	388	1.0E+07	388	1794	320	193.5	
Q	10	00.E+00	1.0E+20	405	0	0.00E+00	475	465	1.0E+20	475	0.001	380	0.0	
Q	11	21.E+04	2.6E+06	400	50000	1.00E+04	405	395	1.0E+07	405	2327	320	0.0	
Q	12	21.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	2327	320	0.0	
Q	13	22.E+04	2.6E+06	400	50000	1.00E+04	415	395	1.0E+07	415	2040	300	0.0	
Q	14	22.E+04	2.6E+06	400	50000	1.00E+04	413	395	1.0E+07	413	2040	300	0.0	
Q	15	22.E+04	2.6E+06	400	50000	1.00E+04	415	395	1.0E+07	415	2040	300	0.0	
Q	16	23.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	1984	300	0.0	
Q	17	23.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	1984	300	0.0	
Q	18	23.E+04	2.6E+06	405	50000	1.00E+04	421	400	1.0E+07	421	1984	300	0.0	
Q	19	23.E+04	2.6E+06	405	50000	1.00E+04	433	400	1.0E+07	433	1984	300	0.0	
Q	20	24.E+04	2.6E+06	405	50000	1.00E+04	416	400	1.0E+07	416	1801	280	0.0	
Q	21	24.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	1801	280	0.0	
Q	22	26.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	1905	280	0.0	
Q	23	26.E+04	2.6E+06	405	50000	1.00E+04	416	400	1.0E+07	416	1905	280	0.0	
Q	24	25.E+04	2.6E+06	400	50000	1.00E+04	413	395	1.0E+07	413	1950	280	0.0	
Q	25	25.E+04	2.6E+06	400	50000	1.00E+04	402	395	1.0E+07	402	1950	280	0.0	
Q	26	24.E+04	2.6E+06	400	50000	1.00E+04	403	395	1.0E+07	403	2168	300	0.0	
Q	27	22.E+04	2.6E+06	400	50000	1.00E+04	411	395	1.0E+07	411	2040	300	0.0	
Q	28	22.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	2040	300	0.0	
Q	29	22.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	2040	300	0.0	
Q	30	22.E+04	2.6E+06	400	50000	1.00E+04	404	395	1.0E+07	404	2040	300	0.0	
Q	31	22.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2198	300	0.0	
Q	32	23.E+04	2.6E+06	405	50000	1.00E+04	420	400	1.0E+07	420	1984	300	0.0	
Q	33	23.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1984	300	0.0	
Q	34	32.E+04	2.6E+06	410	0	2.00E+04	420	415	1.0E+07	420	2641	300	0.0	
Q	35	32.E+04	2.6E+06	410	0	2.00E+04	422	415	1.0E+07	422	2641	300	0.0	
Q	36	32.E+04	2.6E+06	410	0	2.00E+04	422	415	1.0E+07	422	2641	300	0.0	
Q	37	32.E+04	2.6E+06	410	0	2.00E+04	420	415	1.0E+07	420	2641	300	0.0	
Q	38	24.E+04	2.6E+06	405	50000	1.00E+04	420	400	1.0E+07	420	2106	300	0.0	
Q	39	24.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	2106	300	0.0	
Q	40	24.E+04	2.6E+06	405	50000	1.00E+04	413	400	1.0E+07	413	2106	300	0.0	
Q	41	33.E+04	2.6E+06	410	67000	6.70E+03	413	395	1.0E+07	413	2758	300	0.0	
Q	42	33.E+04	2.6E+06	410	67000	6.70E+03	416	395	1.0E+07	416	2758	300	0.0	
Q	43	25.E+04	2.6E+06	410	67000	6.70E+03	414	395	1.0E+07	414	2050	300	0.0	
Q	44	16.E+04	2.6E+06	410	67000	6.70E+03	415	395	1.0E+07	415	1341	300	0.0	
Q	45	16.E+04	2.6E+06	410	0	6.70E+03	430	405	1.0E+07	430	1341	300	0.0	
Q	46	14.E+04	2.6E+06	405	0	7.14E+07	416	389	1.0E+07	389	1547	300	195.2	
R	11	00.E+00	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	0.001	380	0.0	
R	12	12.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	1383	320	0.0	
R	13	21.E+04	2.6E+06	400	50000	1.00E+04	415	395	1.0E+07	415	2327	320	0.0	
R	14	22.E+04	2.6E+06	400	50000	1.00E+04	416	395	1.0E+07	416	2040	300	0.0	
R	15	22.E+04	2.6E+06	400	50000	1.00E+04	415	395	1.0E+07	415	2040	300	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
R	16	22.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	2040	300	0.0	
R	17	23.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	1984	300	0.0	
R	18	23.E+04	2.6E+06	405	50000	1.00E+04	411	400	1.0E+07	411	1984	300	0.0	
R	19	23.E+04	2.6E+06	405	50000	1.00E+04	410	400	1.0E+07	410	1984	300	0.0	
R	20	24.E+04	2.6E+06	405	50000	1.00E+04	414	400	1.0E+07	414	1801	280	0.0	
R	21	24.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	1801	280	0.0	
R	22	26.E+04	2.6E+06	405	50000	1.00E+04	420	400	1.0E+07	420	1905	280	0.0	
R	23	26.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	1905	280	0.0	
R	24	26.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	1905	280	0.0	
R	25	25.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	1950	280	0.0	
R	26	25.E+04	2.6E+06	400	50000	1.00E+04	402	395	1.0E+07	402	1950	280	0.0	
R	27	24.E+04	2.6E+06	400	50000	1.00E+04	406	395	1.0E+07	406	2168	300	0.0	
R	28	22.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	2040	300	0.0	
R	29	22.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	2040	300	0.0	
R	30	22.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	2040	300	0.0	
R	31	22.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2198	300	0.0	
R	32	22.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2198	300	0.0	
R	33	23.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1984	300	0.0	
R	34	23.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1984	300	0.0	
R	35	23.E+04	2.6E+06	410	0	2.00E+04	422	415	1.0E+07	422	1933	300	0.0	
R	36	23.E+04	2.6E+06	410	0	2.00E+04	420	415	1.0E+07	420	1933	300	0.0	
R	37	23.E+04	2.6E+06	410	0	2.00E+04	419	415	1.0E+07	419	1933	300	0.0	
R	38	23.E+04	2.6E+06	410	0	2.00E+04	421	415	1.0E+07	421	1933	300	0.0	
R	39	25.E+04	2.6E+06	410	67000	6.70E+03	419	395	1.0E+07	419	2050	300	0.0	
R	40	25.E+04	2.6E+06	410	67000	6.70E+03	414	395	1.0E+07	414	2050	300	0.0	
R	41	25.E+04	2.6E+06	410	67000	6.70E+03	412	395	1.0E+07	412	2050	300	0.0	
R	42	25.E+04	2.6E+06	410	67000	6.70E+03	413	395	1.0E+07	413	2050	300	0.0	
R	43	25.E+04	2.6E+06	410	67000	6.70E+03	414	395	1.0E+07	414	2050	300	0.0	
R	44	25.E+04	2.6E+06	410	67000	6.70E+03	417	395	1.0E+07	417	2050	300	0.0	
R	45	25.E+04	2.6E+06	410	67000	6.70E+03	429	395	1.0E+07	429	2050	300	0.0	
R	46	25.E+04	2.6E+06	405	0	6.70E+04	430	389	1.0E+07	389	2050	300	195.6	
S	12	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	380	0.0	
S	13	12.E+04	2.6E+06	400	50000	1.00E+04	407	395	1.0E+07	407	1383	320	0.0	
S	14	12.E+04	2.6E+06	400	50000	1.00E+04	407	395	1.0E+07	407	1383	320	0.0	
S	15	12.E+04	2.6E+06	400	50000	1.00E+04	411	395	1.0E+07	411	1383	320	0.0	
S	16	14.E+04	2.6E+06	400	50000	1.00E+04	414	395	1.0E+07	414	1268	300	0.0	
S	17	22.E+04	2.6E+06	400	50000	1.00E+04	411	395	1.0E+07	411	2040	300	0.0	
S	18	22.E+04	2.6E+06	410	0	1.00E+05	412	405	1.0E+07	415	2102	300	0.0	
S	19	23.E+04	2.6E+06	405	50000	1.00E+04	413	400	1.0E+07	413	1984	300	0.0	
S	20	23.E+04	2.6E+06	405	50000	1.00E+04	414	400	1.0E+07	414	1984	300	0.0	
S	21	23.E+04	2.6E+06	405	50000	1.00E+04	412	400	1.0E+07	412	1984	300	0.0	
S	22	26.E+04	2.6E+06	405	50000	1.00E+04	422	400	1.0E+07	422	1905	280	0.0	
S	23	26.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	1905	280	0.0	
S	24	26.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1905	280	0.0	
S	25	26.E+04	2.6E+06	405	50000	1.00E+04	421	400	1.0E+07	421	1905	280	0.0	
S	26	25.E+04	2.6E+06	400	50000	1.00E+04	409	395	1.0E+07	409	1950	280	0.0	
S	27	25.E+04	2.6E+06	400	50000	1.00E+04	405	395	1.0E+07	405	1950	280	0.0	
S	28	25.E+04	2.6E+06	400	50000	1.00E+04	411	395	1.0E+07	411	1950	280	0.0	
S	29	24.E+04	2.6E+06	400	50000	1.00E+04	415	395	1.0E+07	415	1842	280	0.0	
S	30	24.E+04	2.6E+06	400	50000	1.00E+04	412	395	1.0E+07	412	1842	280	0.0	
S	31	24.E+04	2.6E+06	400	50000	1.00E+04	410	395	1.0E+07	410	1842	280	0.0	
S	32	23.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	1953	280	0.0	
S	33	23.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1984	300	0.0	
S	34	23.E+04	2.6E+06	405	50000	1.00E+04	420	400	1.0E+07	420	1984	300	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
S	35	23.E+04	2.6E+06	405	50000	1.00E+04	422	400	1.0E+07	422	1984	300	0.0	
S	36	23.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1984	300	0.0	
S	37	23.E+04	2.6E+06	410	67000	6.70E+03	417	395	1.0E+07	417	1933	300	0.0	
S	38	23.E+04	2.6E+06	410	67000	6.70E+03	420	395	1.0E+07	420	1933	300	0.0	
S	39	23.E+04	2.6E+06	410	67000	6.70E+03	415	395	1.0E+07	415	1933	300	0.0	
S	40	23.E+04	2.6E+06	410	67000	6.70E+03	425	395	1.0E+07	425	1933	300	0.0	
S	41	23.E+04	2.6E+06	410	87000	6.70E+03	420	395	1.0E+07	420	1933	300	0.0	
S	42	23.E+04	2.6E+06	410	67000	6.70E+03	421	395	1.0E+07	421	1933	300	0.0	
S	43	23.E+04	2.6E+06	410	67000	6.70E+03	415	395	1.0E+07	415	1933	300	0.0	
S	44	23.E+04	2.6E+06	410	67000	6.70E+03	415	395	1.0E+07	415	1933	300	0.0	
S	45	24.E+04	2.6E+06	410	67000	6.70E+03	426	400	1.0E+07	426	1886	300	0.0	
S	46	24.E+04	2.6E+06	410	0	6.70E+04	427	389	1.0E+07	389	1886	300	196.1	
T	13	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	320	0.0	
T	14	12.E+04	2.6E+06	400	50000	1.00E+04	403	395	1.0E+07	403	1383	320	0.0	
T	15	21.E+04	2.6E+06	410	0	1.00E+05	407	405	1.0E+07	407	1989	300	0.0	
T	16	21.E+04	2.6E+06	410	0	1.00E+05	407	405	1.0E+07	407	1989	300	0.0	
T	17	21.E+04	8.0E+05	405	50000	1.00E+04	412	400	1.0E+07	406	1863	300	0.0	
T	18	21.E+04	8.0E+05	405	50000	1.00E+04	414	400	1.0E+07	406	1863	300	0.0	
T	19	21.E+04	8.0E+05	405	50000	1.00E+04	415	400	1.0E+07	406	1863	300	0.0	
T	20	23.E+04	2.6E+06	405	50000	1.00E+04	416	400	1.0E+07	416	1984	300	0.0	
T	21	23.E+04	2.6E+06	405	50000	1.00E+04	422	400	1.0E+07	422	1984	300	0.0	
T	22	26.E+04	2.6E+06	405	50000	1.00E+04	420	400	1.0E+07	420	1905	280	0.0	
T	23	26.E+04	2.6E+06	405	50000	1.00E+04	416	400	1.0E+07	416	1905	280	0.0	
T	24	26.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1905	280	0.0	
T	25	26.E+04	2.6E+06	405	50000	1.00E+04	420	400	1.0E+07	420	1905	280	0.0	
T	26	26.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1905	280	0.0	
T	27	26.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	1905	280	0.0	
T	28	24.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2072	280	0.0	
T	29	24.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2072	280	0.0	
T	30	23.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	1953	280	0.0	
T	31	23.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	1953	280	0.0	
T	32	23.E+04	2.6E+06	403	0	1.80E+05	406	398	1.0E+07	398	1953	280	0.0	
T	33	23.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	1953	280	0.0	
T	34	24.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	1801	280	0.0	
T	35	24.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	1801	280	0.0	
T	36	24.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	1801	280	0.0	
T	37	24.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	1801	280	0.0	
T	38	24.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	1801	280	0.0	
T	39	24.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1801	280	0.0	
T	40	25.E+04	2.6E+06	410	67000	6.70E+03	424	395	1.0E+07	424	1764	280	0.0	
T	41	25.E+04	2.6E+06	410	67000	6.70E+03	427	395	1.0E+07	427	1764	280	0.0	
T	42	25.E+04	2.6E+06	410	67000	6.70E+03	426	395	1.0E+07	426	1764	280	0.0	
T	43	25.E+04	2.6E+06	410	67000	6.70E+03	425	395	1.0E+07	425	1764	280	0.0	
T	44	25.E+04	2.6E+06	410	67000	6.70E+03	426	400	1.0E+07	426	1729	280	0.0	
T	45	25.E+04	2.6E+06	410	67000	6.70E+03	426	400	1.0E+07	426	1729	280	0.0	
T	46	25.E+04	2.6E+06	410	0	6.70E+03	427	389	1.0E+07	389	1729	280	196.7	
U	14	00.E+00	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	0.001	380	0.0	
U	15	19.E+04	2.6E+06	405	67000	6.70E+03	409	390	1.0E+07	409	2030	320	0.0	
U	16	19.E+04	8.0E+05	405	67000	6.70E+03	411	390	1.0E+07	406	2030	320	0.0	
U	17	19.E+04	8.0E+05	405	67000	6.70E+03	414	390	1.0E+07	406	2030	320	0.0	
U	18	21.E+04	8.0E+05	405	67000	6.70E+03	412	390	1.0E+07	406	1863	300	0.0	
U	19	21.E+04	8.0E+05	405	50000	1.00E+04	415	400	1.0E+07	406	1863	300	0.0	
U	20	23.E+04	8.0E+05	405	50000	1.00E+04	414	400	1.0E+07	406	1984	300	0.0	
U	21	23.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	1984	300	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
U	22	24.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	2106	300	0.0	
U	23	26.E+04	2.6E+06	405	50000	1.00E+04	416	400	1.0E+07	416	1905	280	0.0	
U	24	26.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	1905	280	0.0	
U	25	26.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1905	280	0.0	
U	26	26.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	1905	280	0.0	
U	27	26.E+04	2.6E+06	405	50000	1.00E+04	412	400	1.0E+07	412	1905	280	0.0	
U	28	26.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	1905	280	0.0	
U	29	24.E+04	2.6E+06	403	0	1.20E+05	408	398	1.0E+07	398	2072	280	0.0	
U	30	26.E+04	2.6E+06	405	50000	1.00E+04	407	400	1.0E+07	407	1905	280	0.0	
U	31	26.E+04	2.6E+06	405	50000	1.00E+04	408	400	1.0E+07	408	1905	280	0.0	
U	32	23.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	1953	280	0.0	
U	33	24.E+04	2.6E+06	405	50000	1.00E+04	410	400	1.0E+07	410	1801	280	0.0	
U	34	24.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	1801	280	0.0	
U	35	24.E+04	2.6E+06	405	50000	1.00E+04	414	400	1.0E+07	414	1801	280	0.0	
U	36	24.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	1801	280	0.0	
U	37	24.E+04	2.6E+06	405	50000	1.00E+04	420	400	1.0E+07	420	1801	280	0.0	
U	38	25.E+04	2.6E+06	410	67000	6.70E+03	419	395	1.0E+07	419	1764	280	0.0	
U	39	25.E+04	2.6E+06	410	67000	6.70E+03	422	395	1.0E+07	422	1764	280	0.0	
U	40	25.E+04	2.6E+06	410	67000	6.70E+03	423	395	1.0E+07	423	1764	280	0.0	
U	41	25.E+04	2.6E+06	410	67000	6.70E+03	424	395	1.0E+07	424	1764	280	0.0	
U	42	25.E+04	2.6E+06	410	67000	6.70E+03	426	395	1.0E+07	426	1764	280	0.0	
U	43	25.E+04	2.6E+06	410	67000	6.70E+03	425	395	1.0E+07	425	1764	280	0.0	
U	44	25.E+04	2.6E+06	410	67000	6.70E+03	423	400	1.0E+07	423	1729	280	0.0	
U	45	25.E+04	2.6E+06	410	67000	6.70E+03	424	400	1.0E+07	424	1729	280	0.0	
U	46	25.E+04	2.6E+06	410	0	6.70E+04	425	389	1.0E+07	389	1729	280	197.2	
V	15	00.E+00	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	0.001	380	0.0	
V	16	11.E+04	8.0E+05	405	50000	1.00E+04	416	390	1.0E+07	406	1202	320	0.0	
V	17	20.E+04	8.0E+05	405	50000	1.00E+04	417	395	1.0E+07	406	2030	320	0.0	
V	18	19.E+04	8.0E+05	405	50000	1.00E+04	421	390	1.0E+07	406	2030	320	0.0	
V	19	21.E+04	8.0E+05	405	50000	1.00E+04	418	400	1.0E+07	406	1863	300	0.0	
V	20	21.E+04	8.0E+05	405	50000	1.00E+04	419	400	1.0E+07	406	1863	300	0.0	
V	21	23.E+04	2.6E+06	405	50000	1.00E+04	423	400	1.0E+07	423	1984	300	0.0	
V	22	23.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1984	300	0.0	
V	23	26.E+04	2.6E+06	405	50000	1.00E+04	413	400	1.0E+07	413	1905	280	0.0	
V	24	34.E+04	2.6E+06	405	50000	1.00E+04	414	400	1.0E+07	414	2535	280	0.0	
V	25	34.E+04	2.6E+06	405	50000	1.00E+04	416	400	1.0E+07	416	2535	280	0.0	
V	26	34.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	2535	280	0.0	
V	27	26.E+04	2.6E+06	405	50000	1.00E+04	409	400	1.0E+07	409	1905	280	0.0	
V	28	26.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	1905	280	0.0	
V	29	26.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	1905	280	0.0	
V	30	26.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	1905	280	0.0	
V	31	23.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2341	300	0.0	
V	32	24.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	2106	300	0.0	
V	33	24.E+04	2.6E+06	405	50000	1.00E+04	420	400	1.0E+07	420	2106	300	0.0	
V	34	24.E+04	2.6E+06	405	50000	1.00E+04	407	400	1.0E+07	407	2106	300	0.0	
V	35	24.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	2106	300	0.0	
V	36	24.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	2106	300	0.0	
V	37	25.E+04	2.6E+06	410	67000	6.70E+03	420	395	1.0E+07	420	2050	300	0.0	
V	38	25.E+04	2.6E+06	410	67000	6.70E+03	421	395	1.0E+07	421	2050	300	0.0	
V	39	25.E+04	2.6E+06	410	67000	6.70E+03	422	395	1.0E+07	422	2050	300	0.0	
V	40	25.E+04	2.6E+06	410	67000	6.70E+03	422	395	1.0E+07	422	2050	300	0.0	
V	41	25.E+04	2.6E+06	410	67000	6.70E+03	424	395	1.0E+07	424	2050	300	0.0	
V	42	25.E+04	2.6E+06	410	67000	6.70E+03	425	395	1.0E+07	425	2050	300	0.0	
V	43	25.E+04	2.6E+06	410	67000	6.70E+03	424	400	1.0E+07	424	1998	300	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
V	44	24.E+04	2.6E+06	410	67000	6.70E+03	424	400	1.0E+07	424	1886	300	0.0	
V	45	24.E+04	2.6E+06	410	67000	6.70E+03	425	400	1.0E+07	425	1886	300	0.0	
V	46	24.E+04	2.6E+06	415	0	6.70E+04	424	389	1.0E+07	389	1886	300	197.8	
W	16	10.E+02	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
W	17	12.E+04	8.0E+05	405	67000	6.70E+03	426	395	1.0E+07	406	1160	320	0.0	
W	18	19.E+04	8.0E+05	405	67000	6.70E+03	425	390	1.0E+07	406	2030	320	0.0	
W	19	19.E+04	8.0E+05	405	67000	6.70E+03	426	390	1.0E+07	406	2030	320	0.0	
W	20	21.E+04	8.0E+05	405	50000	1.00E+04	420	400	1.0E+07	406	1863	300	0.0	
W	21	23.E+04	2.6E+06	405	50000	1.00E+04	420	400	1.0E+07	420	1984	300	0.0	
W	22	23.E+04	2.6E+06	405	50000	1.00E+04	420	400	1.0E+07	420	1984	300	0.0	
W	23	24.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	2106	300	0.0	
W	24	34.E+04	2.6E+06	405	50000	1.00E+04	414	400	1.0E+07	414	2535	280	0.0	
W	25	34.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	2535	280	0.0	
W	26	34.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	2535	280	0.0	
W	27	34.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	2535	280	0.0	
W	28	26.E+04	2.6E+06	405	50000	1.00E+04	410	400	1.0E+07	410	1905	280	0.0	
W	29	26.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1905	280	0.0	
W	30	24.E+04	2.6E+06	405	50000	1.00E+04	415	400	1.0E+07	415	2106	300	0.0	
W	31	24.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	2106	300	0.0	
W	32	25.E+04	2.6E+06	410	67000	6.70E+03	424	399	1.0E+07	424	2008	300	0.0	
W	33	25.E+04	2.6E+06	410	67000	6.70E+03	420	395	1.0E+07	420	2050	300	0.0	
W	34	24.E+04	2.6E+06	405	50000	1.00E+04	409	400	1.0E+07	409	2106	300	0.0	
W	35	25.E+04	2.6E+06	405	67000	6.70E+03	409	395	1.0E+07	409	2050	300	0.0	
W	36	25.E+04	2.6E+06	410	67000	6.70E+03	414	395	1.0E+07	414	2050	300	0.0	
W	37	25.E+04	2.6E+06	410	67000	6.70E+03	420	395	1.0E+07	420	2050	300	0.0	
W	38	25.E+04	2.6E+06	410	67000	6.70E+03	422	395	1.0E+07	422	2050	300	0.0	
W	39	25.E+04	2.6E+06	410	67000	6.70E+03	422	395	1.0E+07	422	2050	300	0.0	
W	40	25.E+04	2.6E+06	410	67000	6.70E+03	422	395	1.0E+07	422	2050	300	0.0	
W	41	25.E+04	2.6E+06	410	67000	6.70E+03	423	395	1.0E+07	423	2050	300	0.0	
W	42	25.E+04	2.6E+06	410	67000	6.70E+03	422	395	1.0E+07	422	2050	300	0.0	
W	43	25.E+04	2.6E+06	410	67000	6.70E+03	422	400	1.0E+07	422	1998	300	0.0	
W	44	25.E+04	2.6E+06	410	67000	6.70E+03	424	400	1.0E+07	424	1998	300	0.0	
W	45	24.E+04	2.6E+06	410	67000	6.70E+03	426	400	1.0E+07	426	1886	300	0.0	
W	46	22.E+04	2.6E+06	415	0	6.70E+04	425	389	1.0E+07	389	2102	320	198.7	
X	17	10.E+02	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
X	18	12.E+04	8.0E+05	405	67000	6.70E+03	420	395	1.0E+07	406	1180	320	0.0	
X	19	12.E+04	8.0E+05	405	67000	6.70E+03	420	395	1.0E+07	406	1180	320	0.0	
X	20	20.E+04	2.6E+06	405	67000	6.70E+03	423	395	1.0E+07	423	2030	320	0.0	
X	21	22.E+04	2.6E+06	405	67000	6.70E+03	410	395	1.0E+07	410	2170	320	0.0	
X	22	23.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	1984	300	0.0	
X	23	24.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	2106	300	0.0	
X	24	33.E+04	2.6E+06	405	50000	1.00E+04	417	400	1.0E+07	417	2845	300	0.0	
X	25	33.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	2845	300	0.0	
X	26	34.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	2535	280	0.0	
X	27	34.E+04	2.6E+06	405	50000	1.00E+04	418	400	1.0E+07	418	2535	280	0.0	
X	28	34.E+04	2.6E+06	405	50000	1.00E+04	408	400	1.0E+07	408	2535	280	0.0	
X	29	34.E+04	2.6E+06	405	50000	1.00E+04	413	400	1.0E+07	413	2535	280	0.0	
X	30	33.E+04	2.6E+06	405	50000	1.00E+04	419	400	1.0E+07	419	2845	300	0.0	
X	31	25.E+04	2.6E+06	410	67000	6.70E+03	420	395	1.0E+07	420	2050	300	0.0	
X	32	25.E+04	2.6E+06	410	67000	6.70E+03	418	395	1.0E+07	418	2050	300	0.0	
X	33	25.E+04	2.6E+06	410	67000	6.70E+03	414	395	1.0E+07	414	2050	300	0.0	
X	34	25.E+04	2.6E+06	410	67000	6.70E+03	412	395	1.0E+07	412	2050	300	0.0	
X	35	25.E+04	2.6E+06	410	67000	6.70E+03	410	395	1.0E+07	410	2050	300	0.0	
X	36	25.E+04	2.6E+06	410	67000	6.70E+03	420	395	1.0E+07	420	2050	300	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
X	37	25.E+04	2.6E+06	410	67000	6.70E+03	420	395	1.0E+07	420	2050	300	0.0	
X	38	25.E+04	2.6E+06	410	67000	6.70E+03	420	395	1.0E+07	420	2050	300	0.0	
X	39	25.E+04	2.6E+06	410	67000	6.70E+03	421	395	1.0E+07	421	2050	300	0.0	
X	40	25.E+04	2.6E+06	410	67000	6.70E+03	421	395	1.0E+07	421	2050	300	0.0	
X	41	25.E+04	2.6E+06	410	67000	6.70E+03	422	395	1.0E+07	422	2050	300	0.0	
X	42	25.E+04	2.6E+06	410	67000	6.70E+03	422	395	1.0E+07	422	2050	300	0.0	
X	43	25.E+04	2.6E+06	410	67000	6.70E+03	422	400	1.0E+07	422	1998	300	0.0	
X	44	25.E+04	2.6E+06	410	67000	6.70E+03	422	400	1.0E+07	422	1998	300	0.0	
X	45	25.E+04	2.6E+06	410	67000	6.70E+03	432	400	1.0E+07	432	2235	320	0.0	
X	46	23.E+04	2.6E+06	410	67000	6.70E+03	427	400	1.0E+07	427	0	320	0.0	
Y	18	10.E+02	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
Y	19	10.E+02	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
Y	20	10.E+02	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
Y	21	22.E+04	2.6E+06	405	67000	6.70E+03	420	395	1.0E+07	420	2170	320	0.0	
Y	22	22.E+04	2.6E+06	405	67000	6.70E+03	418	395	1.0E+07	418	2170	320	0.0	
Y	23	25.E+04	2.6E+06	405	67000	6.70E+03	416	395	1.0E+07	416	2050	300	0.0	
Y	24	25.E+04	2.6E+06	405	67000	6.70E+03	419	395	1.0E+07	419	2050	300	0.0	
Y	25	33.E+04	2.6E+06	405	67000	6.70E+03	418	395	1.0E+07	418	2758	300	0.0	
Y	26	33.E+04	2.6E+06	405	67000	6.70E+03	419	395	1.0E+07	419	2758	300	0.0	
Y	27	33.E+04	2.6E+06	405	67000	6.70E+03	418	395	1.0E+07	418	2758	300	0.0	
Y	28	35.E+04	2.6E+06	410	67000	6.70E+03	416	395	1.0E+07	416	2471	280	0.0	
Y	29	35.E+04	2.6E+06	410	67000	6.70E+03	414	395	1.0E+07	414	2471	280	0.0	
Y	30	33.E+04	2.6E+06	405	67000	6.70E+03	409	395	1.0E+07	409	2758	300	0.0	
Y	31	25.E+04	2.6E+06	410	67000	6.70E+03	413	395	1.0E+07	413	2050	300	0.0	
Y	32	25.E+04	2.6E+06	410	67000	6.70E+03	414	395	1.0E+07	414	2050	300	0.0	
Y	33	25.E+04	2.6E+06	410	67000	6.70E+03	414	395	1.0E+07	414	2050	300	0.0	
Y	34	25.E+04	2.6E+06	410	67000	6.70E+03	412	395	1.0E+07	412	2050	300	0.0	
Y	35	25.E+04	2.6E+06	410	67000	6.70E+03	413	395	1.0E+07	413	2050	300	0.0	
Y	36	25.E+04	2.6E+06	410	67000	6.70E+03	419	395	1.0E+07	419	2050	300	0.0	
Y	37	25.E+04	2.6E+06	410	67000	6.70E+03	419	395	1.0E+07	419	2050	300	0.0	
Y	38	25.E+04	2.6E+06	410	67000	6.70E+03	419	395	1.0E+07	419	2050	300	0.0	
Y	39	25.E+04	2.6E+06	410	67000	6.70E+03	420	395	1.0E+07	420	2050	300	0.0	
Y	40	25.E+04	2.6E+06	410	67000	6.70E+03	421	395	1.0E+07	421	2050	300	0.0	
Y	41	25.E+04	2.6E+06	410	67000	6.70E+03	421	395	1.0E+07	421	2050	300	0.0	
Y	42	25.E+04	2.6E+06	410	67000	6.70E+03	420	395	1.0E+07	420	2050	300	0.0	
Y	43	23.E+04	2.6E+06	410	67000	6.70E+03	421	400	1.0E+07	421	2235	320	0.0	
Y	44	23.E+04	2.6E+06	410	67000	6.70E+03	424	400	1.0E+07	424	2235	320	0.0	
Y	45	23.E+04	2.6E+06	415	67000	6.70E+03	434	400	1.0E+07	434	2235	320	0.0	
Y	46	23.E+04	2.6E+06	415	67000	6.70E+03	428	400	1.0E+07	428	0	320	0.0	
Z	21	10.E+02	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
Z	22	10.E+02	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
Z	23	24.E+04	2.6E+06	405	67000	6.70E+03	423	405	1.0E+07	423	2168	320	0.0	
Z	24	23.E+04	2.6E+06	405	67000	6.70E+03	429	395	1.0E+07	429	2310	320	0.0	
Z	25	25.E+04	2.6E+06	405	67000	6.70E+03	421	395	1.0E+07	421	2050	300	0.0	
Z	26	25.E+04	8.0E+05	405	67000	6.70E+03	421	395	1.0E+07	406	2050	300	0.0	
Z	27	33.E+04	8.0E+05	405	67000	6.70E+03	417	395	1.0E+07	406	2758	300	0.0	
Z	28	33.E+04	8.0E+05	410	67000	6.70E+03	416	395	1.0E+07	406	2758	300	0.0	
Z	29	33.E+04	2.6E+06	410	67000	6.70E+03	418	395	1.0E+07	418	2758	300	0.0	
Z	30	33.E+04	2.6E+06	410	67000	6.70E+03	419	395	1.0E+07	419	2758	300	0.0	
Z	31	25.E+04	2.6E+06	410	67000	6.70E+03	419	395	1.0E+07	419	2050	300	0.0	
Z	32	25.E+04	2.6E+06	410	67000	6.70E+03	414	395	1.0E+07	414	2050	300	0.0	
Z	33	25.E+04	2.6E+06	410	67000	6.70E+03	412	395	1.0E+07	412	2050	300	0.0	
Z	34	25.E+04	2.6E+06	410	67000	6.70E+03	414	395	1.0E+07	414	2050	300	0.0	
Z	35	25.E+04	2.6E+06	410	67000	6.70E+03	418	395	1.0E+07	418	2050	300	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
Z	36	25.E+04	2.6E+06	410	67000	6.70E+03	419	395	1.0E+07	419	2050	300	0.0	
Z	37	23.E+04	2.6E+06	410	67000	6.70E+03	419	395	1.0E+07	419	2310	320	0.0	
Z	38	23.E+04	2.6E+06	410	67000	6.70E+03	419	395	1.0E+07	419	2310	320	0.0	
Z	39	15.E+04	2.6E+06	410	67000	6.70E+03	420	395	1.0E+07	420	1460	320	0.0	
Z	40	23.E+04	2.6E+06	410	67000	6.70E+03	421	400	1.0E+07	421	2310	320	0.0	
Z	41	23.E+04	2.6E+06	410	67000	6.70E+03	422	400	1.0E+07	422	2310	320	0.0	
Z	42	23.E+04	2.6E+06	410	67000	6.70E+03	424	400	1.0E+07	424	2310	320	0.0	
Z	43	23.E+04	2.6E+06	415	67000	6.70E+03	427	400	1.0E+07	427	2235	320	0.0	
Z	44	23.E+04	2.6E+06	415	67000	6.70E+03	432	400	1.0E+07	432	2235	320	0.0	
Z	45	23.E+04	2.6E+06	415	67000	6.70E+03	432	400	1.0E+07	432	2235	320	0.0	
Z	46	22.E+04	2.6E+06	415	67000	6.70E+03	433	405	1.0E+07	433	0	340	0.0	
AA	23	10.E+02	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
AA	24	14.E+04	2.6E+06	405	67000	6.70E+03	439	405	1.0E+07	439	1268	320	0.0	
AA	25	22.E+04	8.0E+05	405	67000	6.70E+03	433	405	1.0E+07	406	2040	320	0.0	
AA	26	22.E+04	8.0E+05	405	67000	6.70E+03	429	403	1.0E+07	406	2064	320	0.0	
AA	27	22.E+04	8.0E+05	405	67000	6.70E+03	418	400	1.0E+07	406	2170	320	0.0	
AA	28	22.E+04	8.0E+05	410	67000	6.70E+03	426	400	1.0E+07	406	2170	320	0.0	
AA	29	32.E+04	8.0E+05	410	67000	6.70E+03	417	400	1.0E+07	406	2641	300	0.0	
AA	30	32.E+04	2.6E+06	410	67000	6.70E+03	424	400	1.0E+07	424	2641	300	0.0	
AA	31	23.E+04	2.6E+06	410	67000	6.70E+03	425	400	1.0E+07	425	1933	300	0.0	
AA	32	25.E+04	2.6E+06	410	67000	6.70E+03	420	400	1.0E+07	420	2050	300	0.0	
AA	33	25.E+04	2.6E+06	410	67000	6.70E+03	417	400	1.0E+07	417	2050	300	0.0	
AA	34	25.E+04	2.6E+06	410	67000	6.70E+03	416	400	1.0E+07	416	2050	300	0.0	
AA	35	25.E+04	2.6E+06	410	67000	6.70E+03	417	400	1.0E+07	417	2050	300	0.0	
AA	36	25.E+04	2.6E+06	410	67000	6.70E+03	420	400	1.0E+07	420	2050	300	0.0	
AA	37	23.E+04	2.6E+06	410	67000	6.70E+03	425	400	1.0E+07	425	2310	320	0.0	
AA	38	23.E+04	2.6E+06	410	67000	6.70E+03	424	400	1.0E+07	424	2310	320	0.0	
AA	39	15.E+04	2.6E+06	410	67000	6.70E+03	424	400	1.0E+07	424	1460	320	0.0	
AA	40	15.E+04	2.6E+06	410	67000	6.70E+03	425	400	1.0E+07	425	1460	320	0.0	
AA	41	15.E+04	2.6E+06	410	67000	6.70E+03	421	400	1.0E+07	421	1460	320	0.0	
AA	42	23.E+04	2.6E+06	415	67000	6.70E+03	430	400	1.0E+07	430	2235	320	0.0	
AA	43	23.E+04	2.6E+06	415	67000	6.70E+03	433	400	1.0E+07	433	2235	320	0.0	
AA	44	23.E+04	2.6E+06	415	67000	6.70E+03	437	400	1.0E+07	437	2235	320	0.0	
AA	45	14.E+04	2.6E+06	415	67000	6.70E+03	445	405	1.0E+07	445	1538	340	0.0	
AA	46	22.E+04	2.6E+06	415	67000	6.70E+03	457	405	1.0E+07	457	0	340	0.0	
BB	24	10.E+02	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	10	320	0.0	
BB	25	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	320	0.0	
BB	26	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	320	0.0	
BB	27	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	320	0.0	
BB	28	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	320	0.0	
BB	29	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	320	0.0	
BB	30	22.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	2040	320	0.0	
BB	31	22.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	2040	320	0.0	
BB	32	24.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	2168	320	0.0	
BB	33	24.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	2168	320	0.0	
BB	34	24.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	2168	320	0.0	
BB	35	24.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	2168	320	0.0	
BB	36	24.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	2168	320	0.0	
BB	37	24.E+04	2.6E+06	415	67000	8.70E+03	430	405	1.0E+07	430	2168	320	0.0	
BB	38	15.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	1395	320	0.0	
BB	39	15.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	1395	320	0.0	
BB	40	15.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	1395	320	0.0	
BB	41	15.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	1395	320	0.0	
BB	42	15.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	1395	320	0.0	

COL	ROW	T	S	INITIAL HEAD	CONST. RATE	RECHARGE FACTOR	LAND ELEV.	STREAM BED ELEV.	S W.T.	TOP AQUIFER	HYDRAU. CONDUC.	BOTTOM AQUIFER	MISS. RIVER MI.	SPECIFIED FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
BB	43	14.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	1538	340	0.0	
BB	44	14.E+04	2.6E+06	415	67000	6.70E+03	430	405	1.0E+07	430	1538	340	0.0	
BB	45	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
BB	46	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	30	10.E+02	1.0E+20	410	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
CC	31	10.E+02	1.0E+20	410	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
CC	32	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	33	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	34	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	35	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	36	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	37	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	38	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	39	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	40	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	41	10.E+02	1.0E+20	420	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	42	10.E+02	1.0E+20	420	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	43	10.E+02	1.0E+20	425	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	44	10.E+02	1.0E+20	420	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	

APPENDIX B

Boundary Data for PLASM Groundwater Model.

Horseshoe Lake Cells				INT.	Const.	RECHARGE	LAND	STREAM	S	TOP AQ.	HYDRAU.	BOTTOM AQ.	MISS.	SPECIFIED
COL	ROW	T	S	HEAD	Rate		ELEV.	BED ELEV	W.T.		CONDUC.	ELEV.	MI.	FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
O	28	22.E+04	2.6E+06	400	0	1.20E+05	406	398	1.0E+07	398	2198	300	0.0	
P	29	22.E+04	2.6E+06	400	0	1.20E+05	406	398	1.0E+07	398	2198	300	0.0	
Q	30	22.E+04	2.6E+06	400	50000	1.00E+04	404	395	1.0E+07	404	2040	300	0.0	
Q	31	22.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2198	300	0.0	
R	31	22.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2198	300	0.0	
R	32	22.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2198	300	0.0	
S	32	23.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	1953	280	0.0	
T	28	24.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2072	280	0.0	
T	29	24.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2072	280	0.0	
T	30	23.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	1953	280	0.0	
T	31	23.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	1953	280	0.0	
T	32	23.E+04	2.6E+06	403	0	1.80E+05	406	398	1.0E+07	398	1953	280	0.0	
T	33	23.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	1953	280	0.0	
U	29	24.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2072	280	0.0	
U	32	23.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	1953	280	0.0	
V	31	23.E+04	2.6E+06	403	0	1.20E+05	406	398	1.0E+07	398	2341	300	0.0	
Frank Holten State Park Lake														
S	18	22.E+04	2.6E+06	410	0	1.00E+05	412	405	1.0E+07	415	2102	300	0.0	
T	15	21.E+04	2.6E+06	410	0	1.00E+05	407	405	1.0E+07	407	1969	300	0.0	
T	16	21.E+04	2.6E+06	410	0	1.00E+05	407	405	1.0E+07	407	1969	300	0.0	
Blue Waters Ditch / Harding Ditch														
G	13	13.E+04	2.6E+06	390	0	6.70E+04	408	372	1.0E+07	372	1637	320	175.3	
H	12	22.E+04	2.6E+06	400	0	1.00E+04	404	372	1.0E+07	372	2585	320	175.1	
I	11	25.E+04	2.6E+06	400	0	1.00E+05	407	372	1.0E+07	372	1998	280	175.4	
J	11	23.E+04	2.6E+06	405	0	1.00E+05	404	372	1.0E+07	372	2235	300	176.1	
K	11	23.E+04	2.6E+06	405	0	1.00E+05	403	372	1.0E+07	372	2235	300	177.2	
L	11	15.E+04	2.6E+06	410	0	1.00E+05	406	373	1.0E+07	373	1426	300	178.2	
M	11	15.E+04	2.6E+06	410	0	1.00E+05	409	373	1.0E+07	373	1426	300	178.7	
Cahokia Diversion Channel														
Q	46	14.E+04	2.6E+06	405	0	7.14E+07	416	389	1.0E+07	389	1547	300	195.2	
R	46	25.E+04	2.6E+06	405	0	6.70E+04	430	389	1.0E+07	389	2050	300	195.6	
S	46	24.E+04	2.6E+06	410	0	6.70E+04	427	389	1.0E+07	389	1886	300	196.1	
T	46	25.E+04	2.6E+06	410	0	6.70E+03	427	389	1.0E+07	389	1729	280	196.7	
U	46	25.E+04	2.6E+06	410	0	6.70E+04	425	389	1.0E+07	389	1729	280	197.2	
V	46	24.E+04	2.6E+06	415	0	6.70E+04	424	389	1.0E+07	389	1886	300	197.8	
W	46	22.E+04	2.6E+06	415	0	6.70E+04	425	389	1.0E+07	389	2102	320	198.7	
Specified Flux														
A	1	22.E+04	2.6E+06	390	0	6.70E+03	400	395	1.0E+07	400	2700	320	0.0	-4.7E+04
B	1	25.E+04	2.6E+06	390	0	6.70E+03	400	385	1.0E+07	400	2050	280	0.0	-1.5E+04
C	1	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	-1.5E+04
E	1	25.E+04	2.6E+06	390	50000	1.00E+04	400	385	1.0E+07	400	2050	280	0.0	-1.5E+04
F	1	23.E+04	2.6E+06	395	50000	1.00E+04	405	390	1.0E+07	405	2235	300	0.0	-1.4E+04

Specified Heads at Eastern Boundary

COL	ROW	T	S	HEAD	Const. Rate	RECHARGE	LAND ELEV.	STREAM BED ELEV	S W.T.	TOP AQ.	HYDRAU. CONDUC.	BOTTOM AQ. ELEV.	MISS. MI.	SP
		gpd/ft		ft. msl	gpd	gpd/ft.	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
G	1	22.E+02	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	370	0.0	
G	2	22.E+02	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	370	0.0	
H	3	22.E+02	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	380	0.0	
H	4	22.E+02	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	380	0.0	
I	5	22.E+02	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	380	0.0	
J	6	00.E+00	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	380	0.0	
K	7	22.E+02	1.0E+20	400	0	0.00E+00	600	590	1.0E+20	600	10	380	0.0	
L	8	17.E+02	1.0E+20	400	0	0.00E+00	550	540	1.0E+20	550	10	380	0.0	
M	9	00.E+00	1.0E+20	400	0	0.00E+00	550	540	1.0E+20	550	0.001	380	0.0	
N	10	00.E+00	1.0E+20	400	0	0.00E+00	475	465	1.0E+20	475	0.001	380	0.0	
O	10	00.E+00	1.0E+20	400	0	0.00E+00	475	465	1.0E+20	475	0.001	380	0.0	
P	10	00.E+00	1.0E+20	400	0	0.00E+00	475	465	1.0E+20	475	0.001	380	0.0	
Q	10	00.E+00	1.0E+20	405	0	0.00E+00	475	465	1.0E+20	475	0.001	380	0.0	
R	11	00.E+00	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	0.001	380	0.0	
S	12	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	380	0.0	
T	13	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	320	0.0	
U	14	00.E+00	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	0.001	380	0.0	
V	15	00.E+00	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	0.001	380	0.0	
W	16	10.E+02	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
X	17	10.E+02	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
Y	18	10.E+02	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
Y	19	10.E+02	1.0E+20	400	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
Y	20	10.E+02	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
Z	21	10.E+02	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
Z	22	10.E+02	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
AA	23	10.E+02	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
BB	24	10.E+02	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	10	320	0.0	
BB	25	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	320	0.0	
BB	26	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	320	0.0	
BB	27	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	320	0.0	
BB	28	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	320	0.0	
BB	29	00.E+00	1.0E+20	405	0	0.00E+00	500	490	1.0E+20	500	0.001	320	0.0	
CC	30	10.E+02	1.0E+20	410	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
CC	31	10.E+02	1.0E+20	410	0	0.00E+00	500	490	1.0E+20	500	10	380	0.0	
CC	32	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	33	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	34	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	35	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	36	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	37	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	38	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	39	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	40	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	41	10.E+02	1.0E+20	420	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	42	10.E+02	1.0E+20	420	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	43	10.E+02	1.0E+20	425	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
CC	44	10.E+02	1.0E+20	420	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
BB	45	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	
BB	46	10.E+02	1.0E+20	415	0	0.00E+00	500	490	1.0E+20	500	10	400	0.0	

Mississippi River Cells														
COL	ROW	T	S	HEAD	Const.	RECHARGE	LAND	STREAM	S	TOP AQ.	HYDRAU.	BOTTOM AQ.	MISS.	SPECIFIED
					Rate		ELEV.	BED ELEV	W.T.		CONDUC.	ELEV.	MI.	FLUX
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	gpd
A	3	16.E+04	2.6E+06	395	0	1.06E+07	395	369	1.0E+07	369	3296	320	169.6	-5.7E+04
A	4	11.E+04	2.6E+06	395	0	1.06E+07	395	370	1.0E+07	370	2170	320	170.1	-3.7E+04
A	5	11.E+04	2.6E+06	395	0	1.06E+07	395	370	1.0E+07	370	2170	320	170.6	
A	6	11.E+04	2.6E+06	395	0	1.06E+07	395	370	1.0E+07	370	2170	320	170.9	
B	7	11.E+04	2.6E+06	395	0	1.06E+07	395	370	1.0E+07	370	2170	320	171.7	
B	8	11.E+04	2.6E+06	395	0	1.06E+07	395	370	1.0E+07	370	2170	320	172.0	
B	9	11.E+04	2.6E+06	395	0	1.06E+07	395	370	1.0E+07	370	2170	320	172.3	
C	10	13.E+04	2.6E+06	395	0	1.06E+07	395	371	1.0E+07	371	2142	320	172.9	
D	11	11.E+04	2.6E+06	395	0	1.06E+07	405	372	1.0E+07	372	2115	320	173.9	
D	12	11.E+04	2.6E+06	395	0	1.06E+07	395	372	1.0E+07	372	2115	320	173.9	
E	12	11.E+04	2.6E+06	395	0	1.06E+07	407	372	1.0E+07	372	2115	320	174.5	
E	13	11.E+04	2.6E+06	395	0	1.06E+07	407	372	1.0E+07	372	2115	320	174.8	
E	14	11.E+04	2.6E+06	395	0	1.06E+07	395	372	1.0E+07	372	2115	320	175.1	
F	14	87.0E+3	2.6E+06	395	0	1.06E+07	409	372	1.0E+07	372	1681	320	175.4	
F	15	87.0E+3	2.6E+06	395	0	1.06E+07	395	372	1.0E+07	372	1681	320	175.4	
G	15	87.0E+3	2.6E+06	395	0	1.06E+07	415	372	1.0E+07	372	1681	320	176.1	
G	16	87.0E+3	2.6E+06	395	0	1.06E+07	395	372	1.0E+07	372	1681	320	176.4	
H	16	87.0E+3	2.6E+06	395	0	1.06E+07	409	372	1.0E+07	372	1681	320	176.9	
H	17	87.0E+3	2.6E+06	395	0	1.06E+07	409	372	1.0E+07	372	1681	320	177.4	
I	18	87.0E+3	2.6E+06	395	0	5.30E+06	395	372	1.0E+07	372	1681	320	177.2	
I	19	87.0E+3	2.6E+06	395	0	5.30E+06	395	372	1.0E+07	372	1681	320	177.5	
J	20	82.0E+3	2.6E+06	395	0	1.06E+07	414	373	1.0E+07	373	2471	340	178.7	
J	21	82.0E+3	2.6E+06	395	0	1.06E+07	400	373	1.0E+07	373	2471	340	179.1	
J	22	82.0E+3	2.6E+06	395	0	1.06E+07	395	373	1.0E+07	373	2471	340	179.5	
J	23	82.0E+3	2.6E+06	395	0	1.06E+07	395	373	1.0E+07	373	2471	340	180.0	
J	24	82.0E+3	2.6E+06	395	0	1.06E+07	395	373	1.0E+07	373	2471	340	180.4	
J	25	82.0E+3	2.6E+06	395	0	1.06E+07	395	373	1.0E+07	373	2471	340	180.9	
J	26	86.0E+3	2.6E+06	395	0	1.06E+07	395	374	1.0E+07	374	2525	340	181.5	
J	27	57.0E+3	2.6E+06	395	0	1.06E+07	395	374	1.0E+07	374	1672	340	182.2	
I	28	04.E+02	2.6E+06	395	0	1.06E+07	395	375	1.0E+07	375	10	340	183.5	
I	29	04.E+02	2.6E+06	395	0	1.06E+07	395	375	1.0E+07	375	10	340	183.8	
I	30	91.0E+3	2.6E+06	395	0	1.06E+07	395	376	1.0E+07	376	2525	340	184.4	
I	31	91.0E+3	2.6E+06	395	0	1.06E+07	395	376	1.0E+07	376	2525	340	184.7	
J	32	92.0E+3	2.6E+06	395	0	5.30E+06	413	377	1.0E+07	377	2477	340	185.5	
J	33	92.0E+3	2.6E+06	395	0	5.30E+06	395	377	1.0E+07	377	2477	340	185.8	
K	34	96.0E+3	2.6E+06	395	0	3.57E+07	420	380	1.0E+07	376	2525	340	194.0	
K	35	10.E+04	2.6E+06	405	0	3.57E+07	405	390	1.0E+07	380	2525	340	194.5	
L	36	12.E+04	2.6E+06	405	0	3.57E+07	418	382	1.0E+07	382	1895	320	194.0	
L	37	12.E+04	2.6E+06	405	0	3.57E+07	405	382	1.0E+07	382	1895	320	194.0	
M	38	12.E+04	2.6E+06	405	0	3.57E+07	405	384	1.0E+07	384	1859	320	194.0	
N	39	21.E+04	2.6E+06	405	0	3.57E+07	405	390	1.0E+07	388	3044	320	194.0	
N	40	26.E+04	2.6E+06	405	0	3.57E+07	405	388	1.0E+07	388	3762	320	194.0	
O	41	26.E+04	2.6E+06	405	0	3.57E+07	415	388	1.0E+07	388	3762	320	194.0	
O	42	26.E+04	2.6E+06	405	0	3.57E+07	405	388	1.0E+07	388	3762	320	194.0	
O	43	12.E+04	2.6E+06	405	0	7.14E+07	405	388	1.0E+07	388	1794	320	192.8	
O	44	12.E+04	2.6E+06	405	0	7.14E+07	405	388	1.0E+07	388	1794	320	193.5	
P	44	26.E+04	2.6E+06	405	0	3.57E+07	414	388	1.0E+07	388	3762	320	194.0	
P	45	21.E+04	2.6E+06	405	0	3.57E+07	405	388	1.0E+07	388	3044	320	194.0	
P	46	12.E+04	2.6E+06	405	0	7.14E+07	405	388	1.0E+07	388	1794	320	193.5	

Extra Mississippi Cells														
COL	ROW	T	S	HEAD	Const. Rate	RECHARGE	LAND ELEV.	STREAM BED ELEV	S W.T.	TOP AQ.	HYDRAU. CONDUC.	BOTTOM AQ. ELEV.	MISS. MI.	SPECIFIED
		gpd/ft		ft. msl	gpd	gpd/ft	ft. msl	ft. msl		ft. msl	gpd/sq. ft.	ft. msl	miles	
I	34	22.0E+3	2.6E+06	400	0	6.70E+03	405	390	1.0E+07	388	458	340	187.7	
I	35	22.0E+3	2.6E+06	400	0	6.70E+03	405	390	1.0E+07	388	458	340	187.9	
I	36	22.0E+3	2.6E+06	400	0	6.70E+03	405	390	1.0E+07	388	458	340	188.3	
L	42	20.0E+3	2.6E+06	405	0	6.70E+03	405	386	1.0E+07	386	445	340	192.1	
M	43	20.0E+3	2.6E+06	405	0	7.14E+07	405	386	1.0E+07	386	445	340	192.1	
N	44	11.E+04	2.6E+06	405	0	7.14E+07	405	388	1.0E+07	388	2229	340	192.8	
O	44	12.E+04	2.6E+06	405	0	7.14E+07	405	388	1.0E+07	388	1794	320	193.5	

APPENDIX C

MODFLOW Groundwater Model Input Data.

COL PLASM	ROW	COL MODFLOW	ROW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft./day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
A	1	1	59	361.80	0.04987	390	0	0.00064	400	395	0.1918	400	320	0.0	6283
A	2	1	58	361.80	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	6283
A	3	1	57	441.66	0.04987	395	0	5.28692	395	369	0.1918	369	320	169.6	7620
A	4	1	56	290.78	0.04987	395	0	5.08358	395	370	0.1918	370	320	170.1	4946
A	5	1	55	290.78	0.04987	395	0	5.08358	395	370	0.1918	370	320	170.6	
A	6	1	54	290.78	0.04987	395	0	5.08358	395	370	0.1918	370	320	170.9	
A	7	1	53	0.00	1.91819E+12	420	0	0.00000	430	420	1.92E+12	430	300	0.0	
A	8	1	52	0.00	1.91819E+12	420	0	0.00000	430	420	1.92E+12	430	300	0.0	
A	9	1	51	0.00	1.91819E+12	420	0	0.00000	430	420	1.92E+12	430	300	0.0	
B	1	2	59	274.70	0.04987	390	0	0.00193	400	385	0.1918	400	280	0.0	2005
B	2	2	58	361.80	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	
B	3	2	57	361.80	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	
B	4	2	56	361.80	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	
B	5	2	55	361.80	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	
B	6	2	54	361.80	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	
B	7	2	53	290.78	0.04987	395	0	5.08358	395	370	0.1918	370	320	171.7	
B	8	2	52	290.78	0.04987	395	0	5.08358	395	370	0.1918	370	320	172.0	
B	9	2	51	290.78	0.04987	395	0	5.08358	395	370	0.1918	370	320	172.3	
B	10	2	50	0.00	1.91819E+12	420	0	0.00000	430	420	1.92E+12	430	300	0.0	
C	1	3	59	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	2005
C	2	3	58	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
C	3	3	57	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
C	4	3	56	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
C	5	3	55	219.36	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	
C	6	3	54	219.36	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	
C	7	3	53	219.36	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	
C	8	3	52	219.36	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	
C	9	3	51	219.36	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	
C	10	3	50	287.03	0.04987	395	0	4.88023	395	371	0.1918	371	320	172.9	
C	11	3	49	0.00	1.91819E+12	420	0	0.00000	430	420	1.92E+12	430	300	0.0	
C	12	3	48	0.00	1.91819E+12	420	0	0.00000	430	420	1.92E+12	430	300	0.0	
D	1	4	59	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	2005
D	2	4	58	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
D	3	4	57	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
D	4	4	56	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
D	5	4	55	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
D	6	4	54	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
D	7	4	53	219.36	0.04987	390	6684.5	0.00288	400	385	0.1918	400	320	0.0	
D	8	4	52	219.36	0.04987	390	6684.5	0.00288	400	385	0.1918	400	320	0.0	
D	9	4	51	219.36	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	
D	10	4	50	219.36	0.04987	390	0	0.00321	410	385	0.1918	410	320	0.0	
D	11	4	49	283.41	0.04987	395	0	6.71032	405	372	0.1918	372	320	173.9	
D	12	4	48	283.41	0.04987	395	0	4.67689	395	372	0.1918	372	320	173.9	
E	1	5	59	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	2005
E	2	5	58	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
E	3	5	57	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
E	4	5	56	369.57	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
E	5	5	55	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
E	6	5	54	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
E	7	5	53	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
E	8	5	52	274.70	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
E	9	5	51	219.36	0.04987	390	6684.5	0.00384	405	385	0.1918	405	320	0.0	
E	10	5	50	219.36	0.04987	390	6684.5	0.00384	405	385	0.1918	405	320	0.0	

COL PLASM	ROW PLASM	COL MODFLOW	ROW MODFLOW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft./day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./da
E	11	5	49	219.36	0.04987	390	0	0.00193	400	385	0.1918	400	320	0.0	
E	12	5	48	283.41	0.04987	395	0	7.11701	407	372	0.1918	372	320	174.5	
E	13	5	47	283.41	0.04987	395	0	7.11701	407	372	0.1918	372	320	174.8	
E	14	5	46	283.41	0.04987	395	0	4.67689	395	372	0.1918	372	320	175.1	
E	15	5	45	0.00	1.91819E+12	420	0	0.00000	430	420	1.918E+12	430	300	0.0	
F	1	6	59	299.49	0.04987	395	6684.5	0.00288	405	390	0.1918	405	300	0.0	1872
F	2	6	58	299.49	0.04987	390	6684.5	0.00288	405	390	0.1918	405	300	0.0	
F	3	6	57	299.49	0.04987	390	6684.5	0.00288	405	390	0.1918	405	300	0.0	
F	4	6	56	369.57	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
F	5	6	55	369.57	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
F	6	6	54	369.57	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
F	7	6	53	369.57	0.04987	390	6684.5	0.00288	400	385	0.1918	400	280	0.0	
F	8	6	52	369.57	0.04987	390	6684.5	0.00384	405	385	0.1918	405	280	0.0	
F	9	6	51	274.70	0.04987	390	6684.5	0.00384	405	385	0.1918	405	280	0.0	
F	10	6	50	346.39	0.04987	395	6684.5	0.00288	405	390	0.1918	405	320	0.0	
F	11	6	49	346.39	0.04987	395	0	0.00193	405	390	0.1918	405	320	0.0	
F	12	6	48	219.36	0.04987	390	0	0.00218	407	390	0.1918	407	320	0.0	
F	13	6	47	219.36	0.04987	390	0	0.00206	406	390	0.1918	406	320	0.0	
F	14	6	46	225.25	0.04987	395	0	7.52369	409	372	0.1918	372	320	175.4	
F	15	6	45	225.25	0.04987	395	0	4.67689	395	372	0.1918	372	320	175.4	
G	1	7	59	1.34	1.91819E+12	400	0	0.00000	600	590	1.918E+12	600	370	0.0	
G	2	7	58	1.34	1.91819E+12	400	0	0.00000	600	590	1.918E+12	600	370	0.0	
G	3	7	57	191.08	0.04987	395	6684.5	0.00671	425	390	0.1918	425	300	0.0	
G	4	7	56	408.03	0.04987	395	6684.5	0.00480	415	390	0.1918	415	300	0.0	
G	5	7	55	408.03	0.04987	390	6684.5	0.00537	418	390	0.1918	418	300	0.0	
G	6	7	54	408.03	0.04987	390	6684.5	0.00537	418	390	0.1918	418	300	0.0	
G	7	7	53	423.44	0.04987	390	6684.5	0.00480	410	385	0.1918	410	300	0.0	
G	8	7	52	423.44	0.04987	390	6684.5	0.00441	408	385	0.1918	408	300	0.0	
G	9	7	51	369.57	0.04987	390	6684.5	0.00441	408	385	0.1918	408	280	0.0	
G	10	7	50	267.73	0.04987	395	6684.5	0.00288	405	390	0.1918	405	280	0.0	
G	11	7	49	346.39	0.04987	395	6684.5	0.00288	405	390	0.1918	405	320	0.0	
G	12	7	48	346.39	0.04987	395	6684.5	0.00384	410	390	0.1918	410	320	0.0	
G	13	7	47	219.36	0.04987	390	0	0.04627	408	372	0.1918	372	320	175.3	
G	14	7	46	346.39	0.04987	395	0	0.00218	407	390	0.1918	407	320	0.0	
G	15	7	45	225.25	0.04987	395	0	8.74375	415	372	0.1918	372	320	176.1	
G	16	7	44	225.25	0.04987	395	0	4.67689	395	372	0.1918	372	320	176.4	
G	17	7	43	0.00	1.91819E+12	430	0	0.00000	430	420	1.918E+12	430	320	0.0	
H	3	8	57	1.34	1.91819E+12	400	0	0.00000	600	590	1.918E+12	600	380	0.0	
H	4	8	56	1.34	1.91819E+12	400	0	0.00000	600	590	1.918E+12	600	380	0.0	
H	5	8	55	408.03	0.04987	395	6684.5	0.00288	405	390	0.1918	405	300	0.0	
H	6	8	54	393.96	0.04987	395	6684.5	0.00384	415	395	0.1918	415	300	0.0	
H	7	8	53	393.96	0.04987	390	6684.5	0.00288	410	395	0.1918	410	300	0.0	
H	8	8	52	393.96	0.04987	390	6684.5	0.00249	408	395	0.1918	408	300	0.0	
H	9	8	51	408.03	0.04987	390	6684.5	0.00345	408	390	0.1918	408	300	0.0	
H	10	8	50	267.73	0.04987	395	6684.5	0.00499	416	390	0.1918	416	280	0.0	
H	11	8	49	267.73	0.04987	395	6684.5	0.00384	410	390	0.1918	410	280	0.0	
H	12	8	48	346.39	0.04987	400	0	0.00614	404	372	0.1918	372	320	175.1	
H	13	8	47	212.39	0.04987	395	0	0.00206	406	390	0.1918	405	320	0.0	
H	14	8	46	190.28	0.04987	395	0	0.00206	406	390	0.1918	405	320	0.0	
H	15	8	45	190.28	0.04987	395	0	0.00231	408	390	0.1918	405	320	0.0	
H	16	6	44	225.25	0.04987	395	0	7.52369	409	372	0.1918	372	320	176.9	
H	17	8	43	225.25	0.04987	395	0	7.52369	409	372	0.1918	372	320	177.4	
I	5	9	55	1.34	1.91819E+12	400	0	0.00000	600	590	1.918E+12	600	380	0.0	

COL PLASM	ROW	COL MODFLOW	ROW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft./day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
I	6	9	54	480.39	0.04987	395	6684.5	0.00288	405	390	0.1918	405	320	0.0	
I	7	9	53	480.39	0.04987	395	6684.5	0.00288	405	390	0.1918	405	320	0.0	
I	8	9	52	480.39	0.04987	395	6684.5	0.00288	405	390	0.1918	405	320	0.0	
I	9	9	51	480.39	0.04987	395	6684.5	0.00480	415	390	0.1918	415	320	0.0	
I	10	9	50	299.49	0.04987	395	6684.5	0.00480	415	390	0.1918	415	300	0.0	
I	11	9	49	267.73	0.04987	400	0	0.06714	407	372	0.1918	372	280	175.4	
I	12	9	48	267.73	0.04987	395	6684.5	0.00230	402	390	0.1918	402	280	0.0	
I	13	9	47	267.73	0.04987	395	6684.5	0.00345	408	390	0.1918	408	280	0.0	
I	14	9	46	267.73	0.04987	395	6684.5	0.00326	407	390	0.1918	407	280	0.0	
I	15	9	45	161.60	0.04987	395	0	0.00193	405	390	0.1918	405	280	0.0	
I	16	9	44	161.60	0.04987	395	0	0.00244	409	390	0.1918	409	280	0.0	
I	17	9	43	161.60	0.04987	395	0	0.00360	418	390	0.1918	418	280	0.0	
I	18	9	42	225.25	0.04987	395	0	2.33845	395	372	0.1918	372	320	177.2	
I	19	9	41	225.25	0.04987	395	0	2.33845	395	372	0.1918	372	320	177.5	
I	28	9	32	1.34	0.04987	395	0	4.06686	395	375	0.1918	375	340	183.5	
I	29	9	31	1.34	0.04987	395	0	4.06686	395	375	0.1918	375	340	183.8	
I	30	9	30	338.35	0.04987	395	0	3.86352	395	376	0.1918	376	340	184.4	
I	31	9	29	338.35	0.04987	395	0	3.86352	395	376	0.1918	376	340	184.7	
I	34	9	26	61.37	0.04987	400	0	0.00193	405	390	0.1918	388	340	187.7	
I	35	9	25	61.37	0.04987	400	0	0.00193	405	390	0.1918	388	340	187.9	
I	36	9	24	61.37	0.04987	400	0	0.00193	405	390	0.1918	388	340	188.3	
I	37	9	23	0.00	1.91819E+12	430	0	0.00000	430	420	1.918E+12	430	320	0.0	
I	44	9	16	0.00	1.91819E+12	430	0	0.00000	430	420	1.918E+12	430	320	0.0	
I	45	9	15	0.00	1.91819E+12	430	0	0.00000	430	420	1.918E+12	430	320	0.0	
I	46	9	14	0.00	1.91819E+12	430	0	0.00000	430	420	1.918E+12	430	320	0.0	
J	6	10	54	1.34	1.91819E+12	400	0	0.00000	600	590	1.918E+12	600	380	0.0	
J	7	10	53	346.39	0.04987	395	6684.5	0.00480	415	390	0.1918	415	320	0.0	
J	8	10	52	346.39	0.04987	395	6684.5	0.00480	415	390	0.1918	415	320	0.0	
J	9	10	51	346.39	0.04987	395	6684.5	0.00480	415	390	0.1918	415	320	0.0	
J	10	10	50	346.39	0.04987	395	6684.5	0.00480	415	390	0.1918	415	320	0.0	
J	11	10	49	299.49	0.04987	405	0	0.06139	404	372	0.1918	372	300	176.1	
J	12	10	48	267.73	0.04987	395	6684.5	0.00288	405	390	0.1918	405	280	0.0	
J	13	10	47	267.73	0.04987	395	6684.5	0.00364	409	390	0.1918	409	280	0.0	
J	14	10	46	267.73	0.04987	395	6684.5	0.00403	411	390	0.1918	411	280	0.0	
J	15	10	45	267.73	0.04987	395	6684.5	0.00326	407	390	0.1918	407	280	0.0	
J	16	10	44	252.72	0.04987	395	6684.5	0.00384	410	390	0.1918	410	280	0.0	
J	17	10	43	252.72	0.04987	395	6684.5	0.00384	410	390	0.1918	410	280	0.0	
J	18	10	42	252.72	0.04987	395	0	0.00308	414	390	0.1918	414	280	0.0	
J	19	10	41	252.72	0.04987	395	0	0.00231	408	390	0.1918	408	280	0.0	
J	20	10	40	331.11	0.04987	395	0	8.33707	414	373	0.1918	373	340	178.7	
J	21	10	39	331.11	0.04987	395	0	5.49026	400	373	0.1918	373	340	179.1	
J	22	10	38	331.11	0.04987	395	0	4.47355	395	373	0.1918	373	340	179.5	
J	23	10	37	331.11	0.04987	395	0	4.47355	395	373	0.1918	373	340	180.0	
J	24	10	36	331.11	0.04987	395	0	4.47355	395	373	0.1918	373	340	180.4	
J	25	10	35	331.11	0.04987	395	0	4.47355	395	373	0.1918	373	340	180.9	
J	26	10	34	338.35	0.04987	395	0	4.27020	395	374	0.1918	374	340	181.5	
J	27	10	33	224.05	0.04987	395	0	4.27020	395	374	0.1918	374	340	182.2	
J	28	10	32	185.32	0.04987	400	6684.5	0.00326	412	395	0.1918	412	320	0.0	
J	29	10	31	185.32	0.04987	400	6684.5	0.00288	410	395	0.1918	410	320	0.0	
J	30	10	30	236.38	0.04987	400	6684.5	0.00326	412	395	0.1918	412	340	0.0	
J	31	10	29	236.38	0.04987	400	6684.5	0.00345	413	395	0.1918	413	340	0.0	
J	32	10	28	331.92	0.04987	395	0	3.66018	413	377	0.1918	377	340	185.5	
J	33	10	27	331.92	0.04987	395	0	1.83009	395	377	0.1918	377	340	185.8	

COL PLASM	ROW PLASM	COL MODFLOW	ROW MODFLOW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST. RATE cu. ft/day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
J	34	10	26	227.26	0.04987	405	0	0.00193	415	400	0.1918	415	340	0.0	
J	35	10	25	75.44	0.04987	405	0	0.00193	415	400	0.1918	415	340	0.0	
J	36	10	24	87.77	0.04987	400	0	0.00321	415	390	0.1918	415	340	0.0	
J	37	10	23	87.77	0.04987	400	0	0.00321	415	390	0.1918	415	340	0.0	
J	38	10	22	0.00	1.91819E+12	430	0	0.00000	430	420	1.918E+12	430	320	0.0	
J	39	10	21	0.00	1.91819E+12	430	0	0.00000	430	420	1.918E+12	430	320	0.0	
J	40	10	20	0.00	1.91819E+12	430	0	0.00000	430	420	1.918E+12	430	320	0.0	
J	43	10	17	0.00	1.91819E+12	430	0	0.00000	430	420	1.918E+12	430	320	0.0	
K	7	11	53	1.34	1.91819E+12	400	0	0.00000	600	590	1.918E+12	600	380	0.0	
K	8	11	52	212.39	0.04987	395	6684.5	0.00288	405	390	0.1918	405	320	0.0	
K	9	11	51	212.39	0.04987	395	6684.5	0.00288	405	390	0.1918	405	320	0.0	
K	10	11	50	212.39	0.04987	395	6684.5	0.00288	405	390	0.1918	405	320	0.0	
K	11	11	49	299.49	0.04987	405	0	0.05947	403	372	0.1918	372	300	177.2	
K	12	11	48	267.73	0.04987	395	6684.5	0.00326	407	390	0.1918	407	280	0.0	
K	13	11	47	267.73	0.04987	395	6684.5	0.00364	409	390	0.1918	409	280	0.0	
K	14	11	46	267.73	0.04987	395	6684.5	0.00403	411	390	0.1918	411	280	0.0	
K	15	11	45	358.85	0.04987	395	6684.5	0.00326	407	390	0.1918	407	280	0.0	
K	16	11	44	358.85	0.04987	395	6684.5	0.00345	408	390	0.1918	408	280	0.0	
K	17	11	43	343.84	0.04987	395	6684.5	0.00403	411	390	0.1918	411	280	0.0	
K	18	11	42	345.72	0.04987	394	6684.5	0.00384	409	389	0.1918	409	280	0.0	
K	19	11	41	252.72	0.04987	395	6684.5	0.00288	405	390	0.1918	405	280	0.0	
K	20	11	40	252.72	0.04987	395	6684.5	0.00499	416	390	0.1918	416	280	0.0	
K	21	11	39	333.79	0.04987	395	6684.5	0.00499	416	390	0.1918	416	320	0.0	
K	22	11	38	333.79	0.04987	395	6684.5	0.00518	417	390	0.1918	417	320	0.0	
K	23	11	37	333.79	0.04987	395	6684.5	0.00480	415	390	0.1918	415	320	0.0	
K	24	11	36	338.35	0.04987	395	6684.5	0.00575	420	390	0.1918	420	320	0.0	
K	25	11	35	338.35	0.04987	395	6684.5	0.00633	423	390	0.1918	423	320	0.0	
K	26	11	34	311.82	0.04987	400	6684.5	0.00460	419	395	0.1918	419	320	0.0	
K	27	11	33	185.32	0.04987	400	6684.5	0.00326	412	395	0.1918	412	320	0.0	
K	28	11	32	185.32	0.04987	400	6684.5	0.00326	412	395	0.1918	412	320	0.0	
K	29	11	31	185.32	0.04987	400	6684.5	0.00326	412	395	0.1918	412	320	0.0	
K	30	11	30	206.09	0.04987	400	6684.5	0.00345	413	395	0.1918	413	320	0.0	
K	31	11	29	206.09	0.04987	400	6684.5	0.00307	411	395	0.1918	411	320	0.0	
K	32	11	28	206.09	0.04987	400	6684.5	0.00345	413	395	0.1918	413	320	0.0	
K	33	11	27	206.09	0.04987	400	6684.5	0.00364	414	395	0.1918	414	320	0.0	
K	34	11	26	338.35	0.04987	395	0	27.39377	420	380	0.1918	378	340	194.0	
K	35	11	25	338.35	0.04987	405	0	10.27266	405	390	0.1918	380	340	194.5	
K	36	11	24	75.44	0.04987	405	0	0.00193	415	400	0.1918	415	340	0.0	
K	37	11	23	75.44	0.04987	405	0	0.00193	415	400	0.1918	415	340	0.0	
K	38	11	22	0.00	1.91819E+12	415	0	0.00000	415	410	1.918E+12	415	320	0.0	
K	41	11	19	0.00	1.91819E+12	430	0	0.00000	430	420	1.918E+12	430	320	0.0	
K	42	11	18	0.00	1.91819E+12	430	0	0.00000	430	420	1.918E+12	430	320	0.0	
K	46	11	14	0.00	0.04987	410	0	0.00193	420	405	0.1918	420	360	0.0	
L	8	12	52	1.34	1.91819E+12	400	0	0.00000	550	540	1.918E+12	550	380	0.0	
L	9	12	51	212.39	0.04987	395	6684.5	0.00288	405	390	0.1918	405	320	0.0	
L	10	12	50	212.39	0.04987	395	6684.5	0.00288	405	390	0.1918	405	320	0.0	
L	11	12	49	191.08	0.04987	410	0	0.06330	406	373	0.1918	373	300	178.2	
L	12	12	48	267.73	0.04987	395	6684.5	0.00403	411	390	0.1918	411	280	0.0	
L	13	12	47	267.73	0.04987	395	6684.5	0.00384	410	390	0.1918	410	280	0.0	
L	14	12	46	267.73	0.04987	395	6684.5	0.00384	410	390	0.1918	410	280	0.0	
L	15	12	45	358.85	0.04987	395	6684.5	0.00403	411	390	0.1918	411	280	0.0	
L	16	12	44	348.80	0.04987	400	6684.5	0.00269	409	395	0.1918	409	280	0.0	
L	17	12	43	348.80	0.04987	400	6684.5	0.00288	410	395	0.1918	410	280	0.0	

COL PLASM	ROW	COL MODFLOW	ROW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST. RATE cu. ft/day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S. W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
L	18	12	42	348.80	0.04987	400	6684.5	0.00288	410	395	0.1918	410	280	0.0	
L	19	12	41	348.80	0.04987	400	6684.5	0.00269	409	395	0.1918	409	280	0.0	
L	20	12	40	348.80	0.04987	400	6684.5	0.00288	410	395	0.1918	410	280	0.0	
L	21	12	39	459.22	0.04987	400	6684.5	0.00326	412	395	0.1918	412	320	0.0	
L	22	12	38	459.22	0.04987	400	6684.5	0.00403	416	395	0.1918	416	320	0.0	
L	23	12	37	459.22	0.04987	400	6684.5	0.00326	412	395	0.1918	412	320	0.0	
L	24	12	36	404.68	0.04987	400	6684.5	0.00288	410	395	0.1918	410	320	0.0	
L	25	12	35	404.68	0.04987	400	6684.5	0.00269	409	395	0.1918	409	320	0.0	
L	26	12	34	404.68	0.04987	400	6684.5	0.00326	412	395	0.1918	412	320	0.0	
L	27	12	33	404.68	0.04987	400	6684.5	0.00403	416	395	0.1918	416	320	0.0	
L	28	12	32	185.32	0.04987	400	6684.5	0.00364	414	395	0.1918	414	320	0.0	
L	29	12	31	185.32	0.04987	400	6684.5	0.00364	414	395	0.1918	414	320	0.0	
L	30	12	30	300.70	0.04987	405	6684.5	0.00269	414	400	0.1918	414	320	0.0	
L	31	12	29	320.53	0.04987	405	6684.5	0.00345	418	400	0.1918	418	320	0.0	
L	32	12	28	320.53	0.04987	405	6684.5	0.00345	418	400	0.1918	418	320	0.0	
L	33	12	27	320.53	0.04987	405	6684.5	0.00326	417	400	0.1918	417	320	0.0	
L	34	12	26	320.53	0.04987	405	6684.5	0.00269	414	400	0.1918	414	320	0.0	
L	35	12	25	320.53	0.04987	405	6684.5	0.00288	415	400	0.1918	415	320	0.0	
L	36	12	24	253.93	0.04987	405	0	24.65439	418	382	0.1918	382	320	194.0	
L	37	12	23	253.93	0.04987	405	0	15.75142	405	382	0.1918	382	320	194.0	
L	38	12	22	75.44	0.04987	405	0	0.00193	415	400	0.1918	415	340	0.0	
L	39	12	21	75.44	0.04987	405	0	0.00193	415	400	0.1918	415	340	0.0	
L	40	12	20	75.44	0.04987	405	0	0.00193	415	400	0.1918	415	340	0.0	
L	41	12	19	75.44	0.04987	405	0	0.00193	415	400	0.1918	415	360	0.0	
L	42	12	18	59.63	0.04987	405	0	0.00244	405	386	0.1918	386	340	192.1	
L	43	12	17	69.14	0.04987	410	0	0.00193	420	405	0.1918	420	360	0.0	
L	44	12	16	53.60	0.04987	410	0	0.00193	420	405	0.1918	420	380	0.0	
L	45	12	15	69.14	0.04987	410	0	0.00193	420	405	0.1918	420	360	0.0	
L	46	12	14	0.00	0.04987	410	0	0.00193	420	405	0.1918	420	360	0.0	
M	9	13	51	0.00	1.91819E+12	400	0	0.00000	550	540	1.918E+12	550	380	0.0	
M	10	13	50	1.34	0.04987	400	8957.2	0.00257	450	430	0.1918	450	360	0.0	
M	11	13	49	191.08	0.04987	410	0	0.06906	409	373	0.1918	373	300	178.7	
M	12	13	48	299.49	0.04987	395	6684.5	0.00384	410	390	0.1918	410	300	0.0	
M	13	13	47	267.73	0.04987	395	6684.5	0.00384	410	390	0.1918	410	280	0.0	
M	14	13	46	267.73	0.04987	395	6684.5	0.00345	408	390	0.1918	408	280	0.0	
M	15	13	45	348.80	0.04987	400	6684.5	0.00249	408	395	0.1918	408	280	0.0	
M	16	13	44	348.80	0.04987	400	6684.5	0.00269	409	395	0.1918	409	280	0.0	
M	17	13	43	348.80	0.04987	400	6684.5	0.00288	410	395	0.1918	410	280	0.0	
M	18	13	42	348.80	0.04987	400	6684.5	0.00307	411	395	0.1918	411	280	0.0	
M	19	13	41	261.30	0.04987	400	6684.5	0.00288	410	395	0.1918	410	280	0.0	
M	20	13	40	348.80	0.04987	400	6684.5	0.00345	413	395	0.1918	413	280	0.0	
M	21	13	39	348.80	0.04987	400	6684.5	0.00384	415	395	0.1918	415	280	0.0	
M	22	13	38	348.80	0.04987	400	6684.5	0.00422	417	395	0.1918	417	280	0.0	
M	23	13	37	459.22	0.04987	400	6684.5	0.00441	418	395	0.1918	418	320	0.0	
M	24	13	36	404.68	0.04987	400	6684.5	0.00422	417	395	0.1918	417	320	0.0	
M	25	13	35	404.68	0.04987	400	6684.5	0.00230	407	395	0.1918	407	320	0.0	
M	26	13	34	404.68	0.04987	400	6684.5	0.00249	408	395	0.1918	408	320	0.0	
M	27	13	33	404.68	0.04987	400	6684.5	0.00326	412	395	0.1918	412	320	0.0	
M	28	13	32	311.82	0.04987	400	6684.5	0.00307	411	395	0.1918	411	320	0.0	
M	29	13	31	311.82	0.04987	400	6684.5	0.00364	414	395	0.1918	414	320	0.0	
M	30	13	30	320.53	0.04987	405	6684.5	0.00249	413	400	0.1918	413	320	0.0	
M	31	13	29	320.53	0.04987	405	6684.5	0.00288	415	400	0.1918	415	320	0.0	
M	32	13	28	320.53	0.04987	405	6684.5	0.00422	422	400	0.1918	422	320	0.0	

COL PLASM	ROW PLASM	COL MODFLOW	ROW MODFLOW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft./day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
M	33	13	27	320.53	0.04987	405	6684.5	0.00384	420	400	0.1918	420	320	0.0	
M	34	13	26	320.53	0.04987	405	6684.5	0.00230	420	400	0.1918	420	320	0.0	
M	35	13	25	346.39	0.04987	405	6684.5	0.00345	420	390	0.1918	420	320	0.0	
M	36	13	24	346.39	0.04987	405	6684.5	0.00391	424	390	0.1918	424	320	0.0	
M	37	13	23	346.39	0.04987	405	6684.5	0.00357	421	390	0.1918	421	320	0.0	
M	38	13	22	249.11	0.04987	405	0	14.38173	405	384	0.1918	384	320	194.0	
M	39	13	21	200.60	0.04987	405	0	0.00193	415	400	0.1918	415	320	0.0	
M	40	13	20	227.26	0.04987	405	0	0.00193	415	400	0.1918	415	340	0.0	
M	41	13	19	75.44	0.04987	405	0	0.00193	415	400	0.1918	415	340	0.0	
M	42	13	18	75.44	0.04987	405	0	0.00193	415	400	0.1918	415	340	0.0	
M	43	13	17	59.63	0.04987	405	0	26.02408	405	386	0.1918	386	340	192.1	
M	44	13	16	259.02	0.04987	410	0	0.00193	420	405	0.1918	420	360	0.0	
M	45	13	15	259.02	0.04987	410	0	0.00193	420	405	0.1918	420	360	0.0	
M	46	13	14	0.00	0.04987	410	0	0.00193	420	405	0.1918	420	360	0.0	
N	10	14	50	0.00	1.91819E+12	400	0	0.00000	475	465	1.918E+12	475	380	0.0	
N	11	14	49	191.08	0.04987	395	6684.5	0.00288	405	390	0.1918	405	300	0.0	
N	12	14	48	299.49	0.04987	395	6684.5	0.00384	410	390	0.1918	410	300	0.0	
N	13	14	47	299.49	0.04987	395	6684.5	0.00288	405	390	0.1918	405	300	0.0	
N	14	14	46	290.51	0.04987	400	6684.5	0.00192	405	395	0.1918	405	300	0.0	
N	15	14	45	290.51	0.04987	400	6684.5	0.00230	407	395	0.1918	407	300	0.0	
N	16	14	44	261.30	0.04987	400	6684.5	0.00288	410	395	0.1918	410	280	0.0	
N	17	14	43	261.30	0.04987	400	6684.5	0.00326	412	395	0.1918	412	280	0.0	
N	18	14	42	261.30	0.04987	400	6684.5	0.00364	414	395	0.1918	414	280	0.0	
N	19	14	41	261.30	0.04987	400	6684.5	0.00384	415	395	0.1918	415	280	0.0	
N	20	14	40	261.30	0.04987	400	6684.5	0.00403	416	395	0.1918	416	280	0.0	
N	21	14	39	261.30	0.04987	400	6684.5	0.00422	417	395	0.1918	417	280	0.0	
N	22	14	38	261.30	0.04987	400	6684.5	0.00480	420	395	0.1918	420	280	0.0	
N	23	14	37	332.72	0.04987	400	6684.5	0.00364	414	395	0.1918	414	320	0.0	
N	24	14	36	332.72	0.04987	400	6684.5	0.00403	416	395	0.1918	416	320	0.0	
N	25	14	35	311.62	0.04987	400	6684.5	0.00249	408	395	0.1918	408	320	0.0	
N	26	14	34	311.82	0.04987	400	6684.5	0.00249	408	395	0.1918	408	320	0.0	
N	27	14	33	311.82	0.04987	400	6684.5	0.00288	410	395	0.1918	410	320	0.0	
N	28	14	32	311.82	0.04987	400	6684.5	0.00288	410	395	0.1918	410	320	0.0	
N	29	14	31	311.82	0.04987	400	6684.5	0.00326	412	395	0.1918	412	320	0.0	
N	30	14	30	320.53	0.04987	405	6684.5	0.00249	413	400	0.1918	413	320	0.0	
N	31	14	29	309.54	0.04987	410	0	0.00460	412	400	0.1918	412	320	0.0	
N	32	14	28	309.54	0.04987	410	0	0.00269	422	415	0.1918	422	320	0.0	
N	33	14	27	309.54	0.04987	410	0	0.00192	420	415	0.1918	420	320	0.0	
N	34	14	26	320.53	0.04987	405	6684.5	0.00364	419	400	0.1918	419	320	0.0	
N	35	14	25	320.53	0.04987	405	6684.5	0.00422	422	400	0.1918	422	320	0.0	
N	36	14	24	320.53	0.04987	405	6684.5	0.00441	423	400	0.1918	423	320	0.0	
N	37	14	23	320.53	0.04987	405	6684.5	0.00460	424	400	0.1918	424	320	0.0	
N	38	14	22	320.53	0.04987	405	6684.5	0.00441	423	400	0.1918	423	320	0.0	
N	39	14	21	407.90	0.04987	405	0	10.27266	405	390	0.1918	388	320	194.0	
N	40	14	20	504.11	0.04987	405	0	11.64235	405	388	0.1918	388	320	194.0	
N	41	14	19	200.60	0.04987	405	0	0.00193	415	400	0.1918	415	320	0.0	
N	42	14	18	227.26	0.04987	405	0	0.00193	415	400	0.1918	415	340	0.0	
N	43	14	17	227.26	0.04987	405	0	0.00193	415	400	0.1918	415	340	0.0	
N	44	14	16	296.69	0.04987	405	0	23.28470	405	388	0.1918	388	340	192.8	
N	45	14	15	219.36	0.04987	410	0	0.00193	420	405	0.1918	420	340	0.0	
N	46	14	14	0.00	0.04987	410	0	0.00193	420	405	0.1918	420	340	0.0	
O	10	15	50	0.00	1.91819E+12	400	0	0.00000	475	465	1.918E+12	475	380	0.0	
O	11	15	49	169.91	0.04987	400	6684.5	0.00269	409	395	0.1918	409	300	0.0	

COL PLASM	ROW PLASM	COL MODFLOW	ROW MODFLOW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft./day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
O	12	15	48	281.67	0.04987	395	6684.5	0.00307	406	390	0.1918	406	300	0.0	
O	13	15	47	273.36	0.04987	400	6684.5	0.00173	404	395	0.1918	404	300	0.0	
O	14	15	46	273.36	0.04987	400	6684.5	0.00230	407	395	0.1918	407	300	0.0	
O	15	15	45	273.36	0.04987	400	6684.5	0.00211	406	395	0.1918	406	300	0.0	
O	16	15	44	273.36	0.04987	400	6684.5	0.00326	412	395	0.1918	412	300	0.0	
O	17	15	43	273.36	0.04987	400	6684.5	0.00364	414	395	0.1918	410	300	0.0	
O	18	15	42	246.83	0.04987	400	6684.5	0.00441	418	395	0.1918	418	280	0.0	
O	19	15	41	261.30	0.04987	400	6684.5	0.00345	413	395	0.1918	413	280	0.0	
O	20	15	40	261.30	0.04987	400	6684.5	0.00364	414	395	0.1918	414	280	0.0	
O	21	15	39	261.30	0.04987	405	6684.5	0.00307	416	400	0.1918	416	280	0.0	
O	22	15	38	261.30	0.04987	405	6684.5	0.00326	417	400	0.1918	417	280	0.0	
O	23	15	37	261.30	0.04987	405	6684.5	0.00326	417	400	0.1918	417	280	0.0	
O	24	15	36	290.51	0.04987	400	6684.5	0.00153	403	395	0.1918	403	300	0.0	
O	25	15	35	290.51	0.04987	400	6684.5	0.00288	410	395	0.1918	410	300	0.0	
O	26	15	34	273.36	0.04987	400	6684.5	0.00269	409	395	0.1918	409	300	0.0	
O	27	15	33	273.36	0.04987	400	6684.5	0.00364	414	395	0.1918	414	300	0.0	
O	28	15	32	294.53	0.04987	400	0	0.01842	406	398	0.1918	398	300	0.0	
O	29	15	31	273.36	0.04987	400	6684.5	0.00249	408	395	0.1918	408	300	0.0	
O	30	15	30	265.86	0.04987	405	6684.5	0.00288	415	400	0.1918	415	300	0.0	
O	31	15	29	320.53	0.04987	405	6684.5	0.00326	417	400	0.1918	417	320	0.0	
O	32	15	28	309.54	0.04987	410	0	0.00153	419	415	0.1918	419	320	0.0	
O	33	15	27	423.44	0.04987	410	0	0.00230	421	415	0.1918	421	320	0.0	
O	34	15	26	423.44	0.04987	410	0	0.00153	419	415	0.1918	419	320	0.0	
O	35	15	25	423.44	0.04987	410	0	0.00192	420	415	0.1918	420	320	0.0	
O	36	15	24	423.44	0.04987	410	0	0.00192	420	415	0.1918	420	320	0.0	
O	37	15	23	423.44	0.04987	410	0	0.00345	424	415	0.1918	424	320	0.0	
O	38	15	22	440.32	0.04987	405	6684.5	0.00441	423	400	0.1918	423	320	0.0	
O	39	15	21	440.32	0.04987	405	6684.5	0.00288	415	400	0.1918	415	320	0.0	
O	40	15	20	440.32	0.04987	405	6684.5	0.00326	417	400	0.1918	417	320	0.0	
O	41	15	19	504.11	0.04987	405	0	18.49079	415	388	0.1918	388	320	194.0	
O	42	15	18	504.11	0.04987	405	0	11.64235	405	388	0.1918	388	320	194.0	
O	43	15	17	240.40	0.04987	405	0	23.28470	405	388	0.1918	388	320	192.8	
O	44	15	16	240.40	0.04987	405	0	23.28470	405	388	0.1918	388	320	193.5	
O	45	15	15	195.64	0.04987	410	0	0.00193	420	405	0.1918	420	320	0.0	
O	46	15	14	0.00	0.04987	410	0	0.00193	420	405	0.1918	420	320	0.0	
P	10	16	50	0.00	1.91819E+12	400	0	0.00000	475	465	1.918E+12	475	380	0.0	
P	11	16	49	185.32	0.04987	400	6684.5	0.00345	413	395	0.1918	413	320	0.0	
P	12	16	48	273.36	0.04987	400	6684.5	0.00364	414	395	0.1918	414	300	0.0	
P	13	16	47	273.36	0.04987	400	6684.5	0.00326	412	395	0.1918	412	300	0.0	
P	14	16	46	273.36	0.04987	400	6684.5	0.00307	411	395	0.1918	411	300	0.0	
P	15	16	45	273.36	0.04987	400	6684.5	0.00364	414	395	0.1918	414	300	0.0	
P	16	16	44	273.36	0.04987	400	6684.5	0.00384	415	395	0.1918	415	300	0.0	
P	17	16	43	273.36	0.04987	400	6684.5	0.00422	417	395	0.1918	417	300	0.0	
P	18	16	42	241.33	0.04987	405	6684.5	0.00403	421	400	0.1918	421	280	0.0	
P	19	16	41	255.27	0.04987	405	6684.5	0.00441	423	400	0.1918	423	280	0.0	
P	20	16	40	255.27	0.04987	405	6684.5	0.00307	416	400	0.1918	416	280	0.0	
P	21	16	39	255.27	0.04987	405	6684.5	0.00326	417	400	0.1918	417	280	0.0	
P	22	16	38	255.27	0.04987	405	6684.5	0.00326	417	400	0.1918	417	280	0.0	
P	23	16	37	281.30	0.04987	400	6684.5	0.00403	416	395	0.1918	416	280	0.0	
P	24	16	36	261.30	0.04987	400	6684.5	0.00173	404	395	0.1918	404	280	0.0	
P	25	16	35	290.51	0.04987	400	6684.5	0.00269	409	395	0.1918	409	300	0.0	
P	26	16	34	290.51	0.04987	400	6684.5	0.00403	416	395	0.1918	418	300	0.0	
P	27	16	33	273.36	0.04987	400	6684.5	0.00326	412	395	0.1918	412	300	0.0	

COL PLASM	ROW	COL MODFLOW	ROW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft./day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
P	26	18	32	273.36	0.04987	400	6684.5	0.00307	411	395	0.1918	411	300	0.0	
P	29	18	31	294.53	0.04987	400	0	0.01842	406	398	0.1918	398	300	0.0	
P	30	18	30	273.36	0.04987	400	6684.5	0.00249	408	395	0.1918	408	300	0.0	
P	31	16	29	265.86	0.04987	405	6684.5	0.00230	412	400	0.1918	412	300	0.0	
P	32	18	28	282.20	0.04987	405	6684.5	0.00288	415	400	0.1918	415	300	0.0	
P	33	16	27	369.57	0.04987	410	0	0.00192	420	415	0.1918	420	300	0.0	
P	34	16	26	369.57	0.04987	410	0	0.00115	418	415	0.1918	418	300	0.0	
P	35	16	25	369.57	0.04987	410	0	0.00153	419	415	0.1918	419	300	0.0	
P	36	16	24	369.57	0.04987	410	0	0.00192	420	415	0.1918	420	300	0.0	
P	37	16	23	440.32	0.04987	405	6684.5	0.00364	419	400	0.1918	419	320	0.0	
P	38	16	22	423.44	0.04987	410	0	0.00230	421	415	0.1918	421	320	0.0	
P	39	16	21	440.32	0.04987	405	6684.5	0.00288	415	400	0.1918	415	320	0.0	
P	40	16	20	440.32	0.04987	405	6684.5	0.00288	415	400	0.1918	415	320	0.0	
P	41	16	19	440.32	0.04987	405	6684.5	0.00249	413	400	0.1918	413	320	0.0	
P	42	16	18	440.32	0.04987	405	6684.5	0.00173	409	400	0.1918	409	320	0.0	
P	43	16	17	440.32	0.04987	405	6684.5	0.00249	413	400	0.1918	413	320	0.0	
P	44	16	16	504.11	0.04987	405	0	17.80595	414	388	0.1918	388	320	194.0	
P	45	18	15	407.90	0.04987	405	0	11.64235	405	388	0.1918	388	320	194.0	
P	46	16	14	240.40	0.04987	405	0	23.28470	405	388	0.1918	388	320	193.5	
Q	10	17	50	0.00	1.91819E+12	405	0	0.00000	475	465	1.918E+12	475	380	0.0	
Q	11	17	49	311.82	0.04987	400	6684.5	0.00182	405	395	0.1918	405	320	0.0	
Q	12	17	48	311.82	0.04987	400	6684.5	0.00326	412	395	0.1918	412	320	0.0	
Q	13	17	47	273.36	0.04987	400	6684.5	0.00384	415	395	0.1918	415	300	0.0	
Q	14	17	46	273.36	0.04987	400	6684.5	0.00345	413	395	0.1918	413	300	0.0	
Q	15	17	45	273.36	0.04987	400	6684.5	0.00384	415	395	0.1918	415	300	0.0	
Q	16	17	44	265.86	0.04987	405	6684.5	0.00345	418	400	0.1918	418	300	0.0	
Q	17	17	43	265.86	0.04987	405	6684.5	0.00345	418	400	0.1918	418	300	0.0	
Q	18	17	42	265.86	0.04987	405	6684.5	0.00403	421	400	0.1918	421	300	0.0	
Q	19	17	41	265.86	0.04987	405	6684.5	0.00633	433	400	0.1918	433	300	0.0	
Q	20	17	40	241.33	0.04987	405	6684.5	0.00307	416	400	0.1918	416	280	0.0	
Q	21	17	39	241.33	0.04987	405	6684.5	0.00326	417	400	0.1918	417	280	0.0	
Q	22	17	38	255.27	0.04987	405	6684.5	0.00288	415	400	0.1918	415	280	0.0	
Q	23	17	37	255.27	0.04987	405	6684.5	0.00307	416	400	0.1918	416	280	0.0	
Q	24	17	36	261.30	0.04987	400	6684.5	0.00345	413	395	0.1918	413	280	0.0	
Q	25	17	35	261.30	0.04987	400	6684.5	0.00134	402	395	0.1918	402	280	0.0	
Q	26	17	34	290.51	0.04987	400	6684.5	0.00153	403	395	0.1918	403	300	0.0	
Q	27	17	33	273.36	0.04987	400	6684.5	0.00307	411	395	0.1918	411	300	0.0	
Q	28	17	32	273.36	0.04987	400	6684.5	0.00326	412	395	0.1918	412	300	0.0	
Q	29	17	31	273.36	0.04987	400	6684.5	0.00364	414	395	0.1918	414	300	0.0	
Q	30	17	30	273.36	0.04987	400	6684.5	0.00173	404	395	0.1918	404	300	0.0	
Q	31	17	29	294.53	0.04987	403	0	0.01842	406	398	0.1918	398	300	0.0	
Q	32	17	28	265.86	0.04987	405	6684.5	0.00384	420	400	0.1918	420	300	0.0	
Q	33	17	27	265.86	0.04987	405	6684.5	0.00364	419	400	0.1918	419	300	0.0	
Q	34	17	26	353.89	0.04987	410	0	0.00192	420	415	0.1918	420	300	0.0	
Q	35	17	25	353.89	0.04987	410	0	0.00269	422	415	0.1918	422	300	0.0	
Q	36	17	24	353.89	0.04987	410	0	0.00269	422	415	0.1918	422	300	0.0	
Q	37	17	23	353.89	0.04987	410	0	0.00192	420	415	0.1918	420	300	0.0	
Q	38	17	22	282.20	0.04987	405	6684.5	0.00384	420	400	0.1918	420	300	0.0	
Q	39	17	21	282.20	0.04987	405	6684.5	0.00288	415	400	0.1918	415	300	0.0	
Q	40	17	20	282.20	0.04987	405	6684.5	0.00249	413	400	0.1918	413	300	0.0	
Q	41	17	19	369.57	0.04987	410	8957.2	0.00231	413	395	0.1918	413	300	0.0	
Q	42	17	18	369.57	0.04987	410	6957.2	0.00270	416	395	0.1918	416	300	0.0	
Q	43	17	17	274.70	0.04987	410	6957.2	0.00244	414	395	0.1918	414	300	0.0	

COL PLASM	ROW	COL MODFLOW	ROW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft./day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
Q	44	17	16	179.69	0.04987	410	8957.2	0.00257	415	395	0.1918	415	300	0.0	
Q	45	17	15	179.69	0.04987	410	0	0.00321	430	405	0.1918	430	300	0.0	
Q	46	17	14	207.30	0.04987	405	0	36.98159	416	389	0.1918	389	300	195.2	
R	11	18	49	0.00	1.91819E+12	400	0	0.00000	500	490	1.918E+12	500	380	0.0	
R	12	18	48	185.32	0.04987	400	6684.5	0.00288	410	395	0.1918	410	320	0.0	
R	13	18	47	311.82	0.04987	400	6684.5	0.00384	415	395	0.1918	415	320	0.0	
R	14	18	46	273.36	0.04987	400	6684.5	0.00403	416	395	0.1918	416	300	0.0	
R	15	18	45	273.36	0.04987	400	6684.5	0.00384	415	395	0.1918	415	300	0.0	
R	16	18	44	273.36	0.04987	400	6684.5	0.00326	412	395	0.1918	412	300	0.0	
R	17	18	43	265.86	0.04987	405	6684.5	0.00288	415	400	0.1918	415	300	0.0	
R	18	18	42	265.86	0.04987	405	6684.5	0.00211	411	400	0.1918	411	300	0.0	
R	19	18	41	265.86	0.04987	405	6684.5	0.00192	410	400	0.1918	410	300	0.0	
R	20	18	40	241.33	0.04987	405	6684.5	0.00269	414	400	0.1918	414	280	0.0	
R	21	18	39	241.33	0.04987	405	6684.5	0.00288	415	400	0.1918	415	280	0.0	
R	22	18	38	255.27	0.04987	405	6684.5	0.00384	420	400	0.1918	420	280	0.0	
R	23	18	37	255.27	0.04987	405	6684.5	0.00288	415	400	0.1918	415	280	0.0	
R	24	18	36	255.27	0.04987	405	6684.5	0.00345	418	400	0.1918	418	280	0.0	
R	25	18	35	261.30	0.04987	400	6684.5	0.00364	414	395	0.1918	414	280	0.0	
R	26	18	34	261.30	0.04987	400	6684.5	0.00134	402	395	0.1918	402	280	0.0	
R	27	18	33	290.51	0.04987	400	6684.5	0.00211	406	395	0.1918	406	300	0.0	
R	28	18	32	273.36	0.04987	400	6684.5	0.00288	410	395	0.1918	410	300	0.0	
R	29	18	31	273.36	0.04987	400	6684.5	0.00364	414	395	0.1918	414	300	0.0	
R	30	18	30	273.36	0.04987	400	6684.5	0.00364	414	395	0.1918	414	300	0.0	
R	31	18	29	294.53	0.04987	403	0	0.01842	406	398	0.1918	398	300	0.0	
R	32	18	28	294.53	0.04987	403	0	0.01842	406	398	0.1918	398	300	0.0	
R	33	18	27	265.86	0.04987	405	6684.5	0.00364	419	400	0.1918	419	300	0.0	
R	34	18	26	265.86	0.04987	405	6684.5	0.00364	419	400	0.1918	419	300	0.0	
R	35	18	25	259.02	0.04987	410	0	0.00269	422	415	0.1918	422	300	0.0	
R	36	18	24	259.02	0.04987	410	0	0.00192	420	415	0.1918	420	300	0.0	
R	37	18	23	259.02	0.04987	410	0	0.00153	419	415	0.1918	419	300	0.0	
R	38	18	22	259.02	0.04987	410	0	0.00230	421	415	0.1918	421	300	0.0	
R	39	18	21	274.70	0.04987	410	8957.2	0.00308	419	395	0.1918	419	300	0.0	
R	40	18	20	274.70	0.04987	410	8957.2	0.00244	414	395	0.1918	414	300	0.0	
R	41	18	19	274.70	0.04987	410	8957.2	0.00218	412	395	0.1918	412	300	0.0	
R	42	18	18	274.70	0.04987	410	8957.2	0.00231	413	395	0.1918	413	300	0.0	
R	43	18	17	274.70	0.04987	410	8957.2	0.00244	414	395	0.1918	414	300	0.0	
R	44	18	16	274.70	0.04987	410	8957.2	0.00283	417	395	0.1918	417	300	0.0	
R	45	18	15	274.70	0.04987	410	8957.2	0.00437	429	395	0.1918	429	300	0.0	
R	46	18	14	274.70	0.04987	405	0	0.05270	430	389	0.1918	389	300	195.6	
S	12	19	48	0.00	1.91819E+12	405	0	0.00000	500	490	1.918E+12	500	380	0.0	
S	13	19	47	185.32	0.04987	400	6684.5	0.00230	407	395	0.1918	407	320	0.0	
S	14	19	46	185.32	0.04987	400	6684.5	0.00230	407	395	0.1918	407	320	0.0	
S	15	19	45	185.32	0.04987	400	6684.5	0.00307	411	395	0.1918	411	320	0.0	
S	16	19	44	169.91	0.04987	400	6684.5	0.00364	414	395	0.1918	414	300	0.0	
S	17	19	43	273.36	0.04987	400	6684.5	0.00307	411	395	0.1918	411	300	0.0	
S	18	19	42	281.67	0.04987	410	0	0.01343	412	405	0.1918	415	300	0.0	
S	19	19	41	265.86	0.04987	405	6684.5	0.00249	413	400	0.1918	413	300	0.0	
S	20	19	40	265.86	0.04987	405	6684.5	0.00269	414	400	0.1918	414	300	0.0	
S	21	19	39	265.86	0.04987	405	6684.5	0.00230	412	400	0.1918	412	300	0.0	
S	22	19	38	255.27	0.04987	405	6684.5	0.00422	422	400	0.1918	422	280	0.0	
S	23	19	37	255.27	0.04987	405	6684.5	0.00288	415	400	0.1918	415	280	0.0	
S	24	19	36	255.27	0.04987	405	6684.5	0.00364	419	400	0.1918	419	280	0.0	
S	25	19	35	255.27	0.04987	405	6684.5	0.00403	421	400	0.1918	421	280	0.0	

COL PLASM	ROW MODFLOW	COL	ROW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft./day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
S	26	19	34	261.30	0.04987	400	6684.5	0.00269	409	395	0.1918	409	280	0.0	
S	27	19	33	261.30	0.04987	400	6684.5	0.00192	405	395	0.1918	405	280	0.0	
S	28	19	32	261.30	0.04987	400	6684.5	0.00307	411	395	0.1918	411	280	0.0	
S	29	19	31	246.83	0.04987	400	6684.5	0.00384	415	395	0.1918	415	280	0.0	
S	30	19	30	246.83	0.04987	400	6684.5	0.00326	412	395	0.1918	412	280	0.0	
S	31	19	29	246.83	0.04987	400	6684.5	0.00288	410	395	0.1918	410	280	0.0	
S	32	19	28	261.70	0.04987	403	0	0.01842	406	398	0.1918	398	280	0.0	
S	33	19	27	265.86	0.04987	405	6684.5	0.00364	419	400	0.1918	419	300	0.0	
S	34	19	26	265.86	0.04987	405	6684.5	0.00384	420	400	0.1918	420	300	0.0	
S	35	19	25	265.86	0.04987	405	6684.5	0.00422	422	400	0.1918	422	300	0.0	
S	36	19	24	265.86	0.04987	405	6684.5	0.00364	419	400	0.1918	419	300	0.0	
S	37	19	23	259.02	0.04987	410	8957.2	0.00283	417	395	0.1918	417	300	0.0	
S	38	19	22	259.02	0.04987	410	8957.2	0.00321	420	395	0.1918	420	300	0.0	
S	39	19	21	259.02	0.04987	410	8957.2	0.00257	415	395	0.1918	415	300	0.0	
S	40	19	20	259.02	0.04987	410	8957.2	0.00386	425	395	0.1918	425	300	0.0	
S	41	19	19	259.02	0.04987	410	8957.2	0.00321	420	395	0.1918	420	300	0.0	
S	42	19	18	259.02	0.04987	410	8957.2	0.00334	421	395	0.1918	421	300	0.0	
S	43	19	17	259.02	0.04987	410	8957.2	0.00257	415	395	0.1918	415	300	0.0	
S	44	19	16	259.02	0.04987	410	8957.2	0.00257	415	395	0.1918	415	300	0.0	
S	45	19	15	252.72	0.04987	410	8957.2	0.00334	426	400	0.1918	426	300	0.0	
S	46	19	14	252.72	0.04987	410	0	0.04884	427	389	0.1918	389	300	196.1	
T	13	20	47	0.00	1.91619E+12	405	0	0.00000	500	490	1.918E+12	500	320	0.0	
T	14	20	46	185.32	0.04987	400	6684.5	0.00153	403	395	0.1918	403	320	0.0	
T	15	20	45	263.85	0.04987	410	0	0.00384	407	405	0.1918	407	300	0.0	
T	16	20	44	263.85	0.04987	410	0	0.00384	407	405	0.1918	407	300	0.0	
T	17	20	43	249.64	0.01535	405	6684.5	0.00230	412	400	0.1918	406	300	0.0	
T	18	20	42	249.64	0.01535	405	6684.5	0.00269	414	400	0.1918	406	300	0.0	
T	19	20	41	249.64	0.01535	405	6684.5	0.00288	415	400	0.1918	406	300	0.0	
T	20	20	40	265.86	0.04987	405	6684.5	0.00307	416	400	0.1918	416	300	0.0	
T	21	20	39	265.86	0.04987	405	6684.5	0.00422	422	400	0.1918	422	300	0.0	
T	22	20	38	255.27	0.04987	405	6684.5	0.00384	420	400	0.1918	420	280	0.0	
T	23	20	37	255.27	0.04987	405	6684.5	0.00307	416	400	0.1918	416	280	0.0	
T	24	20	36	255.27	0.04987	405	6684.5	0.00364	419	400	0.1918	419	280	0.0	
T	25	20	35	255.27	0.04987	405	6684.5	0.00384	420	400	0.1918	420	280	0.0	
T	26	20	34	255.27	0.04987	405	6684.5	0.00364	419	400	0.1918	419	280	0.0	
T	27	20	33	255.27	0.04987	405	6684.5	0.00288	415	400	0.1918	415	280	0.0	
T	28	20	32	277.65	0.04987	403	0	0.01842	406	398	0.1918	398	280	0.0	
T	29	20	31	277.65	0.04987	403	0	0.01842	406	398	0.1918	398	280	0.0	
T	30	20	30	261.70	0.04987	403	0	0.01842	406	398	0.1918	398	280	0.0	
T	31	20	29	261.70	0.04987	403	0	0.01842	406	398	0.1918	398	280	0.0	
T	32	20	28	261.70	0.04987	403	0	0.02762	406	398	0.1918	398	280	0.0	
T	33	20	27	261.70	0.04987	403	0	0.01842	406	398	0.1918	398	280	0.0	
T	34	20	26	241.33	0.04987	405	6684.5	0.00345	418	400	0.1918	418	280	0.0	
T	35	20	25	241.33	0.04987	405	6684.5	0.00345	418	400	0.1918	418	280	0.0	
T	36	20	24	241.33	0.04987	405	6684.5	0.00345	418	400	0.1918	418	280	0.0	
T	37	20	23	241.33	0.04987	405	6684.5	0.00326	417	400	0.1918	417	280	0.0	
T	38	20	22	241.33	0.04987	405	6684.5	0.00326	417	400	0.1918	417	280	0.0	
T	39	20	21	241.33	0.04987	405	6684.5	0.00364	419	400	0.1918	419	280	0.0	
T	40	20	20	236.38	0.04987	410	8957.2	0.00373	424	395	0.1918	424	280	0.0	
T	41	20	19	236.38	0.04987	410	8957.2	0.00411	427	395	0.1918	427	280	0.0	
T	42	20	18	236.38	0.04987	410	8957.2	0.00398	426	395	0.1918	426	280	0.0	
T	43	20	17	236.38	0.04987	410	8957.2	0.00386	425	395	0.1918	425	280	0.0	
T	44	20	16	231.69	0.04987	410	8957.2	0.00334	426	400	0.1918	426	280	0.0	

COL PLASM	ROW	COL MODFLOW	ROW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft./day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
T	45	20	15	231.69	0.04987	410	8957.2	0.00334	426	400	0.1918	426	280	0.0	
T	46	20	14	231.69	0.04987	410	0	0.00488	427	389	0.1918	389	280	196.7	
U	14	21	46	0.00	1.91819E+12	400	0	0.00000	500	490	1.918E+12	500	380	0.0	
U	15	21	45	272.02	0.04987	405	8957.2	0.00244	409	390	0.1918	409	320	0.0	
U	16	21	44	272.02	0.01535	405	8957.2	0.00270	411	390	0.1918	406	320	0.0	
U	17	21	43	272.02	0.01535	405	8957.2	0.00308	414	390	0.1918	406	320	0.0	
U	18	21	42	249.64	0.01535	405	8957.2	0.00283	412	390	0.1918	406	300	0.0	
U	19	21	41	249.64	0.01535	405	6684.5	0.00288	415	400	0.1918	406	300	0.0	
U	20	21	40	265.86	0.01535	405	6684.5	0.00269	414	400	0.1918	406	300	0.0	
U	21	21	39	265.86	0.04987	405	6684.5	0.00288	415	400	0.1918	415	300	0.0	
U	22	21	38	262.20	0.04987	405	6684.5	0.00288	415	400	0.1918	415	300	0.0	
U	23	21	37	255.27	0.04987	405	6684.5	0.00307	416	400	0.1918	416	280	0.0	
U	24	21	36	255.27	0.04987	405	6684.5	0.00345	418	400	0.1918	418	280	0.0	
U	25	21	35	255.27	0.04987	405	6684.5	0.00364	419	400	0.1918	419	280	0.0	
U	26	21	34	255.27	0.04987	405	6684.5	0.00326	417	400	0.1918	417	280	0.0	
U	27	21	33	255.27	0.04987	405	6684.5	0.00230	412	400	0.1918	412	280	0.0	
U	28	21	32	255.27	0.04987	405	6684.5	0.00288	415	400	0.1918	415	280	0.0	
U	29	21	31	277.65	0.04987	403	0	0.01842	406	398	0.1918	398	280	0.0	
U	30	21	30	255.27	0.04987	405	6684.5	0.00134	407	400	0.1918	407	280	0.0	
U	31	21	29	255.27	0.04987	405	6684.5	0.00153	408	400	0.1918	408	280	0.0	
U	32	21	28	261.70	0.04987	403	0	0.01842	406	398	0.1918	398	280	0.0	
U	33	21	27	241.33	0.04987	405	6684.5	0.00192	410	400	0.1918	410	280	0.0	
U	34	21	26	241.33	0.04987	405	6684.5	0.00288	415	400	0.1918	415	280	0.0	
U	35	21	25	241.33	0.04987	405	6684.5	0.00269	414	400	0.1918	414	280	0.0	
U	36	21	24	241.33	0.04987	405	6684.5	0.00345	418	400	0.1918	418	280	0.0	
U	37	21	23	241.33	0.04987	405	6684.5	0.00384	420	400	0.1918	420	280	0.0	
U	38	21	22	236.38	0.04987	410	8957.2	0.00308	419	395	0.1918	419	280	0.0	
U	39	21	21	236.38	0.04987	410	8957.2	0.00347	422	395	0.1918	422	280	0.0	
U	40	21	20	236.38	0.04987	410	8957.2	0.00360	423	395	0.1918	423	280	0.0	
U	41	21	19	236.38	0.04987	410	8957.2	0.00373	424	395	0.1918	424	280	0.0	
U	42	21	18	236.38	0.04987	410	8957.2	0.00398	426	395	0.1918	426	280	0.0	
U	43	21	17	236.38	0.04987	410	8957.2	0.00386	425	395	0.1918	425	280	0.0	
U	44	21	16	231.69	0.04987	410	8957.2	0.00296	423	400	0.1918	423	280	0.0	
U	45	21	15	231.69	0.04987	410	8957.2	0.00308	424	400	0.1918	424	280	0.0	
U	46	21	14	231.69	0.04987	410	0	0.04627	425	389	0.1918	389	280	197.2	
V	15	22	45	0.00	1.91819E+12	400	0	0.00000	500	490	1.918E+12	500	380	0.0	
V	16	22	44	161.07	0.01535	405	6684.5	0.00499	416	390	0.1918	406	320	0.0	
V	17	22	43	272.02	0.01535	405	6684.5	0.00422	417	395	0.1918	406	320	0.0	
V	18	22	42	272.02	0.01535	405	6684.5	0.00595	421	390	0.1918	406	320	0.0	
V	19	22	41	249.64	0.01535	405	6684.5	0.00345	418	400	0.1918	406	300	0.0	
V	20	22	40	249.64	0.01535	405	6684.5	0.00364	419	400	0.1918	406	300	0.0	
V	21	22	39	265.86	0.04987	405	6684.5	0.00441	423	400	0.1918	423	300	0.0	
V	22	22	38	265.86	0.04987	405	6684.5	0.00364	419	400	0.1918	419	300	0.0	
V	23	22	37	255.27	0.04987	405	6684.5	0.00249	413	400	0.1918	413	280	0.0	
V	24	22	36	339.69	0.04987	405	6684.5	0.00269	414	400	0.1918	414	280	0.0	
V	25	22	35	339.69	0.04987	405	6684.5	0.00307	416	400	0.1918	416	280	0.0	
V	26	22	34	339.69	0.04987	405	6684.5	0.00364	419	400	0.1918	419	280	0.0	
V	27	22	33	255.27	0.04987	405	6684.5	0.00173	409	400	0.1918	409	280	0.0	
V	28	22	32	255.27	0.04987	405	6684.5	0.00288	415	400	0.1918	415	280	0.0	
V	29	22	31	255.27	0.04987	405	6684.5	0.00326	417	400	0.1918	417	280	0.0	
V	30	22	30	255.27	0.04987	405	6684.5	0.00288	415	400	0.1918	415	280	0.0	
V	31	22	29	313.69	0.04987	403	0	0.01842	406	398	0.1918	398	300	0.0	
V	32	22	28	282.20	0.04987	405	6684.5	0.00345	418	400	0.1918	418	300	0.0	

COL PLASM	ROW	COL MODFLOW	ROW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft./day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./da
V	33	22	27	282.20	0.04987	405	6684.5	0.00384	420	400	0.1918	420	300	0.0	
V	34	22	26	282.20	0.04987	405	6684.5	0.00134	407	400	0.1918	407	300	0.0	
V	35	22	25	282.20	0.04987	405	6684.5	0.00288	415	400	0.1918	415	300	0.0	
V	36	22	24	282.20	0.04987	405	6684.5	0.00326	417	400	0.1918	417	300	0.0	
V	37	22	23	274.70	0.04987	410	8957.2	0.00321	420	395	0.1918	420	300	0.0	
V	38	22	22	274.70	0.04987	410	8957.2	0.00334	421	395	0.1918	421	300	0.0	
V	39	22	21	274.70	0.04987	410	8957.2	0.00347	422	395	0.1918	422	300	0.0	
V	40	22	20	274.70	0.04987	410	8957.2	0.00347	422	395	0.1918	422	300	0.0	
V	41	22	19	274.70	0.04987	410	8957.2	0.00373	424	395	0.1918	424	300	0.0	
V	42	22	18	274.70	0.04987	410	8957.2	0.00386	425	395	0.1918	425	300	0.0	
V	43	22	17	267.73	0.04987	410	8957.2	0.00308	424	400	0.1918	424	300	0.0	
V	44	22	16	252.72	0.04987	410	8957.2	0.00308	424	400	0.1918	424	300	0.0	
V	45	22	15	252.72	0.04987	410	8957.2	0.00321	425	400	0.1918	425	300	0.0	
V	46	22	14	252.72	0.04987	415	0	0.04498	424	389	0.1918	389	300	197.8	
W	16	23	44	1.34	1.91819E+12	400	0	0.00000	500	490	1.918E+12	500	380	0.0	
W	17	23	43	158.12	0.01535	405	8957.2	0.00398	426	395	0.1918	406	320	0.0	
W	18	23	42	272.02	0.01535	405	8957.2	0.00450	425	390	0.1918	406	320	0.0	
W	19	23	41	272.02	0.01535	405	8957.2	0.00463	426	390	0.1918	406	320	0.0	
W	20	23	40	249.64	0.01535	405	6684.5	0.00384	420	400	0.1918	406	300	0.0	
W	21	23	39	265.86	0.04987	405	6684.5	0.00384	420	400	0.1918	420	300	0.0	
W	22	23	38	265.86	0.04987	405	6684.5	0.00384	420	400	0.1918	420	300	0.0	
W	23	23	37	282.20	0.04987	405	6684.5	0.00345	418	400	0.1918	418	300	0.0	
W	24	23	36	339.69	0.04987	405	6684.5	0.00269	414	400	0.1918	414	280	0.0	
W	25	23	35	339.69	0.04987	405	6684.5	0.00326	417	400	0.1918	417	280	0.0	
W	26	23	34	339.69	0.04987	405	6684.5	0.00326	417	400	0.1918	417	280	0.0	
W	27	23	33	339.69	0.04987	405	6684.5	0.00326	417	400	0.1918	417	280	0.0	
W	28	23	32	255.27	0.04987	405	6684.5	0.00192	410	400	0.1918	410	280	0.0	
W	29	23	31	255.27	0.04987	405	6684.5	0.00364	419	400	0.1918	419	280	0.0	
W	30	23	30	282.20	0.04987	405	6684.5	0.00288	415	400	0.1918	415	300	0.0	
W	31	23	29	282.20	0.04987	405	6684.5	0.00364	419	400	0.1918	419	300	0.0	
W	32	23	28	269.07	0.04987	410	8957.2	0.00321	424	399	0.1918	424	300	0.0	
W	33	23	27	274.70	0.04987	410	8957.2	0.00321	420	395	0.1918	420	300	0.0	
W	34	23	26	282.20	0.04987	405	6684.5	0.00173	409	400	0.1918	409	300	0.0	
W	35	23	25	274.70	0.04987	405	8957.2	0.00180	409	395	0.1918	409	300	0.0	
W	36	23	24	274.70	0.04987	410	8957.2	0.00244	414	395	0.1918	414	300	0.0	
W	37	23	23	274.70	0.04987	410	8957.2	0.00321	420	395	0.1918	420	300	0.0	
W	38	23	22	274.70	0.04987	410	8957.2	0.00347	422	395	0.1918	422	300	0.0	
W	39	23	21	274.70	0.04987	410	8957.2	0.00347	422	395	0.1918	422	300	0.0	
W	40	23	20	274.70	0.04987	410	8957.2	0.00347	422	395	0.1918	422	300	0.0	
W	41	23	19	274.70	0.04987	410	8957.2	0.00360	423	395	0.1918	423	300	0.0	
W	42	23	18	274.70	0.04987	410	8957.2	0.00347	422	395	0.1918	422	300	0.0	
W	43	23	17	267.73	0.04987	410	8957.2	0.00283	422	400	0.1918	422	300	0.0	
W	44	23	16	267.73	0.04987	410	8957.2	0.00308	424	400	0.1918	424	300	0.0	
W	45	23	15	252.72	0.04987	410	8957.2	0.00334	426	400	0.1918	426	300	0.0	
W	46	23	14	281.67	0.04987	415	0	0.04627	425	389	0.1918	389	320	198.7	
X	17	24	43	1.34	1.91819E+12	400	0	0.00000	500	490	1.918E+12	500	380	0.0	
X	18	24	42	158.12	0.01535	405	8957.2	0.00321	420	395	0.1918	406	320	0.0	
X	19	24	41	158.12	0.01535	405	8957.2	0.00321	420	395	0.1918	406	320	0.0	
X	20	24	40	272.02	0.04987	405	8957.2	0.00360	423	395	0.1918	423	320	0.0	
X	21	24	39	290.78	0.04987	405	8957.2	0.00193	410	395	0.1918	410	320	0.0	
X	22	24	38	265.86	0.04987	405	6684.5	0.00364	419	400	0.1918	419	300	0.0	
X	23	24	37	282.20	0.04987	405	6684.5	0.00364	419	400	0.1918	419	300	0.0	
X	24	24	36	381.23	0.04987	405	6684.5	0.00326	417	400	0.1918	417	300	0.0	

COL PLASM	ROW PLASM	COL MODFLOW	ROW MODFLOW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft/day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
X	25	24	35	381.23	0.04987	405	6684.5	0.00364	419	400	0.1918	419	300	0.0	
X	26	24	34	339.69	0.04987	405	6684.5	0.00345	418	400	0.1918	418	280	0.0	
X	27	24	33	339.69	0.04987	405	6684.5	0.00345	418	400	0.1918	418	280	0.0	
X	28	24	32	339.69	0.04987	405	6684.5	0.00153	408	400	0.1918	408	280	0.0	
X	29	24	31	339.69	0.04987	405	6684.5	0.00249	413	400	0.1918	413	280	0.0	
X	30	24	30	381.23	0.04987	405	6684.5	0.00364	419	400	0.1918	419	300	0.0	
X	31	24	29	274.70	0.04987	410	8957.2	0.00321	420	395	0.1918	420	300	0.0	
X	32	24	28	274.70	0.04987	410	8957.2	0.00296	418	395	0.1918	418	300	0.0	
X	33	24	27	274.70	0.04987	410	8957.2	0.00244	414	395	0.1918	414	300	0.0	
X	34	24	26	274.70	0.04987	410	8957.2	0.00218	412	395	0.1918	412	300	0.0	
X	35	24	25	274.70	0.04987	410	8957.2	0.00193	410	395	0.1918	410	300	0.0	
X	36	24	24	274.70	0.04987	410	8957.2	0.00321	420	395	0.1918	420	300	0.0	
X	37	24	23	274.70	0.04987	410	8957.2	0.00321	420	395	0.1918	420	300	0.0	
X	38	24	22	274.70	0.04987	410	8957.2	0.00321	420	395	0.1918	420	300	0.0	
X	39	24	21	274.70	0.04987	410	8957.2	0.00334	421	395	0.1918	421	300	0.0	
X	40	24	20	274.70	0.04987	410	8957.2	0.00334	421	395	0.1918	421	300	0.0	
X	41	24	19	274.70	0.04987	410	8957.2	0.00347	422	395	0.1918	422	300	0.0	
X	42	24	18	274.70	0.04987	410	8957.2	0.00347	422	395	0.1918	422	300	0.0	
X	43	24	17	267.73	0.04987	410	8957.2	0.00283	422	400	0.1918	422	300	0.0	
X	44	24	16	267.73	0.04987	410	8957.2	0.00283	422	400	0.1918	422	300	0.0	
X	45	24	15	299.49	0.04987	410	8957.2	0.00411	432	400	0.1918	432	320	0.0	
X	46	24	14	0.00	0.04987	410	8957.2	0.00347	427	400	0.1918	427	320	0.0	
Y	18	25	42	1.34	1.91819E+12	400	0	0.00000	500	490	1.918E+12	500	380	0.0	
Y	19	25	41	1.34	1.91819E+12	400	0	0.00000	500	490	1.918E+12	500	380	0.0	
Y	20	25	40	1.34	1.91819E+12	405	0	0.00000	500	490	1.918E+12	500	380	0.0	
Y	21	25	39	290.78	0.04987	405	8957.2	0.00321	420	395	0.1918	420	320	0.0	
Y	22	25	38	290.78	0.04987	405	8957.2	0.00296	418	395	0.1918	418	320	0.0	
Y	23	25	37	274.70	0.04987	405	8957.2	0.00270	416	395	0.1918	416	300	0.0	
Y	24	25	36	274.70	0.04987	405	8957.2	0.00308	419	395	0.1918	419	300	0.0	
Y	25	25	35	389.57	0.04987	405	8957.2	0.00296	418	395	0.1918	418	300	0.0	
Y	26	25	34	369.57	0.04987	405	8957.2	0.00308	419	395	0.1918	419	300	0.0	
Y	27	25	33	389.57	0.04987	405	8957.2	0.00296	418	395	0.1918	418	300	0.0	
Y	28	25	32	331.11	0.04987	410	8957.2	0.00270	416	395	0.1918	416	280	0.0	
Y	29	25	31	331.11	0.04987	410	8957.2	0.00244	414	395	0.1918	414	280	0.0	
Y	30	25	30	369.57	0.04987	405	8957.2	0.00180	409	395	0.1918	409	300	0.0	
Y	31	25	29	274.70	0.04987	410	8957.2	0.00231	413	395	0.1918	413	300	0.0	
Y	32	25	28	274.70	0.04987	410	8957.2	0.00244	414	395	0.1918	414	300	0.0	
Y	33	25	27	274.70	0.04987	410	8957.2	0.00244	414	395	0.1918	414	300	0.0	
Y	34	25	26	274.70	0.04987	410	8957.2	0.00218	412	395	0.1918	412	300	0.0	
Y	35	25	25	274.70	0.04987	410	8957.2	0.00231	413	395	0.1918	413	300	0.0	
Y	36	25	24	274.70	0.04987	410	8957.2	0.00308	419	395	0.1918	419	300	0.0	
Y	37	25	23	274.70	0.04987	410	8957.2	0.00308	419	395	0.1918	419	300	0.0	
Y	38	25	22	274.70	0.04987	410	8957.2	0.00308	419	395	0.1918	419	300	0.0	
Y	39	25	21	274.70	0.04987	410	8957.2	0.00321	420	395	0.1918	420	300	0.0	
Y	40	25	20	274.70	0.04987	410	8957.2	0.00334	421	395	0.1918	421	300	0.0	
Y	41	25	19	274.70	0.04987	410	8957.2	0.00334	421	395	0.1918	421	300	0.0	
Y	42	25	18	274.70	0.04987	410	8957.2	0.00321	420	395	0.1918	420	300	0.0	
Y	43	25	17	299.49	0.04987	410	8957.2	0.00270	421	400	0.1918	421	320	0.0	
Y	44	25	16	299.49	0.04987	410	8957.2	0.00308	424	400	0.1918	424	320	0.0	
Y	45	25	15	299.49	0.04987	415	8957.2	0.00437	434	400	0.1918	434	320	0.0	
Y	46	25	14	0.00	0.04987	415	8957.2	0.00360	428	400	0.1918	428	320	0.0	
Z	21	26	39	1.34	1.91819E+12	405	0	0.00000	500	490	1.918E+12	500	380	0.0	
Z	22	26	38	1.34	1.91819E+12	405	0	0.00000	500	490	1.918E+12	500	380	0.0	

COL PLASM	ROW	COL MODFLOW	ROW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft./day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
Z	23	26	37	290.51	0.04987	405	8957.2	0.00231	423	405	0.1918	423	320	0.0	
Z	24	26	36	309.54	0.04987	405	8957.2	0.00437	429	395	0.1918	429	320	0.0	
Z	25	26	35	274.70	0.04987	405	8957.2	0.00334	421	395	0.1918	421	300	0.0	
Z	26	26	34	274.70	0.01535	405	8957.2	0.00334	421	395	0.1918	406	300	0.0	
Z	27	26	33	369.57	0.01535	405	8957.2	0.00283	417	395	0.1918	406	300	0.0	
Z	28	26	32	369.57	0.01535	410	8957.2	0.00270	416	395	0.1918	406	300	0.0	
Z	29	26	31	369.57	0.04987	410	8957.2	0.00296	418	395	0.1918	418	300	0.0	
Z	30	26	30	369.57	0.04987	410	8957.2	0.00308	419	395	0.1918	419	300	0.0	
Z	31	26	29	274.70	0.04987	410	8957.2	0.00308	419	395	0.1918	419	300	0.0	
Z	32	26	28	274.70	0.04987	410	8957.2	0.00244	414	395	0.1918	414	300	0.0	
Z	33	26	27	274.70	0.04987	410	8957.2	0.00218	412	395	0.1918	412	300	0.0	
Z	34	26	26	274.70	0.04987	410	8957.2	0.00244	414	395	0.1918	414	300	0.0	
Z	35	26	25	274.70	0.04987	410	8957.2	0.00296	418	395	0.1918	418	300	0.0	
Z	36	26	24	274.70	0.04987	410	8957.2	0.00308	419	395	0.1918	419	300	0.0	
Z	37	26	23	309.54	0.04987	410	8957.2	0.00308	419	395	0.1918	419	320	0.0	
Z	38	26	22	309.54	0.04987	410	8957.2	0.00308	419	395	0.1918	419	320	0.0	
Z	39	26	21	195.64	0.04987	410	8957.2	0.00321	420	395	0.1918	420	320	0.0	
Z	40	26	20	309.54	0.04987	410	8957.2	0.00270	421	400	0.1918	421	320	0.0	
Z	41	26	19	309.54	0.04987	410	8957.2	0.00283	422	400	0.1918	422	320	0.0	
Z	42	26	18	309.54	0.04987	410	8957.2	0.00308	424	400	0.1918	424	320	0.0	
Z	43	26	17	299.49	0.04987	415	8957.2	0.00347	427	400	0.1918	427	320	0.0	
Z	44	26	16	299.49	0.04987	415	8957.2	0.00411	432	400	0.1918	432	320	0.0	
Z	45	26	15	299.49	0.04987	415	8957.2	0.00411	432	400	0.1918	432	320	0.0	
Z	46	26	14	0.00	0.04987	415	8957.2	0.00360	433	405	0.1918	433	340	0.0	
AA	23	27	37	1.34	1.91819E+12	405	0	0.00000	500	490	1.918E+12	500	380	0.0	
AA	24	27	36	169.91	0.04987	405	8957.2	0.00437	439	405	0.1918	439	320	0.0	
AA	25	27	35	273.36	0.01535	405	8957.2	0.00360	433	405	0.1918	406	320	0.0	
AA	26	27	34	276.58	0.01535	405	8957.2	0.00334	429	403	0.1918	406	320	0.0	
AA	27	27	33	290.78	0.01535	405	8957.2	0.00231	418	400	0.1918	406	320	0.0	
AA	28	27	32	290.78	0.01535	410	8957.2	0.00334	426	400	0.1918	406	320	0.0	
AA	29	27	31	353.89	0.01535	410	8957.2	0.00218	417	400	0.1918	406	300	0.0	
AA	30	27	30	353.89	0.04987	410	8957.2	0.00308	424	400	0.1918	424	300	0.0	
AA	31	27	29	259.02	0.04987	410	8957.2	0.00321	425	400	0.1918	425	300	0.0	
AA	32	27	28	274.70	0.04987	410	8957.2	0.00257	420	400	0.1918	420	300	0.0	
AA	33	27	27	274.70	0.04987	410	8957.2	0.00218	417	400	0.1918	417	300	0.0	
AA	34	27	26	274.70	0.04987	410	8957.2	0.00206	416	400	0.1918	416	300	0.0	
AA	35	27	25	274.70	0.04987	410	8957.2	0.00218	417	400	0.1918	417	300	0.0	
AA	36	27	24	274.70	0.04987	410	8957.2	0.00257	420	400	0.1918	420	300	0.0	
AA	37	27	23	309.54	0.04987	410	8957.2	0.00321	425	400	0.1918	425	320	0.0	
AA	38	27	22	309.54	0.04987	410	8957.2	0.00308	424	400	0.1918	424	320	0.0	
AA	39	27	21	195.64	0.04987	410	8957.2	0.00308	424	400	0.1918	424	320	0.0	
AA	40	27	20	195.64	0.04987	410	8957.2	0.00321	425	400	0.1918	425	320	0.0	
AA	41	27	19	195.64	0.04987	410	8957.2	0.00270	421	400	0.1918	421	320	0.0	
AA	42	27	18	299.49	0.04987	415	8957.2	0.00386	430	400	0.1918	430	320	0.0	
AA	43	27	17	299.49	0.04987	415	8957.2	0.00424	433	400	0.1918	433	320	0.0	
AA	44	27	16	299.49	0.04987	415	8957.2	0.00476	437	400	0.1918	437	320	0.0	
AA	45	27	15	206.09	0.04987	415	8957.2	0.00514	445	405	0.1918	445	340	0.0	
AA	46	27	14	0.00	0.04987	415	8957.2	0.00668	457	405	0.1918	457	340	0.0	
BB	24	28	36	1.34	1.91819E+12	405	0	0.00000	500	490	1.918E+12	500	320	0.0	
BB	25	28	35	0.00	1.91819E+12	405	0	0.00000	500	490	1.918E+12	500	320	0.0	
BB	26	28	34	0.00	1.91819E+12	405	0	0.00000	500	490	1.918E+12	500	320	0.0	
BB	27	28	33	0.00	1.91819E+12	405	0	0.00000	500	490	1.918E+12	500	320	0.0	
BB	28	28	32	0.00	1.91819E+12	405	0	0.00000	500	490	1.918E+12	500	320	0.0	

COL PLASM	ROW	COL MODFLOW	ROW	HYDRAULIC CONDUC. ft/day	S	INTIAL HEAD ft. msl	CONST RATE cu. ft./day	RECHARGE ft/day	LAND ELEV. ft. msl	STREAM BED ELEV. ft. msl	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft. msl	MISS. MILE miles	SPECIFIED FLUX cu. ft./day
BB	29	28	31	0.00	1.91819E+12	405	0	0.00000	500	490	1.918E+12	500	320	0.0	
BB	30	28	30	273.36	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	31	28	29	273.36	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	32	28	28	290.51	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	33	28	27	290.51	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	34	28	26	290.51	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	35	28	25	290.51	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	36	28	24	290.51	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	37	28	23	290.51	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	38	28	22	186.93	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	39	28	21	186.93	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	40	28	20	186.93	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	41	28	19	186.93	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	42	28	18	186.93	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	320	0.0	
BB	43	28	17	206.09	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	340	0.0	
BB	44	28	16	206.09	0.049872817	415	8957.2	0.00321	430	405	0.1918	430	340	0.0	
BB	45	28	15	1.34	1.91819E+12	415	0	0.00000	500	490	1.918E+12	500	400	0.0	
BB	46	28	14	1.34	1.91819E+12	415	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	30	29	30	1.34	1.91819E+12	410	0	0.00000	500	490	1.918E+12	500	380	0.0	
CC	31	29	29	1.34	1.91819E+12	410	0	0.00000	500	490	1.918E+12	500	380	0.0	
CC	32	29	28	1.34	1.91819E+12	415	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	33	29	27	1.34	1.91819E+12	415	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	34	29	26	1.34	1.91819E+12	415	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	35	29	25	1.34	1.91819E+12	415	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	36	29	24	1.34	1.91819E+12	415	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	37	29	23	1.34	1.91819E+12	415	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	38	29	22	1.34	1.91819E+12	415	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	39	29	21	1.34	1.91819E+12	415	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	40	29	20	1.34	1.91819E+12	415	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	41	29	19	1.34	1.91819E+12	420	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	42	29	18	1.34	1.91819E+12	420	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	43	29	17	1.34	1.91819E+12	425	0	0.00000	500	490	1.918E+12	500	400	0.0	
CC	44	29	16	1.34	1.91819E+12	420	0	0.00000	500	490	1.918E+12	500	400	0.0	

APPENDIX D

Boundary Data for MODFLOW Groundwater Model.

Horseshoe Lake Cells					S	S	TOP AQ.	BOTTOM	Recharge	Const.	INT.HEAD	STREAM	LAND	MISS. MI.	SPECIFIE
COL	ROW	COL	ROW	HYDRAU.		W.T.		AQ. ELEV.		Rate		BED ELEV.	ELEV.		FLUX
				CONDUC.(FT/DAY)			ft. msl	ft msl	ft/day	cu. ft/day	ft msl	ft msl	ft msl	miles	cu ft/day
O	28	15	32	294.532	0.04987	0.1918	398	300	0.01842	0	400	398	406	0.0	
P	29	16	31	294.532	0.04987	0.1918	398	300	0.01842	0	400	398	406	0.0	
Q	30	17	30	273.36	0.04987	0.1918	404	300	0.00173	6684.5	400	395	404	0.0	
Q	31	17	29	294.532	0.04987	0.1918	398	300	0.01842	0	403	398	406	0.0	
R	31	18	29	294.532	0.04987	0.1918	398	300	0.01842	0	403	398	406	0.0	
R	32	18	28	294.532	0.04987	0.1918	398	300	0.01842	0	403	398	406	0.0	
S	32	19	28	261.702	0.04987	0.1918	398	280	0.01842	0	403	398	406	0.0	
T	28	20	32	277.648	0.04987	0.1918	398	280	0.01842	0	403	398	406	0.0	
T	29	20	31	277.648	0.04987	0.1918	398	280	0.01842	0	403	398	406	0.0	
T	30	20	30	261.702	0.04987	0.1918	398	280	0.01842	0	403	398	406	0.0	
T	31	20	29	261.702	0.04987	0.1918	398	280	0.01842	0	403	398	406	0.0	
T	32	20	28	261.702	0.04987	0.1918	398	280	0.02762	0	403	398	406	0.0	
T	33	20	27	261.702	0.04987	0.1918	398	280	0.01842	0	403	398	406	0.0	
U	29	21	31	277.648	0.04987	0.1918	398	280	0.01842	0	403	398	406	0.0	
U	32	21	28	261.702	0.04987	0.1918	398	280	0.01842	0	403	398	406	0.0	
V	31	22	29	313.694	0.04987	0.1918	398	300	0.01842	0	403	398	406	0.0	
Frank Holten State Park Lake															
S	18	19	42	281.668	0.04987	0.1918	415	300	0.01343	0	410	405	412	0.0	
T	15	20	45	263.846	0.04987	0.1918	407	300	0.00384	0	410	405	407	0.0	
T	16	20	44	263.846	0.04987	0.1918	407	300	0.00384	0	410	405	407	0.0	
Blue Waters Ditch / Harding Ditch															
G	13	7	47	219.358	0.04987	0.1918	372	320	0.04627	0	390	372	408	175.3	
H	12	8	48	346.39	0.04987	0.1918	372	320	0.00614	0	400	372	404	175.1	
I	11	9	49	267.732	0.04987	0.1918	372	280	0.06714	0	400	372	407	175.4	
J	11	10	49	299.49	0.04987	0.1918	372	300	0.06139	0	405	372	404	176.1	
K	11	11	49	299.49	0.04987	0.1918	372	300	0.05947	0	405	372	403	177.2	
L	11	12	49	191.084	0.04987	0.1918	373	300	0.06330	0	410	373	406	178.2	
M	11	13	49	191.084	0.04987	0.1918	373	300	0.06906	0	410	373	409	178.7	
Cahokia Diversion Channel															
Q	46	17	14	207.298	0.04987	0.1918	389	300	36.98159	0	405	389	416	195.2	
R	46	18	14	274.7	0.04987	0.1918	389	300	0.05270	0	405	389	430	195.6	
S	46	19	14	252.724	0.04987	0.1918	389	300	0.04884	0	410	389	427	196.1	
T	46	20	14	231.686	0.04987	0.1918	389	280	0.00488	0	410	389	427	196.7	
U	46	21	14	231.686	0.04987	0.1918	389	280	0.04627	0	410	389	425	197.2	
V	46	22	14	252.724	0.04987	0.1918	389	300	0.04496	0	415	389	424	197.8	
W	46	23	14	281.668	0.04987	0.1918	389	320	0.04627	0	415	389	425	198.7	
Specified Flux															
A	1	1	59	361.8	0.04987	0.1918	400	320	0.00064	0	390	395	400	0.0	6283
B	1	2	59	274.7	0.04987	0.1918	400	280	0.00193	0	390	385	400	0.0	2005
C	1	3	59	274.7	0.04987	0.1918	400	280	0.00288	6684.5	390	385	400	0.0	2005
E	1	5	59	274.7	0.04987	0.1918	400	280	0.00288	6684.5	390	385	400	0.0	2005
F	1	6	59	299.49	0.04987	0.1918	405	300	0.00288	6684.5	395	390	405	0.0	1872

Specified Heads at Eastern Boundary

COL	ROW	COL	ROW	HYDRAU. CONDUC.(FT/DAY)	S.	S W.T.	TOP AQ. ft. msl	BOTTOM AQ. ELEV. ft msl	ft/day	Const. Rate cu. ft/day	INT.HEAD ft msl	STREAM BED ELEV. ft msl	LAND ELEV. ft msl	MISS. ML. miles	SPECIFIE FLUX cu ft/dai
G	1	7	59	1.34	1.9182E+12	1.918E+12	600	370	0.00	0	400	590	600	0.0	
G	2	7	58	1.34	1.9182E+12	1.918E+12	600	370	0.00	0	400	590	600	0.0	
H	3	8	57	1.34	1.9182E+12	1.918E+12	600	380	0.00	0	400	590	600	0.0	
H	4	8	56	1.34	1.9182E+12	1.918E+12	600	380	0.00	0	400	590	600	0.0	
I	5	9	55	1.34	1.9182E+12	1.918E+12	600	380	0.00	0	400	590	600	0.0	
J	6	10	54	1.34	1.9182E+12	1.918E+12	600	380	0.00	0	400	590	600	0.0	
K	7	11	53	1.34	1.9182E+12	1.918E+12	600	380	0.00	0	400	590	600	0.0	
L	8	12	52	1.34	1.9182E+12	1.918E+12	550	380	0.00	0	400	540	550	0.0	
M	9	13	51	0.000134	1.9182E+12	1.918E+12	550	380	0.00	0	400	540	550	0.0	
N	10	14	50	0.000134	1.9182E+12	1.918E+12	475	380	0.00	0	400	465	475	0.0	
O	10	15	50	0.000134	1.9182E+12	1.918E+12	475	380	0.00	0	400	465	475	0.0	
P	10	16	50	0.000134	1.9182E+12	1.918E+12	475	380	0.00	0	400	465	475	0.0	
Q	10	17	50	0.000134	1.9182E+12	1.918E+12	475	380	0.00	0	405	465	475	0.0	
R	11	18	49	0.000134	1.9182E+12	1.918E+12	500	380	0.00	0	400	490	500	0.0	
S	12	19	48	0.000134	1.9182E+12	1.918E+12	500	380	0.00	0	405	490	500	0.0	
T	13	20	47	0.000134	1.9182E+12	1.918E+12	500	320	0.00	0	405	490	500	0.0	
U	14	21	46	0.000134	1.9182E+12	1.918E+12	500	380	0.00	0	400	490	500	0.0	
V	15	22	45	0.000134	1.9182E+12	1.918E+12	500	380	0.00	0	400	490	500	0.0	
W	16	23	44	1.34	1.9182E+12	1.918E+12	500	380	0.00	0	400	490	500	0.0	
X	17	24	43	1.34	1.918E+12	1.918E+12	500	380	0.00	0	400	490	500	0.0	
Y	18	25	42	1.34	1.9182E+12	1.918E+12	500	380	0.00	0	400	490	500	0.0	
Y	19	25	41	1.34	1.9182E+12	1.918E+12	500	380	0.00	0	400	490	500	0.0	
Y	20	25	40	1.34	1.9182E+12	1.918E+12	500	380	0.00	0	405	490	500	0.0	
Z	21	26	39	1.34	1.9182E+12	1.918E+12	500	380	0.00	0	405	490	500	0.0	
Z	22	26	38	1.34	1.9182E+12	1.918E+12	500	380	0.00	0	405	490	500	0.0	
AA	23	27	37	1.34	1.9182E+12	1.918E+12	500	380	0.00	0	405	490	500	0.0	
BB	24	28	36	1.34	1.9182E+12	1.918E+12	500	320	0.00	0	405	490	500	0.0	
BB	25	28	35	0.000134	1.9182E+12	1.918E+12	500	320	0.00	0	405	490	500	0.0	
BB	26	28	34	0.000134	1.9182E+12	1.918E+12	500	320	0.00	0	405	490	500	0.0	
BB	27	28	33	0.000134	1.9182E+12	1.918E+12	500	320	0.00	0	405	490	500	0.0	
BB	28	28	32	0.000134	1.9182E+12	1.918E+12	500	320	0.00	0	405	490	500	0.0	
BB	29	28	31	0.000134	1.9182E+12	1.918E+12	500	320	0.00	0	405	490	500	0.0	
CC	30	29	30	1.34	1.9182E+12	1.918E+12	500	380	0.00	0	410	490	500	0.0	
CC	31	29	29	1.34	1.9182E+12	1.918E+12	500	380	0.00	0	410	490	500	0.0	
CC	32	29	28	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	415	490	500	0.0	
CC	33	29	27	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	415	490	500	0.0	
CC	34	29	26	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	415	490	500	0.0	
CC	35	29	25	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	415	490	500	0.0	
CC	36	29	24	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	415	490	500	0.0	
CC	37	29	23	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	415	490	500	0.0	
CC	38	29	22	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	415	490	500	0.0	
CC	39	29	21	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	415	490	500	0.0	
CC	40	29	20	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	415	490	500	0.0	
CC	41	29	19	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	420	490	500	0.0	
CC	42	29	18	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	420	490	500	0.0	
CC	43	29	17	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	425	490	500	0.0	
CC	44	29	16	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	420	490	500	0.0	
BB	45	28	15	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	415	490	500	0.0	
BB	46	28	14	1.34	1.9182E+12	1.918E+12	500	400	0.00	0	415	490	500	0.0	

Mississippi River Cells					S	S	TOP AQ.	BOTTOM		Const.	INT.HEAD	STREAM	LAND	MISS. MI.	SPECIFIED
COL	ROW	COL	ROW	HYDRAU.		W.T.		AQ. ELEV.		Rate		BED ELEV.	ELEV.		FLUX
				CONDUC.(FT/DAY)			ft. msl	ft msl	ft/day	cu. ft/day	ft msl	ft msl	ft msl	miles	cu ft/dau
A	3	1	57	441.664	0.04987	0.1918	369	320	5.28692	0	395	369	395	169.6	7620
A	4	1	56	290.78	0.04987	0.1918	370	320	5.08358	0	395	370	395	170.1	4946
A	5	1	55	290.78	0.04987	0.1918	370	320	5.08358	0	395	370	395	170.6	
A	6	1	54	290.78	0.04987	0.1918	370	320	5.08358	0	395	370	395	170.9	
B	7	2	53	290.78	0.04987	0.1918	370	320	5.08358	0	395	370	395	171.7	
B	8	2	52	290.78	0.04987	0.1918	370	320	5.08358	0	395	370	395	172.0	
B	9	2	51	290.78	0.04987	0.1918	370	320	5.08358	0	395	370	395	172.3	
C	10	3	50	287.028	0.04987	0.1918	371	320	4.88023	0	395	371	395	172.9	
D	11	4	49	283.41	0.04987	0.1918	372	320	6.71032	0	395	372	405	173.9	
D	12	4	48	283.41	0.04987	0.1918	372	320	4.67689	0	395	372	395	173.9	
E	12	5	48	283.41	0.04987	0.1918	372	320	7.11701	0	395	372	407	174.5	
E	13	5	47	283.41	0.04987	0.1918	372	320	7.11701	0	395	372	407	174.8	
E	14	5	46	283.41	0.04987	0.1918	372	320	4.67689	0	395	372	395	175.1	
F	14	6	46	225.254	0.04987	0.1918	372	320	7.52369	0	395	372	409	175.4	
F	15	6	45	225.254	0.04987	0.1918	372	320	4.67689	0	395	372	395	175.4	
G	15	7	45	225.254	0.04987	0.1918	372	320	8.74375	0	395	372	415	176.1	
G	16	7	44	225.254	0.04987	0.1918	372	320	4.67689	0	395	372	395	176.4	
H	16	8	44	225.254	0.04987	0.1918	372	320	7.52369	0	395	372	409	176.9	
H	17	8	43	225.254	0.04987	0.1918	372	320	7.52369	0	395	372	409	177.4	
I	18	9	42	225.254	0.04987	0.1918	372	320	2.33845	0	395	372	395	177.2	
I	19	9	41	225.254	0.04987	0.1918	372	320	2.33845	0	395	372	395	177.5	
J	20	10	40	331.114	0.04987	0.1918	373	340	8.33707	0	395	373	414	178.7	
J	21	10	39	331.114	0.04987	0.1918	373	340	5.49026	0	395	373	400	179.1	
J	22	10	38	331.114	0.04987	0.1918	373	340	4.47355	0	395	373	395	179.5	
J	23	10	37	331.114	0.04987	0.1918	373	340	4.47355	0	395	373	395	180.0	
J	24	10	36	331.114	0.04987	0.1918	373	340	4.47355	0	395	373	395	180.4	
J	25	10	35	331.114	0.04987	0.1918	373	340	4.47355	0	395	373	395	180.9	
J	26	10	34	338.35	0.04987	0.1918	374	340	4.27020	0	395	374	395	181.5	
J	27	10	33	224.048	0.04987	0.1918	374	340	4.27020	0	395	374	395	182.2	
I	28	9	32	1.34	0.04987	0.1918	375	340	4.06686	0	395	375	395	183.5	
I	29	9	31	1.34	0.04987	0.1918	375	340	4.06686	0	395	375	395	183.8	
I	30	9	30	338.35	0.04987	0.1918	376	340	3.86352	0	395	376	395	184.4	
I	31	9	29	338.35	0.04987	0.1918	376	340	3.86352	0	395	376	395	184.7	
J	32	10	28	331.918	0.04987	0.1918	377	340	3.66018	0	395	377	413	185.5	
J	33	10	27	331.918	0.04987	0.1918	377	340	1.83009	0	395	377	395	185.8	
K	34	11	26	338.35	0.04987	0.1918	378	340	27.39377	0	395	380	420	194.0	
K	35	11	25	338.35	0.04987	0.1918	380	340	10.27266	0	405	390	405	194.5	
L	36	12	24	253.93	0.04987	0.1918	382	320	24.65439	0	405	382	418	194.0	
L	37	12	23	253.93	0.04987	0.1918	382	320	15.75142	0	405	382	405	194.0	
M	38	13	22	249.106	0.04987	0.1918	384	320	14.38173	0	405	384	405	194.0	
N	39	14	21	407.896	0.04987	0.1918	388	320	10.27266	0	405	390	405	194.0	
N	40	14	20	504.108	0.04987	0.1918	388	320	11.64235	0	405	388	405	194.0	
O	41	15	19	504.108	0.04987	0.1918	388	320	18.49079	0	405	388	415	194.0	
O	42	15	18	504.108	0.04987	0.1918	388	320	11.64235	0	405	388	405	194.0	
O	43	15	17	240.396	0.04987	0.1918	388	320	23.28470	0	405	388	405	192.8	
O	44	15	16	240.396	0.04987	0.1918	388	320	23.28470	0	405	388	405	193.5	
P	44	16	16	504.108	0.04987	0.1918	388	320	17.80595	0	405	388	414	194.0	
P	45	16	15	407.896	0.04987	0.1918	388	320	11.64235	0	405	388	405	194.0	
P	46	16	14	240.396	0.04987	0.1918	388	320	23.28470	0	405	388	405	193.5	

Extra Mississippi Cells					S	S	TOP AQ.	BOTTOM		Const.	INT.HEAD	STREAM	LAND	MISS. MI.	SPECIFIED
COL	ROW	COL	ROW	HYDRAU.		W.T.		AQ. ELEV.		Rate		BED ELEV.	ELEV.		FLUX
				CONDUC.(FT/DAY)			ft. msl	ft msl	ft/day	cu. ft/day	ft msl	ft msl	ft msl	miles	cu ft/dau
I	34	9	26	61.372	0.04987	0.1918	388	340	0.00193	0	400	390	405	187.7	
I	35	9	25	61.372	0.04987	0.1918	388	340	0.00193	0	400	390	405	187.9	
I	36	9	24	61.372	0.04987	0.1918	388	340	0.00193	0	400	390	405	188.3	
L	42	12	18	59.63	0.04987	0.1918	386	340	0.00244	0	405	386	405	192.1	
M	43	13	17	59.63	0.04987	0.1918	386	340	26.02408	0	405	386	405	192.1	
N	44	14	16	298.686	0.04987	0.1918	388	340	23.28470	0	405	388	405	192.8	
O	44	15	16	240.396	0.04987	0.1918	388	320	23.28470	0	405	388	405	193.5	

APPENDIX E

Input Data Used to Generate Water Level Exceedance Probability Curves.

		Ordered Elevations of Annual Highs		
ORDER	Freq.	SWS #3	CORPS 70	BLSTFRN
		(24, 16)	(25, 12)	(28, 170
1	.011	415.00	422.97	410.57
2	.022	413.98	418.66	409.40
3	.033	412.31	418.30	409.23
4	.044	412.00	418.00	409.10
5	.055	411.94	416.23	408.54
6	.066	411.81	415.87	408.48
7	.077	411.64	414.62	408.40
8	.088	411.58	414.59	408.31
9	.099	411.57	414.56	408.16
10	.110	411.43	414.48	408.13
11	.121	411.35	413.85	408.08
12	.132	411.04	413.80	408.06
13	.143	411.02	413.51	407.91
14	.154	410.95	413.09	407.76
15	.165	410.91	413.00	407.67
16	.176	410.84	412.96	407.66
17	.187	410.80	412.94	407.45
18	.198	410.70	412.84	407.41
19	.209	410.61	412.73	407.12
20	.220	410.52	412.26	407.08
21	.231	410.45	411.80	407.01
22	.242	410.37	411.77	406.91
23	.253	410.03	411.64	406.64
24	.264	409.93	411.49	406.58
25	.275	409.89	410.99	406.48
26	.286	409.87	410.81	406.44
27	.297	409.82	410.57	406.36
28	.308	409.65	410.38	406.22
29	.319	409.61	410.32	406.18
30	.330	409.57	409.93	406.14
31	.341	409.51	409.88	406.09
32	.352	409.21	409.85	405.91
33	.363	409.10	409.65	405.72
34	.374	409.02	409.35	405.62
35	.385	408.99	409.13	405.57
36	.396	408.92	409.01	405.35
37	.407	408.92	408.85	405.28
38	.418	408.85	408.72	405.21
39	.429	408.67	408.49	404.83
40	.440	408.60	408.48	404.68
41	.451	408.59	408.45	404.33
42	.462	408.59	408.42	404.28
43	.473	408.51	408.36	403.86
44	.484	408.32	408.20	403.82
45	.495	408.28	408.11	403.75
46	.505	408.14	408.06	403.68
47	.516	408.11	408.05	403.53
48	.527	408.04	407.85	403.46
49	.538	408.02	407.71	403.31
50	.549	407.93	407.61	403.09
51	.560	407.83	407.07	403.05
52	.571	407.75	406.91	402.82
53	.582	407.61	406.89	402.74

		Ordered Elevations of Annual Highs		
ORDER	Freq.	SWS #3	CORPS 70	BLSTFRN
54	.593	407.55	406.86	402.70
55	.604	407.55	406.78	402.68
56	.615	407.47	406.72	402.67
57	.626	407.47	406.42	402.66
58	.637	407.43	406.25	402.63
59	.648	407.36	406.14	402.62
60	.659	407.31	406.13	402.59
61	.670	407.29	406.04	402.58
62	.681	407.19	406.03	402.46
63	.692	407.04	405.95	402.41
64	.703	406.94	405.65	402.39
65	.714	406.84	405.25	402.36
66	.725	406.82	405.12	402.33
67	.736	406.81	405.10	402.32
68	.747	406.77	405.07	402.25
69	.758	406.68	404.97	402.20
70	.769	406.67	404.90	402.17
71	.780	406.63	404.79	402.08
72	.791	406.45	404.79	402.06
73	.802	406.38	404.66	401.88
74	.813	406.34	404.52	401.80
75	.824	406.23	404.51	401.80
76	.835	406.10	404.42	401.76
77	.846	405.83	404.26	401.60
78	.857	405.64	403.70	401.43
79	.868	405.60	403.58	401.37
80	.879	405.45	403.48	401.34
81	.890	405.36	402.91	401.33
82	.901	405.31	402.80	401.28
83	.912	405.28	402.71	401.16
84	.923	405.16	402.64	401.04
85	.934	405.12	402.27	400.83
86	.945	404.73	402.08	400.61
87	.956	404.66	401.71	400.53
88	.967	404.06	401.49	400.41
89	.978	403.82	401.04	400.38
90	.989	403.32	400.50	400.00

	Annual Max. G.W. Elev.		
Date	SWS #3	CORPS 70	BLSTFRN
	(24, 16)	(25, 12)	(28, 17)
1905	409.82	409.01	407.41
1906	410.37	409.88	407.45
1907	410.52	410.57	407.66
1908	411.57	418.00	408.48
1909	412.31	414.48	409.23
1910	411.04	406.89	408.08
1911	409.61	403.48	407.67
1912	412.00	412.96	408.40
1913	411.58	409.65	409.10
1914	408.04	404.51	406.64
1915	409.87	414.56	406.09
1916	411.02	413.51	407.08
1917	409.21	412.26	407.01
1918	408.51	406.14	406.44
1919	408.60	409.85	405.91
1920	409.57	412.84	406.58
1921	410.80	406.78	408.16
1922	410.91	414.59	408.13
1923	410.03	406.13	407.76
1924	409.65	408.72	407.12
1925	408.28	405.12	406.18
1926	408.67	410.38	406.48
1927	413.98	418.66	409.40
1928	411.35	408.06	408.31
1930	410.84	405.07	408.54
1931	407.61	402.80	406.22
1932	407.04	404.79	405.21
1933	409.51	407.71	406.91
1934	406.82	401.04	405.62
1935	408.99	411.77	406.36
1936	407.29	405.10	405.57
1937	408.11	406.42	405.72
1938	410.70	408.36	408.06
1939	410.45	408.45	407.91
1940	407.36	401.71	406.14
1941	405.83	406.86	404.33
1942	407.43	408.48	402.63
1943	408.92	413.09	403.46
1944	409.02	411.64	403.82
1945	410.61	412.73	404.83
1946	408.92	405.65	403.75
1947	408.59	413.85	403.31
1948	407.93	406.91	403.05
1949	406.68	406.04	402.32
1950	407.55	408.05	402.70

	Annual Max. G.W. Elev.		
Date	SWS #3	CORPS 70	BLSTFRN
	(24, 16)	(25, 12)	(28, 17)
1951	407.47	415.87	402.08
1952	408.14	410.32	402.62
1953	406.81	404.42	402.39
1954	403.82	402.71	400.53
1955	404.06	402.27	400.38
1956	403.32	400.50	400.00
1957	406.84	404.52	402.17
1958	405.28	404.66	401.43
1959	404.73	403.58	400.83
1960	405.12	409.35	400.41
1961	407.55	407.85	402.41
1962	408.02	408.49	402.66
1963	406.77	402.64	402.59
1964	405.36	402.08	401.76
1965	404.66	407.07	400.61
1966	406.10	404.90	401.28
1967	405.31	406.25	401.16
1968	405.16	402.91	401.04
1969	406.94	411.49	401.37
1970	407.19	408.11	402.25
1971	406.38	404.79	401.60
1972	405.45	406.03	401.33
1973	409.89	418.30	402.46
1974	409.93	411.80	403.68
1975	409.10	408.85	403.86
1976	407.47	405.95	402.82
1977	406.34	403.70	402.33
1978	407.75	409.13	402.67
1979	408.59	413.00	403.09
1980	406.63	404.97	402.20
1981	406.67	407.61	402.06
1982	408.32	410.99	402.36
1983	411.43	414.62	404.28
1984	411.94	413.80	405.35
1985	411.64	410.81	405.28
1986	408.85	412.94	403.53
1987	407.83	406.72	402.58
1988	407.31	404.26	402.74
1989	405.64	401.49	401.88
1990	406.23	408.20	401.80
1991	406.45	408.42	401.80
1992	405.60	405.25	401.34
1993	410.95	422.97	402.68
1994	411.81	409.93	404.68

			Ordered Monthly Highs			
ORDER	Month-Year	SWS No. 3		CORPS 70		BLSTFRN
		(24,16)		(25,12)		(28,17)
1	May-1929	415.00	Jul-1993	422.97	May-1929	410.57
2	Jun-1929	414.99	Aug-1993	420.34	Jun-1929	410.28
3	Jul-1929	414.61	Apr-1927	418.66	Jul-1929	409.79
4	Aug-1929	414.01	Apr-1973	418.30	Aug-1929	409.42
5	Jun-1927	413.98	Jun-1927	418.18	May-1927	409.40
6	May-1927	413.92	Jun-1908	418.00	Apr-1929	409.33
7	Jul-1927	413.90	Sep-1993	417.64	May-1909	409.23
8	Aug-1927	413.42	May-1973	417.40	Apr-1913	409.10
9	Sep-1929	413.07	May-1927	416.72	Jun-1927	409.09
10	Apr-1929	412.84	May-1929	416.23	Sep-1929	409.01
11	Sep-1927	412.66	Jul-1951	415.87	Jun-1909	408.84
12	May-1909	412.31	Apr-1929	415.02	Mar-1913	408.79
13	Oct-1929	412.23	May-1993	414.96	Apr-1909	408.77
14	Jul-1909	412.19	May-1983	414.62	Oct-1929	408.76
15	Jun-1909	412.03	Apr-1922	414.59	May-1913	408.72
16	Jun-1912	412.00	Jul-1915	414.56	Jul-1927	408.71
17	Oct-1927	411.96	Jul-1909	414.48	Jul-1909	408.62
18	May-1984	411.94	Jul-1908	414.43	Jan-1930	408.54
19	Aug-1909	411.90	Aug-1915	414.19	May-1908	408.48
20	Apr-1927	411.88	Jun-1929	414.03	Feb-1930	408.48
21	Jun-1984	411.85	Apr-1983	413.86	Nov-1929	408.46
22	Apr-1994	411.81	Jun-1947	413.85	Jun-1912	408.40
23	Jul-1912	411.78	May-1984	413.80	Aug-1927	408.39
24	Jun-1985	411.64	Jun-1993	413.68	May-1912	408.34
25	Apr-1913	411.58	Jun-1984	413.60	Mar-1930	408.31
26	Jul-1908	411.57	Jun-1916	413.51	Apr-1928	408.31
27	May-1912	411.56	Jun-1915	413.46	Dec-1929	408.29
28	Nov-1927	411.52	Apr-1993	413.36	Jun-1928	408.28
29	Aug-1908	411.51	Jun-1973	413.19	Apr-1927	408.24
30	Apr-1909	411.49	Jun-1943	413.09	Jun-1913	408.23
31	Jul-1984	411.49	Apr-1979	413.00	Jun-1908	408.18
32	Jun-1983	411.43	Apr-1912	412.96	May-1921	408.16
33	May-1908	411.41	Oct-1986	412.94	May-1928	408.16
34	Sep-1909	411.41	Apr-1984	412.93	Apr-1922	408.13
35	Nov-1929	411.36	May-1920	412.84	Aug-1909	408.12
36	Jun-1928	411.35	Jun-1945	412.73	Jul-1912	408.12
37	May-1994	411.34	Oct-1993	412.33	Apr-1930	408.11
38	May-1913	411.33	Jun-1917	412.26	Apr-1912	408.11
39	Jun-1908	411.31	May-1916	412.25	Sep-1927	408.10
40	May-1983	411.30	Jun-1974	411.80	Mar-1929	408.10
41	Jul-1928	411.25	Apr-1945	411.80	Jul-1928	408.09
42	Mar-1913	411.20	Jun-1935	411.77	May-1910	408.08
43	May-1985	411.19	Mar-1973	411.75	May-1938	408.06
44	Apr-1984	411.13	May-1979	411.72	Mar-1928	408.04
45	Apr-1928	411.13	Jul-1984	411.68	Jun-1910	407.95
46	Aug-1912	411.13	May-1944	411.64	Oct-1927	407.94
47	May-1928	411.06	Apr-1920	411.53	Apr-1921	407.93
48	Sep-1908	411.04	Jul-1969	411.49	Apr-1939	407.91
49	May-1910	411.04	Apr-1916	411.48	Nov-1927	407.90
50	Aug-1916	411.02	Jul-1927	411.31	Jun-1938	407.86
51	Jun-1916	411.01	Jul-1947	411.30	Jul-1908	407.85
52	Apr-1985	410.97	May-1943	411.15	May-1930	407.84
53	Aug-1984	410.97	Jun-1982	410.99	Jul-1913	407.83
54	Jun-1913	410.96	May-1912	410.98	May-1939	407.83

			Ordered Monthly Highs			
ORDER	Month-Year	SWS No. 3		CORPS 70		BLSTFRN
55	Oct-1993	410.95	Jun-1909	410.97	May-1922	407.82
56	Mar-1929	410.93	May-1945	410.95	Aug-1928	407.79
57	Jun-1910	410.93	May-1922	410.94	Jun-1921	407.78
58	May-1922	410.91	Mar-1985	410.81	Sep-1909	407.77
59	Jul-1983	410.91	Jun-1983	410.75	Mar-1909	407.77
60	Dec-1927	410.91	Jun-1944	410.66	May-1923	407.76
61	Aug-1928	410.91	Jul-1907	410.57	Aug-1912	407.74
62	Apr-1912	410.88	Oct-1926	410.38	Dec-1927	407.73
63	Sep-1993	410.88	Jul-1929	410.37	Feb-1928	407.68
64	Jul-1916	410.88	Apr-1952	410.32	Apr-1911	407.67
65	Apr-1922	410.87	Sep-1915	410.16	May-1907	407.66
66	Jun-1994	410.86	Apr-1985	410.00	Jul-1910	407.65
67	Jul-1985	410.85	Mar-1929	409.96	Jan-1928	407.64
68	Jan-1930	410.84	May-1994	409.93	Apr-1910	407.63
69	Nov-1993	410.82	Apr-1906	409.88	Apr-1908	407.63
70	May-1921	410.80	May-1951	409.87	Jun-1907	407.60
71	Oct-1909	410.78	Jul-1916	409.87	Jun-1923	407.59
72	Dec-1929	410.77	May-1919	409.85	Sep-1928	407.58
73	Mar-1928	410.76	Dec-1982	409.79	Apr-1938	407.56
74	Jun-1922	410.72	May-1974	409.76	Jun-1939	407.56
75	May-1938	410.70	May-1952	409.75	Jun-1930	407.54
76	May-1916	410.67	Jun-1920	409.67	Oct-1909	407.54
77	Jul-1913	410.62	Jun-1951	409.66	May-1911	407.47
78	Jun-1938	410.61	Apr-1913	409.65	Aug-1908	407.46
79	May-1945	410.61	Nov-1993	409.60	Apr-1906	407.45
80	Sep-1928	410.57	Jul-1982	409.58	Jul-1938	407.44
81	Feb-1930	410.57	Jun-1919	409.48	Nov-1909	407.43
82	Jul-1910	410.56	May-1986	409.36	May-1906	407.43
83	Apr-1921	410.55	May-1960	409.35	May-1905	407.41
84	Jun-1907	410.52	Apr-1994	409.35	Feb-1910	407.40
85	Jan-1928	410.48	Nov-1986	409.28	Aug-1913	407.39
86	Aug-1993	410.47	Apr-1944	409.27	Sep-1912	407.38
87	May-1939	410.45	May-1909	409.26	Jun-1922	407.38
88	May-1907	410.45	Apr-1947	409.18	Mar-1906	407.35
89	Jul-1922	410.43	Jul-1920	409.15	Jul-1921	407.30
90	Sep-1916	410.40	May-1978	409.13	Oct-1928	407.28
91	Jun-1921	410.39	Jun-1912	409.06	Aug-1910	407.27
92	Apr-1939	410.37	Jul-1905	409.01	Jul-1907	407.26
93	May-1906	410.37	Jul-1943	409.00	Dec-1909	407.26
94	Sep-1912	410.36	Jul-1935	408.86	Jan-1910	407.25
95	Feb-1928	410.34	May-1975	408.85	Apr-1923	407.25
96	Mar-1930	410.33	Apr-1978	408.83	Jun-1906	407.21
97	Mar-1927	410.33	May-1982	408.81	Sep-1908	407.17
98	Sep-1984	410.32	May-1908	408.78	Jun-1905	407.15
99	Jul-1907	410.32	May-1969	408.73	Apr-1907	407.14
100	Apr-1910	410.32	Jul-1924	408.72	Jul-1939	407.14
101	Oct-1908	410.31	Jun-1907	408.66	Mar-1908	407.12
102	Jul-1938	410.31	Aug-1927	408.66	Jun-1924	407.12
103	Mar-1985	410.31	Apr-1982	408.63	Jul-1923	407.11
104	Jun-1945	410.30	Jul-1983	408.61	Oct-1912	407.11
105	Dec-1993	410.29	Jul-1917	408.57	Mar-1938	407.10
106	Jun-1906	410.28	Aug-1907	408.56	Apr-1905	407.09
107	Aug-1983	410.27	Apr-1960	408.53	May-1916	407.08
108	Nov-1909	410.26	Mar-1984	408.49	Mar-1910	407.08
109	Jun-1939	410.24	Apr-1962	408.49	Nov-1928	407.07

			Ordered Monthly Highs			
ORDER	Month-Year	SWS No. 3		CORPS 70		BLSTFRN
110	Aug-1907	410.21	Jun-1942	408.48	Jun-1916	407.07
111	Apr-1916	410.19	Oct-1973	408.47	Sep-1913	407.05
112	Apr-1930	410.18	Aug-1951	408.46	Jul-1922	407.05
113	Jul-1994	410.16	Apr-1939	408.45	Aug-1939	407.05
114	Apr-1906	410.12	Apr-1969	408.45	Jul-1930	407.04
115	Apr-1983	410.10	May-1991	408.42	Aug-1907	407.01
116	Aug-1985	410.09	May-1947	408.39	May-1917	407.01
117	Mar-1909	410.06	Jun-1938	408.36	Aug-1938	407.00
118	Apr-1945	410.05	Sep-1951	408.34	May-1924	406.98
119	Apr-1908	410.05	Jul-1945	408.27	Mar-1927	406.98
120	May-1923	410.03	Jun-1990	408.20	Jun-1911	406.97
121	Oct-1928	410.03	Aug-1908	408.16	Sep-1910	406.97
122	Aug-1913	410.01	Jul-1973	408.15	Mar-1905	406.97
123	Mar-1916	410.00	Apr-1951	408.13	Apr-1916	406.96
124	Jul-1945	409.99	Jun-1970	408.11	Mar-1939	406.95
125	May-1930	409.99	Jul-1928	408.06	Feb-1909	406.95
126	Mar-1906	409.99	Jun-1991	408.06	Oct-1908	406.94
127	Aug-1922	409.94	Jul-1944	408.06	Aug-1921	406.93
128	May-1974	409.93	May-1950	408.05	Dec-1928	406.93
129	Jun-1923	409.92	Jun-1928	408.02	May-1933	406.91
130	Jul-1921	409.92	Apr-1928	408.01	Mar-1923	406.90
131	Jul-1993	409.91	Jun-1922	407.99	Jul-1905	406.90
132	Aug-1910	409.90	Jun-1924	407.99	Nov-1912	406.88
133	Apr-1938	409.90	Jun-1906	407.96	Mar-1916	406.87
134	Jun-1973	409.89	Feb-1916	407.94	Aug-1923	406.87
135	Jul-1906	409.87	Jun-1986	407.93	Jan-1929	406.87
136	Sep-1915	409.87	Mar-1974	407.89	Aug-1916	406.86
137	Aug-1915	409.85	May-1961	407.85	Jul-1924	406.84
138	Sep-1907	409.84	May-1985	407.83	Jul-1906	406.83
139	Feb-1910	409.84	Nov-1985	407.83	Nov-1908	406.83
140	Jun-1974	409.83	Jul-1986	407.79	Apr-1917	406.82
141	Jan-1994	409.83	May-1917	407.77	Oct-1913	406.81
142	May-1905	409.82	Apr-1986	407.77	Jan-1905	406.81
143	Jan-1916	409.82	May-1906	407.74	Jul-1916	406.78
144	Jul-1973	409.81	May-1933	407.71	Mar-1922	406.76
145	Jun-1930	409.75	Jun-1979	407.68	Aug-1922	406.75
146	Aug-1938	409.75	May-1970	407.68	Jan-1913	406.75
147	Dec-1909	409.74	Mar-1993	407.64	Feb-1929	406.74
148	Jul-1939	409.72	Mar-1983	407.64	Oct-1910	406.73
149	Nov-1908	409.71	Apr-1909	407.63	Sep-1907	406.72
150	Oct-1915	409.68	Jul-1981	407.61	Jan-1909	406.71
151	Oct-1984	409.68	Oct-1927	407.46	Mar-1907	406.70
152	Oct-1912	409.67	Sep-1926	407.46	Sep-1939	406.69
153	Oct-1916	409.67	May-1935	407.46	Sep-1916	406.68
154	Jun-1924	409.65	Apr-1974	407.44	Dec-1908	406.65
155	Aug-1945	409.65	Mar-1927	407.42	Feb-1905	406.64
156	Jun-1905	409.62	Aug-1909	407.42	Apr-1914	406.64
157	Feb-1916	409.61	May-1913	407.40	Aug-1930	406.64
158	Jan-1910	409.61	Jul-1974	407.39	Sep-1938	406.64
159	Apr-1911	409.61	Oct-1915	407.36	Jun-1917	406.63
160	Mar-1984	409.58	Jun-1905	407.35	Mar-1914	406.63
161	Nov-1928	409.58	Jun-1950	407.35	Nov-1913	406.62
162	Jun-1920	409.57	Apr-1975	407.34	Mar-1921	406.62
163	Apr-1907	409.56	Jun-1985	407.34	Dec-1912	406.62
164	Aug-1939	409.56	Mar-1979	407.33	Feb-1913	406.62

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165	Jun-1993	409.56	Jun-1975	407.30	Sep-1921	406.62
166	Mar-1938	409.55	Mar-1916	407.26	Feb-1914	406.61
167	Aug-1906	409.54	Jun-1969	407.22	Aug-1906	406.61
168	Jul-1905	409.53	Nov-1926	407.22	Aug-1905	406.60
169	May-1933	409.51	Jun-1994	407.20	Feb-1916	406.59
170	Aug-1994	409.50	Jul-1942	407.19	Mar-1912	406.58
171	May-1920	409.49	Aug-1924	407.16	May-1920	406.58
172	Jul-1924	409.48	May-1907	407.15	Aug-1924	406.57
173	Jul-1923	409.45	Jul-1990	407.14	Jun-1933	406.55
174	Jul-1974	409.45	Jul-1922	407.14	Sep-1923	406.54
175	Aug-1921	409.44	May-1938	407.13	Oct-1907	406.53
176	Sep-1983	409.42	Dec-1986	407.13	Oct-1916	406.51
177	Oct-1907	409.41	Jun-1960	407.11	Jan-1916	406.50
178	Mar-1910	409.39	Nov-1928	407.09	Sep-1922	406.50
179	Jul-1920	409.39	May-1990	407.09	Jul-1911	406.49
180	Nov-1915	409.37	May-1965	407.07	Apr-1926	406.48
181	Dec-1928	409.36	Mar-1982	407.00	Nov-1910	406.46
182	Sep-1913	409.35	Dec-1985	406.99	May-1918	406.44
183	May-1924	409.32	Dec-1993	406.98	Jan-1907	406.42
184	Aug-1924	409.32	Apr-1965	406.96	Feb-1908	406.42
185	Feb-1994	409.32	Apr-1948	406.91	Dec-1913	406.42
186	Aug-1905	409.32	May-1910	406.89	Apr-1924	406.41
187	Sep-1922	409.31	Aug-1905	406.89	Nov-1916	406.40
188	Mar-1908	409.31	Feb-1974	406.88	May-1926	406.39
189	May-1973	409.31	Oct-1941	406.86	Nov-1907	406.38
190	May-1911	409.30	Jul-1978	406.82	Oct-1939	406.38
191	Aug-1973	409.28	Mar-1986	406.81	Mar-1911	406.38
192	Jan-1929	409.26	Apr-1919	406.81	Sep-1906	406.37
193	Aug-1920	409.25	Mar-1948	406.80	May-1935	406.36
194	Sep-1910	409.25	Jun-1921	406.78	Jun-1920	406.36
195	May-1917	409.21	Apr-1907	406.73	Jun-1935	406.35
196	Aug-1923	409.21	Nov-1941	406.73	Jul-1917	406.35
197	Sep-1985	409.20	May-1921	406.72	Sep-1905	406.35
198	Apr-1923	409.19	Apr-1987	406.72	Feb-1907	406.34
199	Jul-1930	409.18	Jun-1913	406.66	Oct-1938	406.34
200	Apr-1905	409.17	Jan-1910	406.62	Sep-1930	406.34
201	Nov-1984	409.15	Sep-1905	406.61	Jan-1914	406.33
202	Jul-1915	409.13	Jul-1906	406.58	Mar-1924	406.33
203	Sep-1938	409.12	Jul-1938	406.56	Jul-1933	406.33
204	Aug-1974	409.12	Mar-1994	406.53	Oct-1921	406.33
205	Jul-1917	409.10	Jun-1952	406.52	Feb-1939	406.31
206	May-1975	409.10	Apr-1938	406.48	Dec-1916	406.31
207	Jun-1975	409.09	Mar-1906	406.46	May-1914	406.31
208	Dec-1915	409.09	Mar-1945	406.45	Oct-1923	406.30
209	Mar-1994	409.07	Jun-1937	406.42	Sep-1924	406.29
210	Nov-1912	409.06	Apr-1908	406.39	Feb-1906	406.28
211	Nov-1916	409.05	Jul-1919	406.38	Dec-1907	406.28
212	Sep-1945	409.05	Aug-1928	406.38	Oct-1922	406.28
213	Sep-1905	409.04	Apr-1917	406.36	Apr-1918	406.26
214	Sep-1906	409.03	Jun-1910	406.36	Jan-1908	406.25
215	May-1944	409.02	Aug-1916	406.31	Jan-1917	406.24
216	Dec-1908	409.01	Jun-1967	406.25	Oct-1905	406.24
217	Feb-1909	409.01	May-1928	406.23	Dec-1910	406.23
218	Apr-1974	409.01	Jun-1962	406.23	May-1931	406.22
219	Jun-1933	409.00	May-1942	406.23	Mar-1917	406.22

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220	Jul-1935	408.99	Jun-1918	406.14	Nov-1938	406.20
221	Jun-1935	408.97	Jun-1923	406.13	Nov-1921	406.20
222	Jun-1917	408.97	Aug-1984	406.12	Jun-1918	406.18
223	Feb-1929	408.97	Mar-1910	406.08	Apr-1925	406.18
224	Sep-1924	408.96	Nov-1973	406.08	Jan-1939	406.17
225	Mar-1939	408.95	Aug-1929	406.07	Aug-1911	406.17
226	Aug-1917	408.94	Mar-1952	406.07	Nov-1939	406.16
227	Mar-1921	408.94	Jul-1912	406.06	Apr-1920	406.16
228	Mar-1945	408.93	Jan-1983	406.05	Dec-1922	406.15
229	May-1946	408.92	Apr-1949	406.04	Oct-1906	406.14
230	Mar-1905	408.92	May-1972	406.03	Apr-1940	406.14
231	Nov-1907	408.92	Mar-1922	406.01	Jun-1926	406.13
232	Jun-1943	408.92	Dec-1973	405.99	Jan-1906	406.13
233	Sep-1921	408.91	Apr-1910	405.98	Nov-1922	406.12
234	Oct-1905	408.90	Jun-1978	405.98	Jul-1935	406.10
235	Dec-1984	408.88	Jan-1916	405.97	Jan-1923	406.10
236	Jun-1944	408.88	Aug-1982	405.96	Feb-1911	406.09
237	Jun-1986	408.85	May-1976	405.95	Aug-1915	406.09
238	Apr-1975	408.84	Jul-1979	405.95	Nov-1923	406.09
239	Jan-1909	408.84	Apr-1943	405.93	Nov-1905	406.09
240	May-1935	408.84	Apr-1991	405.91	Dec-1921	406.09
241	Sep-1939	408.83	May-1905	405.90	Jun-1925	406.08
242	Jul-1944	408.83	Jun-1933	405.88	Nov-1906	406.07
243	Jul-1933	408.82	Mar-1908	405.86	Aug-1917	406.07
244	Mar-1922	408.81	May-1962	405.86	May-1925	406.07
245	Mar-1907	408.81	Apr-1950	405.84	Oct-1930	406.07
246	Sep-1920	408.80	May-1937	405.83	Apr-1931	406.06
247	Oct-1913	408.78	Jun-1965	405.82	Dec-1938	406.05
248	Aug-1935	408.77	Feb-1973	405.81	Jun-1915	406.04
249	Mar-1923	408.77	Apr-1933	405.78	Feb-1917	406.04
250	Apr-1917	408.76	May-1939	405.75	Jul-1920	406.04
251	May-1993	408.75	Jul-1975	405.75	Oct-1924	406.03
252	Jul-1975	408.75	Oct-1951	405.73	Feb-1923	406.02
253	Mar-1912	408.74	Jan-1974	405.73	Jan-1911	406.01
254	Oct-1983	408.73	Jul-1994	405.71	Jul-1915	406.01
255	Jul-1943	408.73	Aug-1981	405.71	Dec-1923	406.00
256	Mar-1974	408.72	Sep-1928	405.70	Dec-1905	406.00
257	Jun-1915	408.72	May-1924	405.69	Jun-1931	405.98
258	Jan-1905	408.69	Dec-1928	405.66	Aug-1933	405.98
259	Oct-1910	408.67	Mar-1946	405.65	May-1940	405.97
260	Sep-1923	408.67	Jul-1950	405.64	Dec-1906	405.96
261	Apr-1926	408.67	Sep-1924	405.63	Dec-1939	405.94
262	May-1986	408.66	Apr-1924	405.61	Apr-1933	405.93
263	Oct-1922	408.65	Apr-1946	405.60	Mar-1925	405.93
264	Sep-1994	408.65	Oct-1985	405.60	Sep-1915	405.91
265	Jun-1946	408.63	Jul-1921	405.59	Jun-1919	405.91
266	Jun-1911	408.62	Sep-1927	405.59	Dec-1924	405.90
267	Jul-1986	408.61	Jul-1923	405.59	Jun-1914	405.89
268	Jun-1919	408.60	Jan-1993	405.56	Jan-1922	405.89
269	May-1979	408.59	Aug-1969	405.55	Aug-1920	405.88
270	Jun-1947	408.59	Apr-1976	405.54	Nov-1930	405.87
271	Jun-1979	408.58	Nov-1951	405.54	Sep-1911	405.86
272	Aug-1930	408.58	Apr-1926	405.53	Nov-1924	405.86
273	Nov-1905	408.55	Aug-1973	405.48	Mar-1926	405.86
274	Sep-1973	408.55	Mar-1949	405.45	Jan-1924	405.84

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275	Oct-1938	408.55	Jun-1939	405.44	Sep-1917	405.83
276	May-1943	408.54	Nov-1972	405.43	Aug-1935	405.82
277	Feb-1986	408.53	Apr-1921	405.42	Dec-1915	405.81
278	Jul-1947	408.52	Apr-1905	405.41	May-1919	405.80
279	Mar-1986	408.51	Dec-1926	405.40	Jul-1925	405.79
280	Dec-1916	408.51	Mar-1909	405.40	Jul-1918	405.78
281	May-1918	408.51	Jul-1952	405.37	Feb-1925	405.78
282	Nov-1985	408.51	Aug-1920	405.37	Oct-1915	405.77
283	Sep-1917	408.50	Nov-1984	405.36	Jan-1940	405.77
284	May-1926	408.50	Feb-1928	405.30	Jun-1940	405.75
285	Oct-1924	408.50	Aug-1943	405.30	Jul-1926	405.75
286	Feb-1985	408.49	Dec-1992	405.25	Feb-1924	405.75
287	Jan-1913	408.49	Jan-1973	405.23	Jan-1925	405.74
288	Oct-1985	408.48	Mar-1939	405.23	Nov-1915	405.73
289	Dec-1907	408.47	Aug-1986	405.20	Mar-1940	405.73
290	Aug-1944	408.47	Apr-1923	405.19	Mar-1931	405.73
291	Jul-1919	408.46	Mar-1928	405.19	Feb-1922	405.73
292	Oct-1906	408.45	Mar-1937	405.18	Jun-1937	405.72
293	Apr-1944	408.45	Apr-1961	405.18	Mar-1920	405.72
294	Apr-1986	408.45	Sep-1907	405.16	Jul-1931	405.71
295	Dec-1985	408.44	May-1915	405.15	Sep-1920	405.71
296	Aug-1943	408.43	Nov-1927	405.15	Sep-1933	405.68
297	Dec-1912	408.42	Mar-1907	405.13	Feb-1940	405.65
298	Apr-1979	408.42	Jun-1925	405.12	Oct-1917	405.65
299	Jul-1979	408.41	Mar-1975	405.11	Dec-1930	405.65
300	Jan-1907	408.41	Nov-1961	405.10	Jul-1919	405.64
301	Feb-1906	408.40	Mar-1936	405.10	Apr-1934	405.62
302	Sep-1974	408.40	Apr-1936	405.09	May-1937	405.62
303	Aug-1975	408.40	Oct-1905	405.07	Apr-1935	405.61
304	Feb-1905	408.39	May-1930	405.07	May-1915	405.59
305	Apr-1920	408.39	Mar-1930	405.06	Oct-1911	405.58
306	Oct-1921	408.39	May-1948	405.06	Aug-1918	405.58
307	Aug-1946	408.38	Apr-1925	405.05	Apr-1936	405.57
308	Nov-1983	408.38	Apr-1915	405.03	Sep-1935	405.57
309	Aug-1947	408.36	Mar-1962	405.03	Oct-1920	405.57
310	Jul-1982	408.32	Apr-1942	405.02	Aug-1925	405.55
311	Jan-1906	408.31	Mar-1912	405.00	Jul-1914	405.52
312	Feb-1908	408.30	Sep-1986	405.00	Apr-1937	405.51
313	Nov-1913	408.30	May-1987	404.98	Nov-1917	405.48
314	Jan-1985	408.29	Apr-1937	404.97	Oct-1933	405.46
315	Apr-1918	408.28	Apr-1980	404.97	Aug-1926	405.45
316	Apr-1925	408.28	Jul-1967	404.96	Aug-1919	405.45
317	Jun-1925	408.27	Aug-1983	404.95	Mar-1934	405.45
318	Oct-1920	408.27	Apr-1992	404.95	Aug-1931	405.43
319	Sep-1935	408.27	Mar-1987	404.93	Nov-1920	405.42
320	Dec-1905	408.26	Jun-1930	404.92	Jan-1931	405.41
321	Oct-1945	408.26	Aug-1979	404.92	Nov-1911	405.41
322	Aug-1986	408.24	May-1923	404.91	Jul-1937	405.40
323	Apr-1924	408.23	Feb-1983	404.90	Oct-1935	405.40
324	Aug-1919	408.21	May-1966	404.90	Mar-1933	405.39
325	Aug-1933	408.21	Sep-1982	404.89	Feb-1931	405.37
326	Nov-1938	408.21	Mar-1905	404.89	May-1936	405.37
327	Oct-1923	408.20	Apr-1935	404.88	Dec-1920	405.36
328	Jun-1926	408.20	Jul-1913	404.88	May-1984	405.35
329	Aug-1982	408.20	Jun-1981	404.85	Jul-1940	405.34

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330	Apr-1973	408.18		Jul-1991	404.83	Aug-1914	405.33
331	Jan-1908	408.18		Jun-1926	404.80	May-1934	405.33
332	May-1925	408.17		Apr-1971	404.79	Sep-1918	405.33
333	Feb-1907	408.15		Jul-1925	404.79	Mar-1935	405.32
334	May-1952	408.14		Jun-1932	404.79	Sep-1925	405.32
335	Apr-1952	408.14		Mar-1971	404.78	Dec-1917	405.31
336	Feb-1913	408.14		May-1946	404.72	Mar-1936	405.31
337	Jun-1918	408.13		Jul-1948	404.71	Jun-1985	405.28
338	Nov-1906	408.13		Sep-1909	404.71	Jan-1921	405.27
339	Apr-1946	408.12		Jul-1960	404.70	Sep-1919	405.27
340	May-1919	408.12		Nov-1915	404.68	Nov-1933	405.26
341	Nov-1922	408.11		Aug-1906	404.67	Feb-1918	405.26
342	Oct-1939	408.11		Aug-1945	404.67	Oct-1919	405.25
343	Jun-1937	408.11		Jul-1958	404.66	Jun-1936	405.24
344	Jan-1917	408.10		Nov-1982	404.66	Dec-1911	405.24
345	Mar-1924	408.10		Sep-1908	404.66	Nov-1935	405.23
346	Nov-1921	408.09		Apr-1970	404.65	Jan-1918	405.23
347	Jun-1952	408.09		Oct-1928	404.64	Sep-1926	405.23
348	Mar-1983	408.07		Dec-1927	404.60	Feb-1912	405.22
349	Nov-1924	408.06		Jun-1946	404.59	May-1932	405.21
350	Jan-1986	408.04		Sep-1921	404.58	Feb-1921	405.20
351	Apr-1914	408.04		Feb-1986	404.56	Dec-1935	405.19
352	Nov-1910	408.03		Feb-1910	404.56	Apr-1932	405.19
353	Feb-1914	408.03		Apr-1932	404.55	Mar-1918	405.18
354	Dec-1983	408.02		Aug-1944	404.55	Oct-1925	405.17
355	Jun-1962	408.02		Mar-1915	404.54	Oct-1918	405.17
356	Sep-1930	408.02		May-1926	404.54	Jun-1934	405.17
357	Feb-1974	408.01		Jun-1957	404.52	Nov-1919	405.15
358	Jul-1946	408.01		Aug-1947	404.52	Sep-1931	405.14
359	Feb-1939	408.00		Jun-1949	404.51	Aug-1937	405.14
360	Jun-1982	407.99		Jul-1965	404.51	Feb-1936	405.12
361	Mar-1914	407.97		Jul-1914	404.51	Jun-1984	405.12
362	Jul-1911	407.96		Aug-1917	404.49	Jan-1912	405.11
363	Oct-1917	407.95		Mar-1925	404.49	Sep-1914	405.11
364	May-1947	407.93		May-1971	404.48	Dec-1933	405.11
365	May-1948	407.93		Sep-1965	404.47	Jan-1936	405.11
366	Dec-1922	407.93		May-1936	404.45	Nov-1925	405.10
367	Aug-1979	407.92		Jul-1985	404.43	Aug-1940	405.08
368	Dec-1924	407.92		Mar-1913	404.43	Jun-1932	405.08
369	Apr-1993	407.91		Apr-1953	404.42	Jan-1927	405.08
370	Jul-1925	407.91		Mar-1976	404.42	Oct-1926	405.06
371	Jan-1939	407.90		Dec-1983	404.41	May-1985	405.05
372	Oct-1994	407.90		Jan-1994	404.41	Nov-1918	405.04
373	Sep-1944	407.89		Jul-1962	404.41	Oct-1914	405.03
374	Oct-1973	407.88		Jul-1932	404.40	Feb-1927	405.03
375	Mar-1975	407.87		Jun-1914	404.40	Feb-1926	405.02
376	Sep-1943	407.86		Jul-1918	404.40	Dec-1919	405.02
377	Jun-1948	407.85		Apr-1966	404.38	Jan-1934	405.02
378	Sep-1947	407.85		Jan-1907	404.38	Apr-1985	405.01
379	Apr-1933	407.84		Aug-1935	404.37	Apr-1984	405.00
380	Apr-1987	407.83		Nov-1983	404.36	Jan-1920	405.00
381	May-1915	407.82		Feb-1994	404.35	Jul-1932	404.99
382	Mar-1987	407.82		Mar-1935	404.35	Aug-1932	404.98
383	Dec-1938	407.82		Apr-1918	404.32	Dec-1925	404.98
384	Dec-1913	407.82		Aug-1912	404.31	Apr-1919	404.97

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385	Sep-1986	407.82	Oct-1969	404.30	Nov-1926	404.96
386	Jul-1952	407.82	Jul-1939	404.30	Dec-1918	404.96
387	May-1987	407.80	Sep-1938	404.29	Feb-1915	404.95
388	May-1937	407.79	Jan-1985	404.29	Feb-1934	404.93
389	Dec-1921	407.78	Jun-1980	404.29	Jan-1926	404.93
390	Mar-1917	407.77	Dec-1909	404.26	Jul-1936	404.92
391	Nov-1923	407.77	Apr-1988	404.26	Oct-1931	404.90
392	Mar-1925	407.76	May-1981	404.24	Mar-1919	404.90
393	Sep-1975	407.75	Jul-1937	404.24	Feb-1920	404.90
394	Sep-1919	407.75	Jan-1928	404.24	Sep-1937	404.89
395	May-1978	407.75	Aug-1922	404.23	Jul-1985	404.88
396	Apr-1947	407.74	Jun-1961	404.23	Nov-1914	404.88
397	Jul-1948	407.74	Nov-1905	404.23	Jan-1915	404.87
398	Dec-1906	407.74	Mar-1920	404.22	Dec-1926	404.87
399	Sep-1982	407.73	Dec-1984	404.22	May-1945	404.83
400	Oct-1935	407.73	Apr-1967	404.22	Mar-1915	404.83
401	Jul-1918	407.73	Jun-1971	404.21	Mar-1932	404.82
402	Jul-1926	407.72	Feb-1984	404.21	Jul-1934	404.82
403	Jul-1937	407.71	Aug-1974	404.21	Dec-1914	404.80
404	Jul-1962	407.69	May-1953	404.19	Nov-1931	404.80
405	Mar-1946	407.69	Apr-1972	404.19	Apr-1915	404.78
406	Mar-1926	407.69	Aug-1938	404.18	Sep-1932	404.78
407	Jun-1978	407.68	Feb-1993	404.18	Sep-1940	404.77
408	Nov-1920	407.68	Sep-1929	404.14	Jan-1919	404.74
409	Jul-1978	407.68	Nov-1909	404.14	Jan-1932	404.74
410	Jun-1987	407.67	Jul-1957	404.13	Oct-1937	404.74
411	May-1914	407.67	Jan-1987	404.13	Dec-1931	404.74
412	Mar-1911	407.66	Aug-1950	404.12	Feb-1932	404.71
413	Feb-1984	407.66	Jul-1946	404.11	Feb-1919	404.70
414	Mar-1920	407.66	Aug-1978	404.11	Feb-1938	404.68
415	Oct-1974	407.65	Apr-1930	404.10	Apr-1994	404.68
416	Jan-1923	407.63	Feb-1930	404.08	Jul-1984	404.67
417	Apr-1962	407.63	Jul-1910	404.08	Oct-1932	404.66
418	Jan-1974	407.62	Jan-1986	404.07	May-1994	404.66
419	Jan-1914	407.62	Aug-1952	404.06	Mar-1985	404.65
420	May-1962	407.62	Mar-1942	404.04	Aug-1936	404.64
421	May-1931	407.61	Mar-1938	404.04	Apr-1945	404.64
422	Sep-1946	407.61	Jun-1987	404.04	Mar-1937	404.60
423	Apr-1948	407.61	Aug-1923	404.03	Dec-1937	404.58
424	Nov-1945	407.60	Jun-1941	404.02	Nov-1937	404.57
425	Nov-1973	407.60	Jan-1929	404.02	Aug-1934	404.55
426	Sep-1933	407.60	Sep-1920	404.01	Nov-1932	404.55
427	Oct-1986	407.59	Sep-1973	404.01	Aug-1985	404.54
428	Nov-1986	407.59	Oct-1965	403.99	Oct-1940	404.51
429	Aug-1925	407.58	Oct-1907	403.96	Jan-1938	404.49
430	Jan-1987	407.58	Oct-1970	403.95	Jun-1945	404.44
431	Nov-1939	407.56	Nov-1919	403.95	Sep-1936	404.44
432	Dec-1986	407.56	May-1932	403.92	Jun-1994	404.41
433	Feb-1917	407.56	Feb-1985	403.92	Jan-1933	404.39
434	Apr-1937	407.56	Sep-1979	403.91	Nov-1940	404.38
435	Dec-1923	407.55	Jun-1966	403.91	Dec-1932	404.38
436	Jun-1950	407.55	Mar-1978	403.90	Jan-1937	404.38
437	Aug-1918	407.55	Jul-1949	403.89	Feb-1937	404.38
438	Jun-1961	407.55	May-1992	403.89	Jan-1941	404.33
439	May-1982	407.53	Aug-1921	403.89	Dec-1940	404.33

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440	Dec-1973	407.53	Aug-1990	403.89	Feb-1933	404.32
441	Jan-1927	407.53	Mar-1944	403.88	Sep-1934	404.30
442	Jan-1984	407.53	Sep-1978	403.87	Aug-1984	404.28
443	May-1961	407.52	Apr-1941	403.86	May-1983	404.28
444	Dec-1910	407.51	Jun-1972	403.85	Oct-1936	404.27
445	Aug-1911	407.50	Jul-1933	403.85	Jun-1983	404.27
446	Oct-1930	407.48	Feb-1907	403.83	Nov-1936	404.18
447	Feb-1925	407.48	Sep-1916	403.77	Sep-1985	404.12
448	May-1950	407.48	Nov-1946	403.77	Mar-1984	404.11
449	May-1976	407.47	May-1918	403.75	Dec-1936	404.09
450	Aug-1951	407.47	Sep-1984	403.71	Oct-1934	404.08
451	Oct-1919	407.46	Nov-1977	403.70	Feb-1941	404.08
452	Apr-1935	407.45	Jun-1948	403.70	Apr-1941	404.06
453	Jan-1925	407.44	Aug-1942	403.69	Jul-1994	404.02
454	Jul-1942	407.43	Aug-1958	403.69	Sep-1984	404.02
455	Nov-1994	407.43	Aug-1985	403.68	Jul-1945	403.98
456	Mar-1952	407.40	Jan-1946	403.66	Jul-1983	403.94
457	Feb-1987	407.40	Mar-1951	403.66	Nov-1934	403.93
458	Jul-1987	407.40	Oct-1982	403.65	Mar-1945	403.87
459	Mar-1948	407.38	Jun-1953	403.64	Oct-1984	403.87
460	Nov-1917	407.38	Nov-1970	403.62	Jan-1935	403.86
461	Feb-1923	407.38	Jan-1932	403.60	May-1975	403.86
462	Aug-1952	407.36	May-1925	403.59	Feb-1935	403.83
463	Apr-1940	407.36	Feb-1906	403.59	Dec-1934	403.82
464	Oct-1947	407.36	Mar-1926	403.59	May-1944	403.82
465	Jan-1922	407.35	May-1949	403.58	May-1941	403.81
466	Sep-1951	407.35	Mar-1992	403.58	Apr-1975	403.79
467	Apr-1931	407.33	May-1959	403.58	Nov-1984	403.77
468	Mar-1988	407.31	Oct-1984	403.56	Oct-1985	403.76
469	Sep-1979	407.30	Feb-1987	403.56	May-1946	403.75
470	Aug-1926	407.30	Sep-1912	403.56	Dec-1984	403.73
471	Dec-1920	407.29	Mar-1969	403.54	Nov-1985	403.73
472	Apr-1936	407.29	Mar-1924	403.54	Mar-1941	403.72
473	Aug-1937	407.29	Sep-1970	403.52	Aug-1994	403.72
474	Jul-1951	407.28	Feb-1925	403.50	Jun-1975	403.72
475	Jul-1961	407.27	Sep-1969	403.48	Aug-1945	403.69
476	Jun-1976	407.27	Oct-1911	403.48	May-1974	403.68
477	Jan-1924	407.24	Feb-1952	403.48	Aug-1983	403.66
478	Feb-1927	407.24	Aug-1972	403.47	Apr-1983	403.64
479	Jun-1914	407.23	May-1967	403.47	Jun-1974	403.61
480	Jun-1931	407.23	Feb-1942	403.46	Feb-1985	403.61
481	Jul-1950	407.22	Oct-1972	403.46	Jun-1946	403.60
482	Apr-1978	407.22	Jan-1925	403.46	Dec-1985	403.59
483	Oct-1943	407.20	Aug-1994	403.45	Apr-1944	403.56
484	Jun-1942	407.19	Sep-1906	403.44	Mar-1986	403.53
485	Oct-1982	407.19	Nov-1992	403.43	Feb-1986	403.52
486	Oct-1944	407.19	Sep-1981	403.43	Jun-1944	403.50
487	Jun-1970	407.19	Aug-1932	403.43	Jan-1985	403.47
488	Aug-1978	407.19	Sep-1942	403.42	Jun-1943	403.46
489	Apr-1970	407.18	Mar-1943	403.42	Jun-1941	403.46
490	Dec-1926	407.18	Sep-1972	403.42	Sep-1945	403.44
491	Aug-1942	407.17	Oct-1929	403.42	Apr-1986	403.42
492	Feb-1911	407.16	Apr-1914	403.37	Aug-1946	403.40
493	Sep-1925	407.15	Jul-1926	403.36	Jul-1975	403.39
494	Nov-1926	407.14	Dec-1915	403.36	May-1943	403.38

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495	Nov-1935	407.14	Mar-1988	403.35	Sep-1983	403.37
496	May-1936	407.14	Apr-1959	403.35	Sep-1994	403.37
497	Aug-1950	407.14	May-1957	403.35	May-1986	403.36
498	Aug-1962	407.14	Jul-1970	403.34	Apr-1946	403.35
499	Jun-1936	407.13	Mar-1919	403.33	Jun-1947	403.31
500	Oct-1951	407.13	Aug-1919	403.33	Jun-1986	403.30
501	May-1940	407.12	Aug-1975	403.32	Jan-1986	403.28
502	Aug-1948	407.12	Oct-1909	403.30	Jul-1974	403.28
503	Sep-1918	407.10	Feb-1962	403.29	Mar-1975	403.24
504	Dec-1982	407.10	Jun-1976	403.29	Apr-1947	403.23
505	Jun-1951	407.09	Oct-1938	403.27	Jul-1944	403.20
506	Nov-1974	407.09	Jan-1927	403.26	Jul-1946	403.20
507	Mar-1933	407.08	Dec-1941	403.26	May-1947	403.20
508	Mar-1935	407.08	Feb-1975	403.25	Oct-1983	403.19
509	Oct-1975	407.07	Sep-1983	403.24	Nov-1983	403.18
510	Oct-1933	407.06	Jul-1953	403.24	Aug-1975	403.16
511	Nov-1930	407.06	Jul-1992	403.22	Apr-1974	403.15
512	Feb-1924	407.06	Jul-1930	403.21	Oct-1945	403.14
513	Apr-1988	407.05	Mar-1950	403.21	Mar-1946	403.14
514	Jan-1911	407.05	Dec-1951	403.18	Dec-1983	403.11
515	Dec-1939	407.05	Nov-1929	403.17	Jul-1943	403.10
516	May-1932	407.04	Oct-1923	403.16	Aug-1974	403.10
517	Dec-1945	407.04	Sep-1945	403.16	Apr-1979	403.09
518	Aug-1932	407.03	Oct-1983	403.14	Oct-1994	403.08
519	Nov-1919	407.03	Oct-1924	403.14	Sep-1946	403.06
520	Feb-1922	407.03	Jan-1906	403.13	May-1948	403.05
521	Sep-1911	407.03	Sep-1961	403.13	Jul-1986	403.03
522	Feb-1975	407.01	Sep-1975	403.13	May-1979	403.00
523	Mar-1976	407.01	Dec-1972	403.12	Feb-1984	403.00
524	May-1970	407.00	Mar-1932	403.11	Jul-1941	403.00
525	Apr-1950	407.00	Feb-1915	403.10	Apr-1948	403.00
526	Aug-1987	406.99	Oct-1977	403.08	Mar-1974	402.95
527	Jul-1932	406.98	Oct-1908	403.08	Mar-1994	402.94
528	Jul-1914	406.98	Jul-1987	403.05	Jun-1948	402.93
529	Sep-1926	406.97	Feb-1982	403.05	Nov-1994	402.92
530	Oct-1946	406.97	Sep-1944	403.04	Jul-1947	402.92
531	Jun-1932	406.96	Mar-1961	403.02	Nov-1945	402.91
532	Oct-1926	406.96	Feb-1929	403.02	Aug-1944	402.90
533	Apr-1976	406.94	Sep-1985	403.01	Jan-1984	402.89
534	Jul-1969	406.94	Oct-1945	403.00	Mar-1948	402.87
535	Jan-1983	406.93	Aug-1913	403.00	Sep-1975	402.85
536	Jul-1931	406.92	Aug-1992	402.93	Sep-1974	402.84
537	Aug-1914	406.92	Mar-1923	402.93	May-1976	402.82
538	Dec-1994	406.90	Oct-1921	402.93	Jul-1948	402.82
539	Jun-1940	406.90	Aug-1925	402.92	Aug-1943	402.81
540	Apr-1932	406.90	Jul-1968	402.91	Feb-1994	402.79
541	Jan-1921	406.89	Jul-1971	402.91	Oct-1946	402.77
542	Nov-1951	406.89	Mar-1921	402.91	Aug-1986	402.76
543	Jan-1975	406.86	Aug-1939	402.90	Jun-1979	402.75
544	Dec-1917	406.85	Apr-1911	402.90	Mar-1988	402.74
545	Dec-1935	406.84	Feb-1949	402.87	Feb-1975	402.73
546	Jun-1957	406.84	Feb-1946	402.85	Dec-1945	402.72
547	Feb-1983	406.83	Jun-1936	402.83	Jan-1994	402.72
548	Jul-1976	406.83	Nov-1994	402.82	May-1950	402.70
549	Nov-1982	406.82	May-1914	402.80	Nov-1946	402.70

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550	Apr-1934	406.82	Dec-1931	402.80		Mar-1976	402.69
551	Mar-1973	406.81	Aug-1970	402.78		Dec-1994	402.69
552	May-1953	406.81	Feb-1969	402.77		Jun-1993	402.68
553	May-1988	406.80	Feb-1937	402.77		May-1978	402.67
554	Oct-1925	406.79	Aug-1948	402.76		Apr-1988	402.66
555	Jul-1970	406.79	Sep-1922	402.74		Jun-1962	402.66
556	Mar-1931	406.79	Feb-1932	402.73		Apr-1976	402.66
557	Feb-1946	406.78	Aug-1946	402.72		Feb-1946	402.65
558	May-1963	406.77	Oct-1912	402.71		Sep-1944	402.63
559	Sep-1937	406.75	Jun-1954	402.71		Jul-1942	402.63
560	Nov-1946	406.75	Oct-1961	402.71		Jun-1942	402.62
561	Nov-1947	406.74	Mar-1991	402.69		Apr-1952	402.62
562	Sep-1952	406.73	Sep-1943	402.69		Jul-1993	402.61
563	Mar-1978	406.73	Jul-1961	402.68		Aug-1941	402.61
564	Oct-1979	406.72	Feb-1909	402.68		Oct-1974	402.61
565	Oct-1918	406.72	Jan-1952	402.68		Jun-1976	402.61
566	Aug-1969	406.71	Nov-1911	402.68		Aug-1947	402.60
567	Jun-1969	406.70	Nov-1938	402.68		Dec-1993	402.59
568	Mar-1936	406.70	Jan-1984	402.67		Jun-1950	402.59
569	Dec-1919	406.70	Nov-1907	402.67		May-1963	402.59
570	May-1942	406.68	Oct-1916	402.66		Apr-1987	402.58
571	Jul-1949	406.68	Nov-1908	402.66		Apr-1962	402.58
572	Sep-1978	406.68	Dec-1905	402.66		Jan-1946	402.58
573	Jan-1940	406.67	Sep-1917	402.66		May-1987	402.56
574	Jul-1981	406.67	Oct-1914	402.65		Jan-1975	402.55
575	Jul-1936	406.67	Nov-1974	402.64		Oct-1975	402.54
576	Jan-1946	406.64	May-1963	402.64		Sep-1986	402.54
577	Sep-1942	406.64	May-1941	402.62		Nov-1993	402.54
578	Dec-1951	406.63	Aug-1926	402.62		Sep-1943	402.54
579	May-1980	406.63	Sep-1914	402.60		Dec-1946	402.54
580	Oct-1911	406.62	Sep-1974	402.60		May-1952	402.53
581	Dec-1930	406.62	Sep-1935	402.58		Jul-1979	402.52
582	Feb-1921	406.62	Oct-1942	402.58		Aug-1993	402.52
583	Sep-1932	406.61	Aug-1910	402.58		Apr-1950	402.52
584	Sep-1914	406.60	May-1980	402.57		May-1962	402.51
585	Sep-1950	406.60	Jul-1972	402.56		Apr-1978	402.50
586	Mar-1940	406.60	Dec-1987	402.56		Jun-1987	402.49
587	Dec-1974	406.59	Oct-1978	402.55		Mar-1987	402.48
588	Nov-1975	406.59	Oct-1906	402.53		Feb-1974	402.47
589	Mar-1934	406.58	Oct-1981	402.53		Nov-1974	402.47
590	Nov-1933	406.57	Sep-1923	402.52		Jun-1978	402.47
591	Nov-1925	406.57	Aug-1987	402.50		Mar-1983	402.46
592	Aug-1961	406.57	Feb-1927	402.50		Jun-1973	402.46
593	Apr-1982	406.56	Feb-1943	402.50		Aug-1948	402.46
594	Jan-1920	406.55	Nov-1942	402.50		May-1988	402.46
595	May-1949	406.55	Oct-1920	402.48		May-1993	402.45
596	Aug-1970	406.55	Mar-1918	402.47		Jul-1962	402.45
597	Nov-1944	406.54	Jan-1905	402.46		Sep-1993	402.44
598	Apr-1953	406.54	Jan-1930	402.44		Jan-1947	402.42
599	Jan-1918	406.53	Oct-1919	402.43		Mar-1947	402.42
600	Nov-1943	406.53	Sep-1947	402.42		May-1942	402.42
601	Aug-1931	406.52	Dec-1994	402.42		Aug-1942	402.42
602	Sep-1948	406.51	Sep-1992	402.41		Oct-1993	402.41
603	Jan-1936	406.51	Dec-1923	402.40		Jun-1961	402.41
604	Jun-1953	406.50	Nov-1969	402.40		Oct-1986	402.40

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605	Oct-1914	406.48	Sep-1977	402.39	May-1973	402.39
606	Mar-1962	406.48	Aug-1918	402.37	May-1953	402.39
607	Feb-1918	406.48	Nov-1924	402.37	May-1961	402.38
608	Nov-1911	406.47	Feb-1988	402.36	Apr-1963	402.38
609	Sep-1962	406.47	Mar-1953	402.36	Oct-1944	402.37
610	Dec-1946	406.47	Sep-1980	402.36	Jun-1952	402.36
611	May-1991	406.45	Nov-1912	402.34	Jul-1982	402.36
612	Feb-1940	406.44	Nov-1925	402.34	Jun-1963	402.35
613	Aug-1981	406.44	Jan-1943	402.34	Jun-1982	402.35
614	May-1951	406.42	May-1988	402.34	Jul-1987	402.35
615	Feb-1936	406.41	Mar-1959	402.32	Nov-1975	402.35
616	Apr-1963	406.41	Aug-1968	402.31	Jul-1973	402.35
617	Mar-1980	406.41	Jun-1968	402.31	Sep-1947	402.34
618	Feb-1912	406.41	Nov-1979	402.31	Oct-1943	402.33
619	Jul-1940	406.40	Sep-1950	402.30	Jul-1978	402.33
620	Jun-1988	406.40	Feb-1908	402.30	Mar-1977	402.33
621	Jun-1963	406.40	Sep-1994	402.29	Mar-1949	402.32
622	May-1971	406.38	Mar-1980	402.28	Dec-1974	402.31
623	Jun-1949	406.37	Apr-1955	402.27	Jul-1976	402.29
624	Sep-1969	406.36	Oct-1925	402.26	Apr-1953	402.29
625	Apr-1943	406.36	Jun-1959	402.26	Aug-1979	402.27
626	Jul-1991	406.36	Nov-1921	402.24	Apr-1943	402.27
627	Sep-1987	406.34	Nov-1981	402.24	Jul-1961	402.25
628	May-1934	406.34	Mar-1966	402.23	May-1982	402.25
629	Mar-1977	406.34	Nov-1916	402.22	May-1970	402.25
630	Nov-1918	406.33	Mar-1933	402.22	Sep-1941	402.24
631	Jul-1957	406.33	Nov-1906	402.21	Nov-1986	402.24
632	Jun-1981	406.33	Sep-1910	402.21	Apr-1977	402.24
633	Jun-1980	406.32	Aug-1953	402.21	Oct-1947	402.24
634	Mar-1949	406.32	Mar-1963	402.21	Jun-1970	402.24
635	Apr-1980	406.31	Aug-1937	402.20	Apr-1970	402.23
636	Mar-1918	406.30	Nov-1923	402.19	Aug-1973	402.23
637	Dec-1911	406.30	Mar-1990	402.18	Jul-1950	402.22
638	Apr-1919	406.30	Dec-1908	402.18	Aug-1982	402.22
639	Mar-1963	406.29	Feb-1905	402.18	May-1980	402.20
640	Oct-1937	406.28	Aug-1991	402.17	Dec-1975	402.20
641	Dec-1925	406.27	Mar-1911	402.17	Aug-1987	402.19
642	Feb-1920	406.27	Dec-1991	402.16	Mar-1943	402.19
643	Feb-1926	406.27	Aug-1962	402.16	Mar-1944	402.19
644	Oct-1932	406.27	Dec-1970	402.15	Jun-1988	402.19
645	Jan-1947	406.26	Dec-1929	402.14	Mar-1963	402.18
646	Mar-1947	406.26	Feb-1926	402.14	Feb-1947	402.18
647	Dec-1947	406.24	May-1911	402.13	Jun-1957	402.17
648	Jan-1952	406.24	Dec-1921	402.12	Aug-1962	402.16
649	May-1990	406.23	Sep-1919	402.12	Jan-1974	402.16
650	Jun-1991	406.23	Mar-1965	402.12	Sep-1942	402.15
651	Mar-1979	406.22	May-1968	402.11	Nov-1944	402.15
652	Mar-1943	406.22	May-1954	402.10	Jul-1952	402.14
653	Apr-1949	406.21	Sep-1987	402.09	Jun-1953	402.14
654	Nov-1979	406.21	Jun-1964	402.08	Nov-1943	402.14
655	Oct-1978	406.21	Oct-1994	402.07	Mar-1952	402.14
656	Dec-1975	406.21	Sep-1913	402.07	Sep-1948	402.13
657	Jan-1931	406.20	Aug-1933	402.06	Apr-1993	402.13
658	Aug-1976	406.20	Dec-1961	402.03	Apr-1949	402.13
659	Jun-1990	406.20	Feb-1924	402.03	May-1977	402.12

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660	Jun-1934	406.19	Dec-1924	402.03	May-1949	402.12
661	Apr-1951	406.19	Nov-1978	402.03	Feb-1987	402.11
662	Nov-1914	406.19	Aug-1914	402.02	Mar-1978	402.11
663	Aug-1949	406.18	Dec-1907	402.01	Jan-1987	402.11
664	Jan-1912	406.17	Dec-1911	402.00	Nov-1947	402.10
665	Feb-1952	406.17	Jan-1988	401.99	Dec-1986	402.10
666	Jul-1990	406.17	Dec-1919	401.99	Aug-1950	402.09
667	May-1981	406.17	Feb-1912	401.98	Sep-1973	402.09
668	Dec-1933	406.16	Dec-1975	401.98	Apr-1980	402.09
669	Mar-1950	406.16	Sep-1990	401.97	Apr-1951	402.08
670	Jun-1971	406.16	Apr-1990	401.96	Jun-1977	402.07
671	Oct-1942	406.15	Jan-1975	401.95	Mar-1980	402.07
672	Feb-1931	406.14	Sep-1952	401.92	Jul-1981	402.06
673	Aug-1936	406.14	May-1964	401.91	Apr-1973	402.06
674	Jan-1926	406.13	Nov-1920	401.91	Oct-1941	402.05
675	Mar-1932	406.13	Dec-1906	401.90	Mar-1950	402.04
676	Apr-1915	406.12	Dec-1971	401.89	Apr-1942	402.03
677	May-1966	406.10	Oct-1975	401.89	Mar-1951	402.02
678	Aug-1940	406.09	Apr-1963	401.88	Sep-1982	402.02
679	Mar-1993	406.08	Jul-1954	401.87	Jun-1951	402.02
680	Dec-1918	406.08	Sep-1962	401.85	Aug-1978	402.01
681	Sep-1981	406.06	Aug-1930	401.85	Dec-1973	402.01
682	Sep-1931	406.06	Oct-1979	401.83	Sep-1979	402.00
683	Oct-1952	406.06	Oct-1922	401.83	Jun-1981	402.00
684	Feb-1915	406.04	Jun-1911	401.83	Dec-1947	402.00
685	Oct-1969	406.04	Sep-1925	401.82	Dec-1943	401.99
686	Jul-1980	406.03	Mar-1947	401.82	Jan-1976	401.99
687	May-1969	406.02	Nov-1975	401.81	Dec-1982	401.98
688	Mar-1951	406.01	Jan-1950	401.80	Jun-1980	401.98
689	Sep-1970	406.01	Feb-1939	401.79	Jul-1949	401.98
690	Mar-1982	406.00	Apr-1981	401.79	Oct-1973	401.98
691	Mar-1919	406.00	Dec-1938	401.78	Dec-1944	401.98
692	Dec-1944	406.00	Feb-1938	401.78	Jul-1963	401.97
693	Aug-1990	405.99	Mar-1958	401.77	Feb-1949	401.97
694	Dec-1943	405.99	Dec-1977	401.76	Feb-1945	401.96
695	Oct-1950	405.99	Feb-1971	401.75	Aug-1976	401.96
696	Jan-1915	405.99	Mar-1972	401.75	May-1951	401.95
697	Jul-1953	405.99	Nov-1945	401.74	Oct-1942	401.94
698	Dec-1914	405.98	Nov-1922	401.72	Nov-1973	401.94
699	Jul-1971	405.97	Oct-1959	401.72	Aug-1952	401.93
700	Mar-1953	405.97	Sep-1941	401.72	Mar-1962	401.93
701	Mar-1944	405.96	Dec-1942	401.72	May-1981	401.93
702	Aug-1991	405.95	Apr-1940	401.71	Mar-1979	401.93
703	Nov-1978	405.92	Feb-1966	401.71	Aug-1961	401.92
704	Mar-1915	405.92	Sep-1918	401.70	Sep-1987	401.91
705	Apr-1977	405.91	Oct-1944	401.70	Jan-1949	401.91
706	Jan-1934	405.91	Jul-1966	401.69	Mar-1953	401.91
707	Oct-1948	405.91	Nov-1965	401.69	Jun-1949	401.91
708	Jul-1988	405.91	Jan-1912	401.69	Apr-1982	401.90
709	Sep-1961	405.90	Aug-1960	401.68	Jul-1988	401.89
710	Oct-1962	405.88	Jan-1922	401.67	Dec-1942	401.88
711	Nov-1932	405.87	Jun-1940	401.66	May-1989	401.88
712	Apr-1991	405.86	Nov-1935	401.66	Mar-1942	401.87
713	Nov-1942	405.86	Jun-1958	401.66	Sep-1962	401.87
714	Jul-1963	405.84	Jul-1941	401.66	Jul-1957	401.87

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715	May-1941	405.83	Sep-1932	401.66	Oct-1948	401.86
716	Aug-1957	405.82	Dec-1946	401.66	Nov-1942	401.86
717	Apr-1942	405.81	Mar-1917	401.64	Feb-1976	401.86
718	Feb-1947	405.80	Jan-1908	401.64	Nov-1941	401.85
719	Jan-1976	405.78	Oct-1913	401.64	Jan-1983	401.85
720	Oct-1987	405.78	Nov-1968	401.63	Jul-1970	401.85
721	Dec-1979	405.77	Oct-1910	401.63	Oct-1982	401.84
722	Nov-1937	405.77	Oct-1917	401.63	Jan-1948	401.83
723	Jun-1966	405.77	Feb-1950	401.63	Feb-1944	401.83
724	Feb-1932	405.76	Dec-1925	401.62	Sep-1950	401.82
725	Dec-1942	405.76	Dec-1974	401.62	Jan-1945	401.81
726	May-1957	405.76	Mar-1955	401.60	Jul-1977	401.80
727	Mar-1937	405.76	Aug-1967	401.60	Jul-1951	401.80
728	Jun-1977	405.76	Oct-1974	401.59	May-1990	401.80
729	Apr-1941	405.76	Jun-1955	401.59	May-1991	401.80
730	May-1977	405.74	Jan-1939	401.57	Aug-1981	401.80
731	Jun-1941	405.73	Jul-1976	401.57	Oct-1979	401.79
732	Oct-1981	405.73	Jun-1992	401.57	Feb-1988	401.78
733	Jan-1948	405.72	Dec-1912	401.56	Nov-1982	401.77
734	Jul-1934	405.71	Mar-1914	401.55	Jan-1944	401.77
735	Feb-1934	405.70	Jul-1980	401.55	May-1964	401.76
736	Feb-1945	405.69	Jun-1963	401.54	Feb-1983	401.76
737	Sep-1936	405.66	Jan-1909	401.54	Apr-1964	401.76
738	Oct-1931	405.66	Jan-1924	401.54	Feb-1948	401.76
739	May-1989	405.64	Sep-1946	401.53	Jan-1988	401.76
740	Jan-1919	405.64	May-1955	401.52	Sep-1978	401.74
741	Jan-1932	405.63	Jun-1931	401.52	Jul-1953	401.74
742	Sep-1949	405.62	Oct-1946	401.51	Jul-1980	401.72
743	Jan-1949	405.62	Dec-1979	401.51	Jan-1943	401.72
744	Feb-1988	405.61	Dec-1920	401.50	Aug-1949	401.70
745	Feb-1949	405.61	Apr-1989	401.49	Dec-1987	401.70
746	Apr-1992	405.60	Oct-1943	401.49	Jun-1990	401.70
747	Sep-1940	405.60	Apr-1958	401.49	Mar-1982	401.69
748	Jan-1988	405.60	Jul-1911	401.48	Apr-1989	401.69
749	Dec-1937	405.59	May-1940	401.47	Sep-1952	401.68
750	Feb-1919	405.58	Nov-1913	401.46	Aug-1970	401.68
751	Feb-1938	405.57	Oct-1947	401.46	Nov-1948	401.67
752	Dec-1987	405.57	Jan-1942	401.45	Oct-1987	401.67
753	Oct-1970	405.55	Aug-1961	401.44	Dec-1941	401.64
754	Aug-1980	405.55	Jan-1969	401.44	Sep-1976	401.64
755	Nov-1950	405.55	Sep-1930	401.44	Jun-1989	401.64
756	Sep-1990	405.55	Aug-1949	401.44	Nov-1979	401.64
757	Nov-1952	405.55	Aug-1965	401.43	Oct-1962	401.63
758	Nov-1969	405.54	Dec-1913	401.42	Feb-1951	401.62
759	Sep-1976	405.54	Oct-1935	401.40	Feb-1943	401.62
760	May-1992	405.53	Jan-1920	401.39	Aug-1963	401.61
761	Jan-1945	405.52	Sep-1958	401.39	Feb-1942	401.61
762	Feb-1976	405.52	Feb-1922	401.39	Sep-1961	401.60
763	Dec-1978	405.52	Jan-1913	401.38	May-1971	401.60
764	Nov-1931	405.51	Sep-1933	401.37	Apr-1991	401.59
765	Aug-1953	405.49	Oct-1962	401.37	Mar-1989	401.58
766	Apr-1966	405.47	Dec-1967	401.37	Aug-1951	401.58
767	Dec-1931	405.47	Aug-1957	401.36	Aug-1988	401.57
768	Feb-1948	405.47	Feb-1992	401.36	Oct-1950	401.56
769	Apr-1972	405.45	Jan-1926	401.34	Nov-1987	401.55

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770	Jul-1941	405.45	Dec-1935	401.33	Aug-1977	401.55
771	Mar-1942	405.44	Oct-1980	401.33	Jun-1964	401.54
772	Nov-1987	405.43	Nov-1914	401.32	Oct-1978	401.53
773	Feb-1951	405.43	Sep-1939	401.32	Jan-1942	401.53
774	Nov-1948	405.42	Dec-1965	401.31	May-1957	401.52
775	Sep-1991	405.41	Feb-1913	401.30	Aug-1957	401.52
776	Dec-1932	405.40	Dec-1916	401.30	Dec-1979	401.51
777	Oct-1961	405.39	Apr-1957	401.29	Dec-1948	401.50
778	Apr-1969	405.39	Apr-1964	401.27	Jun-1991	401.49
779	Jul-1977	405.39	Aug-1980	401.27	Sep-1981	401.48
780	Jan-1944	405.38	Aug-1911	401.25	Jun-1971	401.47
781	Nov-1981	405.38	Feb-1921	401.25	Nov-1978	401.45
782	Feb-1944	405.38	Jan-1933	401.24	Oct-1952	401.45
783	Aug-1971	405.37	Nov-1917	401.24	Aug-1980	401.44
784	Apr-1964	405.36	May-1931	401.23	Sep-1949	401.44
785	Apr-1971	405.35	Dec-1981	401.23	Jul-1990	401.44
786	Jan-1943	405.35	Feb-1920	401.23	May-1958	401.43
787	May-1964	405.35	Dec-1922	401.23	Sep-1970	401.43
788	Jan-1933	405.34	Oct-1950	401.22	Nov-1950	401.42
789	Aug-1934	405.33	Feb-1911	401.22	Oct-1976	401.42
790	Jan-1982	405.33	Sep-1911	401.21	Nov-1962	401.40
791	Nov-1962	405.31	Apr-1968	401.19	Jul-1989	401.39
792	Jul-1966	405.31	Nov-1931	401.18	Feb-1950	401.39
793	Jan-1980	405.31	Jul-1936	401.17	Sep-1951	401.39
794	Jun-1967	405.31	Nov-1967	401.14	Mar-1973	401.38
795	Aug-1988	405.29	Jan-1921	401.13	Aug-1953	401.37
796	May-1958	405.28	Oct-1954	401.12	Jul-1969	401.37
797	Jan-1938	405.28	Nov-1947	401.12	Jun-1969	401.37
798	Feb-1962	405.28	Apr-1931	401.11	Apr-1958	401.36
799	Jun-1989	405.27	Jan-1992	401.11	Apr-1992	401.34
800	Jan-1979	405.26	Jul-1931	401.10	Feb-1952	401.34
801	Jan-1937	405.26	Aug-1971	401.10	Jul-1991	401.34
802	Oct-1936	405.25	Jan-1962	401.08	Dec-1978	401.33
803	Aug-1963	405.25	Oct-1992	401.07	Jan-1980	401.33
804	Mar-1991	405.24	Nov-1943	401.05	Oct-1961	401.33
805	Apr-1989	405.24	Feb-1991	401.05	Apr-1972	401.33
806	Oct-1949	405.24	Feb-1976	401.05	Jul-1971	401.32
807	Jul-1967	405.24	Jan-1937	401.05	Nov-1952	401.32
808	May-1972	405.24	Jul-1959	401.04	Oct-1949	401.30
809	Jan-1962	405.22	Dec-1978	401.04	Jun-1958	401.30
810	Jun-1992	405.22	Apr-1934	401.04	Apr-1981	401.30
811	Mar-1992	405.22	Mar-1970	401.04	Jan-1979	401.28
812	Feb-1937	405.21	Oct-1930	401.03	May-1966	401.28
813	Oct-1940	405.20	Dec-1934	401.02	Feb-1980	401.28
814	May-1967	405.19	Sep-1948	401.01	Oct-1951	401.28
815	Apr-1961	405.19	Dec-1969	401.00	Oct-1981	401.28
816	Feb-1933	405.19	Nov-1944	401.00	Dec-1951	401.27
817	Oct-1990	405.18	Oct-1968	401.00	Jan-1950	401.27
818	Dec-1969	405.18	Feb-1918	400.99	Nov-1951	401.26
819	Sep-1957	405.18	May-1958	400.98	Jul-1964	401.26
820	Mar-1971	405.17	Jan-1960	400.97	Sep-1963	401.26
821	Nov-1970	405.17	Feb-1914	400.96	Dec-1962	401.25
822	May-1968	405.16	Feb-1919	400.96	Jan-1982	401.24
823	Mar-1989	405.16	Dec-1945	400.96	Sep-1988	401.24
824	Feb-1943	405.16	Jan-1949	400.96	Feb-1962	401.24

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825	Apr-1958	405.14	Oct-1933	400.95	Feb-1979	401.24
826	Jul-1958	405.13	Oct-1990	400.94	Jan-1952	401.23
827	Jun-1960	405.12	Sep-1991	400.93	Mar-1991	401.23
828	Feb-1980	405.12	Nov-1971	400.92	Jul-1958	401.23
829	Jun-1958	405.10	Jan-1971	400.91	Dec-1950	401.22
830	Jun-1964	405.10	Jan-1923	400.91	May-1992	401.22
831	Aug-1941	405.10	Nov-1918	400.91	Sep-1977	401.21
832	Feb-1982	405.08	Oct-1987	400.90	Feb-1982	401.20
833	Jul-1960	405.08	Dec-1918	400.89	Dec-1952	401.20
834	Nov-1961	405.08	Jan-1947	400.89	Oct-1970	401.20
835	Dec-1952	405.07	Feb-1935	400.87	Aug-1990	401.18
836	Dec-1981	405.06	Feb-1923	400.86	Jan-1951	401.18
837	Jun-1972	405.05	Feb-1951	400.85	Nov-1976	401.18
838	Apr-1981	405.04	Jan-1966	400.85	May-1967	401.16
839	Feb-1979	405.03	Jul-1940	400.85	May-1972	401.16
840	Sep-1980	405.02	Feb-1933	400.84	Sep-1980	401.15
841	Oct-1976	405.02	Jan-1917	400.83	Mar-1993	401.15
842	Jul-1992	405.02	Aug-1940	400.81	Sep-1957	401.15
843	Aug-1977	405.02	Feb-1968	400.80	May-1969	401.15
844	Nov-1936	405.01	Jun-1988	400.79	Jan-1962	401.15
845	Feb-1950	405.01	Jan-1976	400.77	Jun-1967	401.14
846	Dec-1950	405.00	Sep-1960	400.76	Apr-1961	401.13
847	Oct-1991	405.00	Jan-1980	400.76	Jan-1953	401.13
848	Dec-1948	404.99	Nov-1910	400.76	Nov-1961	401.13
849	Sep-1953	404.97	Dec-1968	400.76	Nov-1981	401.13
850	Nov-1940	404.95	Sep-1937	400.75	Mar-1992	401.12
851	Jul-1989	404.95	Jan-1935	400.74	Aug-1969	401.12
852	Dec-1961	404.93	Oct-1952	400.74	Apr-1971	401.12
853	Sep-1934	404.91	Dec-1947	400.73	Mar-1958	401.11
854	Dec-1962	404.91	Oct-1949	400.71	Jun-1966	401.11
855	Aug-1967	404.91	Sep-1949	400.71	Mar-1964	401.10
856	Dec-1970	404.91	Jan-1919	400.71	Feb-1953	401.10
857	Feb-1942	404.90	May-1977	400.68	Aug-1989	401.08
858	Aug-1966	404.89	Dec-1990	400.68	Nov-1949	401.07
859	Jul-1972	404.89	May-1989	400.68	Mar-1971	401.07
860	Jul-1964	404.86	Dec-1944	400.67	Aug-1991	401.06
861	Aug-1958	404.85	Jan-1991	400.66	Jan-1963	401.05
862	Jan-1950	404.84	Oct-1918	400.66	Sep-1953	401.05
863	Jan-1941	404.84	Sep-1989	400.66	May-1968	401.04
864	Dec-1940	404.84	Nov-1930	400.65	Dec-1949	401.02
865	Sep-1971	404.82	Mar-1931	400.65	Dec-1981	401.01
866	Dec-1990	404.81	Jan-1982	400.64	Aug-1971	401.00
867	Aug-1960	404.80	Oct-1932	400.64	Dec-1961	400.99
868	Dec-1936	404.80	Apr-1977	400.64	Nov-1970	400.98
869	Feb-1971	404.79	Nov-1987	400.63	Dec-1976	400.97
870	Nov-1990	404.79	Jun-1989	400.63	Oct-1988	400.96
871	Feb-1993	404.79	Feb-1960	400.63	Jun-1992	400.96
872	Jan-1953	404.77	Sep-1953	400.61	Oct-1963	400.96
873	Mar-1958	404.76	Mar-1934	400.60	Aug-1958	400.96
874	Jan-1951	404.76	Sep-1968	400.60	Oct-1977	400.96
875	Mar-1961	404.76	Aug-1976	400.60	Feb-1977	400.95
876	May-1959	404.73	Jan-1918	400.60	Apr-1969	400.94
877	Jan-1942	404.71	Feb-1945	400.59	Sep-1969	400.93
878	Aug-1992	404.70	Jan-1978	400.59	Aug-1964	400.93
879	Mar-1970	404.70	Jul-1955	400.58	Oct-1980	400.93

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880	Dec-1941	404.69	May-1934	400.58	Jan-1977	400.92
881	Jun-1968	404.68	Mar-1940	400.58	Jul-1967	400.92
882	Nov-1949	404.67	Jan-1914	400.57	Mar-1981	400.91
883	May-1960	404.66	Aug-1966	400.56	Nov-1988	400.90
884	Jul-1965	404.66	Nov-1950	400.56	Sep-1990	400.90
885	Sep-1988	404.65	Oct-1939	400.56	Jun-1972	400.89
886	Feb-1941	404.64	Mar-1977	400.56	Oct-1957	400.87
887	Sep-1941	404.64	Feb-1936	400.55	Apr-1966	400.87
888	Oct-1980	404.64	Dec-1932	400.55	Oct-1969	400.87
889	Jun-1965	404.63	Feb-1917	400.53	Sep-1991	400.83
890	Jan-1970	404.63	Oct-1931	400.53	May-1959	400.83
891	Nov-1941	404.63	Dec-1943	400.53	Apr-1967	400.83
892	Sep-1963	404.62	Jan-1936	400.52	Dec-1970	400.83
893	Oct-1957	404.62	Jul-1977	400.51	Feb-1963	400.83
894	Jan-1971	404.60	Oct-1936	400.51	Feb-1971	400.82
895	Feb-1953	404.59	Nov-1962	400.51	Oct-1953	400.82
896	Nov-1991	404.58	Nov-1936	400.50	Nov-1977	400.82
897	Mar-1941	404.58	Jul-1963	400.50	Dec-1990	400.82
898	Aug-1972	404.57	May-1956	400.50	Mar-1961	400.82
899	Oct-1941	404.56	Oct-1991	400.50	Jul-1966	400.81
900	Oct-1934	404.53	Feb-1947	400.49	Dec-1988	400.80
901	Oct-1953	404.52	Jun-1977	400.49	Jun-1968	400.80
902	Sep-1958	404.52	Jan-1972	400.47	Jul-1992	400.79
903	Sep-1977	404.51	Nov-1990	400.47	Jan-1989	400.77
904	Aug-1965	404.51	Feb-1980	400.46	Apr-1990	400.77
905	Jan-1991	404.49	Nov-1980	400.45	Oct-1990	400.77
906	Nov-1976	404.48	Nov-1959	400.45	Nov-1963	400.76
907	Dec-1949	404.48	Jul-1964	400.45	Sep-1971	400.76
908	Sep-1967	404.47	Dec-1917	400.43	Jan-1991	400.75
909	Jul-1968	404.46	Sep-1967	400.43	Sep-1989	400.75
910	Mar-1981	404.46	Mar-1941	400.42	Mar-1970	400.74
911	Apr-1967	404.45	Feb-1979	400.41	Oct-1991	400.73
912	Aug-1989	404.44	Feb-1941	400.38	Nov-1969	400.73
913	Jan-1963	404.43	Sep-1957	400.38	Mar-1972	400.73
914	Mar-1972	404.42	Sep-1940	400.37	Nov-1980	400.72
915	Jan-1993	404.39	Sep-1971	400.37	Dec-1977	400.69
916	Apr-1957	404.38	Nov-1952	400.37	Nov-1957	400.68
917	Aug-1964	404.37	Jun-1934	400.36	Feb-1989	400.68
918	Feb-1935	404.36	Nov-1932	400.34	Nov-1990	400.68
919	Jun-1959	404.36	Apr-1954	400.34	Jan-1971	400.67
920	May-1965	404.36	Jan-1948	400.33	Sep-1958	400.67
921	Jan-1935	404.36	Dec-1930	400.33	Dec-1969	400.67
922	Mar-1969	404.35	Mar-1960	400.33	Apr-1957	400.66
923	Sep-1992	404.32	Mar-1989	400.33	Feb-1991	400.65
924	Mar-1964	404.31	Sep-1936	400.32	Jul-1972	400.65
925	Nov-1934	404.31	Jan-1941	400.32	Nov-1991	400.65
926	Sep-1960	404.30	Jan-1979	400.31	Aug-1967	400.62
927	Sep-1966	404.30	Dec-1914	400.30	Apr-1965	400.61
928	Sep-1972	404.27	Nov-1939	400.28	Jan-1978	400.61
929	Oct-1971	404.27	Aug-1954	400.28	Nov-1953	400.61
930	Nov-1980	404.26	Nov-1991	400.27	Aug-1966	400.60
931	Oct-1977	404.25	Oct-1948	400.27	Sep-1964	400.60
932	Apr-1990	404.24	Nov-1933	400.26	Jul-1968	400.59
933	Feb-1970	404.24	Aug-1977	400.26	Aug-1992	400.58
934	Dec-1991	404.23	Feb-1931	400.25	Feb-1978	400.58

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935	Nov-1977	404.22	Sep-1931	400.24	Dec-1957	400.56
936	Feb-1991	404.22	Dec-1910	400.20	Dec-1991	400.55
937	Aug-1968	404.20	Mar-1968	400.20	Mar-1959	400.55
938	Feb-1973	404.19	Oct-1967	400.20	Jun-1959	400.54
939	Dec-1977	404.19	Jan-1915	400.18	Oct-1971	400.53
940	Apr-1965	404.18	Oct-1958	400.16	May-1954	400.53
941	Nov-1957	404.18	Jul-1934	400.15	Dec-1980	400.52
942	Dec-1972	404.17	Jan-1970	400.14	Jan-1970	400.51
943	Dec-1934	404.17	Jan-1945	400.13	Apr-1959	400.51
944	Jan-1973	404.15	Aug-1931	400.13	Dec-1963	400.50
945	Jan-1978	404.14	Feb-1948	400.13	Jan-1958	400.49
946	Sep-1965	404.12	Oct-1971	400.13	Dec-1971	400.48
947	Oct-1988	404.11	Feb-1953	400.12	May-1965	400.48
948	Oct-1958	404.10	Nov-1948	400.12	Feb-1993	400.47
949	Feb-1978	404.09	Oct-1937	400.12	Mar-1990	400.46
950	Oct-1963	404.09	Aug-1959	400.12	Apr-1954	400.46
951	May-1955	404.06	Feb-1944	400.11	Mar-1967	400.45
952	Dec-1976	404.06	Jan-1968	400.11	Feb-1992	400.45
953	Nov-1972	404.05	Nov-1934	400.10	Jan-1992	400.44
954	Oct-1967	404.04	Dec-1939	400.09	Oct-1989	400.44
955	Apr-1955	404.02	Aug-1941	400.07	Oct-1958	400.42
956	Nov-1953	404.02	Jan-1911	400.06	Aug-1972	400.42
957	Feb-1992	404.01	Oct-1960	400.05	Mar-1969	400.42
958	Jul-1959	404.01	Dec-1980	400.04	Jun-1960	400.41
959	Dec-1971	404.00	Jul-1988	400.04	May-1960	400.41
960	Mar-1968	403.99	Feb-1970	400.03	Feb-1970	400.40
961	Jan-1992	403.99	Mar-1981	400.03	Apr-1968	400.40
962	Apr-1968	403.99	Aug-1936	400.02	Dec-1953	400.39
963	Oct-1972	403.99	Dec-1952	400.02	Apr-1955	400.38
964	Dec-1992	403.98	Aug-1963	399.98	Feb-1964	400.38
965	Nov-1988	403.96	Sep-1976	399.98	Mar-1968	400.38
966	Feb-1977	403.95	Dec-1933	399.97	Feb-1981	400.38
967	Jan-1977	403.95	Nov-1949	399.96	Sep-1967	400.38
968	Oct-1992	403.94	Jan-1944	399.96	Jun-1954	400.38
969	Feb-1963	403.94	Nov-1960	399.95	Jan-1964	400.37
970	Jun-1955	403.93	Apr-1956	399.94	Aug-1968	400.37
971	Nov-1992	403.92	Feb-1978	399.92	Jun-1965	400.36
972	Apr-1959	403.90	Sep-1954	399.92	Nov-1971	400.36
973	Sep-1989	403.88	Dec-1949	399.91	Sep-1966	400.35
974	Mar-1959	403.88	Oct-1957	399.89	May-1955	400.35
975	Dec-1957	403.87	Feb-1959	399.89	Sep-1992	400.35
976	Jul-1955	403.87	Sep-1966	399.89	Jan-1981	400.34
977	Oct-1960	403.86	Feb-1972	399.88	Jan-1972	400.34
978	Dec-1980	403.86	Nov-1954	399.88	Nov-1958	400.33
979	Nov-1958	403.84	Feb-1934	399.87	Feb-1958	400.31
980	Mar-1966	403.84	Dec-1940	399.86	Jan-1993	400.31
981	Nov-1971	403.83	Jan-1931	399.86	Oct-1964	400.29
982	Apr-1960	403.83	Nov-1937	399.84	Jan-1954	400.28
983	May-1954	403.82	Oct-1934	399.81	Jul-1959	400.23
984	Sep-1968	403.82	Mar-1967	399.81	Sep-1972	400.22
985	Mar-1967	403.82	Jul-1989	399.79	Jun-1955	400.21
986	Sep-1964	403.81	Oct-1953	399.79	Jul-1960	400.21
987	Oct-1965	403.81	Nov-1958	399.77	Feb-1972	400.21
988	Dec-1967	403.78	Jan-1951	399.77	Oct-1967	400.21
989	Oct-1966	403.78	Jan-1940	399.76	Feb-1959	400.20

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990	Jun-1954	403.75		Jan-1953	399.74	Dec-1967	400.19
991	Dec-1988	403.75		Oct-1976	399.74	Mar-1965	400.19
992	Feb-1966	403.75		Dec-1950	399.74	Nov-1989	400.18
993	Aug-1959	403.71		Jan-1934	399.72	Jul-1965	400.17
994	Jan-1972	403.71		Dec-1936	399.71	Jan-1968	400.16
995	Nov-1963	403.71		Dec-1959	399.69	Apr-1960	400.16
996	Jan-1958	403.69		Nov-1957	399.67	Dec-1992	400.16
997	Jan-1989	403.68		Sep-1959	399.67	Oct-1966	400.15
998	Sep-1954	403.66		Aug-1988	399.65	Nov-1964	400.15
999	Jan-1968	403.65		Dec-1962	399.65	Nov-1992	400.15
1000	Nov-1967	403.65		Jan-1938	399.65	Mar-1954	400.14
1001	Apr-1954	403.61		Feb-1940	399.63	Oct-1992	400.14
1002	Mar-1990	403.61		Jun-1956	399.62	Feb-1954	400.13
1003	Nov-1965	403.59		Jul-1956	399.60	Dec-1958	400.12
1004	Feb-1981	403.56		Oct-1940	399.59	Sep-1968	400.11
1005	Dec-1965	403.55		Dec-1937	399.58	Jul-1955	400.11
1006	Jul-1954	403.54		Dec-1957	399.56	Mar-1955	400.10
1007	Feb-1989	403.54		Feb-1955	399.55	Jul-1954	400.09
1008	Dec-1953	403.53		Aug-1955	399.54	Feb-1968	400.09
1009	Jan-1981	403.52		Nov-1940	399.54	Jan-1959	400.09
1010	Feb-1969	403.51		Oct-1966	399.53	Nov-1967	400.07
1011	Mar-1955	403.49		Nov-1976	399.53	Dec-1972	400.06
1012	Feb-1972	403.48		Dec-1948	399.52	Jan-1973	400.04
1013	Feb-1968	403.47		Sep-1963	399.48	Aug-1959	400.04
1014	Nov-1960	403.44		Feb-1981	399.47	Sep-1954	400.04
1015	Aug-1955	403.44		Sep-1934	399.47	Nov-1972	400.03
1016	Jan-1969	403.43		Sep-1964	399.47	Oct-1972	400.03
1017	Mar-1960	403.43		Aug-1934	399.45	Aug-1960	400.03
1018	Oct-1989	403.42		Aug-1964	399.44	Nov-1966	400.02
1019	Oct-1968	403.39		Nov-1988	399.43	Feb-1990	400.02
1020	Nov-1966	403.38		Jan-1955	399.43	Feb-1973	400.02
1021	Aug-1954	403.36		Dec-1960	399.42	Aug-1965	400.01
1022	Dec-1958	403.34		Jan-1981	399.42	May-1956	400.00
1023	Feb-1958	403.33		Aug-1989	399.42	Feb-1969	399.99
1024	Nov-1968	403.33		Sep-1988	399.40	Jan-1967	399.99
1025	May-1956	403.32		Nov-1966	399.39	Feb-1967	399.98
1026	Oct-1954	403.29		Oct-1989	399.38	Dec-1964	399.96
1027	Oct-1964	403.29		Nov-1953	399.34	Dec-1989	399.96
1028	Jan-1954	403.28		Dec-1966	399.32	Mar-1960	399.95
1029	Feb-1959	403.27		Dec-1954	399.29	Jan-1965	399.94
1030	Jan-1966	403.27		Jan-1958	399.27	Dec-1966	399.94
1031	Sep-1959	403.24		Jan-1989	399.26	Jan-1969	399.94
1032	Mar-1965	403.24		Oct-1988	399.26	Mar-1966	399.93
1033	Dec-1963	403.24		Dec-1988	399.24	Oct-1954	399.89
1034	Dec-1968	403.20		Aug-1956	399.23	Aug-1954	399.88
1035	Jan-1959	403.17		Feb-1965	399.22	Oct-1968	399.88
1036	Dec-1960	403.17		Feb-1967	399.20	Aug-1955	399.86
1037	Jul-1956	403.16		Feb-1989	399.18	Nov-1968	399.85
1038	Dec-1966	403.10		Oct-1955	399.18	Jun-1956	399.85
1039	Jun-1956	403.10		Dec-1976	399.13	Sep-1960	399.85
1040	Jan-1967	403.10		Feb-1977	399.07	Feb-1965	399.84
1041	Feb-1964	403.05		Dec-1953	399.07	Jan-1990	399.84
1042	Nov-1964	403.03		Oct-1964	399.06	Sep-1965	399.83
1043	Jan-1964	403.02		Jan-1965	399.06	Jul-1956	399.82
1044	Nov-1989	403.01		Nov-1964	399.03	Feb-1966	399.82

			Ordered Monthly Highs			
ORDER	Month-Year	SWS No. 3		CORPS 70		BLSTFRN
1045	Feb-1967	403.01	Sep-1955	399.02	Sep-1959	399.81
1046	Mar-1954	402.99	Oct-1963	399.02	Dec-1968	399.80
1047	Sep-1955	402.96	Nov-1989	399.02	Oct-1960	399.76
1048	Feb-1954	402.96	Feb-1990	399.01	Feb-1955	399.74
1049	Nov-1954	402.96	Mar-1964	398.98	Nov-1954	399.73
1050	Oct-1959	402.88	Nov-1963	398.98	Nov-1960	399.68
1051	Feb-1955	402.87	Jan-1963	398.93	Feb-1961	399.68
1052	Feb-1961	402.86	Feb-1961	398.92	Dec-1960	399.66
1053	Nov-1959	402.80	Mar-1954	398.90	Oct-1965	399.65
1054	Feb-1990	402.78	Jan-1967	398.89	Apr-1956	399.65
1055	Jan-1961	402.76	Feb-1958	398.88	Oct-1959	399.64
1056	Dec-1954	402.76	Dec-1958	398.77	Dec-1954	399.64
1057	Aug-1956	402.71	Feb-1963	398.68	Sep-1955	399.63
1058	Dec-1959	402.71	Sep-1956	398.67	Nov-1959	399.60
1059	Dec-1964	402.68	Mar-1957	398.66	Jan-1955	399.57
1060	Jan-1965	402.66	Jan-1977	398.66	Dec-1959	399.56
1061	Dec-1989	402.64	Jan-1961	398.60	Dec-1965	399.56
1062	Apr-1956	402.63	Nov-1955	398.56	Aug-1956	399.56
1063	Oct-1955	402.62	Jan-1954	398.52	Jan-1961	399.55
1064	Jan-1955	402.60	Jan-1959	398.44	Nov-1965	399.53
1065	Jan-1960	402.58	Mar-1956	398.42	Mar-1957	399.53
1066	Feb-1960	402.55	Jan-1990	398.36	Jan-1960	399.51
1067	Feb-1965	402.51	Dec-1964	398.35	Jan-1966	399.49
1068	Mar-1957	402.45	Dec-1963	398.32	Feb-1960	399.48
1069	Jan-1990	402.45	Feb-1954	398.29	Oct-1955	399.48
1070	Nov-1955	402.29	Dec-1989	398.19	Nov-1955	399.35
1071	Sep-1956	402.23	Jan-1964	398.07	Mar-1956	399.34
1072	Mar-1956	402.08	Feb-1964	398.05	Sep-1956	399.28
1073	Feb-1956	401.90	Dec-1955	397.97	Feb-1956	399.21
1074	Dec-1955	401.86	Oct-1956	397.94	Dec-1955	399.15
1075	Oct-1956	401.78	Feb-1957	397.93	Feb-1957	399.06
1076	Feb-1957	401.66	Feb-1956	397.82	Oct-1956	399.03
1077	Jan-1956	401.56	Nov-1956	397.71	Jan-1956	399.00
1078	Nov-1956	401.49	Jan-1956	397.71	Nov-1956	398.89
1079	Dec-1956	401.43	Dec-1956	397.62	Dec-1956	398.89
1080	Jan-1957	401.18	Jan-1957	397.49	Jan-1957	398.78

APPENDIX F

Potentiometric Surface Maps (provisional) for 1985, 1990 and 1995.

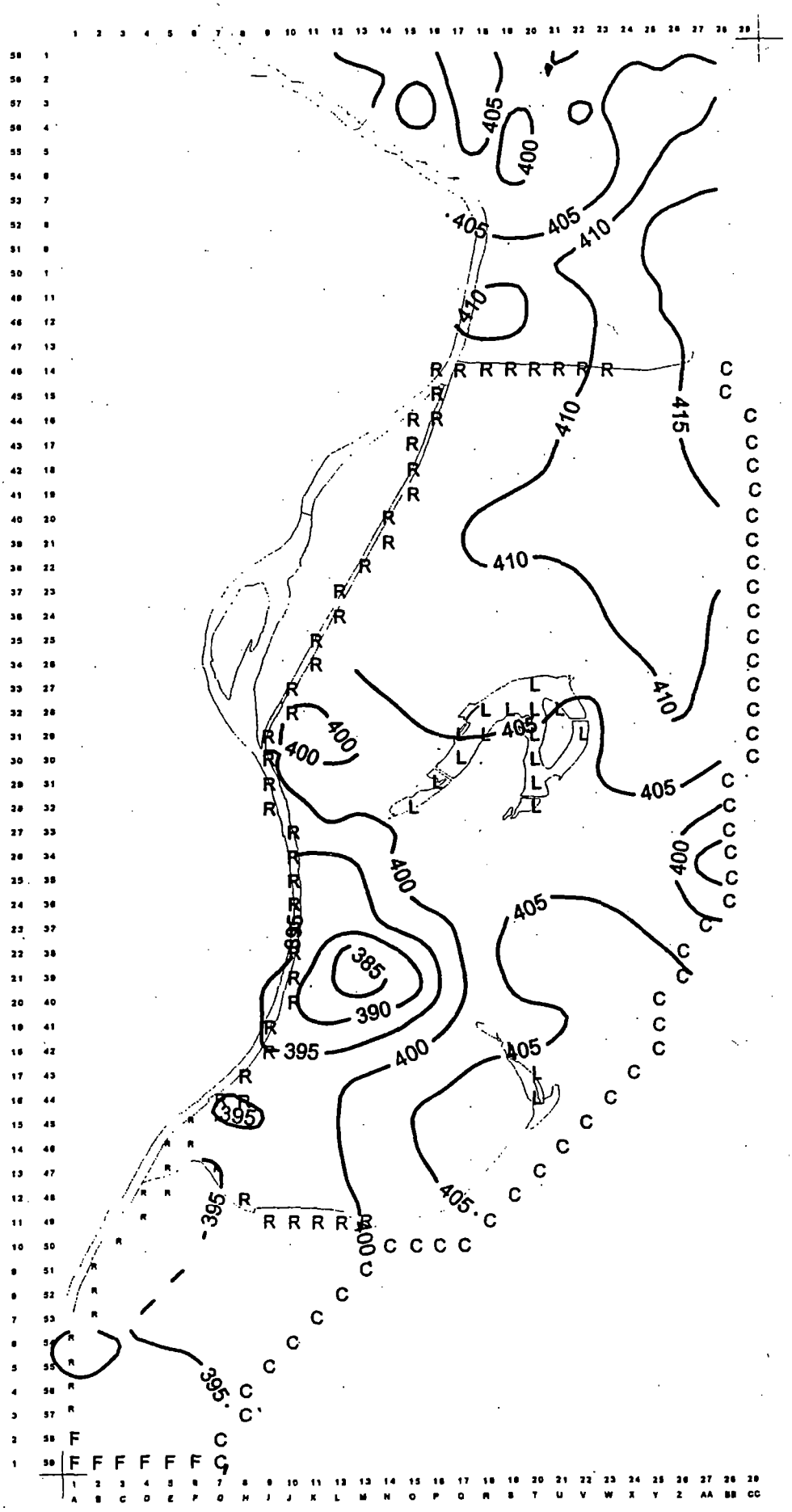

Figure F1. Approximate elevation of potentiometric surface, 1985, (provisional data from Hlinka, 1997).

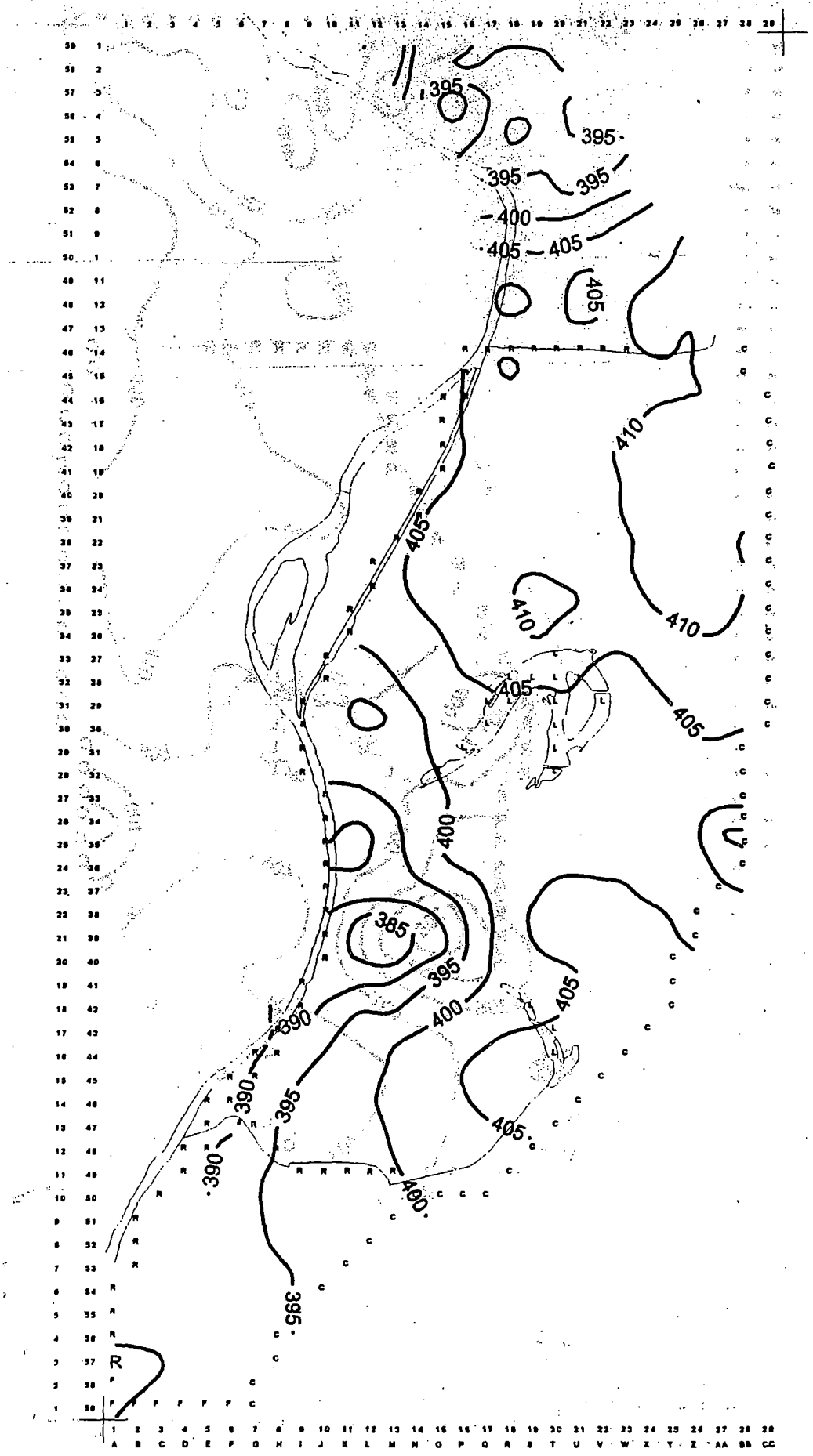

Figure F2. Approximate elevation of potentiometric surface, 1990, (provisional data from Hlinka, 1997).

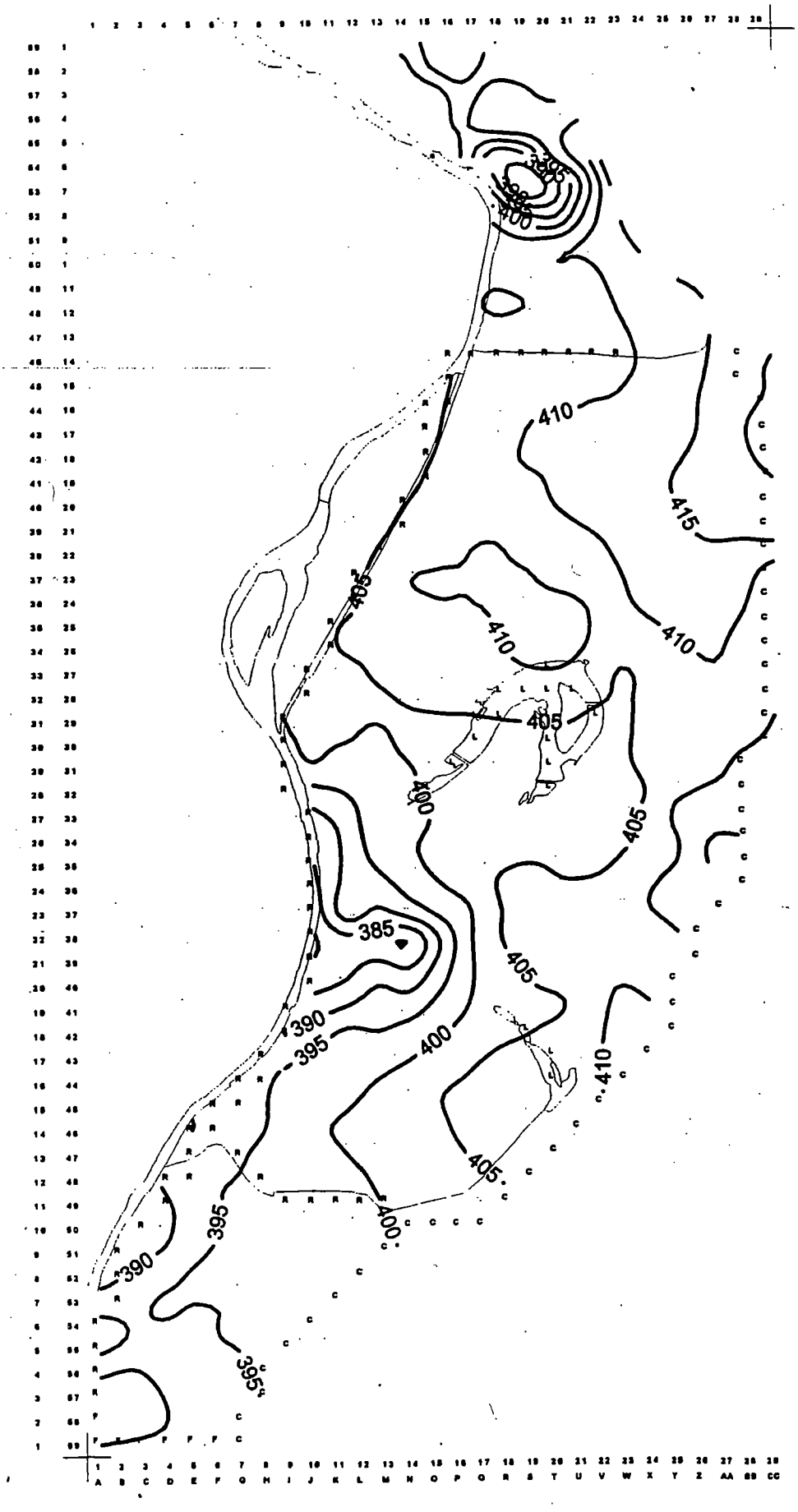

Figure F3. Approximate elevation of potentiometric surface, 1995, (provisional data from Hlinka, 1997).

DRAFT
GSI Job No. G-2561
March 31, 2002

GROUNDWATER
SERVICES, INC.

Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

ATTACHMENT 12

Ground-Water Management in the American Bottoms, Illinois

by Joseph D. Ritchey and Richard J. Schicht

The American Bottoms (Figure 1) is a 450-sq km (175-sq mi) area of the Mississippi River valley lowlands that includes the urban industrial areas of East St. Louis, Granite City and Alton. Ground water is a major source of water for the area and is used for industrial, public and irrigation supplies. Ground-water levels prior to industrial and urban development were near land surface. Intensive industrial development and construction of a system of drainage ditches, levees and canals to protect developed areas have altered the water resources in the area. According to Bruin and Smith (1953), the surface area of natural lakes between 1907 and 1950 was reduced by more than 40 percent. Development of ground water has caused problems in the past, including water-level declines in response to overdraft conditions. In recent years, water-level rises due to reductions in pumpage, high river stages and high precipitation producing favorable recharge conditions have caused damage to underground structures. Other related management problems include short well life, causing added maintenance expense, and ground-water contamination, causing added pretreatment and cleanup expense.

This paper describes the hydrogeology, past ground-water problems and their solutions, current and future ground-water activities, and ground-water management for the American Bottoms.

Hydrogeologic Setting

Ground-water supplies are obtained from permeable sand and gravel deposits in unconsolidated valley fill. The valley fill has an average thickness of 37m (120 ft) and is composed of recent alluvium and glacial valley-train material (Figures 2 and 3). The valley-train materials are predominantly medium-to-coarse sand and gravel, increasing in grain size with depth. The coarsest deposits most favorable for development are commonly encountered near bedrock and often average 9 to 12m (30 to 40 ft) in thickness. The valley fill is underlain by Mississippian and Pennsylvanian System bedrock consisting primarily of limestone and dolomite with some sandstone and shale. Because of the low permeability of

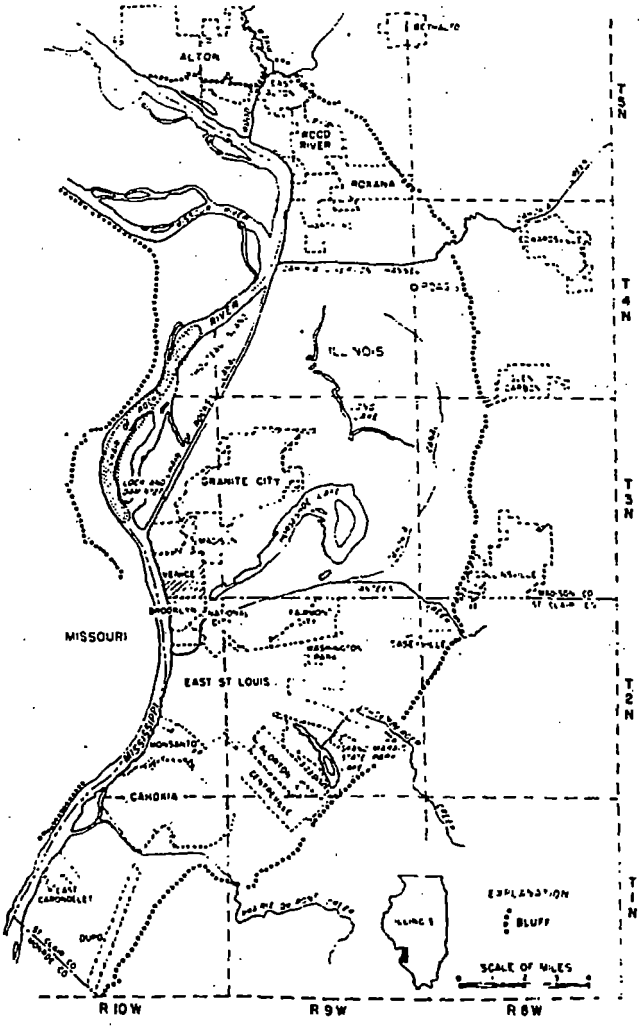

Figure 1. Location of the American Bottoms, showing developed areas

these formations and poor water quality with depth, the bedrock does not constitute an important aquifer in the area.

Ground water in the valley fill occurs under leaky artesian and water-table conditions. Leaky artesian conditions exist in most of the area because fine-grained alluvium in the upper deposits impedes the vertical movement of water. Water-table conditions prevail in the northeastern portion of the area, where the upper deposits consist of coarser alluvium, and at the major pumping centers where deep cones of depression have dewatered upper sediments.

Hydraulic properties of the aquifer were determined from approximately 10 aquifer tests and 100 specific-capacity tests conducted on industrial, municipal, irrigation and relief wells. Based on aquifer test data, the coefficient of storage ranged from 0.002 to 0.155 and the hydraulic conductivity ranged from 5×10^{-4}m/sec (1,060 gpd/ft^2) to 1.4×10^{-3}m/sec (2,900 gpd/ft^2).

Recharge occurs from four sources: precipitation, infiltration from the Mississippi River, subsurface flow from the uplands (bluffs) and inflow from the Mississippi River buried valley channel.

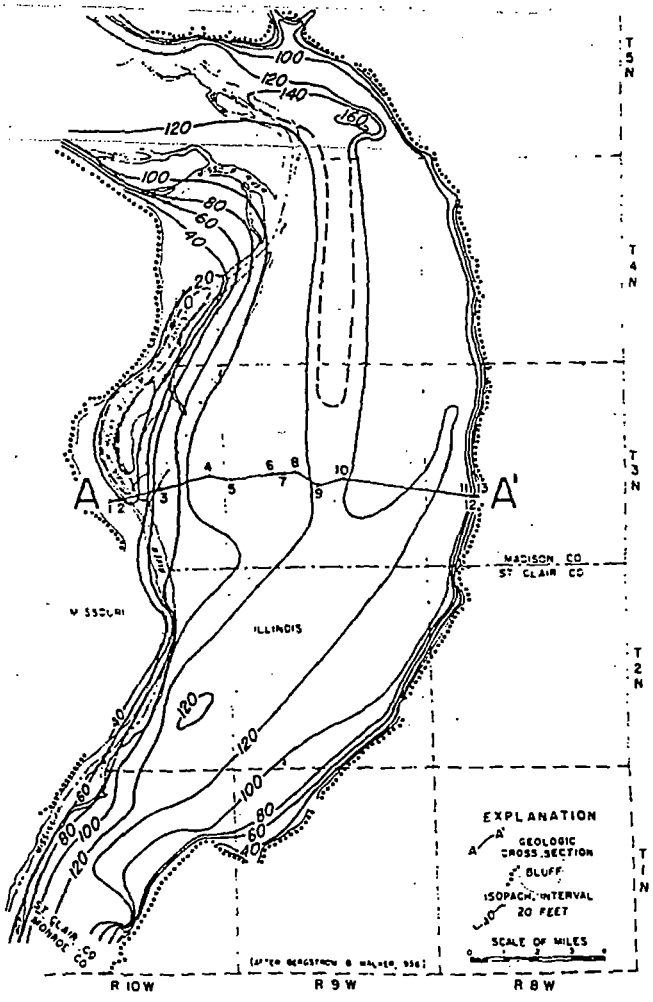

Figure 2. Thickness of the valley fill

Historical Perspective

Low Ground-Water Levels

The first significant withdrawal of ground water in the American Bottoms area started in the late 1890s (Figure 4). Pumpage increased from 0.044m^3/sec (1.0 mgd) in 1900 to 4.6m^3/sec (104 mgd) in 1956. Increased industrial output was mainly responsible for the tremendous growth in water withdrawal. Industries in the area include oil refineries, steel plants, meat packing plants and ore refining plants. As a result of increased withdrawals, ground-water levels by 1956 declined to critical stages in major pumping centers, particularly in the Granite City area. As shown in Figure 5, water levels in 1900 ranged from less than 122m (400 ft) to more than 128m (420 ft) near the bluffs. By 1956, water levels in major pumping centers declined to elevations ranging from 105m (345 ft) at Granite City to 119m (375 ft) at Wood River, as shown in Figure 6.

In addition to increased withdrawals, a severe drought from 1952 to 1956 affected ground-water levels. During this period, development of ground water for irrigation reached significant proportions.

The most serious decline in water levels occurred at Granite City, where the water levels dropped 17m (55 ft) to 105m (345 ft) above mean sea level. As a result, the

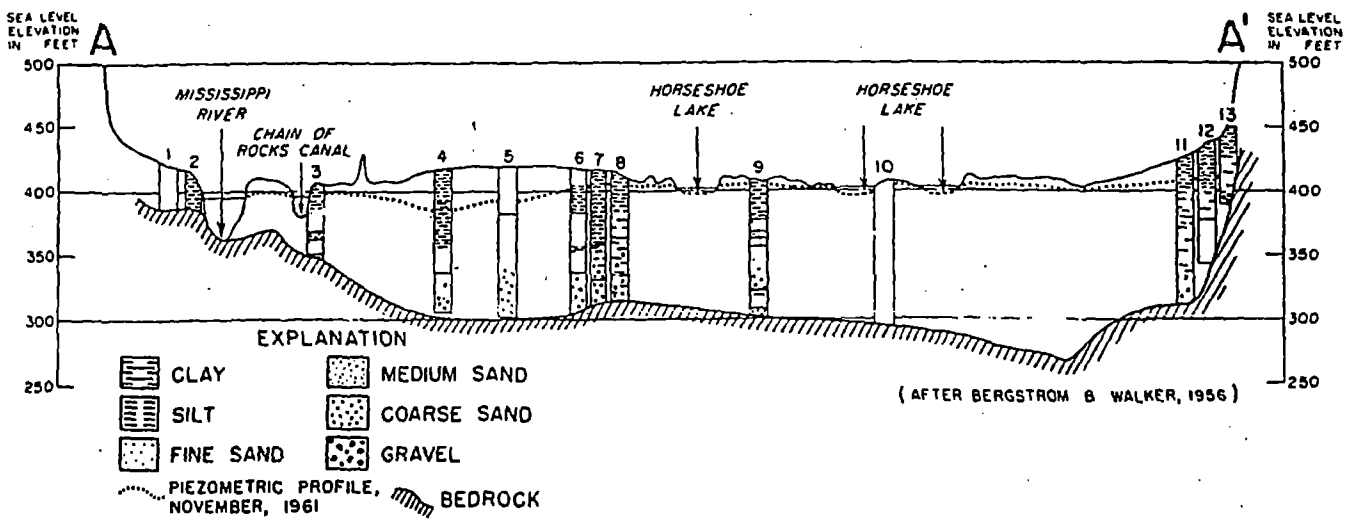

Figure 3. Geologic cross section and piezometric profile of the valley fill

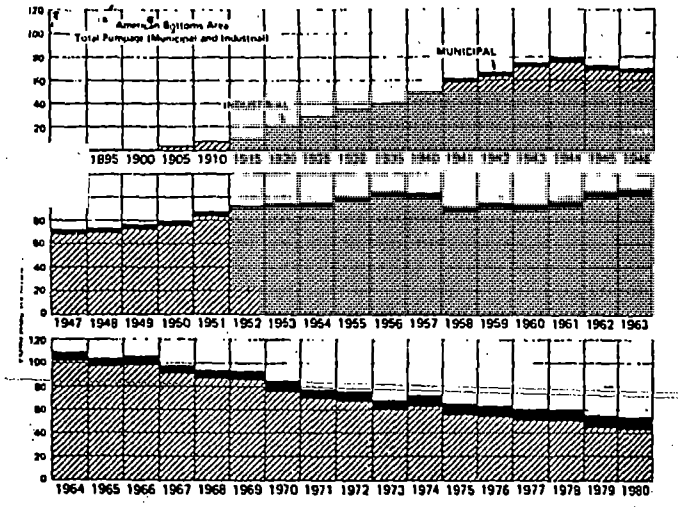

Figure 4. Total pumpage in the American Bottoms from 1890-1980 (Ritchey et al. 1982)

Granite City Steel Co., the major water user in the area, abandoned its wells in 1957 and began obtaining water from the Mississippi River.

Withdrawals in the Granite City area in 1956 were 1.3m³/sec (30 mgd). Schicht (1965) estimated the practical sustained yield of the pumping center in Granite City to be 0.66m³/sec (15 mgd). He also demonstrated that the American Bottoms area could sustain a yield of 8.2m³/sec (187 mgd), 3.1m³/sec (70 mgd) more than the maximum amount withdrawn in 1964. The selected scheme of development suggested by Schicht would locate well-fields so that the area of influence is extended over the area (Figure 7).

High Ground-Water Levels

Since 1970, high ground-water levels in many areas of the American Bottoms have caused damage to basements, sewers and other underground utilities and sections of roadway in recent years. These elevated levels are due to reduction in ground-water withdrawals in the area, high Mississippi River stages and above-normal precipitation. In 1973, for example, extensive damage occurred due to high ground-water levels because high Mississippi River stages occurred following a year of moderately high ground-water levels (Figure 8).

Controlling high ground-water levels is a primary objective of water resources management in the American Bottoms. Ongoing activities and a look to the future are addressed in the following sections.

Short Well Life

According to a study by Bruin and Smith (1953), the median service life of municipal wells terminating in sand and gravel formations in the American Bottoms is about half that for similar wells in other parts of Illinois. Nearly all the wells retired in the area were taken out of service either because the screens had become partially clogged or because the wells had filled with sand. Schicht (1965) made a study of mechanical analyses presented by Bergstrom and Walker (1956) which indicated the presence of a high percentage of fine material that under heavy pumpage could migrate

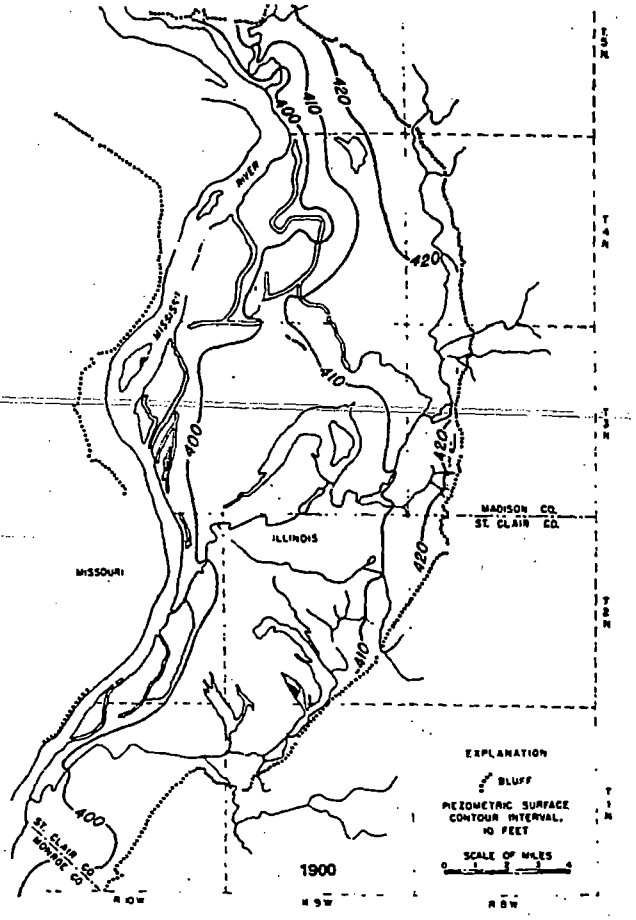

Figure 5. Elevation of the piezometric surface in 1900 (Schicht 1965)

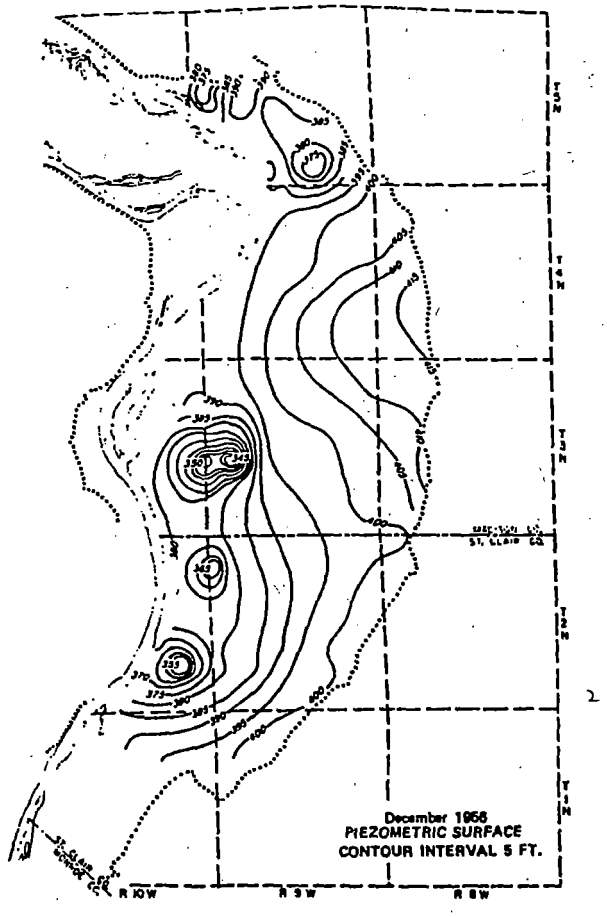

Figure 6. Elevation of the piezometric surface in December 1956 (Schicht 1965)

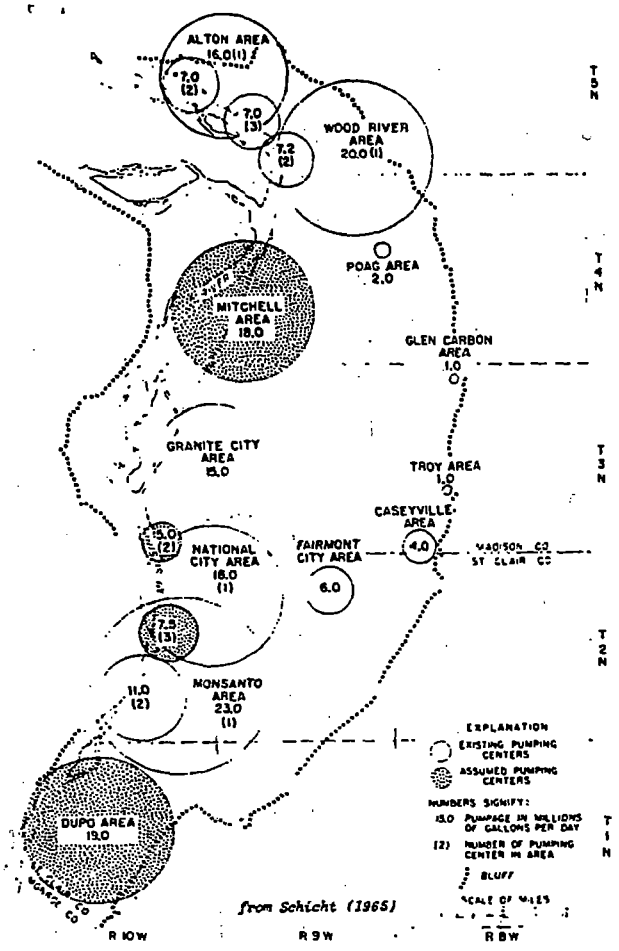

Figure 7. A selected scheme of development

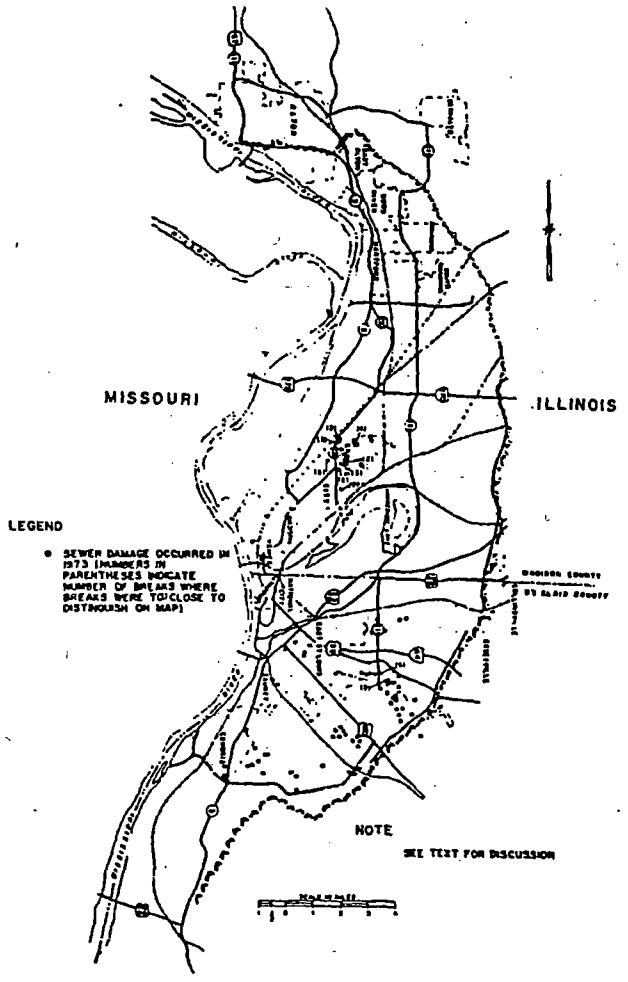

Figure 8. Location of sewer damage that occurred in 1976 (U.S. Army Corps of Engineers 1979)

toward a screen and partially clog the well screen openings. A recent study of large-volume water users in the American Bottoms (Planning and Management Consultants Ltd. 1982) revealed that sand in the well water was among the reasons given for well abandonment.

Though encrustation of well openings is not a significant problem, water quality has affected ground-water use. Studies of ground-water quality by Bruin and Smith (1953) and Schicht (1965) noted that water samples from wells in heavily pumped areas often have high sulfate and iron contents and a high degree of hardness. High iron content, generally poor water quality and high water treatment costs were also given as reasons for well abandonment (Planning and Management Consultants Ltd. 1982).

Ground-Water Contamination

Ground-water contamination in the American Bottoms is certain though its extent and seriousness have not been well defined. Permits to dispose of refuse have been issued for more than 75 sites in the American Bottoms by the Illinois EPA. In addition to ground-water contamination resulting from land disposal of wastes, long-term increases in chloride and sulfate content have occurred near pumping centers due to other factors (Schicht 1977). Industrial activity and the use of road salt was considered the probable major contributor to the increase in chloride content. Increase in the sulfate content also was attributed to industrial activity; however, an additional source may be the upward movement of high-sulfate water from the bedrock. Deposition of sulfate from the atmosphere, resulting partially from the burning of coal and oil and by the smelting of ores, was suggested to be a contributor to ground-water quality degradation, but has not been quantified.

Current Activities

A variety of operation, surveillance, research and legal activities are currently in progress in the American Bottoms. Activities in each area are highlighted below, based on available information.

Operation

The Illinois Department of Transportation (IDOT) operates and maintains a deep-well dewatering system at an interstate highway junction in National City (Figure 9). The juncture of interstate highways 55, 64 and 70 includes a section of roadbed 11m (35 ft) below the original land surface. The interchange was constructed in the early 1960s when water levels were near the level of the roadbed. As pumpage at nearby industries declined, water levels rose damaging pavement and making it necessary to construct a dewatering system. What began as a small system operating intermittently

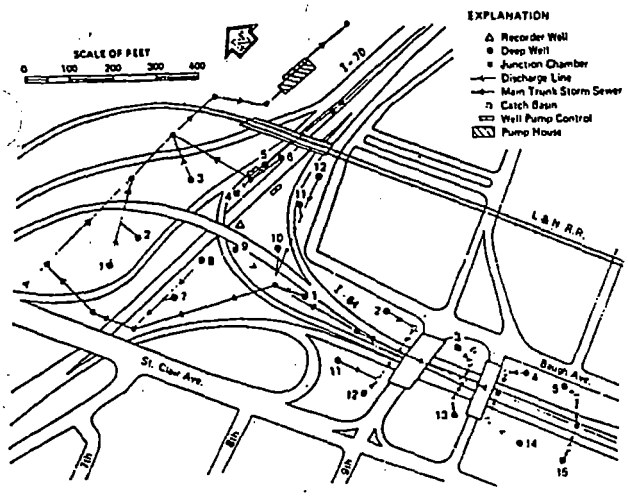

Figure 9a. Location of wells at the junction of I-70 and I-64 in East St. Louis, Illinois (Courtesy of Illinois Department of Transportation)

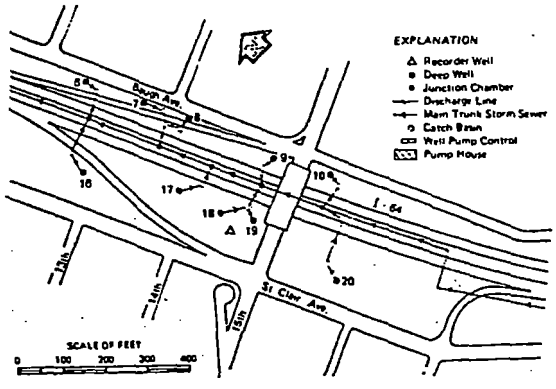

Figure 9b. Location of wells at the junction of I-64 and 15th Street in East St. Louis, Illinois (Courtesy of Illinois Department of Transportation)

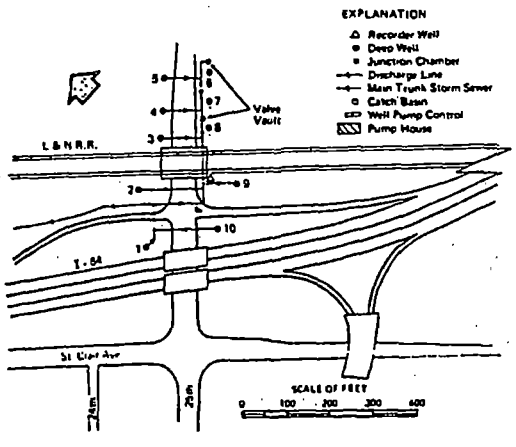

Figure 9c. Location of wells at the junction of I-64 and 25th Street in East St. Louis, Illinois (Courtesy of Illinois Department of Transportation)

presently includes 40 deep wells withdrawing an average of more than 3.1m³/sec (7 mgd) per year. The water is discharged to a drainage ditch that flows into the Mississippi River. The water is fairly high in iron content and therefore is not a marketable commodity in an area of abundant water supplies.

Currently, a second dewatering site is proposed by IDOT for a section of State Route 3, which passes beneath railroad tracks in Venice, Illinois. Six deep wells are proposed to remove approximately 0.04m³/sec (0.9 mgd) in order to prevent pavement damage caused by the heaving and settling of roadbeds resulting from freezing and thawing of ground water near the surface.

Beginning in 1966, three oil companies in the Hartford and Wood River areas received reports of odors and fires near their refineries. Results of investigations beginning in 1978 indicated that man-made and natural sources of combustible air/gas mixtures in the ground were present in and near Hartford (Mathes 1982). Some of the odors emanated from an underground accumulation of spilled petroleum products floating on top of the water table. The petroleum products, including gasoline, diesel fuel, light oil and jet fuel, were presumably lost from underground pipelines which traverse the Village of Hartford between the refineries and shipping terminals on the Mississippi River. The combined volume of the three pools of petroleum products illustrated in Figure 10 was estimated to be about 3,785m³ (1,000,000 gal). By November 1981 more than 2,130m³ (564,000 gal) of pure petroleum product had been recovered by installation of two shallow pumping wells and two deep wells (Figure 11). The information presented here is based on the work of the consulting engineering firm of John Mathes and Associates Inc. No information was available as to whether or not leaks in underground pipelines were repaired and whether all of the spilled petroleum would eventually be recovered.

Surveillance

The U.S. Army Corps of Engineers is currently sponsoring a ground-water quality surveillance project in the American Bottoms. The purpose of this project conducted by the USGS is to determine the quality of ground water that may be encountered in future dewatering projects under consideration. A wide variety of parameters are to be determined, including cations, anions, heavy metals and selected organics.

A second surveillance project being considered by the Corps of Engineers is to determine if any contamination has occurred at a small Army installation in Madison near the Chain of Rocks canal. Seven sites were identified that may contain various heavy metals, hydrocarbons and organics.

Research

The U.S. Army Corps of Engineers is currently funding a study related to high ground-water levels. The aim of this project, conducted by the Illinois State Water Survey, is to design and evaluate different systems to lower ground-water levels. Proposed systems include

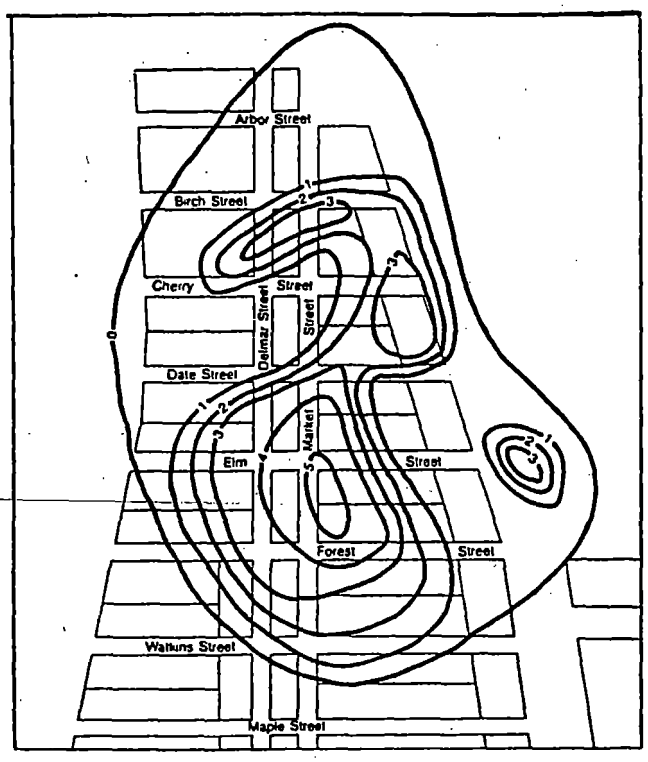

Figure 10. Thickness of petroleum products occurring on the top of the piezometric surface in Hartford, Illinois (Mathes 1982)

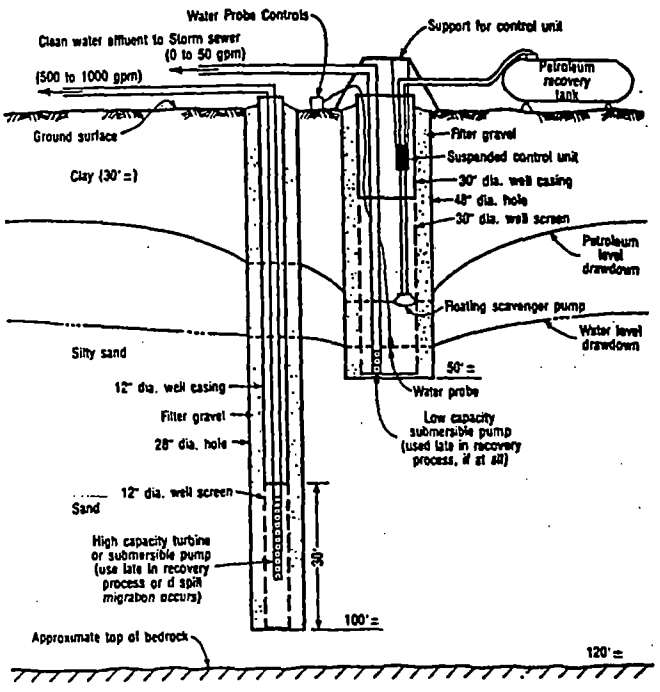

Figure 11. Diagram of the petroleum products recovery system (Mathes 1982)

extending the current drainage ditch system or installing high-capacity wells. Various configurations of each system will be evaluated with the aid of a digital computer model.

Legal

Legal action against companies operating in the American Bottoms has been considered. Ground-water contamination has been suspected at a number of industrial waste disposal sites, and the Illinois EPA and the Illinois attorney general's office are investigating a number of different cases.

Future Outlook

The American Bottoms area has experienced many problems over the past decade. Increased energy and labor costs as well as aging facilities and declining sales have adversely affected many industries in the area. The result has been a reduction in water use because of decreased demand and the initiation of conservation efforts.

Positive economic changes are not expected in the immediate future, and the long-term outlook is uncertain. Forecasts of ground-water use, based on correspondence with industry and municipal representatives, are presented in Figure 12. The Southwest Illinois Metropolitan Area Planning Commission, which forecasts population growth, expects population to grow slowly. Municipal water use of ground water is expected to increase at a fairly constant rate. Industrial water use of ground water is expected to remain constant for a number of years until major industrial growth can be re-established in the area. When this occurs, new industry with large water requirements may locate in the area with the potential of increasing total water use significantly.

Two major construction projects are being considered by the U.S. Army Corps of Engineers in the American Bottoms area, which may affect ground-water levels and quality. One is the construction of Lock and Dam 26 on the Mississippi River at Alton, Illinois. This multi-year, multi-million dollar project would be constructed approximately two miles downriver from the old lock and dam. Aspects of this project that could affect ground-water levels include the proposed dewatering during installation of pilings for the lock, and then, after completion of the lock and dam, the impact of the pool on sediment deposition and therefore on riverbed infiltration and on water levels in the riverbank adjacent to the new pool.

The second major construction project under consideration is a dewatering/drainage system to reduce or maintain ground-water levels in areas having a high risk of flooding. Two systems that have been considered are a deep-well system and a line-collector system (U.S. Army Corps of Engineers 1979). Figures 13 and 14 illustrate possible design and the resulting water-level decline contours after one year, as determined by a two-dimensional digital ground-water flow model. Other designs have also been simulated (Ritchey et al. 1982). The cost-effectiveness of each design will be determined as part of another project yet to be conducted.

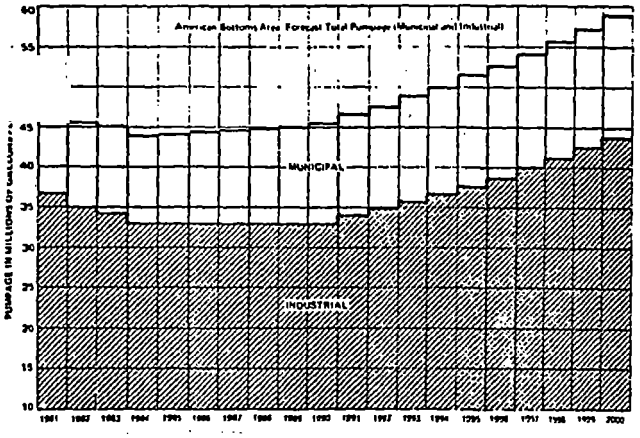

Figure 12. Forecast of total pumpage in the American Bottoms from 1981-2000 (Ritchey et al. 1982)

Ground-Water Management

Management of ground-water resources in the American Bottoms has consisted primarily of individual responses to specific problems as they arise. Planning, research and construction activities are undertaken by federal, state and local agencies depending upon the scope of the problem and the jurisdiction of agencies. Each problem is dealt with separately because no comprehensive water authority exists.

The St. Louis District of the U.S. Army Corps of Engineers has done most of the recent work in the American Bottoms. This group is authorized by Congress to focus its efforts on flood-related problems. Because many of the problems in the American Bottoms are either directly or indirectly caused by flooding, the Corps of Engineers has assumed responsibility for remedial and preventive efforts.

The Illinois EPA is responsible for protection of the quality of ground-water resources. The functions of this agency include supervision of public water supplies, issuance of permits and monitoring of waste disposal sites.

The Illinois Department of Transportation is responsible for the state's highways and shipping channels. Because of damage to roadbeds due to high ground-water levels, IDOT has responded by constructing dewatering systems to maintain low ground-water levels.

The Illinois State Water Survey of the Department of Energy and Natural Resources conducts scientific studies of ground-water resources, including those studies mentioned here: Schicht (1965), Schicht (1977), U.S. Army Corps of Engineers (1979), and Ritchey et al. (1982). Ground-water information collected and maintained by the survey includes data on water levels, water quality and water withdrawals.

The Southwestern Illinois Metropolitan Area Planning Commission (SIMAPC) is a regional agency encompassing all of the American Bottoms area as well as additional areas to the northeast and south. SIMAPC has the authority to assist local governments with plans, policies and proposals concerning flood control and drainage and to conduct research required for planning, including the collection of data on population trends.

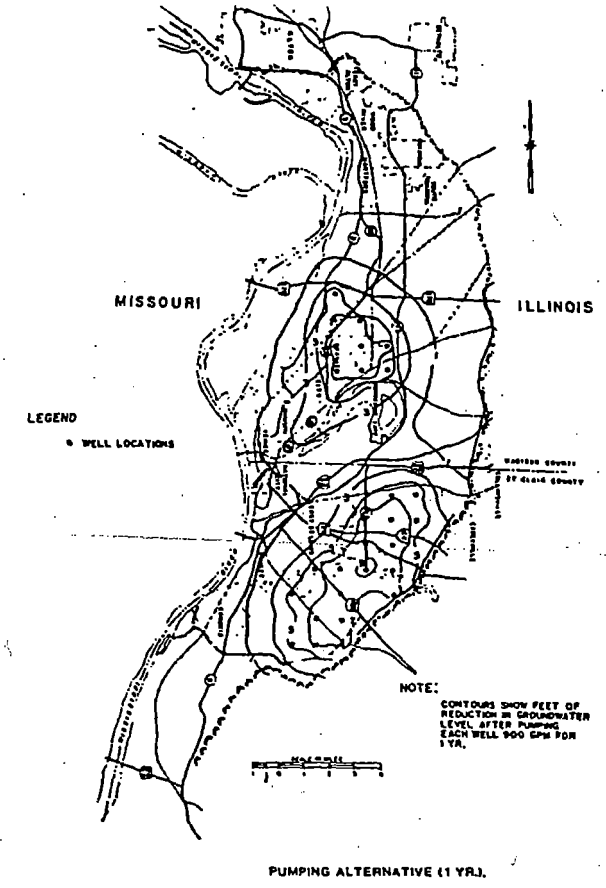

Figure 13. Locations of possible deep wells and the resulting water-level decline contours (U.S. Army Corps of Engineers 1979)

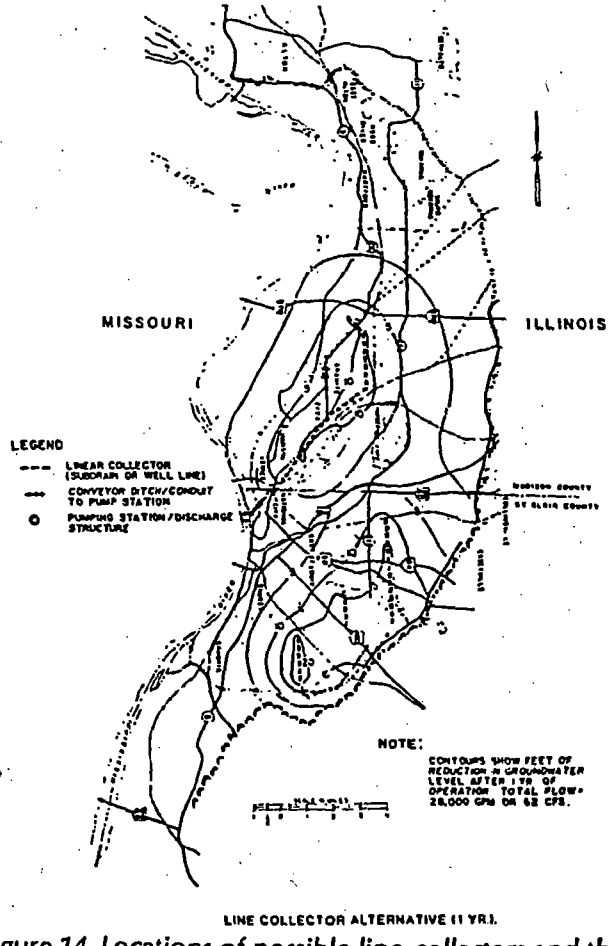

Figure 14. Locations of possible line-collectors and the resulting water-level decline contours (U.S. Army Corps of Engineers 1979)

Conclusions

Industrial growth in the American Bottoms area was in part caused by an abundant supply of ground water. Decline in the use of ground water by industry was caused by the combined effects of a deteriorating economic environment; poor design of many wells, resulting in premature failure; adverse changes in ground-water quality; and efforts to conserve water.

Ground-water use in the future may not exceed use in the past because of the poor water quality and the unfavorable conditions that must be overcome to attract industries that use larger quantities of water. Industrial revival by small ground-water users will not appreciably affect ground-water levels.

It may be possible to use ground water withdrawn from the dewatering systems for consumption rather than to discharge it to the Mississippi River. This may be the only way to obtain favorable benefit/cost ratios for such a system. However, this in effect would be subsidizing ground water, which may not be possible because it could be considered discriminatory against surface-water suppliers.

Protection of existing ground-water supplies, particularly public supplies, is essential. Public ground-water use has continued to increase despite the conditions affecting industrial water use.

The American Bottoms is a prime example of an area with regional ground-water problems that require a regional management approach to achieve solutions. Though many agencies have been involved in operations, surveillance, research, and legal activities, institutional and legal constraints have prohibited the formulation of a regional management authority. Until all those involved in ground-water related activities unify to support a regional ground-water management program in the American Bottoms and other areas experiencing similar difficulties, problems will remain difficult to solve.

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Biographical Sketches

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Richard J. Schicht has been assistant chief for research and services of the Illinois State Water Survey since 1980. He served as head of the hydrology section of the survey from 1977 to 1980. He was a ground-water hydrologist and engineer from 1957 to 1977 and he directed all water survey ground-water research studies from 1964 to 1977. He served as a meteorologist for the U.S. Navy from 1956 to 1957.

Schicht has authored more than 10 major research reports and 25 scientific papers. He is a registered professional engineer (by examination) in Illinois. As a member of the American Geophysical Union, he serves on the committee on ground water. He is a member of the Hydraulics Division Committee of the American Society of Civil Engineers and is presently chairman of the Hydraulics Review Committee on Ground-Water Hydrology. He is also a member of Sigma Xi, the American Water Works Association and the National Water Well Association.

Schicht obtained his bachelor of science degree in agricultural engineering from Purdue University. He attended the U.S. Naval Post Graduate School in Monterey, California, and graduated in 1955 with a certificate of meteorology. He then served as a meteorologist in the Navy for two years.

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ATTACHMENT 13

DEVELOPMENT OF A THREE-DIMENSIONAL GROUND-WATER FLOW MODEL FOR SAUGET SITE R, SAUGET, ILLINOIS

February 1993

Prepared for

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DEVELOPMENT OF A THREE-DIMENSIONAL GROUND-WATER FLOW MODEL FOR SAUGET SITE R, SAUGET, ILLINOIS

1.0 INTRODUCTION

Monsanto Company retained Geraghty & Miller, Inc. to construct a three-dimensional ground-water flow model for Sauget Site R and surrounding area in Sauget, Illinois. The purpose of the project was to develop a calibrated model to simulate ground-water flow at Sauget Site R, which is shown on Figure 1. Contained in this report is the documentation of the model construction and calibration. The model was calibrated successfully to low flow conditions representing base flow to the Mississippi River using water-level data measured in November 1988. This time period represented a prolonged period of base flow conditions in the Mississippi River. The model was further tested by calibrating to high river stage conditions which occurred in November 1985.

A ground-water model is a powerful tool for analyzing current ground-water flow conditions and for predicting the impacts of remedial actions on the ground-water system. Development of an accurate model requires the integration of all available data defining the flow system. The current Sauget model incorporates all ground-water data collected through August 1992, including results from the June 1992 aquifer test conducted at Site R.

The scope of the ground-water flow modeling analysis included three main tasks: (1) data review and organization, (2) conceptual model development, and (3) model calibration. The purpose of the ground-water flow analysis was to develop a calibrated steady-state, ground-water model suitable for predicting water levels over a wide range of future conditions and potential system stresses.

The data review phase of the ground-water flow analysis examined all data pertinent to the ground-water system. In general, four fundamental types of information are required

for modeling a particular hydrogeologic system: (1) geologic framework, (2) hydraulic properties, (3) water levels, and (4) fluid sources and sinks (pumping rates, recharge, etc.). The data review and organization phase resulted in development of a modeling database. This database facilitates the integration and analysis of data about the hydrogeologic system. The database forms the foundation of the conceptual model and provides the necessary information used during the model construction and calibration.

The Monsanto database includes geologic information from the extensive work Geraghty & Miller and other consultants have conducted at Sauget Site R and Monsanto's W.G. Krummrich plant in Sauget, Illinois. Water levels have been monitored biannually since 1984 and water-level recording instruments have provided continuous water-level data at nine locations in the area during that time period. Sources and sinks in the ground-water system include the Mississippi River, the Harding Ditch and associated tributaries, as well as the small lakes located to the north of the Harding Ditch.

The conceptual model, a succinct description of the important components of the ground-water system, was developed on the basis of the data review. The conceptual model formulates input data for the mathematical model by identifying initial values for hydraulic parameters. The conceptual model also guides calibration of the numerical model and aids in interpreting model results. The conceptual model of the ground-water flow system is presented in the next section.

After developing a conceptual understanding of the ground-water flow system, the numerical model was constructed. Model construction consisted of discretizing the flow system into rectangular blocks, assigning aquifer properties to each block, and estimating ground-water sources and sinks. Model data sets were constructed for the USGS Modular Three-Dimensional Flow Model, also know as MODFLOW (McDonald and Harbaugh 1988). MODFLOW is a well-accepted public domain software package for modeling three-dimensional ground-water flow.

Model calibration refers to the process of adjusting hydraulic parameters to obtain a reasonable match between water levels measured in the field and water levels calculated by the model. The Site R model was calibrated to water levels measured in November 1988 (base flow conditions) and to water levels measured during a flood event in November 1985. The 1988 calibration is termed a steady-state calibration and represents base flow conditions in the ground-water basin. The 1985 calibration was performed transiently to a short-term flood event where ground-water gradients were reversed in the vicinity of the Mississippi River and Sauget Site R. Quantitative or statistical comparisons were made between the site water-level data and model-computed heads for the steady-state calibration, while only a qualitative comparison was made for the transient calibration. The transient calibration was evaluated qualitatively because only one set of measurements was available for a large transient event. Thus, there was more uncertainty involved in the transient analysis than in the steady-state calibration.

2.0 CONCEPTUAL MODEL

2.1 GEOLOGY

Sauget Site R and the surrounding area included in the ground-water flow model are located in southwestern Illinois on the flood plain of the Mississippi River, named the American Bottoms (Figure 1). The flood plain is surrounded by bedrock bluffs on the eastern boundary of the model and across the Mississippi on the western boundary of the model. The flood plain contains unconsolidated deposits composed of recent alluvium (Cahokia Alluvium) which overlies glacial material (Henry Formation). Underlying the unconsolidated deposits is Mississippian and Pennsylvanian limestone and dolomite with lesser amounts of sandstone and shale. The average thickness of the unconsolidated material across the model area is approximately 130 ft.

To simplify the flow system and thus the model, the unconsolidated deposits were categorized into three hydrogeologic zones. They are as follows: the water-table (shallow)

zone (Layer 1), intermediate zone (Layer 2), and deep zone (Layer 3). The following discussion will be limited to these three zones. The bedrock is not included in the model because it is not an important aquifer due to low permeability. Although the water-table, intermediate, and deep zones have variable thicknesses, a uniform thickness and depth interval was assigned to each subsurface zone for the purpose of modeling. The thickness of each zone is provided later in this section. These zone distinctions are based on the differences in subsurface lithologic conditions. Wells used to guide the modeling effort are shown on the site location map (Figure 2). Delineation of the three zones and their relationships to the layers are shown on the generalized east-west cross section found on Figure 3. The cross section lies in the western portion of the model area, which has good geologic control due to extensive drilling by Geraghty & Miller and others at Sauget Site R and at the W.G. Krummrich plant. The geology is fairly uniform throughout the model area and, therefore, only an east-west cross section is necessary.

The water-table zone consists of the Cahokia Alluvium (recent deposits), which is an unconsolidated, fine-grained silty sand. For the purposes of the model, the layer is considered to be 30 ft thick, starting at the water table and continuing down to the medium sand deposits of the Henry Formation (bottom elevation of the layer is 365 ft msl). The cross section (Figure 3) shows Layer 1 to be a low permeability zone with fine-grained silty sand deposits predominating.

The intermediate zone is much coarser than the overlying water-table zone. This zone contains medium-grained sand representing the upper portion of the Henry Formation, a Wisconsinan glacial outwash in the form of valley-train deposits. Valley train deposits are long narrow bodies of outwash, deposited by meltwater streams far beyond the terminal moraine and confined within the walls of a valley. The Henry Formation is characterized by medium to coarse sand becoming coarser with depth. Thickness of model Layer 2 is 45 ft. This corresponds well to the range of thickness in the cross section (Figure 3).

Below the intermediate zone is the deep zone (Layer 3) which is marked by coarser deposits of the lower portion of the Henry Formation. In some areas, till and/or boulder zones were encountered 10 to 15 ft above the bedrock. The coarser deposits are delineated by the model to be 35 ft thick (Figure 3).

2.2 PREVIOUS NUMERICAL MODELING STUDY

A modeling study of the entire American Bottoms ground-water flow system was conducted by the Illinois State Water Survey Division (Ritchey et al. 1984). The purpose of this study was to conduct a detailed investigation of the flow systems in the area. Then current hydrologic data pertaining to the area were compiled, a computer model was developed to simulate the movement of the ground water, ground-water levels in the area were analyzed, and future ground-water levels were predicted. Documentation of the model, including a user guide, was also included.

The compilation of hydrogeologic information included the distribution of pumpage in the area including the major and minor pumping centers and pumpage from wells adjacent to the Mississippi River. A series of hydrographs from the years 1940 to 1981 were plotted and included in the report.

The ground-water model used was a modified form of the Illinois State Water Survey aquifer model (Prickett and Lonnquist 1971). Modifications were made to incorporate river stage and precipitation. The model was calibrated by history matching two 5-year periods with constant 1-month time steps. Hydrographs of actual and simulated water levels of ten observation wells and the nearest model well for the two 5-year periods were presented. The model was found to consistently calculate water levels within 2 ft of the actual measured water level within a specified area of interest.

Ground-water levels were evaluated with the aid of ground-water level exceedance plots. Ground-water level exceedance probability plots were constructed for ten model wells

by compiling the maximum yearly water levels from monthly simulated values. Plots were based on simulation of the 30-year period from 1951-1980. Mississippi River stage was also simulated during the 30-year period from 1951-1980.

2.3 HYDRAULIC PROPERTIES

In 1986, Geraghty & Miller compiled hydraulic properties that were determined from aquifer tests and slug tests run by Geraghty & Miller and other consultants (Geraghty & Miller, 1986b). These data are listed in Table 1. In general, the hydraulic conductivities of the intermediate and deep zones are much greater than that of the shallow water-table zone.

A detailed aquifer test was conducted by Geraghty & Miller in June 1992. The results from this test indicate that the intermediate and deep zones have approximately equal permeability with an average of 315 ft/d (Table 2). The storage coefficient was calculated to be 0.007. The overall transmissivity of the combined intermediate and deep zones was found to be about 30,000 ft^2/d which was used in the model, and 15,000 ft^2/d was applied to each zone. The construction of the model is described in Section 3.0.

2.4 RECHARGE

Average annual rainfall in the Sauget area is approximately 34 inches. Based on a 30-year average (1951 to 1980) for precipitation in the Sauget area, 13 inches of precipitation are estimated to infiltrate into the ground as recharge to the aquifer system. The calibrated steady-state model represents base flow conditions, so a lower value of recharge was used (about 9 inches/year).

3.0 GROUND-WATER FLOW MODEL CONSTRUCTION

3.1 CODE SELECTION

Ground-water flow in the Sauget area was modeled with the USGS Modular Three-Dimensional Finite-Difference Ground-Water Flow Model (McDonald and Harbaugh 1988), also known as MODFLOW. The three-dimensional capabilities of this code are appropriate for the proper treatment of the vertically variable hydrostratigraphy (three distinct aquifer zones) and boundary conditions at the study site. MODFLOW is also well documented, publicly available, and generally accepted within the scientific community.

Prior to the simulation of ground-water flow at Sauget Site R and vicinity using MODFLOW, the model was calibrated using an automatic (inverse) parameter estimation algorithm incorporated into the MODFLOW code by Duffield (1988). The inverse algorithm systematically selects a set of user-specified hydraulic parameter values that provide a least-squares match between observed and calculated water levels. Hydraulic parameters estimated in the Sauget model include: (1) hydraulic conductivity in the water-table zone (Layer 1), (2) vertical leakance across the water-table/intermediate and intermediate/deep boundaries, (3) vertical leakance of the Mississippi River bottom sediments, and (4) precipitation recharge. The transmissivity of the intermediate and deep zones (Layers 2 and 3, respectively) was maintained at the value estimated from the June 1992 aquifer test and was not changed during calibration.

3.2 MODEL CONFIGURATION

3.2.1 Discretization

The Monsanto model includes Sauget Site R, the entire W.G. Krummrich facility, and a large amount of the surrounding area, as shown on Figure 4. The model grid covers 58 square miles around the Sauget area with an east-west dimension of 44,000 ft and a north-

south dimension of 37,000 ft. The model is much larger than the area of interest to incorporate regional ground-water flow effects at the site scale. The model extends to the bedrock bluffs east and west of the site (across the Mississippi River) and to Old Prairie Dupont Creek south of the site. The northern boundary of the model coincides with the center of a pumping cone of depression caused by dewatering efforts near the Poplar Street Bridge.

In the finite-difference modeling technique used in MODFLOW, the aquifer is divided into rectangular regions known as cells. The maximum cell dimension in the Sauget model is 1,000 ft. These large cells were placed away from the areas of interest. Finer grid spacings were used near Sauget Site R and the W.G. Krummrich Plant. The smallest cells measure 250 ft on a side. A portion of the finite-difference grid covering Site R and the Krummrich Plant is shown on Figure 5. This figure is provided to illustrate the finer detail used to model these areas.

The model contains three layers representing the Cahokia Alluvium (Layer 1) and Henry Formation (Layers 2 and 3). The upper model layer is unconfined and the lower two layers are semiconfined, although there are no continuous aquitards separating any of the model layers. The flow of ground water between model layers is represented in the model using a leakance term. The leakance term incorporates the lower vertical permeability characteristic of most glaciofluvial deposits to retard the movement of ground water between the three aquifer zones.

3.2.2 Boundary Conditions

To represent the variety of physical boundaries to the aquifer system in the Sauget area, several types of boundary conditions were prescribed in the ground-water flow model. A boundary condition is a numerical representation of a physical boundary or process effecting the aquifer system. These physical boundaries and processes include: (1) surface-water bodies and streams (Mississippi River and the lakes northeast of the site), (2)

production wells in the Sauget area, (3) the vertical and lateral limit of the unconsolidated aquifer system, and (4) precipitation recharge.

Two primary types of numerical boundary conditions were used in the Monsanto model to represent these physical boundaries to the system. The model boundary conditions are termed constant head and flux boundaries. A third type of boundary condition, called a head-dependent flux boundary condition, was not employed in this model. The latter may be used to represent drains, for example, but there are no such features in the area.

For the purposes of calibration, constant head boundaries in the upper model layer were used to represent all surface water features including the Mississippi River, the Harding Ditch, and other small streams. In a constant head boundary cell, the ground-water level is fixed at a specified point for the duration of the simulation. This provides a continuous source or sink for ground water in the surrounding aquifer. The water-level value specified in a constant head which represents a surface stream is equal to the water elevation on the stream. A river stage of 381 ft msl was estimated for the Mississippi River from the gauging station at St. Louis, Market Street (Mile 179.6). Elevations for the remaining surface-water bodies were estimated from USGS topographic maps of the area.

A constant flux boundary condition represents a continuous and constant inflow or outflow of water within a model cell. Rather than specifying a constant water elevation, a constant discharge or recharge rate is used. Constant flux boundary conditions typically represent wells, recharge, or areas of no ground-water flow (the flux is zero). The latter are termed no-flow boundaries. Boundary conditions in Layer 1 are shown on Figure 5. The outer edge of cells on Figure 5 are assumed to be no-flow boundaries, except where specified as another type of boundary condition.

Constant flux boundary conditions were used in the model to represent: (1) recharge from precipitation, (2) production wells north of the site, and (3) the limit of the unconsolidated deposits (no-flow boundaries). A special form of no-flow boundary was used

to represent the northern boundary of the model in all layers and the southern boundary in Layers 2 and 3. These boundaries were selected so as to be parallel with the regional ground-water flow directions. In this manner, the ground-water flow lines (or stream tubes) represent the model boundary. In theory, ground water does not flow across stream lines, and thus a no-flow boundary is formed.

The northern boundary was also selected to bisect the cone of depression surrounding a production center. This boundary takes advantage of the symmetry of the cone of depression and uses streamlines entering the production zone from the east and west. Thus, only half of the cone of depression around the pumping center is simulated. Consequently, only half of the pumping rate for these wells was used in the model.

The remaining no-flow boundaries included the eastern and western boundaries in all layers and the base of the model (bedrock surface). The eastern and western boundaries represent the bedrock bluffs as shown on Figure 4. It is assumed that the volume of water entering or leaving the unconsolidated aquifer system from the bedrock is insignificant compared to the volume of water entering from precipitation and induced leakage from the river.

Three discrete zones of recharge were defined in the model in Layer 1. The primary recharge zone covers most of the model and received 8.8 inches per year (in/yr)(2.0×10^{-3} ft/day). A second recharge zone was used to simulate an anomalous ground-water mound in the W.G. Krummrich Plant. This zone received 370 in/yr (8.4×10^{-2} ft/day). The third recharge zone represents the clay cap which was installed over the landfill at Site R, adjacent to the Mississippi River. Recharge zone No. 3 received 2.63 in/yr (6.00×10^{-4} ft/day). The location of these zones is shown on Figure 6.

The model recharge rate of 8.8 in/yr is below the average of 13 in/yr, as discussed in Section 2.5. The lower rate was used to simulate base flow conditions during dry periods

of the year. The model was calibrated to the period around November 1988 when the Mississippi River was at a relatively constant but low stage.

3.3 AQUIFER PARAMETERS

Aquifer parameters such as hydraulic conductivity, transmissivity, and vertical leakance were assigned to each cell in the model using the concept of parameter zonation. This philosophy of modeling specifies several discrete values of each parameter which are assigned to groups (zones) of cells. Aquifer properties defined in the model include: (1) hydraulic conductivity of Layer 1, (2) transmissivity of Layers 2 and 3, (3) vertical leakance between model layers, and (4) bottom elevation for Layer 1.

Layer 1 was divided into three separate hydraulic conductivity zones having values of 6.2, 1.0, and 0.4 ft per day (ft/day). These values were determined during the calibration process. These hydraulic conductivity zones are referred to as Zones 1, 4, and 5, respectively. Zone 1 represents the hydraulic conductivity of the Cahokia Alluvium, Zone 4 represents the hydraulic conductivity of the landfill, and Zone 5 represents the bottom sediments of the Mississippi River. The distribution of hydraulic conductivity zones in Layer 1 is shown on Figure 7.

The calibrated values of hydraulic conductivity in Layer 1 for Zones 1, 4, and 5 fall within the range of field measurements for the Cahokia Alluvium (0.25 to 17.01 ft/day). Hydraulic conductivity Zone 1 (6.2 ft/day) represents average conditions across Layer 1, the low permeability layer. The value given by the model is very close to the average field hydraulic conductivity (4.42 ft/day) which is the range of silty sand. Hydraulic conductivity Zone 4 (1.0 ft/day) represents the filled portion of Sauget Site R which is estimated to be less permeable than the surrounding area due to the reworked nature of the landfill material. Zone 5 has a hydraulic conductivity of 0.4 which represents the Mississippi River bottom sediments, which are finer grained than the Cahokia Alluvium.

The transmissivity of the Henry Formation was defined in two zones (2 and 3 in the model). Zone 2 represents Layer 2 which has a transmissivity of 15,000 ft^2/day. Zone 3 represents Layer 3 which also has a transmissivity of 15,000 ft^2/day. Both Zones 2 and 3 are part of the glacial Henry Formation. The transmissivity values used in layers 2 and 3 were derived from the results of the June 1992 aquifer test. These values were not refined during calibration.

The leakage of water between the three layers was treated using a leakance term. The leakance term was calculated using the vertical permeabilities and the thickness of the layers. Five leakance zones were determined during the calibration. The vertical leakances between Layers 1 and 2 are 0.0063 day^{-1} (Zone 1, Water-table Layer/Intermediate Layer), 1.0×10^{-9} day^{-1} (Zone 3, water-table layer/intermediate layer in the landfill area), and 0.42 day^{-1} (Zone 4, Mississippi River/Intermediate Layer). Figure 8 depicts the vertical leakance zones in Layer 1. The leakance between Layer 2 and 3 has a value of 1.00 day^{-1} (Zone 2, Intermediate Layer/Deep Layer). The leakance for Zone 5 in the intermediate layer/deep layer (Mississippi River) is 0.25 day^{-1}.

A summary of the hydraulic parameter zones and their model calibrated values are shown in Table 3, which also includes the recharge values discussed in the previous section. All values were estimated using the automatic calibration procedure which is described in the next section.

4.0 STEADY-STATE MODEL CALIBRATION

4.1 CALIBRATION TECHNIQUE

A ground-water model is calibrated by adjusting aquifer properties (hydraulic conductivity, transmissivity, and vertical leakance) and boundary conditions within reasonable limits to obtain an acceptable match between observed and calculated ground-water levels. The reasonable limits within which parameters may be varied is determined by field testing

and by values reported in the scientific literature. Many single-well aquifer tests and slug tests were used to set reasonable limits for hydraulic conductivity in the Sauget area. An acceptable match between water levels measured in the field and those calculated by the model is determined through graphical and statistical analysis of residuals. A residual is the difference between observed water levels (field measurements) and water levels calculated by the model.

The model was calibrated using a nonlinear least-squares technique known as the Marquardt Algorithm (Marquardt 1963). This technique is often referred to as "automatic calibration" or inverse modeling. Inverse techniques determine optimum aquifer parameter values for a given model configuration (grid spacing and boundary conditions) which provide the best statistical calibration. The calibration for the model was arrived at through an iterative procedure involving inverse model runs and subsequent redefinition of aquifer parameter zones and boundary conditions. Parameter values for the final calibrated model were described in the previous section.

Two types of calibrations were performed on the Sauget model. The first step consisted of calibrating the model to base flow (steady-state) conditions in the Mississippi River. The steady-state calibration was performed by comparing model-calculated water levels to those measured in the field during November 1988. This period represents a prolonged base flow period. The second calibration compared model calculations to a flood event in the Mississippi River in November 1985. The latter was a transient calibration which is discussed in Section 5.0.

4.2 CALIBRATION TARGETS

A critical component of any model calibration is a set of measured ground-water levels to compare with model calculations. These observed or measured ground-water levels are known as calibration targets. The goal in selecting calibration targets is to define a set of targets that are reliable and well distributed throughout the area of the model.

Calibration targets were selected for the model using a three-step procedure. In the first step, the November and December water levels for the years 1984 through 1988 were compiled to chose the year that would most closely represent steady state ground-water flow conditions in the area. The months of November and December were chosen because they are typically closest to base flow conditions in the Mississippi River. The standard deviation for each of the wells was also computed to assess the variability in water level measurements. Water levels from 1988 were chosen during the first phase of target selection because of prolonged base flow conditions in the Mississippi River which imparted a low standard deviation in water level measurements.

During the second phase of target selection, wells with a 1988 reading and a low standard deviation were included in the list of targets. Wells exhibiting a large standard deviation (> 3 ft) were not included in the list of targets; however, it was necessary to choose some wells near the Mississippi River with a high standard deviation. The high standard deviation is due to the extreme fluctuation in water levels near the Mississippi River because of the river's variation over time. Most of the standard deviations away from the river were less than 3 ft and near the river the deviation was approximately 6 ft. The 1988 readings were chosen because these measurements were made during a prolonged period of base flow conditions in the Mississippi River.

During the third phase of target selection, clusters of wells were reduced in number. Many of the wells are closely grouped around the landfill, for example. In order not to significantly bias the automatic calibration procedure, not all wells around the landfill were used in the calibration. Wells were chosen to provide an even distribution over the study area. Using this three-step approach, 69 target wells were chosen from a total of 164 wells. The water-table zone (Layer 1) contains the greatest number of calibration targets (30). The calibration targets in the intermediate (23) and deep (16) zones are fewer in number, but well distributed. The locations of calibration targets within the model are shown on Figures 9 through 11. These wells are also summarized in Table 4.

4.3 STEADY-STATE CALIBRATION RESULTS

One of the most important parameters used in evaluating a calibration is the residual. A residual is calculated for each calibration target by subtracting the model-calculated water level from the observed water level. A residual near zero signifies a close match between the model and observed field conditions. The sign of the residual, positive or negative, is just as important as the magnitude of the residual. Negative residuals occur where the model-calculated water levels are higher than observed. Conversely, positive residuals indicate that the model-calculated water levels are too low.

In discussing the quality of a model calibration, the following criteria must be considered: (1) the average of all residuals (residual mean) should be close to zero; (2) the variation in residuals (residual standard deviation) should be low; (3) the distribution of residuals within the model should be random; and (4) the flow patterns predicted by the model should match field observations. Most of these factors are subjective; however, all must be evaluated when determining the quality of a calibration.

All criteria listed above were satisfied in the model calibration. The residual mean (0.03 ft) was close to zero. The residual standard deviation (1.04 ft) is very low. The residuals are fairly well distributed and ground-water flow directions match field observations. Flow is toward the Mississippi River in all three layers with ground-water mounding in Layer 1 at the landfill and W.G. Krummrich plant. Figures 9 through 11 illustrate the potentiometric surfaces for the three model layers in the vicinity of Site R.

A statistical analysis of residuals quantifies the match between the simulated water levels and actual water-level measurements. The two important statistics discussed above include the residual mean and the residual standard deviation. For good calibration, the residual mean should be close to zero. This implies that positive residuals (areas where the model water levels are too low) and negative residuals (model water levels are higher than observed) are equally balanced within the model domain. In the model, the residual mean

is 0.03 ft. In addition to a residual mean close to zero, the residual standard deviation should be low. The model residual standard deviation was 1.04 ft. This means that most model residuals are in error by no more than 1.04 ft. In fact, 27 of the 69 residuals are less than 0.5 ft. The residual standard deviation should also be much less than the total change in head across the site. In this case, the total water-level change across the modeled area is about 23 ft. The residual standard deviation is less than five percent of this number. Residuals for each well are listed in Table 4.

The next test of a good calibration is the spatial distribution of residuals. There are two ways of looking at spatial distribution. The first involves plotting the observed versus calculated water levels. In a perfect calibration, the calculated water levels would equal the observed water levels. The scatter of actual residuals around this perfect line is a graphical means of evaluating spatial distribution of residuals. Such a plot is presented in Figure 12. This plot illustrates that residuals at high and low points in the flow system have a random error of ± 1.0 ft. That is, there is an even scatter among the residuals and the errors are evenly distributed between high and low water levels.

The second type of spatial analysis involves plotting the residuals on a site map. Positive or negative residuals should not cluster in any area, i.e., they should be randomly distributed. Figures 9 through 11 show the residuals in Layers 1 through 3 for the areas near Site R and the W.G. Krummrich plant. There are no wells and associated residuals located outside the area displayed by Figures 9 through 11.

Residuals in Layer 1 are well distributed around Site R, however, there is minor clustering of negative residuals around the ground-water mound located in the W.G. Krummrich plant, and the overall distribution of residuals is slightly biased toward higher water levels. Layer 1 also has a number of high residuals located in the landfill. This is due to the destabilizing effect of the Mississippi River on water levels.

5.0 TRANSIENT CALIBRATION RESULTS

The steady-state calibration discussed in the previous section compared model-calculated results to water levels measured in November 1988. This calibration represents average base-flow conditions in the Mississippi River. In order to demonstrate that the ground-water flow model constructed for Sauget is valid for higher water-level events as well, a transient calibration was also performed.

The transient calibration compared model-calculated water levels to those measured in November 1985 when the Mississippi River was at a much higher stage than in November 1988. The Mississippi River stage used in the transient calibration was 410 ft msl, compared to a stage of 381 ft msl used in the steady-state calibration. The November 1985 water levels are contoured in Figures 13 through 15 for the shallow, intermediate, and deep zones, respectively.

The transient calibration differed from the steady-state calibration in that ground-water levels in the aquifer were not at equilibrium. The Mississippi River was rising for about 1 week prior to the ground-water level measurements. Consequently, the ground-water levels were also still rising. To simulate these conditions, the water-level distribution calculated by the steady-state model was used as initial conditions in the transient calibration. Next, the Mississippi River stage was increased to 410 ft msl. This was the river stage reached just prior to the round of ground-water level measurements. The model was then run for 7 days and the model-calculated heads were contoured.

Only a qualitative comparison was made between model-calculated heads and observed heads because only one round of water-level measurements (November 1985) were available for comparison during a period when high river stage lasted for several weeks. In an ideal transient calibration, water levels are available at numerous times for comparison with the model results.

The model configuration for the transient calibration was identical to the steady-state model, with two exceptions: (1) a uniform storage coefficient was assumed in each layer (no storage coefficient is necessary in a steady-state model), and (2) the recharge rate was increased 10 percent because there was a significant amount of precipitation during the week prior to the water-level measurements. The storage coefficients were adjusted during the calibration to obtain a qualitative match between the observed and calculated water levels. The final storage coefficients were 0.1 in Layer 1, and 0.03 in Layers 2 and 3. These storage values are close to those obtained from pumping test analyses (0.07, 0.04 and 0.09) as discussed in Section 2.

The final calibrated ground-water levels simulated in the transient model one week after raising the Mississippi River level are presented in Figures 16 through 18 for model Layers 1 through 3, respectively. Both the model-generated figures and those contoured from observed data (Figures 22 to 24) show a reversal in ground-water flow directions near the Mississippi River. During this time frame, ground water flowed away from the river into the aquifer. A point of converging ground-water flow is clearly identified between the Krummrich Plant and Site R. This reversal in gradient near the river occurs in all three aquifer zones. In addition to the reversal in gradient, both model results and observed water levels increased to levels above 400 ft msl between Site R and the Mississippi River.

The two methods used to calibrate the model each clearly illustrate that the numerical ground-water flow model accurately represents the aquifer system at Site R and its vicinity for both high and low flow conditions.

6.0 REFERENCES

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Table 1. Summary of Hydraulic Conductivity and Storage Coefficient Data Available Prior to the June 1992 Aquifer Test.

Zone	Source	Well Number or Location	Depth (ft)	Hydraulic Conductivity (ft/day)	Storage Coefficient (dimensionless)	
Upper	Slug Test (G&M, 1986b)	GM-1	36	3.07	0.01	
(Layer 1)	Slug Test (G&M, 1986b)	GM-2	44	0.25	0.1	
	Slug Test (G&M, 1986b)	GM-3	36	0.47	0.1	
	Aquifer Test (G&M, 1986b)	B-1	10.5	0.51	--	
	Aquifer Test (G&M, 1986b)	B-10	35.5	17.01	--	
	Aquifer Test (G&M, 1986b)	B-11	25.5	5.67	--	
	Aquifer Test (G&M, 1986b)	B-15	45.5	3.97	--	
				4.42	0.07	Avg.
Intermediate	Aquifer Test (1986b)	WGK Plant	65	441.18	0.04	
(Layer 2)						
Deep	Aquifer Test (G&M, 1986b)	Mobil Oil Corp.	114	387.70	0.1	
(Layer 3)		St. Clair County				
		T2N, R10W				
		Section 25				
	Aquifer Test (G&M, 1986b)	Ranney Well	99	374.33	0.082	
		Sauget Site R				
				381.02	0.09	Avg.

MONSANTOMODEL RPT\TABLE1\WY4211

Table 2. Estimates of aquifer characteristics obtained through interpretation of data from the June 1992 aquifer test, Sauget, Illinois.

Well Number	Method	T* (ft^2/d)	K** (ft/d)	S***
Intermediate Zone				
P-5	Theis	38,000	420	0.012
P-9	Theis	22,000	240	0.013
P-10	Theis	22,000	240	0.0083
B-24C	Theis	34,000	380	0.0042
	Cooper-Jacob	32,000	360	0.0045
B-26B	Theis	32,000	360	0.0065
	Neuman	29,000	320	0.007
Deep Zone				
GM-57C	Hantush	31,000	340	0.0004
	Neuman	30,000	330	0.0055
GM-56C	Hantush	16,000	170	0.0013
	Neuman	24,000	270	0.016
GM-28C	Hantush	31,000	350	0.0001
	Average:	28,400	315	0.007

* Transmissivity (for the combined Intermediate/Deep Zones)

** Hydraulic Conductivity

*** Storage Coefficient (Specific Yield for Neuman Method)

MONSANTO\MODEL RPT\TABLE.2\NY642.11

Table 3. Summary of Hydraulic Parameters used in the Monsanto Model.

Parameter Type	Zone	Value	Representation
Hydraulic Conductivity			
K(ft/day)	1	6.2	Water-table Layer (Cahokia Alluvium)
K(ft/day)	4	1.0	Site R (Water-Table Layer)
K(ft/day)	5	0.4	Mississippi River (Water-Table Layer)
Transmissivity			
T(ft^2/day)	2	15,000	Intermediate Layer (Henry Formation)
T(ft^2/day)	3	15,000	Deep Layer (Henry Formation)
Vertical Leakance			
Kv(day^{-1})	1	0.0063	Water-Table Layer/Intermediate Layer
Kv(day^{-1})	2	1.00	Intermediate Layer/Deep Layer
Kv(day^{-1})	3	1.0×10^{-9}	Water-table/Intermediate Layer in the landfill area
Kv(day^{-1})	4	0.42	Mississippi River/Intermediate Layer
Kv(day^{-1})	5	0.23	Intermediate Layer/Deep Layer (Mississippi River)
Recharge			
R(ft/day)	1	0.002	Water-Table Layer
R(ft/day)	2	0.084	Mounding at the W.G. Krummrich Plant (Water-Table Layer)
R(ft/day)	3	0.0006	Landfill Cap at Site R (Water-Table Layer)

Table 4. Comparison between observed and computed water levels in the steady-state calibration.

Well	Row	Column	Layer	Observed Head (ft above msl)	Computed Head (ft above msl)	Residual (ft)
GM-1	26	32	1	393.970	395.077	-1.11
GM-4A	33	22	1	388.720	389.829	-1.11
GM-4B	33	22	2	388.510	389.493	-0.983
GM-4C	33	22	3	390.530	389.492	1.04
GM-5	36	20	1	387.930	388.705	-0.775
GM-6A	38	24	1	389.390	390.439	-1.05
GM-6B	38	24	2	389.440	390.124	-0.684
GM-9A	35	29	1	394.810	392.909	1.90
GM-9B	35	29	2	391.540	392.427	-0.887
GM-9C	35	29	3	391.250	392.426	-1.18
GM-10A	28	28	1	395.740	396.468	-0.728
GM-10B	28	28	2	391.520	392.023	-0.503
GM-10C	28	28	3	391.400	392.009	-0.609
GM-11	24	30	1	392.530	392.929	-0.399
GM-12A	30	32	1	393.270	394.804	-1.53
GM-12B	30	32	2	393.600	394.465	-0.865
GM-12C	30	32	3	393.260	394.464	-1.20
GM-15	29	30	1	392.960	393.918	-0.958
GM-16A	25	27	1	391.480	391.800	-0.320
GM-16B	25	27	2	391.420	391.468	-0.048
GM-17A	36	23	1	389.220	390.255	-1.03
GM-17B	36	23	2	389.220	389.936	-0.716
GM-17C	36	23	3	388.950	389.935	-0.985
GM-18A	42	21	1	388.530	389.171	-0.641
GM-18B	42	21	2	388.610	388.855	-0.245
GM-19B	41	17	2	386.530	386.498	0.03192
GM-19C	41	17	3	386.580	386.497	0.08292
GM-20A	38	18	1	387.260	387.388	-0.128
GM-20B	38	18	2	387.110	387.072	0.03819
GM-22A	40	16	1	386.710	386.691	0.01897
GM-26A	31	16	1	386.380	386.443	-0.063
GM-26B	31	16	2	386.110	386.127	-0.017
GM-27C	36	8	3	383.040	382.213	0.827
GM-28B	42	10	3	382.700	382.531	0.169
GM-29	32	28	1	397.480	397.467	0.01282
GM-30	32	31	1	395.380	394.289	1.09
GM-31C	44	22	3	388.780	389.080	-0.300

Table 4. Comparison between observed and computed water levels in the steady-state calibration. (continued)

Well	Row	Column	Layer	Observed Head (ft above msl)	Computed Head (ft above msl)	Residual (ft)
GM-45	30	28	1	403.040	402.708	-0.332
GM-46	31	29	1	401.520	401.128	0.392
GM-54B	44	20	2	388.390	388.417	-0.027
GM-55C	44	9	3	383.260	382.366	0.894
GM-56C	40	9	3	383.490	382.359	1.13
GM-57C	38	9	3	382.910	382.208	0.702
GM-58A	44	21	1	388.540	389.212	-0.672
GM-60A	36	15	1	386.010	385.941	0.06854
GM-60B	36	15	2	386.020	385.612	0.408
GM-60C	36	15	3	386.080	385.611	0.469
GM-61A	43	17	1	386.810	387.066	-0.256
GM-62A	41	13	1	385.420	385.365	0.05530
GM-62B	41	13	2	385.420	384.728	0.692
GM-62C	41	13	3	385.440	384.726	0.714
GM-63A	42	15	1	386.170	386.155	0.01485
B-21B	37	11	2	384.250	383.640	0.610
B-22A	37	9	1	396.300	392.283	4.02
B-24A	39	10	1	398.110	395.667	2.44
B-24B	39	10	2	383.170	382.659	0.511
B-24C	39	10	3	383.270	382.668	0.602
B-26A	41	10	1	392.500	394.400	-1.90
B-26B	41	10	2	383.200	382.645	0.555
B-27B	44	11	2	383.800	383.171	0.629
B-28A	43	10	1	394.320	393.846	0.474
B-28B	43	10	2	383.280	382.774	0.506
B-29A	43	12	1	394.090	390.691	3.40
B-31C	37	12	3	384.880	384.315	0.565
P-1	44	9	2	383.000	382.342	0.658
P-8	40	9	2	382.730	382.317	0.413
P-12	37	8	2	382.530	382.194	0.336
BK-3	30	29	2	392.130	392.614	-0.484
WB-6	39	33	1	394.240	396.027	-1.79

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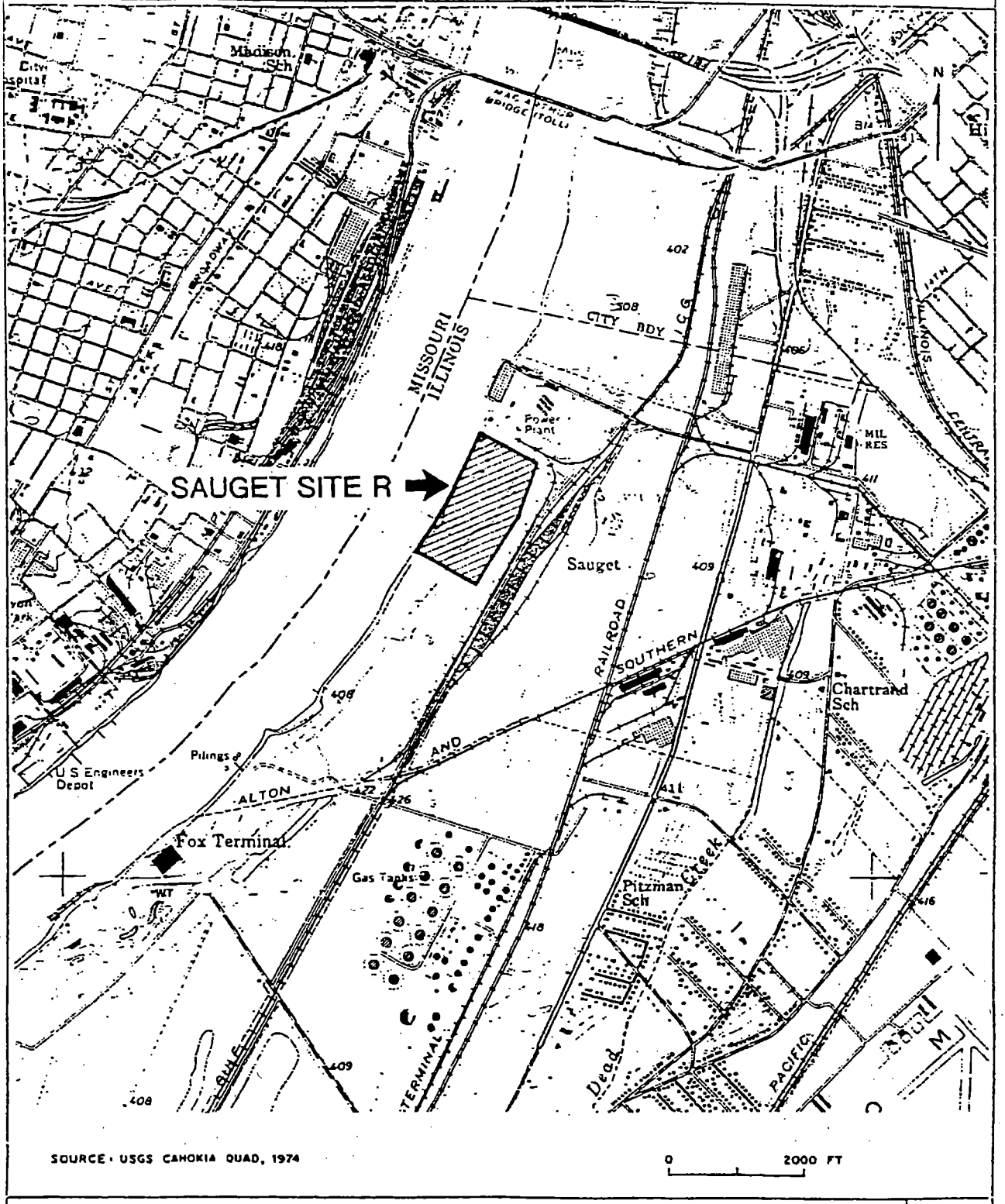

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LOCATION OF SAUGET SITE R, Monsanto Company, Sauget, Illinois

FIGURE 1

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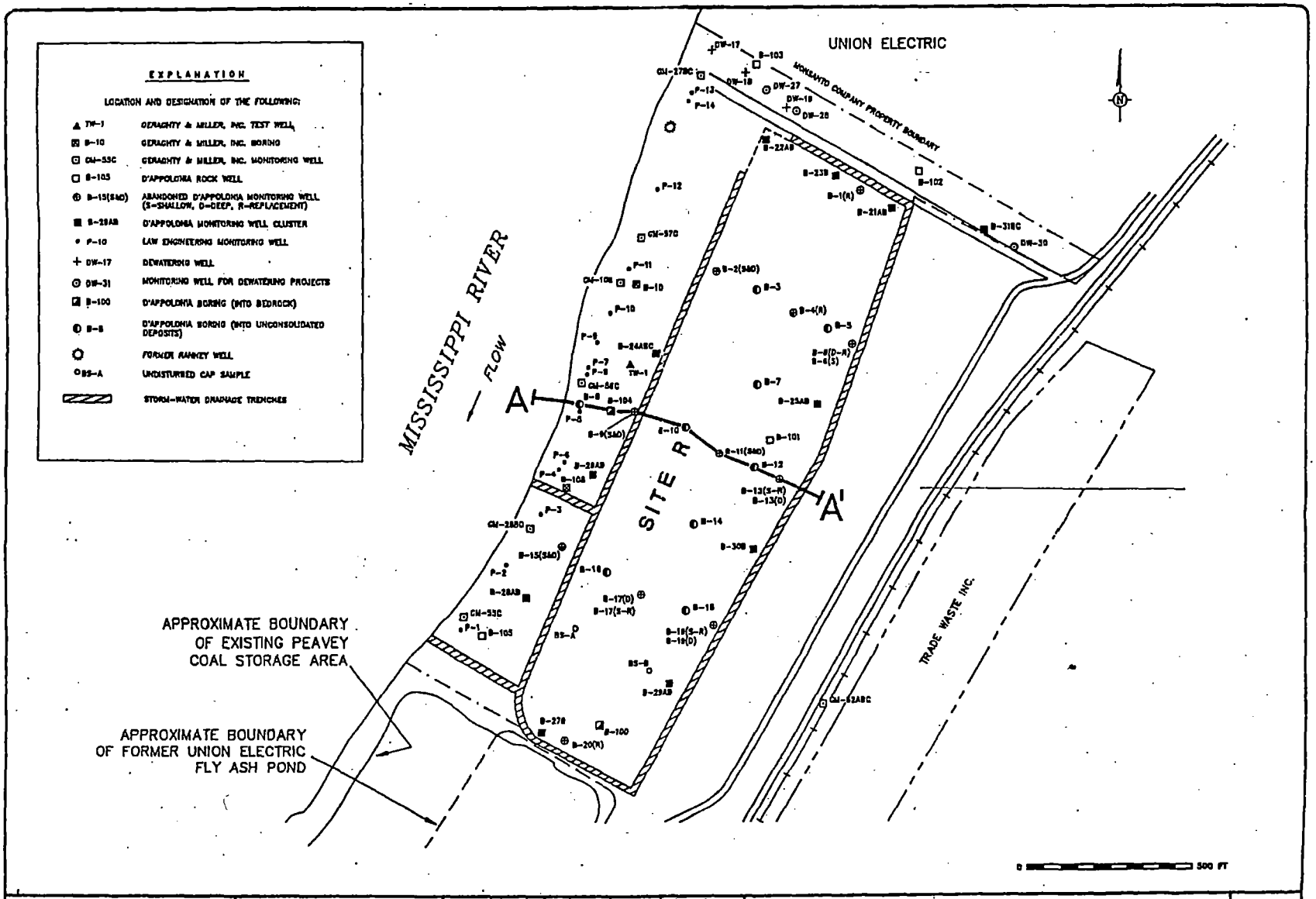

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LOCATIONS OF MONITORING WELLS AND SOIL BORINGS, SAUGET SITE R

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 2

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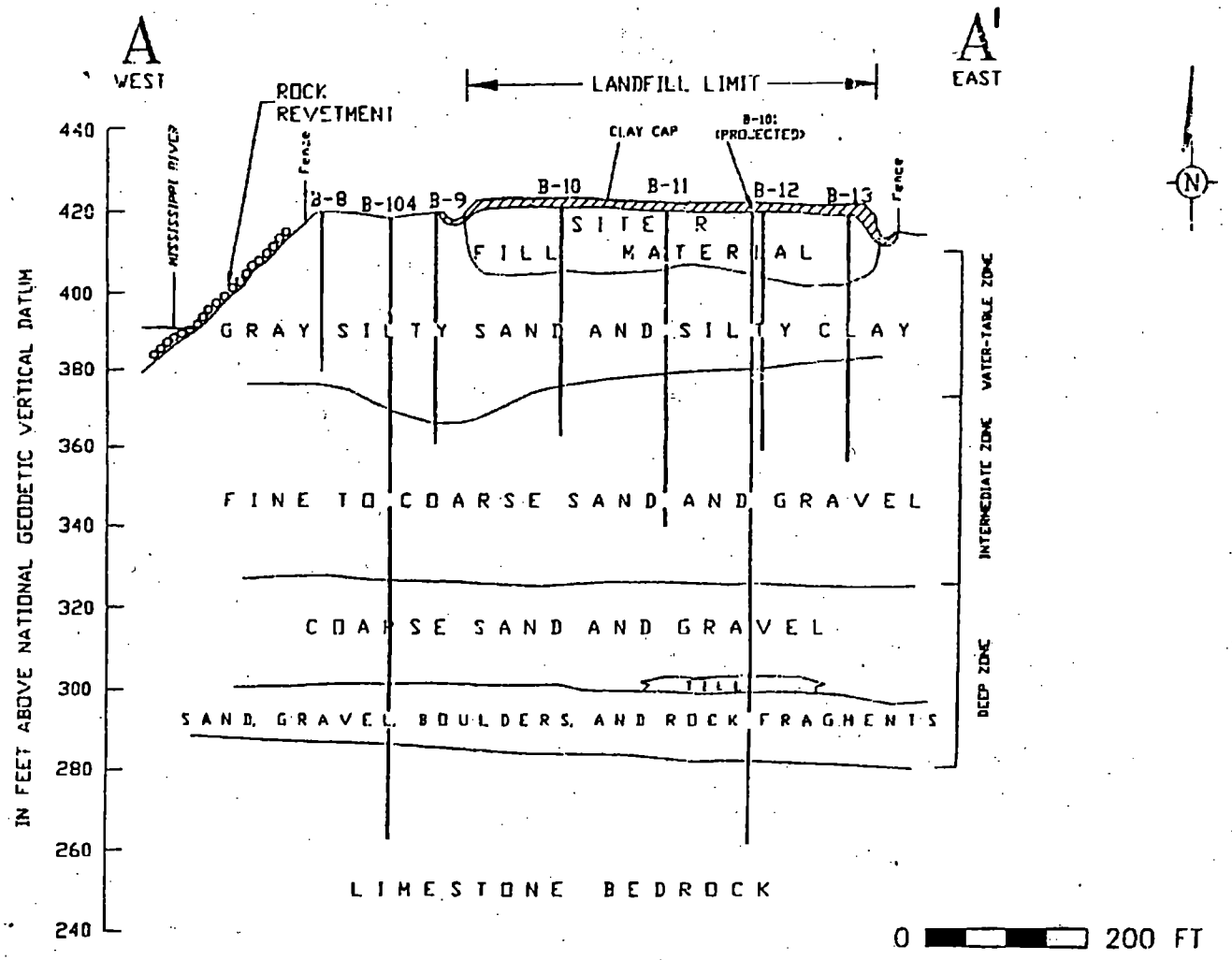

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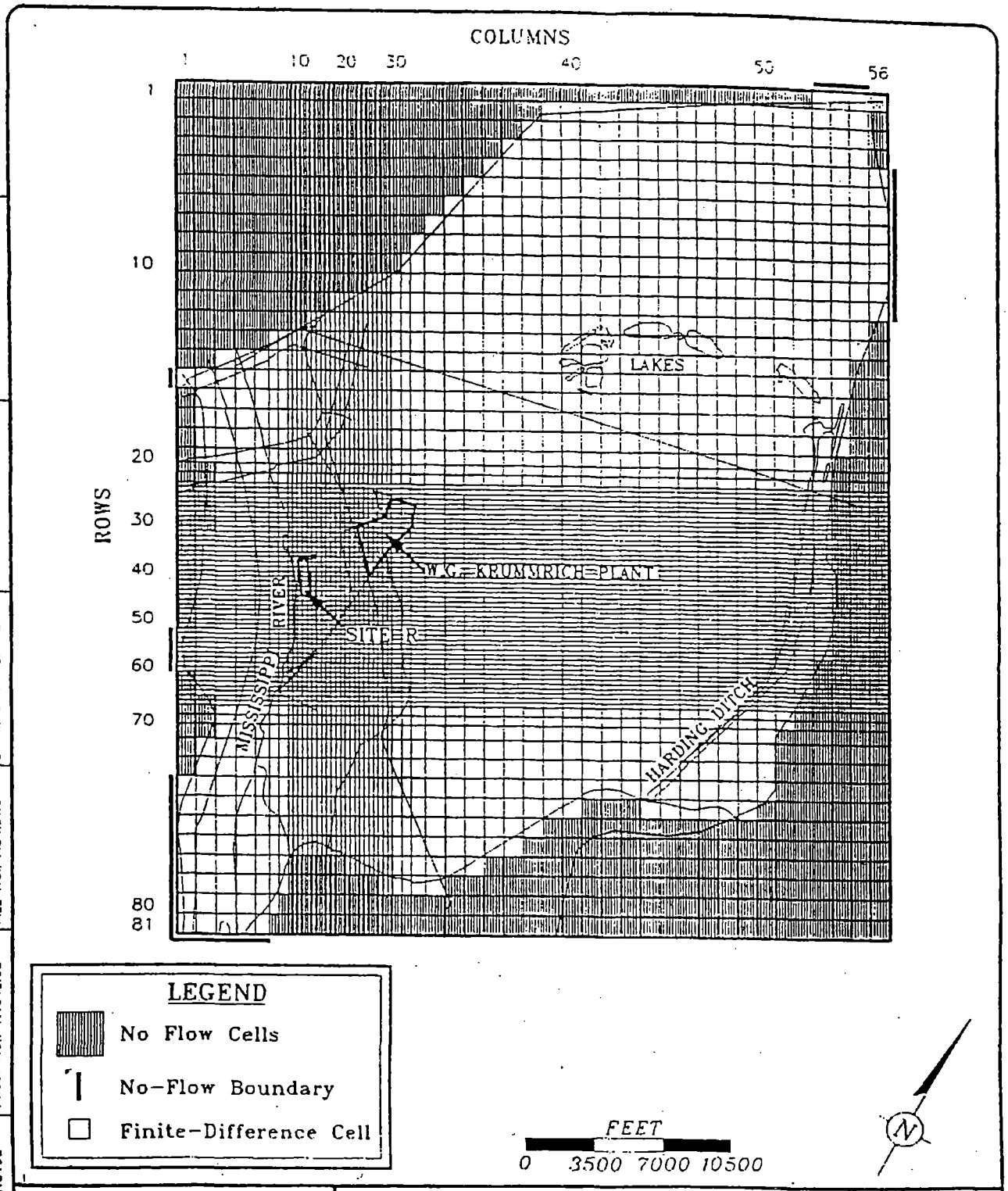
CROSS SECTION A-A', SAUGET SITE R, Monsanto Company, Sauget, Illinois

FIGURE 3



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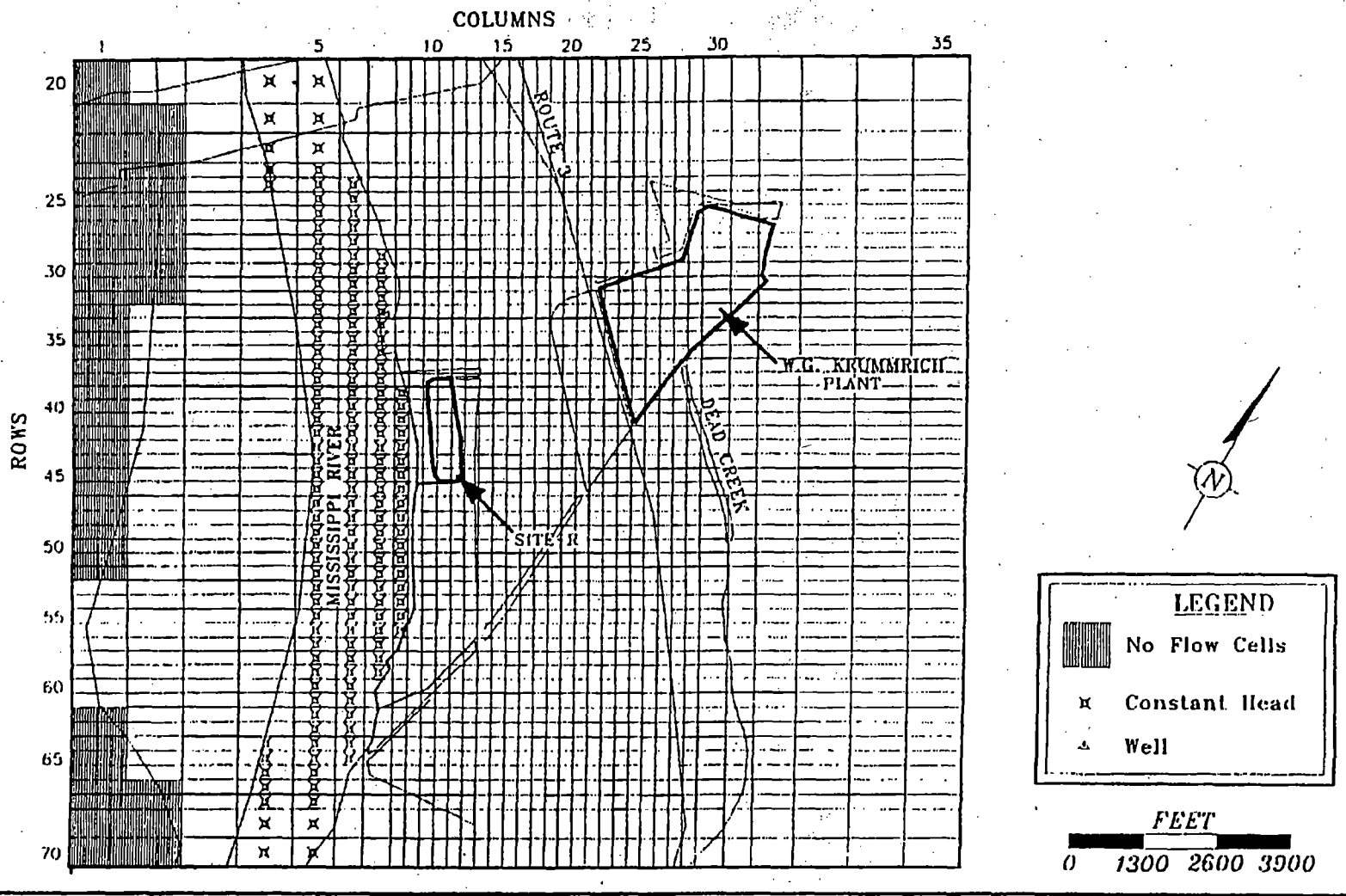
Finite-Difference Mesh of the Monsanto Ground-Water Flow Model.

Monsanto Company
Sauget, Illinois

FIGURE 4

DWG DATE: 18NOV92 | PRJCT NO.: NY642 08 | FILE NO: FIG5 DWG | DRAWING: MONSANTO | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



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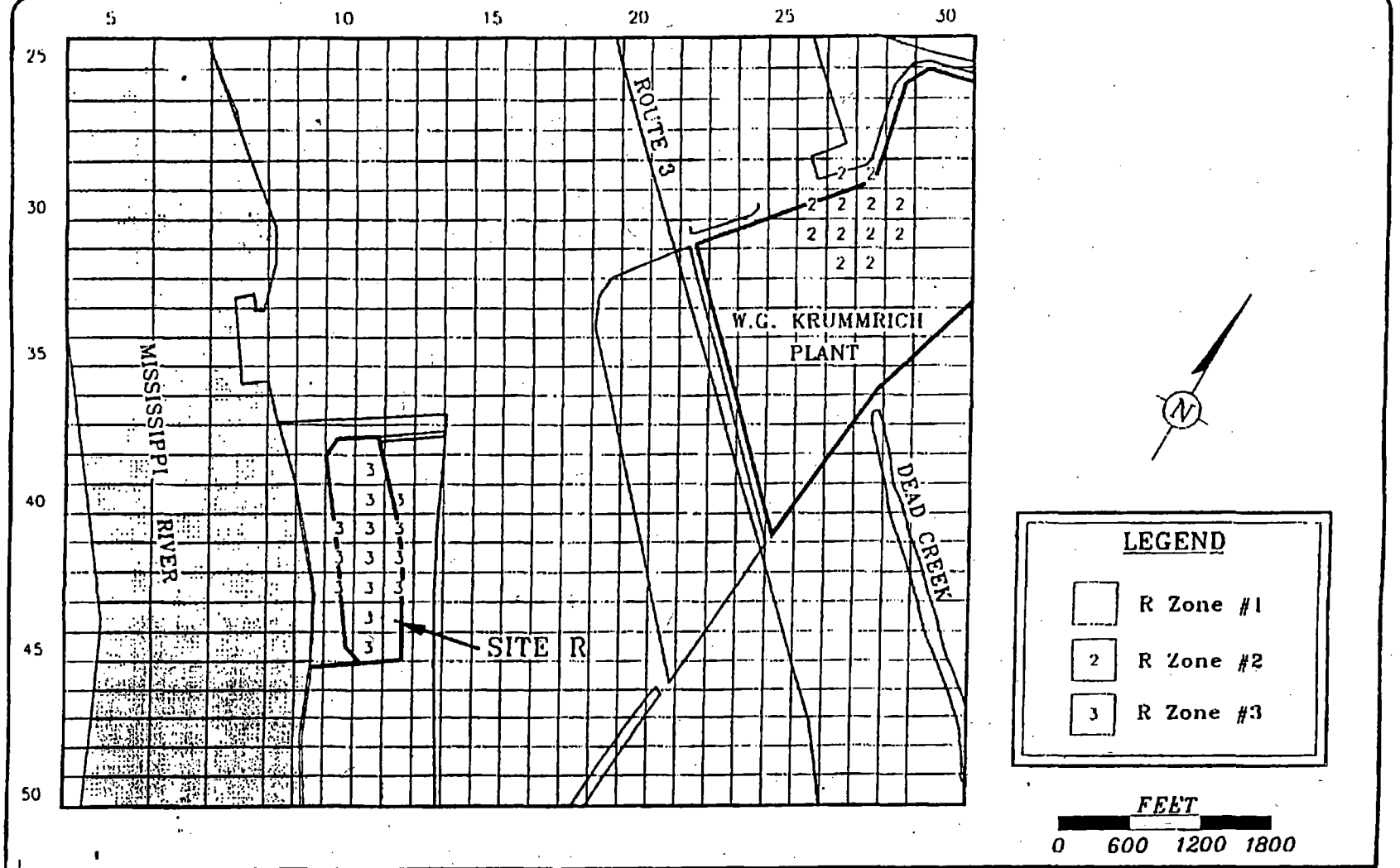
Portion of the Finite-Difference Mesh near Sauget Site R Illustrating Placement of Boundary Conditions in Layer 1.

Monsanto Company
Sauget, Illinois

FIGURE
5

DWG . 18NOV92 | PRJCT NO: NY642.08 | FILE NO.: FIG6.DWG | DRAWING: MONS...ION | CHECKED: JOR | APPROVED: JOR | DRAFTER: CM



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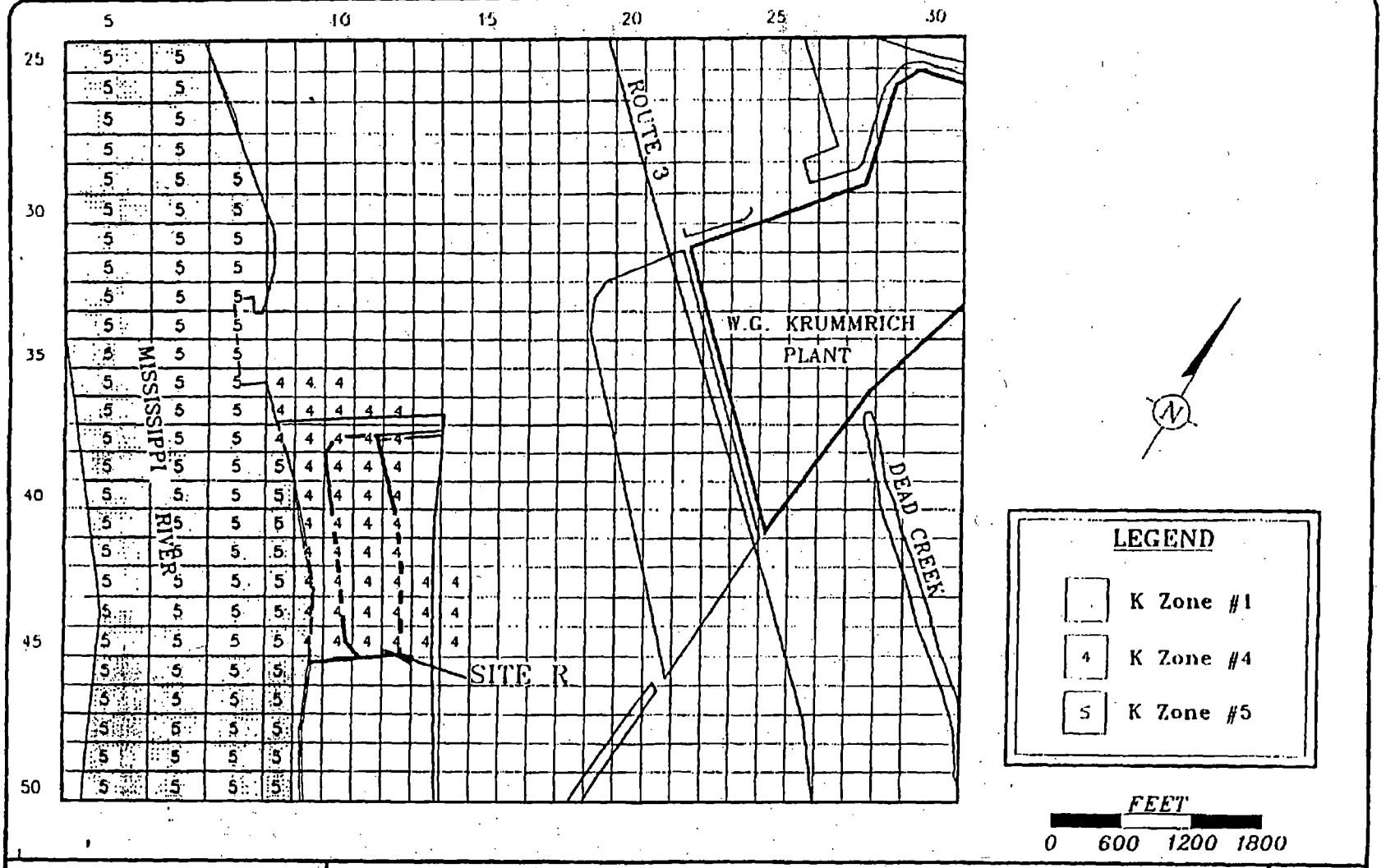
Portion of the Finite-Difference Mesh near Sauget Site R Illustrating Placement of Recharge Zones in Layer 1.

Monsanto Company
Sauget, Illinois

FIGURE 6

DWG DATE: 18NOV92 | PRJCT NO.: NY642.08 | FILE NO.: FIG7.DWG | DRAWING: MONSANTO | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



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Portion of the Finite-Difference Mesh near Sauget Site R Illustrating Placement of Hydraulic Conductivity Zones in Layer 1.

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Sauget, Illinois

FIGURE 7

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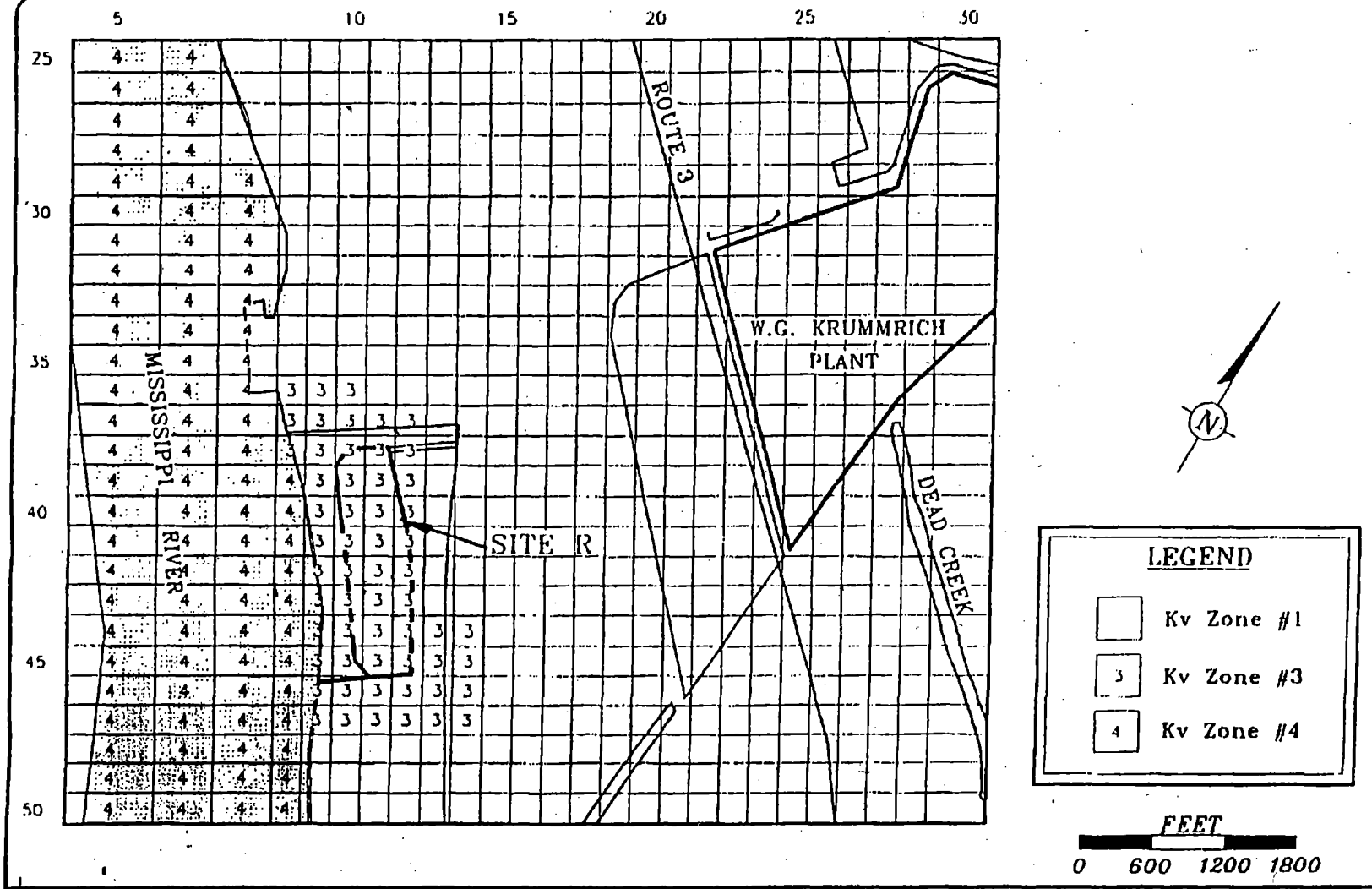

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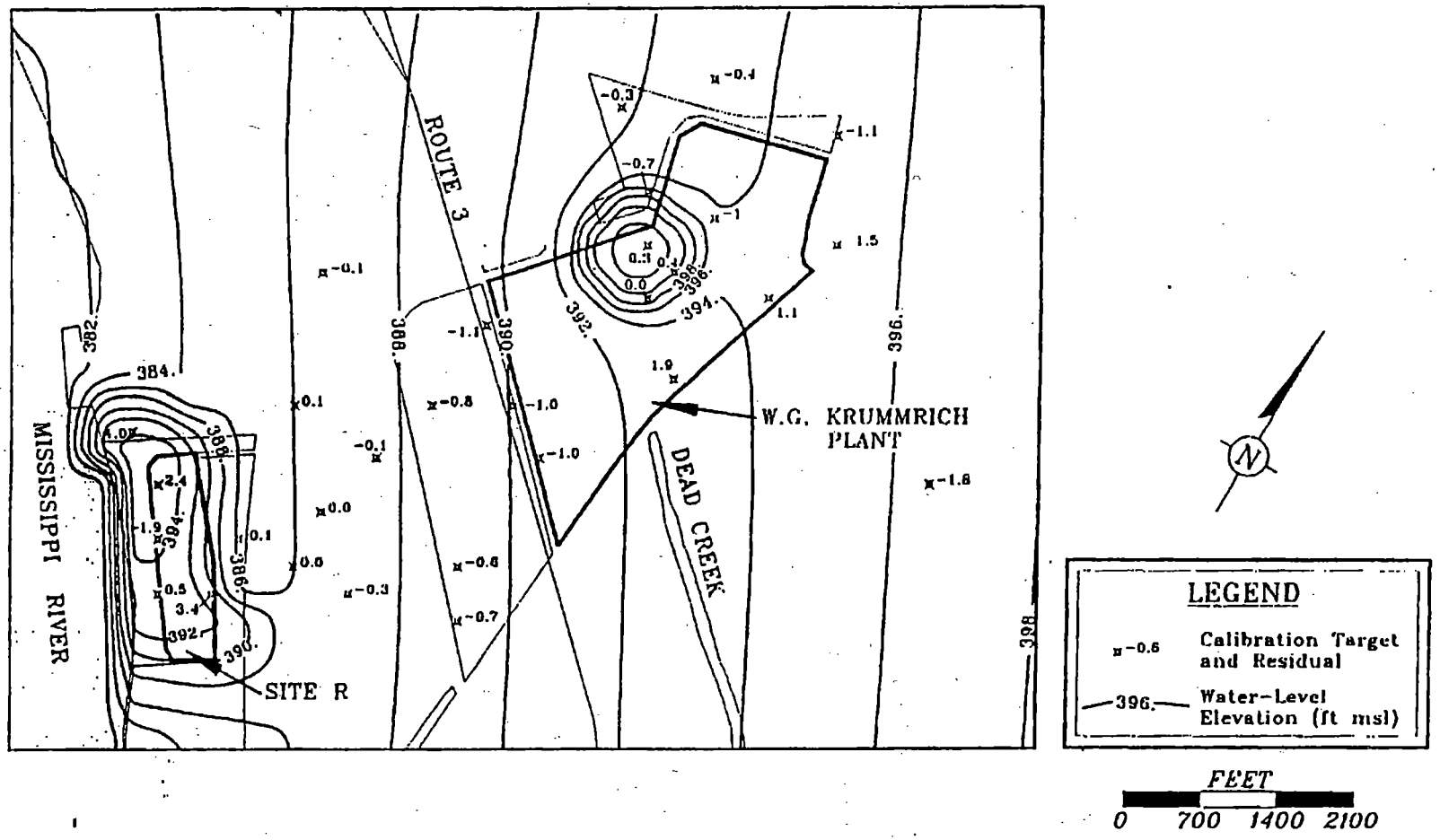
Portion of the Finite-Difference Mesh near Sauget Site R Illustrating Placement of Leakance Zones in Layer 1.

Monsanto Company
Sauget, Illinois

FIGURE
8

DWG D..... 19NOV92 | PRJCT NO · NY642.08 | FILE NO.: FIG9 DWG | DRAWING: MONSANTO | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



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Calibrated Water-Table Contours in Layer 1 Showing Residuals for the Steady-State Calibration.

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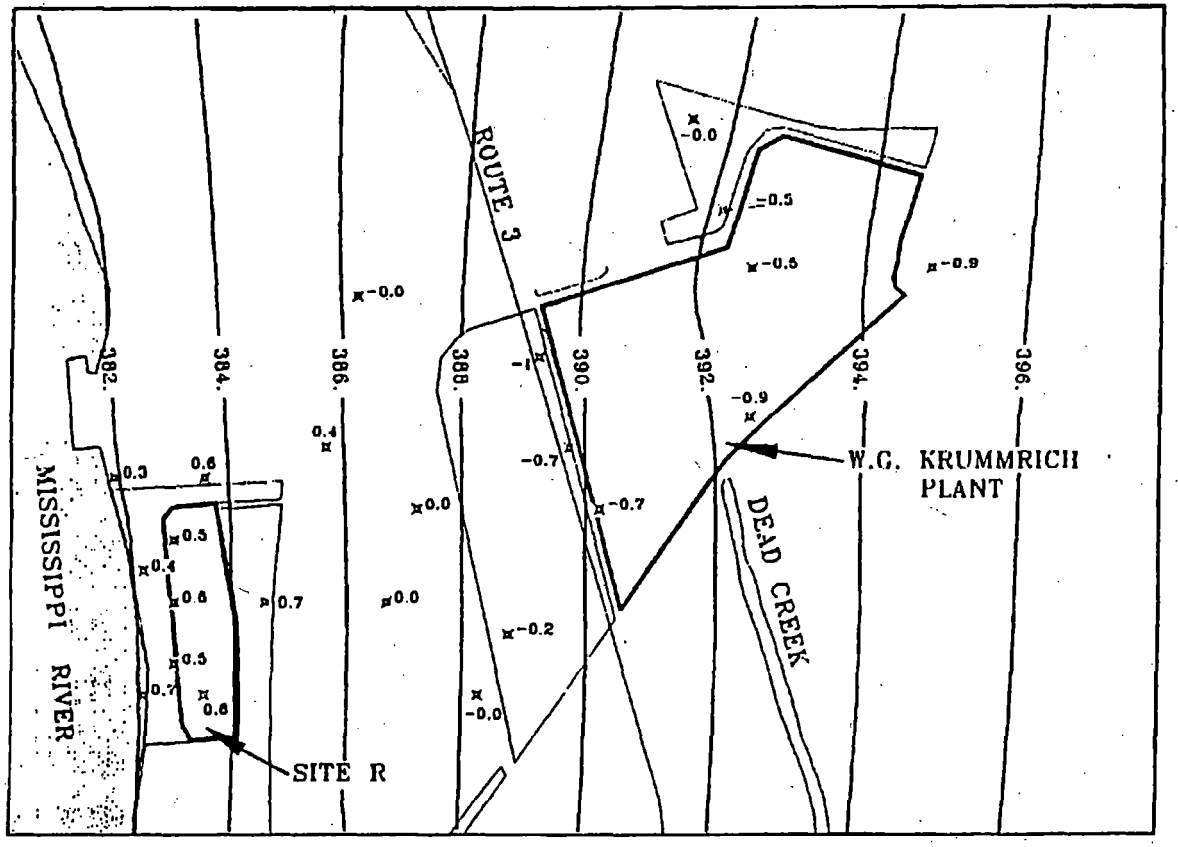
FIGURE 9

DWG DATE: 19NOV92	PRJCT NO.: NY642 08	FILE NO.: FIG10.DWG	DRAWING: MONSANTO	CHECKED: JOR	APPROVED: JOR	DRAFTER: CMS



LEGEND

x -0.6	Calibration Target and Residual
—396.—	Water-Level Elevation (ft msl)

FEET

0 700 1400 2100

GERAGHTY & MILLER, INC.
Environmental Services

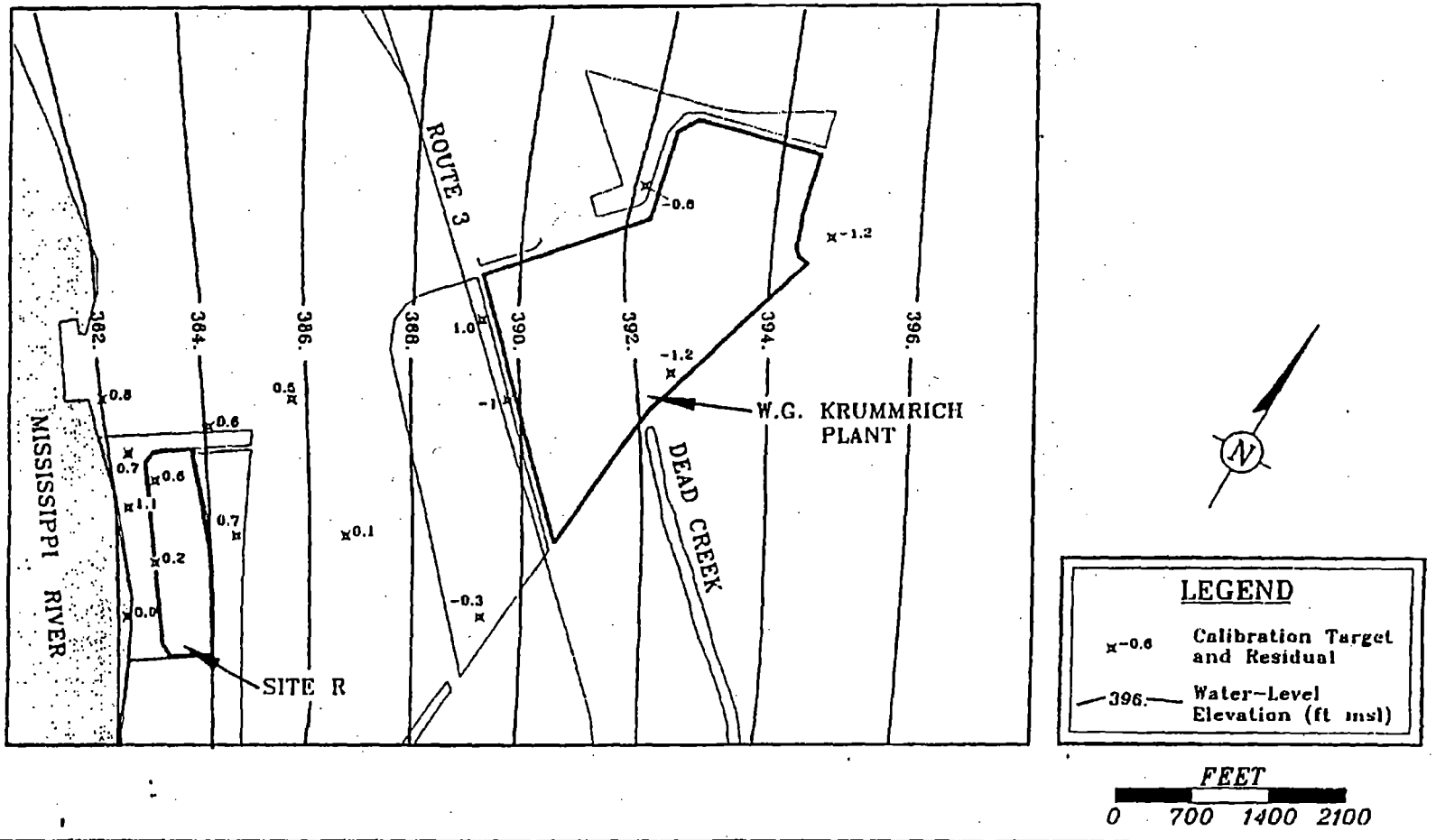
Calibrated Potentiometric Surface in Layer 2 Showing Residuals for the Steady-State Calibration.

Monsanto Company
Sauget, Illinois

FIGURE

10

DWG D. - 19NOV92 | PRJCT NO: NY642.08 | FILE NO: FIG11.DWG | DRAWING: HUNSMITH | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS

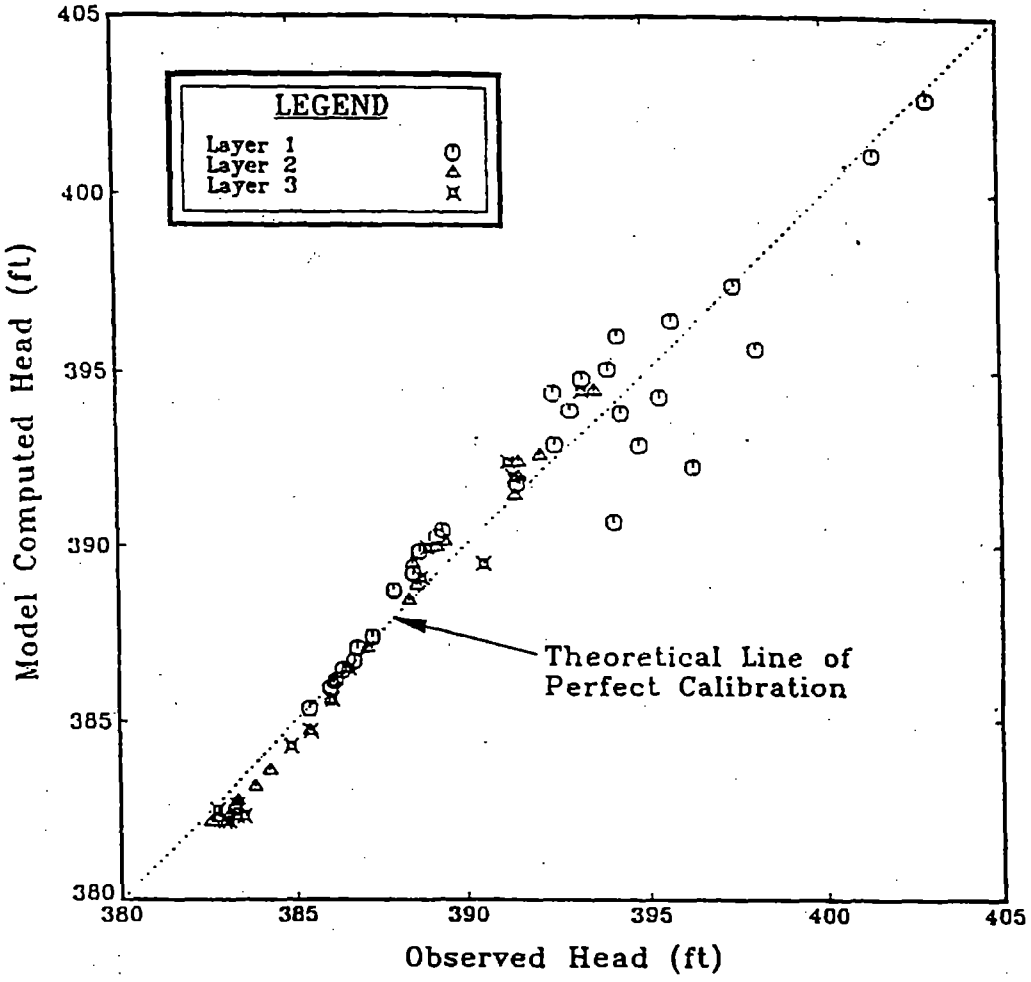


GERAGHTY & MILLER, INC.
Environmental Services

Calibrated Potentiometric Surface in Layer 3 Showing Residuals for the Steady-State Calibration.

Monsanto Company
Sauget, Illinois

FIGURE
11



GERAGHTY & MILLER, INC.
Environmental Services

Comparison Between Observed and Computed Water Levels.

Monsanto Company
Sauget, Illinois

FIGURE 12

DRAFTER: CMS | APPROVED: JOR | CHECKED: JOR | DRAWING: MONSANTO | FILE NO.: FIG12.DWG | PRJCT NO.: NY642.08 | DWG DATE: 19NOV92

CONFIGURATION OF THE WATER TABLE

NOVEMBER 1985

MONSANTO COMPANY
Sauget, Illinois

FIGURE 13

POTENTIOMETRIC SURFACE OF THE INTERMEDIATE ZONE
NOVEMBER 1988

MONSANTO COMPANY
Sauget, Illinois

FIGURE 14

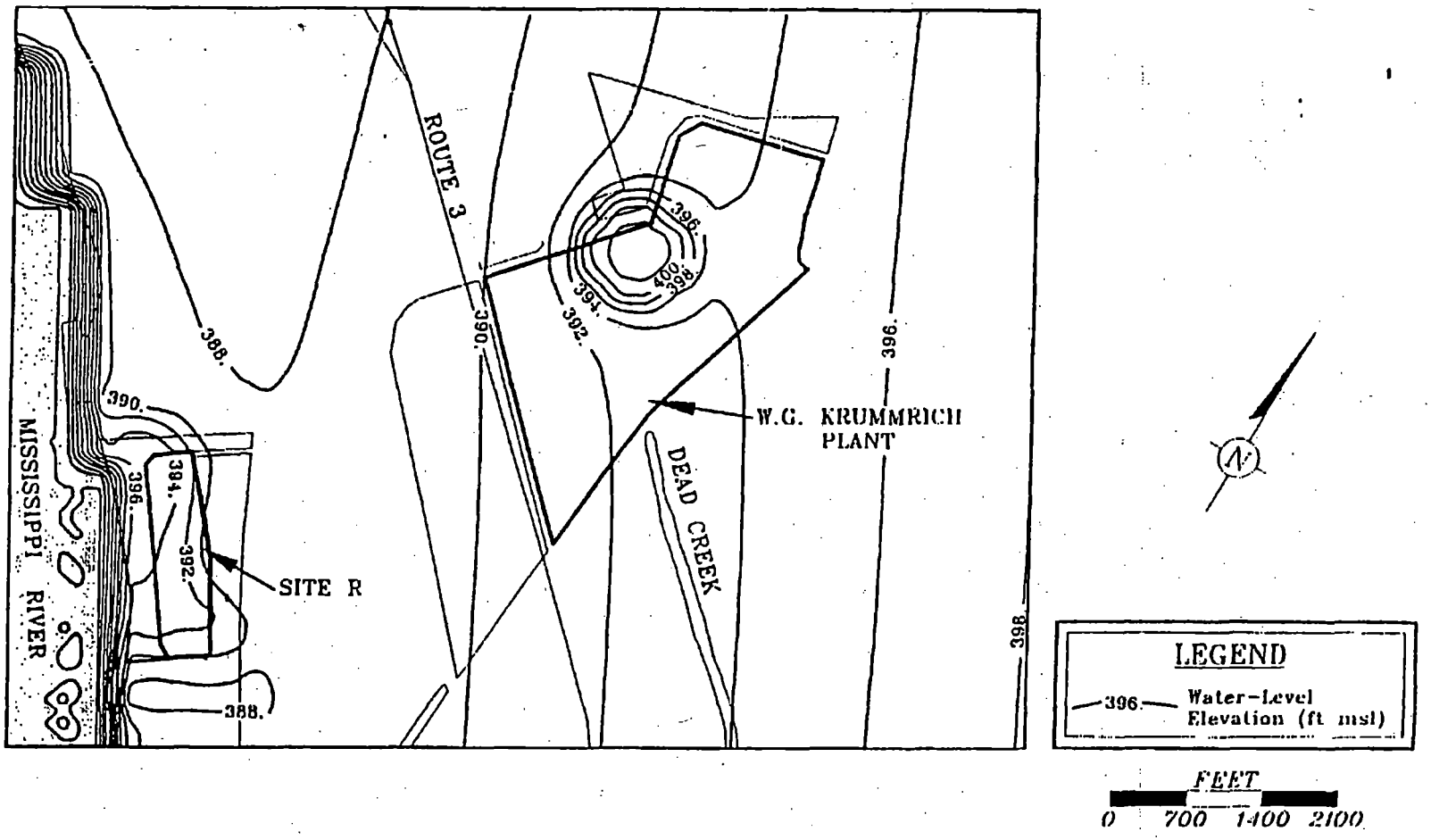
POTENTIOMETRIC SURFACE
OF THE DEEP ZONE

NOVEMBER 1985

MONSANTO COMPANY
Sauget, Illinois

FIGURE 15

DWG DATE: 19NOV92	PRJCT NO.: NY642.08	FILE NO.: FIG16.DWG	DRAWING: MONSANTO	CHECKED: JOR	APPROVED: JOR	DRAFTER: CMS



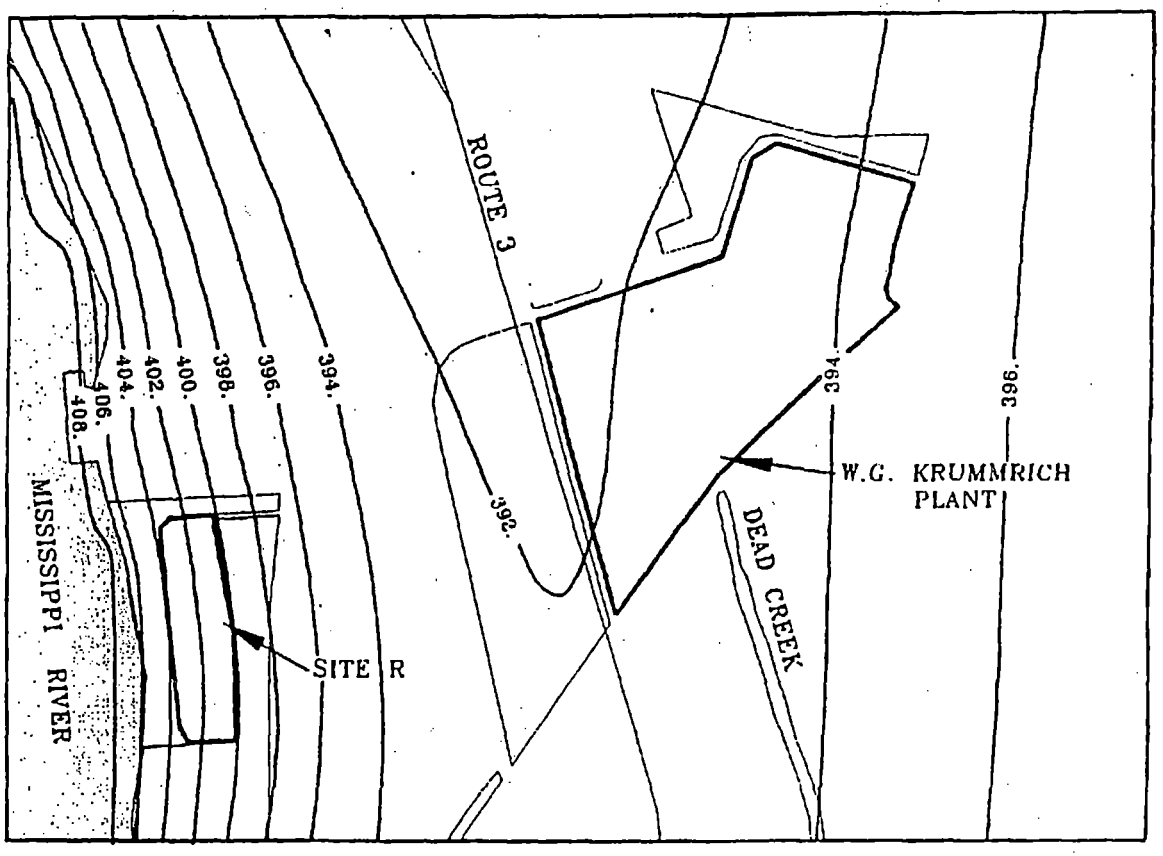
GERAGHTY & MILLER, INC.
Environmental Services

Model Calculated Water-Table Contours in Layer 1 for the Transient High-Water Analysis.

Monsanto Company
Sauget, Illinois

FIGURE 16

DWG DATE: 19NOV92 | PRJCT NO.: NY642.08 | FILE NO.: FIG17.DWG | DRAWING: MONSANTO | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



LEGEND

396. — Water-Level Elevation (ft msl)

FEET

0 700 1400 2100

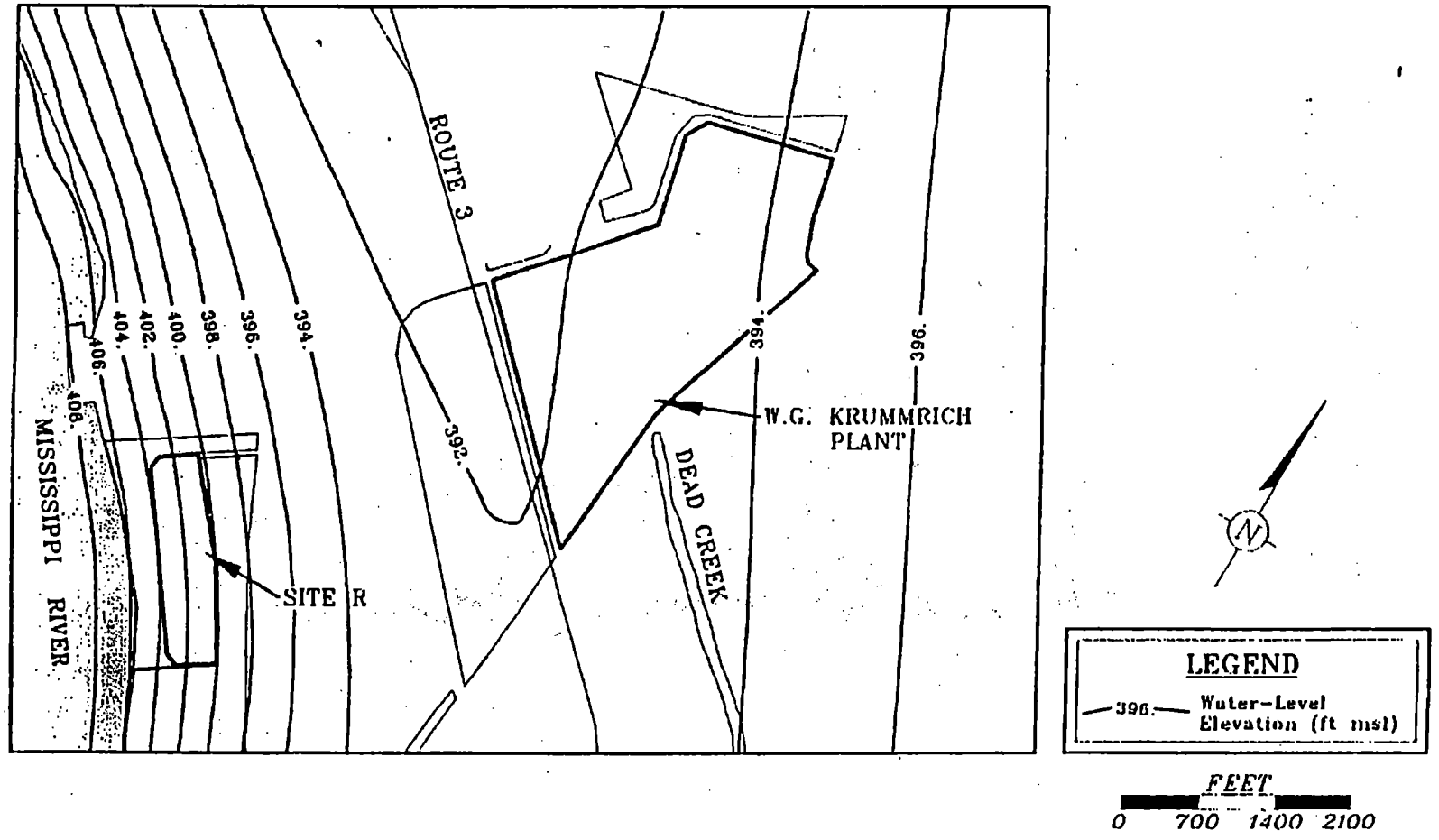
GERAGHTY & MILLER, INC.
Environmental Services

Model Calculated Potentiometric Surface in Layer 2 for the Transient High-Water Analysis.

Monsanto Company
Sauget, Illinois

FIGURE 17

DWG DATE: 19NOV92 | PRJCT NO.: NY642.08 | FILE NO.: FIG18.DWG | DRAWING: HGHSANTD | CHECKED: JOR | APPROVED: JOR | DRAFTER: CMS



GERAGHTY & MILLER, INC.
Environmental Services

Model Calculated Potentiometric Surface in Layer 3 for the Transient High-Water Analysis.

Monsanto Company
Sauget, Illinois

FIGURE 18

DRAFT
GSI Job No. G-2561
March 31, 2002

GROUNDWATER
SERVICES, INC.

Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

ATTACHMENT 14

Appendix 3

Groundwater Flow Conditions

(Geraghty & Miller, 1994)

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APPENDIX G

GROUNDWATER FLOW CONDITIONS

Groundwater flow at Site R has been monitored routinely since 1983. Additional information on groundwater flow and aquifer characteristics of the three hydrogeologic zones within the unconsolidated aquifer was developed during RI activities in 1992. These activities included collecting water-level measurements under static conditions and conducting an aquifer test. This information was used to supplement previous data and to calibrate a three-dimensional groundwater flow model (Appendix H). Section 1 discusses groundwater flow conditions; Section 2 provides results of the aquifer test; and Section 3 provides a discussion of groundwater discharge calculations.

1.0 GROUNDWATER FLOW

Section 1.1 provides a description of groundwater flow conditions based on data collected prior to December 1992. Section 1.2 discusses results of modeling performed to assess the impact of the 1993 Mississippi River flood on the groundwater system.

1.1 NORMAL RIVER STAGES

As discussed in Section 2.6 of the RI Report (Historical Groundwater Use and Flow Patterns), regional groundwater flow in the three hydrogeologic zones is to the west, towards the Mississippi River. Water levels measured on June 3, 1992 in the shallow, intermediate, and deep zones are shown on Figures 1, 2, and 3, respectively. These data are summarized in Table 1.

Figure 1 shows that a groundwater mound exists in the shallow zone at Site R. The existence of this mound has been previously documented in the RI work plan. It is apparently due to low permeability units beneath the area that reduce drainage rates from the shallow zone after periods of precipitation or high river stage. Groundwater flows to the east and south from the mound, but must eventually flow west toward the river. Historical data and the groundwater model (Appendix H) indicate that the eastern flow reaches a stagnation point (where the eastward

flow meets the regional westward flow) which is generally between Site R and the levee. Its exact location depends on the magnitude of the regional westward flow and river stage. At the stagnation point, water from the shallow zone flows downward into the intermediate zone. Water which flows south from the mound eventually turns to the west under the influence of the regional flow patterns.

Both the easterly and southerly flow from the mound are included in the model. The easterly flow is included in the intermediate zone estimate of groundwater discharge to the river. Wells screened in the intermediate zone adjacent to the river encounter this flow. Shallow wells along the river in the southern portion of Site R and in the Expanded Study Area encounter the southern flow.

Figures 2 and 3 show that groundwater flow in the intermediate and deep zones on June 3, 1992 was toward the river. Water-level data from well clusters screened in the intermediate and deep zones (GM-27B and GM-27C, P-8 and GM-56C, and GM-28B and GM-28C) indicates that there is an upward gradient from the deep zone to the intermediate zone (Table 1). This is to be expected because these wells are adjacent to the Mississippi River, which is a major groundwater discharge boundary. Groundwater flows from the lower portion of the aquifer up toward the river.

During periods of high river stage, when the river rises higher than the water table, gradients in the intermediate and deep zones are reversed. Flow in all three zones is toward the east, but eventually reaches a stagnation point where the eastward gradient equals the westward regional gradient. This "riverbank storage effect" can last from several days to a few weeks. The response of all three zones to varying river stages was demonstrated in hydrographs provided in the RI Work Plan (Geraghty & Miller 1990).

Analytical data from the well cluster located adjacent to the flood control levee (GM-62A, GM-62B, and GM-62C) indicate that there has been little, if any, transport of constituents from Site R to the east. The concentrations of total VOCs and total SVOCs are less than 150 ug/L

in each of these wells. These concentrations are several orders of magnitude lower than the concentrations detected in Site R wells.

1.2 FLOOD CONDITIONS

In order to assess the impact of extreme conditions, such as those in the flood of 1993, a scenario which simulated even worse conditions was run on the model. A flood stage of 48 ft was assumed to last for 60 days. The flow field at the end of the 60-day period was then used to estimate the flow velocities to the east. The actual flood crest was 49.5 ft on August 1, 1993, and river levels dropped by 10 ft (to 39.5 ft) within two weeks.

The modeling results estimate that under the extreme conditions simulated, groundwater in the intermediate zone would travel approximately 6.5 ft/day. In the deep zone groundwater would travel approximately 8.3 ft/day. Water levels in the shallow zone did not reach equilibrium in the 60-day period modeled. Water-level measurements obtained from wells east of the flood wall on July 24, 1993 (when the river stage was 46.5 ft) were used to calculate a groundwater velocity of 0.06 ft/day in the shallow zone.

Within the actual groundwater flow environment, constituents dissolved in the groundwater would move more slowly than the predicted groundwater velocities because various factors such as adsorption and biodegradation can retard their movement. No retardation coefficients were considered in the modeled scenario.

2.0 AQUIFER TEST

An aquifer test was conducted to provide site-specific hydraulic characteristics necessary to calibrate the three-dimensional groundwater flow model for the area and to calculate concentrations of constituents discharging to the Mississippi River for use in the risk assessment. During June 15 through 19, 1992, a step-drawdown test, constant-rate aquifer test, and recovery

test were conducted. The site-specific aquifer coefficients determined from this testing include transmissivity, hydraulic conductivity, and storage coefficient.

2.1 FIELD TESTING

Prior to testing, two 6,000-gallon Calgon carbon adsorption units were delivered to the site, set up in series, and filled with 40,000 pounds of activated carbon to treat the discharge water on site. Piping was then installed from the well to the carbon units and from the carbon unit discharge line to a line which was connected to the American Bottoms treatment facility. The American Bottoms facility issued a permit for this discharge before testing was initiated.

A step-drawdown test was conducted to evaluate the optimum pumping rate for the constant-rate aquifer test. Based on this test, a flow rate of 350 gallons per minute (gpm) was selected for the constant-rate test. A network of 22 wells was monitored on a regular basis using three different types of monitoring equipment. Pressure transducers were used to monitor water-level changes in 16 monitoring wells, automatic Steven's water-level recorders were used on three wells, and manual measurements were collected in three wells. Table 2 provides a summary of the method used to monitor each well. The water-level measurements collected during the test are provided in Attachment A.

During the test, water samples were collected from the carbon unit influent, lead vessel effluent, and final effluent after 6, 24, and 48 hours, for laboratory analysis of volatile organic compounds (VOCs), semivolatile organic compounds (SVOCs), and herbicides. The final effluent sample collected 24 hours into the test was also analyzed for cyanide, ammonia, metals, and pesticides. Analysis was performed by Savannah Laboratories, Savannah, Georgia. Field analyses of the phenol in lead vessel effluent were conducted to monitor for breakthrough.

At approximately 12 hours into the test, the river stage began to rise as a result of a storm event that had occurred upriver several days earlier (Figure 4). Water levels within the wells

began to rise in response to the river, and the cone of depression that had been established began to diminish.

Drawdown in the intermediate and deep zones was plotted after 550 minutes of pumping to show the effect of pumping prior to the impact of rising river stage on these zones (Figure 5). Review of Figure 5 shows that approximately 1 ft of drawdown was induced at a distance of 100 to 150 ft from pumping well TW-1, and drawdown appeared to extend to the site boundaries. Approximately 0.2 ft of drawdown was observed in wells along the eastern border of Site R, approximately 0.4 ft of drawdown was observed in wells along the northern boundary, and approximately 0.1 ft of drawdown was observed in wells along the southern boundary of the site.

After approximately 1000 minutes of pumping, the rising river stage reduced drawdown in intermediate and deep zone wells, and the cone of influence decreased in size (Figure 6). Along the southern boundary of the area of influence, water levels rose to 0.2 ft above the static level in well GM-55C and 0.59 ft in well GM-28C. Along the northern boundary, water levels rose to 1.2 ft above the static level. The effect of the rising river stage is less apparent in the intermediate and deep zone wells in the vicinity of well TW-1, where drawdown data did not change significantly (Figure 6). Eastern perimeter wells exhibited increased drawdowns at 1000 minutes and were apparently unaffected by elevated river stage. This is most likely due to their distance from the river.

After 51 hours of pumping, the constant-rate drawdown test was completed, and recovery measurements were collected for 4 hours. This information was used to confirm the results of the drawdown test. Recovery water-level measurements are provided in Attachment A.

2.2 METHODS OF EVALUATION

Different types of aquifers respond to pumping in different ways. Several analytical solutions were used to evaluate the test data, to determine whether the aquifers could be characterized as confined or semi-confined.

Unconfined aquifer conditions were analyzed using the non-equilibrium method of Neuman or the methods of Theis and Cooper-Jacob with Jacob's correction for reduction in saturated thickness. The applicability of the semi-confined (leaky) solution of aquifer conditions was analyzed using the non-equilibrium method of Hantush, with storage in the overlying unpumped aquifer zone.

Except for the Jacob distance-drawdown solution, all of the methods were applied with the support of AQTESOLV, a Geraghty & Miller Modeling Group aquifer test analysis software package. Data utilized by the Jacob distance-drawdown solution were plotted on semilog paper.

AQTESOLV is an interactive, menu driven program that provides graphical curve matching techniques for quick and efficient analysis of aquifer test data. The option was utilized in which the analyst interactively matches type curves to the time-drawdown data directly on the computer screen. Data relevant to the configuration of the aquifer test are presented in Table 3.

2.2.1 Theis Method

If an unconfined aquifer does not exhibit a delayed water-table response, then the Theis Method for unsteady flow in confined aquifers can be applied once the drawdown data are corrected as follows:

$$S' = S - \frac{S^2}{2m}$$

where S' = equivalent confined aquifer drawdown

S = observed drawdown under unconfined conditions

m = aquifer thickness (pretesting)

Correction of drawdown data was unnecessary, however, because dewatering of the aquifer was insignificantly small in relation to the total saturated thickness of the aquifer. The Theis type-curve exhibited a close match with the log-log plots of drawdown versus time for the intermediate zone wells.

2.2.2 Neuman Method

Water levels near a pumping well in unconfined aquifers often tend to decline at a slower rate than that described by the Theis solution. Log-log plots of time-drawdown exhibit a three phase S-shape curve due to the phenomenon of "delayed water-table response." The second phase is characterized by gravity drainage of the pore spaces that is not instantaneous. A delay in the release of this stored water causes the increase of drawdown to slow with time, and thus deviate from the Theis curve (Kruseman and de Ridder 1990). Data from the three deep zone wells corresponded to the flow regime described by the Neuman solution. The applicability of the Neuman method to this aquifer is based on the premise that slow drainage from low permeability zones and horizontal-to-vertical anisotropy cause a delayed yield of water released from storage.

2.2.3 Hantush Method

When a well is pumped in a leaky aquifer, the well discharge comes from storage within the aquifer, vertical leakage from stored water in the aquitard, and leakage through the aquitard from the overlying unpumped aquifer. The leaky-confined aquifer analytical solution was also applied to the time-drawn data from the deep zone wells because aquifer deposits in the water-table zone beneath Site R consist of poorly sorted, fine grained material of low permeability, and drawdown in this zone was negligible.

2.2.4 Jacob (Distance-Drawdown) Method

Simultaneous drawdown measurements in several observation wells, each at a different distance from the pumped well, were plotted on semilog paper to show the straight line distance-drawdown relationship. This distance-drawdown graph was used to calculate the aquifer transmissivity and storativity. Distance-drawdown graphs were plotted for data from groups of intermediate and deep zone wells to determine the aquifer characteristics for those zones.

2.3 INTERPRETATION OF RESULTS

2.3.1 Water-Table Zone

Observation wells screened in the poorly sorted, fine grained material of this zone recorded maximum drawdown ranging from 0.08 to 0.15 ft. This small drawdown was not sufficient to establish drawdown behavior from natural fluctuations in water-level elevations. Thus, the aquifer characteristics of the water-table zone were not estimated through the analysis of aquifer test data.

2.3.2 Intermediate Zone

Time-drawdown measurements in the intermediate zone wells exhibited the characteristic shape of the Theis type-curve (Figures 7 through 11). The decline in measured drawdown beginning about 800 minutes after the start of pumping indicates the recovery in water-levels induced by aquifer recharge from the river. Table 4 presents trasmissivity values for all observation wells that produced a sufficient drawdown response; values ranged from about 22,000 to 38,200 square feet per day (ft^2/day). Values of storativity calculated with the Theis method range from .004 to .013 (Table 4). Estimates of transmissivity obtained with the Theis solution were closely reproduced with the Cooper-Jacob (semilog) method for Well B-24C (Figure 12) and the Neuman method for Well B-26B (Figure 13).

The Jacob distance-drawdown method was applied to a group of intermediate zone wells (P-5, P-9, P-10, B-26B) at 10, 100, and 500 minutes into the aquifer test. This method yielded inconsistent results that ranged from 24,702 ft^2/day to 51,463 ft^2/day (Figure 14). These estimates were not considered to be as reliable as individual well analysis.

2.3.3 Deep Zone

Several minutes into the aquifer test, the increase in drawdown in the deep zone wells began to slow and deviate from the Theis type-curve. This phenomenon, in which the drawdown curve approaches horizontal, is characteristic of the aquifer response to delayed gravity drainage of water released from storage (Figure 15 and 16). Calculation of transmissivities with the Neuman solution for deep zone Wells GM-56C and GM-57C were 23,961 ft^2/day and 29,736 ft^2/day. Under normal circumstances, the time-drawdown curve increases in slope and once again conforms to the Theis curve. However, the water-level recovery in the observation wells induced by recharge from the river masked the typical third phase of the Neuman curve.

The effect of leakage through an overlying confining unit on drawdown is comparable to that of delayed drainage. Therefore, the Hantush (leaky confined) solution was used to determine if it was the appropriate analytical model for this aquifer. The deep zone transmissivities calculated with the Hantush method yielded one low estimate of 15,580 ft^2/day (Well GM-56C), and two more representative estimates of 30,859 ft^2/day for Well GM-57C, and 31,162 ft^2/day for Well GM-28C (Figures 17 through 19). However, the Hantush type-curve did not fit the time-drawdown data as well as the Neuman type-curve. Although the Hantush solution yielded similar results to the Neuman solution, its applicability to this aquifer system for analysis of the aquifer test is not the appropriate selection. The Neuman theoretical model identifies most closely with this aquifer system and provides the best interpretation of the time-drawdown data.

The Jacob distance-drawdown analysis was also performed on the group of deep zone wells (GM-28C, GM-56C, and GM-57C) at 10, 100, and 500 minutes into the aquifer test (Figure 20). The method yielded consistent results but the transmissivities were lower than estimates computed

for individual time-drawdown plots. Values of transmissivity with the Jacob method ranged from 17,154 ft^2/day to 22,055 ft^2/day. The distance-drawdown results for the intermediate and deep zone wells were lower than individual well estimates. However, the groundwater flow model was calibrated with transmissivity values based on the higher estimates obtained from individual well plots. Thus, simulated remedial pumping rates will produce conservative estimates of capture zones since they are based on values in the higher range of transmissivity estimates.

3.0 GROUNDWATER DISCHARGE CALCULATIONS

As one of the first steps of the risk assessment, a list of chemicals of concern (COC) was selected for the groundwater at Site R. In order to complete the evaluation of risks associated with exposure to river water affected by the ground water, predicted concentrations of the COCs in the river were calculated. Geraghty & Miller used the groundwater model described in Appendix H and the concentrations of the COCs in the wells to complete these calculations.

Several steps were involved in the process. First, because the rate of groundwater discharge to the river changes with varying river stage, data were obtained from the U.S. Army Corps of Engineers (COE) which show the daily percent frequency of occurrence for every river stage on record in 1-ft increments, i.e., the percent of days in a given period that each river stage occurred. The data included the 130-year period from January 1861 to December 1991.

Using these data, a range of river stages was selected for the discharge calculations. The lower limit of this range was 374 ft above mean sea level (msl), the lowest river stage on record. The upper limit of the range was 410 ft above msl. Groundwater level data and the model indicate that the hydraulic gradient in the aquifer reverses above this level, so there would be no discharge to the river. These river stages and their frequency of occurrence are shown in columns 1 and 2 of Table 5.

The model was used to predict the groundwater discharge to the river at each river stage in the range. A separate calculation was done for each of the three hydrogeologic zones (Columns

3, 5, and 7 of Table 5). These predicted discharge rates at each river stage were then multiplied by the frequency of occurrence for that stage. These products (columns 4, 6, and 8 of Table 5) were summed to obtain a weighted average daily discharge for each aquifer zone. This represents the average volume of ground water which flows into the river each day from each aquifer zone along the entire length of the landfill (2,000 ft). In the next step, the length of the river frontage was divided into segments. Each hydrogeologic zone was treated separately and was divided into one segment for each well screened in that zone. The percent of river frontage represented by each well segment was multiplied by the average daily discharge for that aquifer zone and then by the concentration in that well of each COC. These products were summed to obtain a weighted average daily loading of each COC to the river for each aquifer zone. These were then summed across the three zones to obtain a total average daily loading to the river for each COC.

To obtain the predicted concentration of each COC in the river, these daily loadings will be divided by the flow rate in the river. Both average exposure and reasonable maximum exposure (RME) scenarios will be considered in the risk assessment. Calculations of the river concentrations of each COC will be shown in the risk assessment.

The discharge across all zones for all river stages was summed in Column 9 of Table 5. This number (795,000 gallons/day) will be used for calculating percent dilution in the evaluation of aquatic hazard indices in the risk assessment).

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Table 1. Water-Level Elevations in Monitoring Wells at Sauget Site R, June 3, 1992, Monsanto Company, Sauget, Illinois.

Well No.	Measuring Point Elevation (1)	Depth to Water (2)	Water Level Elevation (1)
Shallow Wells			
B-21A	428.53	29.93	398.60
B-22A	428.16	28.79	399.37
B-24A	422.49	22.17	400.32
B-25A	428.47	30.95	397.52
B-26A	423.71	26.37	397.34
B-28A	423.04	25.25	397.79
B-29A	429.03	31.97	397.06
B-31B	421.68	28.45	393.23
P-6	421.78	24.92	396.86
P-7	421.82	22.96	398.86
P-14	424.36	24.93	399.43
GM-62A	425.75	31.99	393.76
GM-65A	424.11	32.66	391.45
GM-66A	422.73	27.79	394.94
Intermediate Wells			
GM-27B	426.04	36.63	389.41
GM-28B	423.88	34.15	389.73
B-21B	428.37	36.12	392.25
B-22B	428.16	37.55	390.61
B-23B	428.17	33.97	394.20
B-24B	422.28	31.70	390.58
B-24C	422.52	32.02	390.50
B-25B	427.35	34.84	392.51
B-26B	423.62	33.29	390.33
B-27B	425.83	34.26	391.57
B-28B	423.08	32.55	390.53
B-29B	429.06	36.69	392.37
B-30B	430.52	38.02	392.50
B-31C	421.88	28.70	393.18
P-1	423.11	33.18	389.93
P-2	423.15	33.26	389.89
P-3	423.43	33.27	390.16
P-4	421.82	31.70	390.12
P-5	422.12	32.31	389.81
P-8	421.79	32.02	389.77
P-9	423.14	33.38	389.76
P-10	423.43	33.19	390.24
P-11	422.30	32.68	389.62
P-12	423.75	34.45	389.30
P-13	424.32	34.90	389.42
GM-62B	426.16	32.42	393.74
GM-66B	423.20	34.05	389.15
TW-1	423.14	32.47	390.67
Deep Wells			
GM-27C	426.76	36.60	390.16
GM-28C	423.78	33.98	389.80
GM-55C	422.88	32.32	390.56
GM-56C	422.16	31.16	391.00
GM-57C	424.02	34.06	389.96
GM-62C	427.03	33.30	393.73
GM-66C	423.46	34.37	389.09
Bedrock Wells			
GM-106	424.82	29.73	395.09
B-102	423.84	31.74	392.10

(1) Elevation in feet above mean sea level.
(2) Depth to water in feet below measuring point.
The water-level for Well B-21A may be representative of a water level in the 2-ft section of blank casing at the bottom of the well, and not representative of the water table zone. This water level was not used in the groundwater model.

Table 2. Wells Monitored During the June 1992 Aquifer Test, Sauget Site R, Monsanto Company, Sauget, Illinois.

Well Number	Pressure Transducer	Stevens Recorder	Manual Measurement
Water-Table Zone			
P-7	X	-	-
B-24A	X	-	-
B-25A	X	-	-
B-26A	X	-	-
Intermediate Zone			
P-5	X	-	-
P-8	X*	-	-
P-9	X	-	-
P-10	X	-	-
B-24C	X*	-	-
B-25B	X	-	-
B-26B	X*	-	-
B-30B	-	-	X
B-31C	-	-	X
GM-27B	-	X	-
GM-28B	X	-	-
Test Well 1	X	-	-
Deep Zone			
GM-27C	-	X	-
GM-28C	X	-	-
GM-55C	-	-	X
GM-56C	X*	-	-
GM-57C	X	-	-
Bedrock Zone			
GM-106	-	X	-

* Backup transducer was installed

Table 3. Data Used to Define the Configuration of the June 1992 Aquifer Test, Sauget, Illinois.

Well Number	Distance From Pumped Well to Observation Well	Maximum Drawdown	Saturated Thickness	Well Depth	Depth From Static Water Level to Top of Screen	Depth From Static Water Level to Bottom of Screen
Water-Table Zone						
B-24A	118	.08	90	27.5	0	2.6
B-25A	625	.07	90	35.2	0	1.7
B-26A	355	.15	90	33.2	0	3.8
P-7	102	.05	90	33	2.7	7.7
Intermediate Zone						
B-24C	118	.98	90	69	22.9	32.9
P-10	143	.97	90	54	11.4	16.4
P-9	104	.99	90	50	7.7	12.7
B-25B	625	.33	90	49.5	0.8	10.8
P-5	272	.36	90	54.5	13.3	18.3
B-26B	433	.46	90	49.8	2.4	12.4
GM-28B	772	.22	90	93	34.4	54.4
P-8	112	–	90	53.5	12.5	17.5
TW-1*	0	34.8	90	108	41.5	73.5
Deep Zone						
GM-57C	368	.77	90	116	60	80
GM-28C	772	.21	90	107	51	71
GM-56C	150	1.21	90	111	58.1	78.1

* Pumping Rate = 350 gpm; screened interval is from the lower intermediate zone to the upper deep zone.
– Not reported due to problems with pressure transducer.

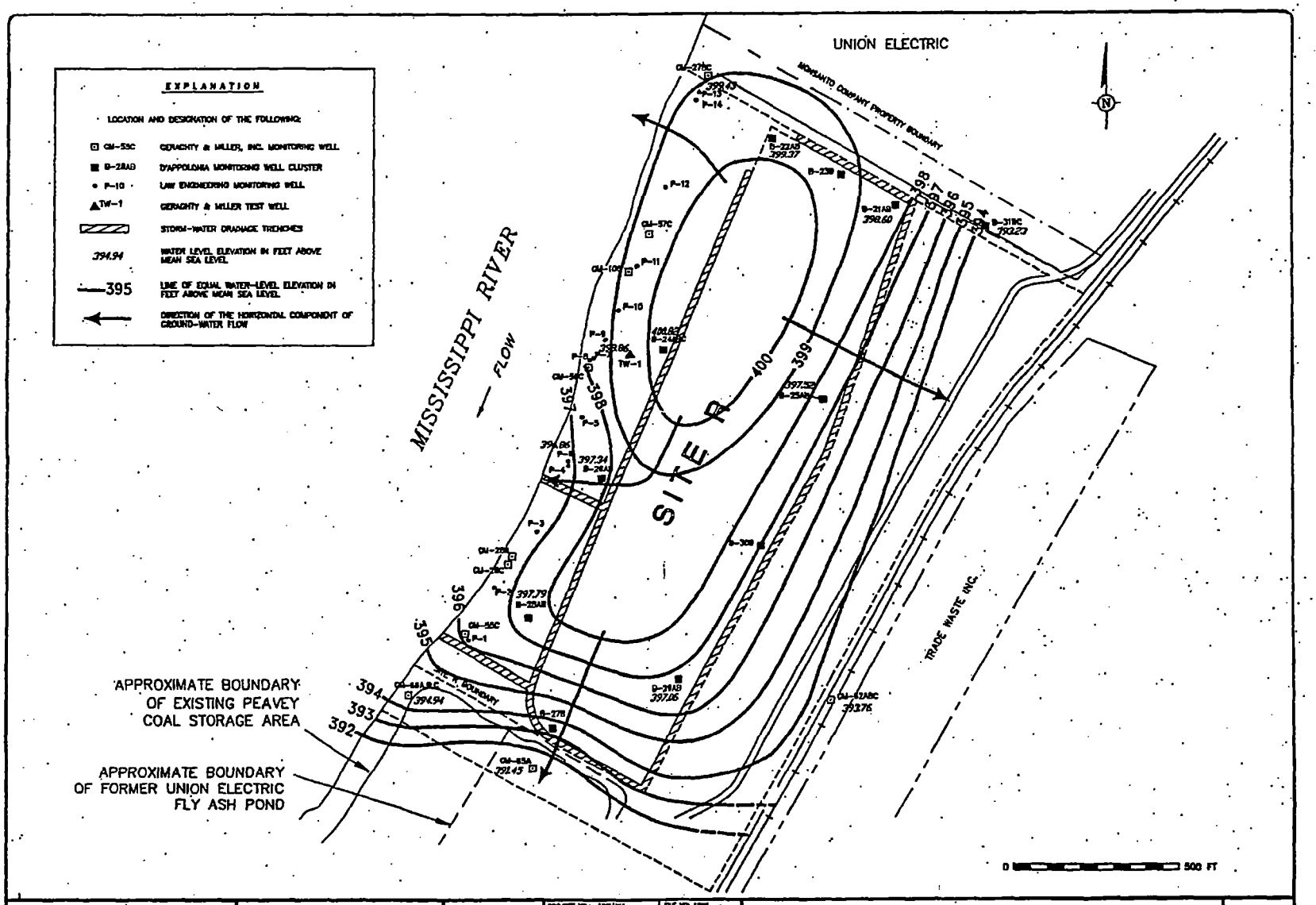
Table 4. Estimates of Aquifer Characteristics Obtained Through Interpretation of Observation Well Drawdown Data From the June 1992 Aquifer Test, Sauget Site R, Monsanto Company, Sauget, Illinois.

Well Number	Method	Aquifer Test	Transmissivity Ft²/Min	Transmissivity Ft²/Day	Hydraulic Conductivity Ft/Day	Storage Coefficient
Intermediate Zone						
P-5	Theis	Unconfined	26.55	38,232	425	.012
P-9	Theis	Unconfined	15.28	22,003	244	.0134
P-10	Theis	Unconfined	15.19	21,874	243	.0083
B-24C	Theis	Unconfined	23.8	34,272	381	.0042
	Cooper-Jacob	Unconfined	22.53	32,443	360	.0045
B-26B	Theis	Unconfined	22.22	31,996	356	.0065
	Neuman	Unconfined	20.3	29,232	325	.007 (Sy)
Deep Zone						
GM-57C	Hantush	Leaky	21.43	30,859	343	.0004
	Neuman	Unconfined	20.65	29,736	330	.0055 (Sy)
GM-56C	Hantush	Leaky	10.82	15,580	173	.0013
	Neuman	Unconfined	16.64	23,961	266	.016 (Sy)
GM-28C	Hantush	Leaky	21.64	31,162	346	.0001
Distance-Drawdown Evaluation						
Intermediate Wells	Jacob	Unconfined				
10 minutes			35.74	51,463	572	.0055
100 minutes			23.8	34,309	381	.0025
500 minutes			17.2	24,702	274	.0146
Deep Wells	Jacob	Unconfined				
10 minutes			15.32	22,055	245	.0005
100 minutes			12.61	18,163	202	.0028
500 minutes			11.91	17,154	191	.0104

Table 5. Estimate of Average Daily Ground-Water Discharge to Mississippi River, Sauget Site R, Monsanto Company, Sauget, Illinois.

1 River-Stage Elevation (ft above MSL)	2 Frequency of Occurence (% of days)	3 Q Shallow	4 Q x % Shallow	5 Q Intermediate	6 Q x % Intermediate	7 Q Deep	8 Q x % Deep	9 Total Q x %
374	0.2	0.00024	0.00000048	0.6496	0.0012992	0.6228	0.0012456	0.00254528
375	0.5	0.00037	0.00000185	0.634	0.00317	0.608	0.00304	0.00621185
376	1.5	0.0005	0.0000075	0.6184	0.009276	0.5932	0.008898	0.0181815
377	2.2	0.00063	0.00001386	0.6028	0.0132616	0.5784	0.0127248	0.02600026
378	2.3	0.00076	0.00001748	0.5872	0.0135056	0.5636	0.0129628	0.02648588
379	3	0.00089	0.0000267	0.5716	0.017148	0.5488	0.016464	0.0336387
380	3.8	0.00102	0.00003876	0.556	0.021128	0.534	0.020292	0.04145876
381	3.9	0.00109	0.00004251	0.5406	0.0210834	0.5194	0.0202566	0.04138251
382	4.6	0.00116	0.00005336	0.5252	0.0241592	0.5048	0.0232208	0.04743336
383	4.2	0.00124	0.00005208	0.5098	0.0214116	0.4902	0.0205884	0.04205208
384	4.9	0.00131	0.00006419	0.4944	0.0242256	0.4756	0.0233044	0.04759419
385	4.9	0.00138	0.00006762	0.479	0.023471	0.461	0.022589	0.04612762
386	5.5	0.00142	0.0000781	0.4638	0.025509	0.4462	0.024541	0.0501281
387	4.5	0.00145	0.00006525	0.4486	0.020187	0.4314	0.019413	0.03966525
388	4.4	0.00149	0.00006556	0.4334	0.0190696	0.4166	0.0183304	0.03746556
389	4.1	0.00152	0.00006232	0.4182	0.0171462	0.4018	0.0164738	0.03368232
390	4.1	0.00156	0.00006396	0.403	0.016523	0.387	0.015867	0.03245396
391	3.5	0.00158	0.0000553	0.3878	0.013573	0.3724	0.013034	0.0266623
392	3.5	0.00159	0.00005565	0.3726	0.013041	0.3578	0.012523	0.02561965
393	3.5	0.00161	0.00005635	0.3574	0.012509	0.3432	0.012012	0.02457735
394	3.6	0.00162	0.00005832	0.3422	0.0123192	0.3286	0.0118296	0.02420712
395	2.7	0.00164	0.00004428	0.327	0.008829	0.314	0.008478	0.01735128
396	2.9	0.00164	0.00004756	0.3116	0.0090364	0.2992	0.0086768	0.01776076
397	2.5	0.00164	0.000041	0.2962	0.007405	0.2844	0.00711	0.014556
398	2.2	0.00164	0.00003608	0.2808	0.0061776	0.2696	0.0059312	0.01214488
399	2.1	0.00164	0.00003444	0.2654	0.0055734	0.2548	0.0053508	0.01095864
400	1.8	0.00164	0.00002952	0.25	0.0045	0.24	0.00432	0.00884952
401	1.8	0.00163	0.00002934	0.235	0.00423	0.2256	0.0040608	0.00832014
402	1.3	0.00161	0.00002093	0.22	0.00286	0.2112	0.0027456	0.00562653
403	1.4	0.0016	0.0000224	0.205	0.00287	0.1968	0.0027552	0.0056476
404	1.1	0.00158	0.00001738	0.19	0.00209	0.1824	0.0020064	0.00411378
405	1.2	0.00157	0.00001884	0.175	0.0021	0.168	0.002016	0.00413484
406	0.9	0.00154	0.00001386	0.1602	0.0014418	0.1537	0.0013833	0.00283896
407	0.9	0.00152	0.00001368	0.1454	0.0013086	0.1394	0.0012546	0.00257688
408	1.1	0.00149	0.00001639	0.1306	0.0014366	0.1251	0.0013761	0.00282909
409	0.9	0.00147	0.00001323	0.1158	0.0010422	0.1108	0.0009972	0.00205263
410	0.8	0.00144	0.00001152	0.101	0.000808	0.0965	0.000772	0.00159152
			0.1%		50%		50%	
Weighted Average Daily Discharge	98.3		0.00135765 (Shallow)		0.4047248 (Intermediate)		0.3888442 (Deep)	0.79492665 (Total)

[1] - Rate of ground-water discharge to river (million gallons per day).



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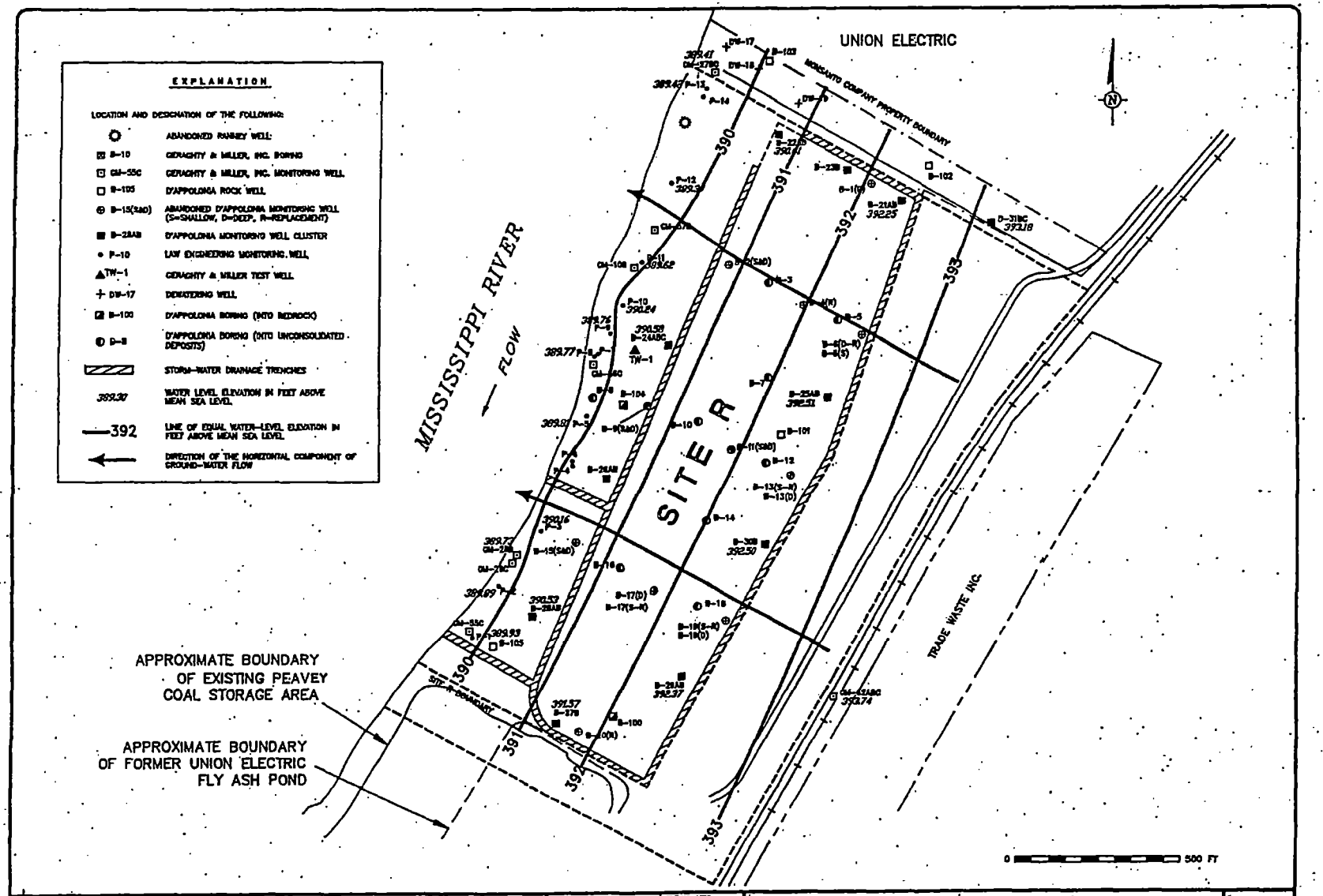
APPROVED BY	DC	DATE	

WATER LEVEL ELEVATIONS IN THE SHALLOW ZONE ON JUNE 3, 1992

MONSANTO CHEMICAL COMPANY
SAUGET SITE R, ILLINOIS

FIGURE 1



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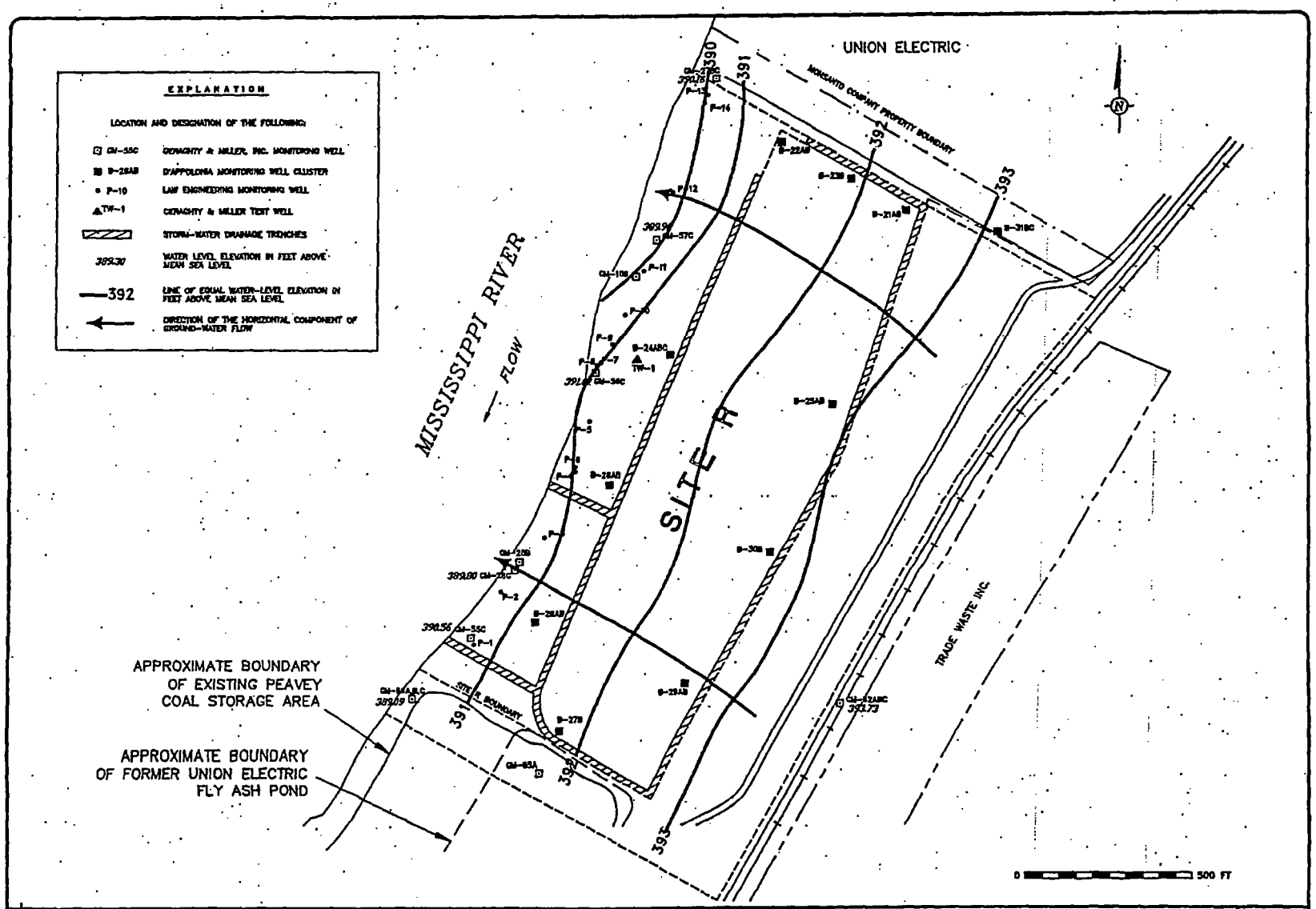
DRAWING	PLOT SCO
DRAFTED BY	DATE
CHECKED BY	DATE
APPROVED BY	DATE

WATER-LEVEL ELEVATIONS IN THE INTERMEDIATE ZONE ON JUNE 3, 1992

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 2



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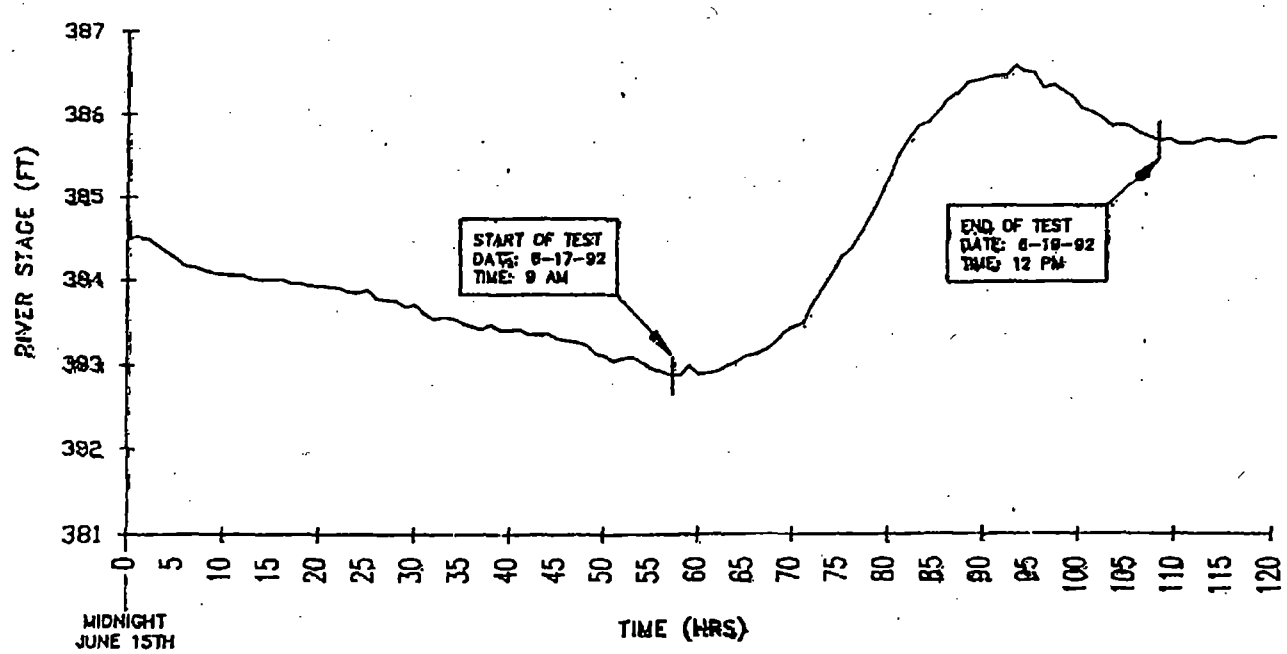
CHECKED BY: [illegible]	DATE:
APPROVED BY: [illegible]	DATE:

WATER-LEVEL ELEVATIONS IN THE DEEP ZONE ON JUNE 3, 1992

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 3

DWG DATE: 2-9-93 | PRJCT NO.: NY84207 | FILE NO.: 1558 | DRAWING: MISSRIV | CHECKED: S. FELDMAN | APPROVED: T. KRAMER | DRAFTER: V.C./G.S.



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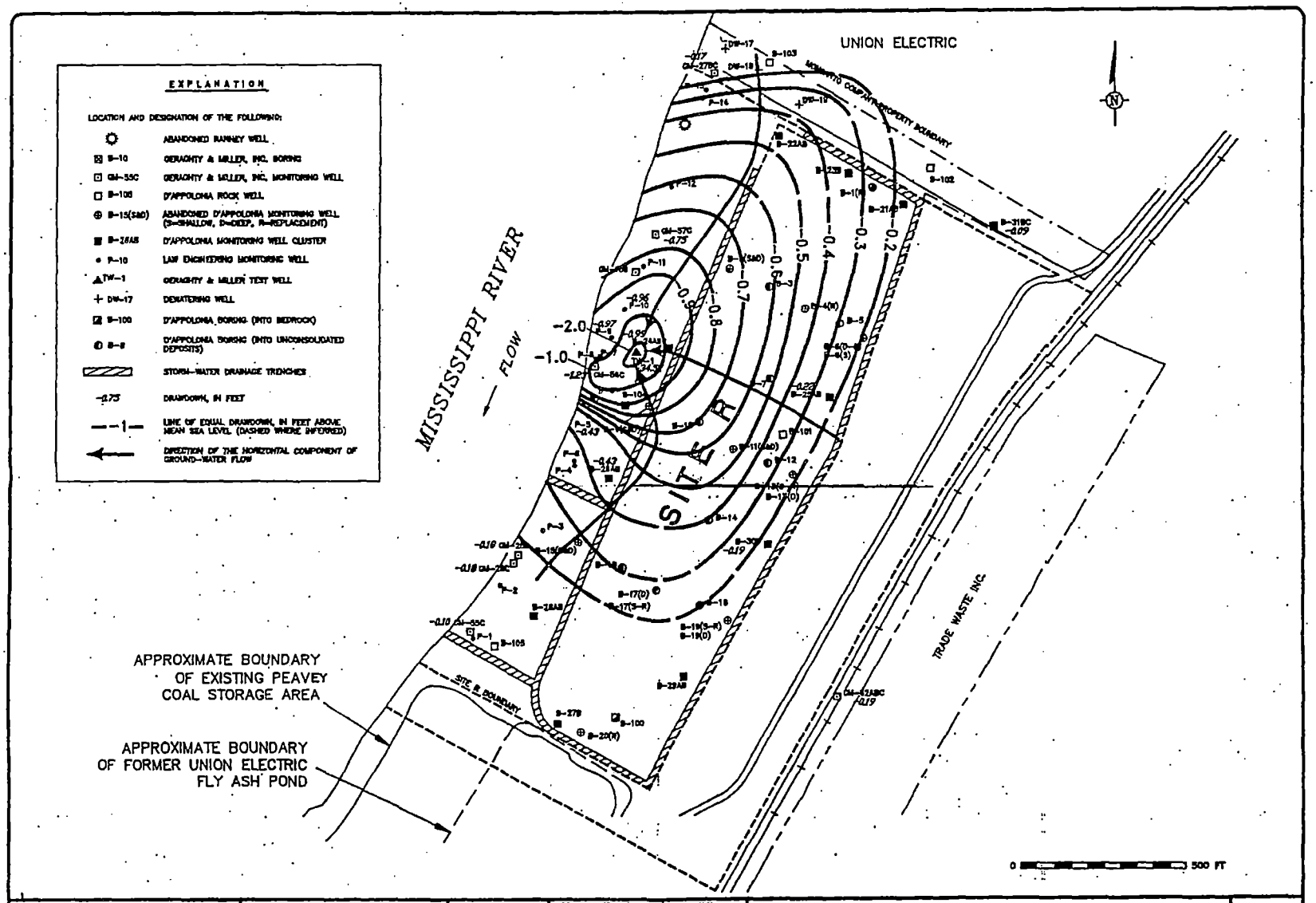
MISSISSIPPI RIVER STAGE AT THE ST. LOUIS STATION
FROM JUNE 15 THROUGH 19, 1992

MONSANTO
SAUGET, ILLINOIS

FIGURE
4



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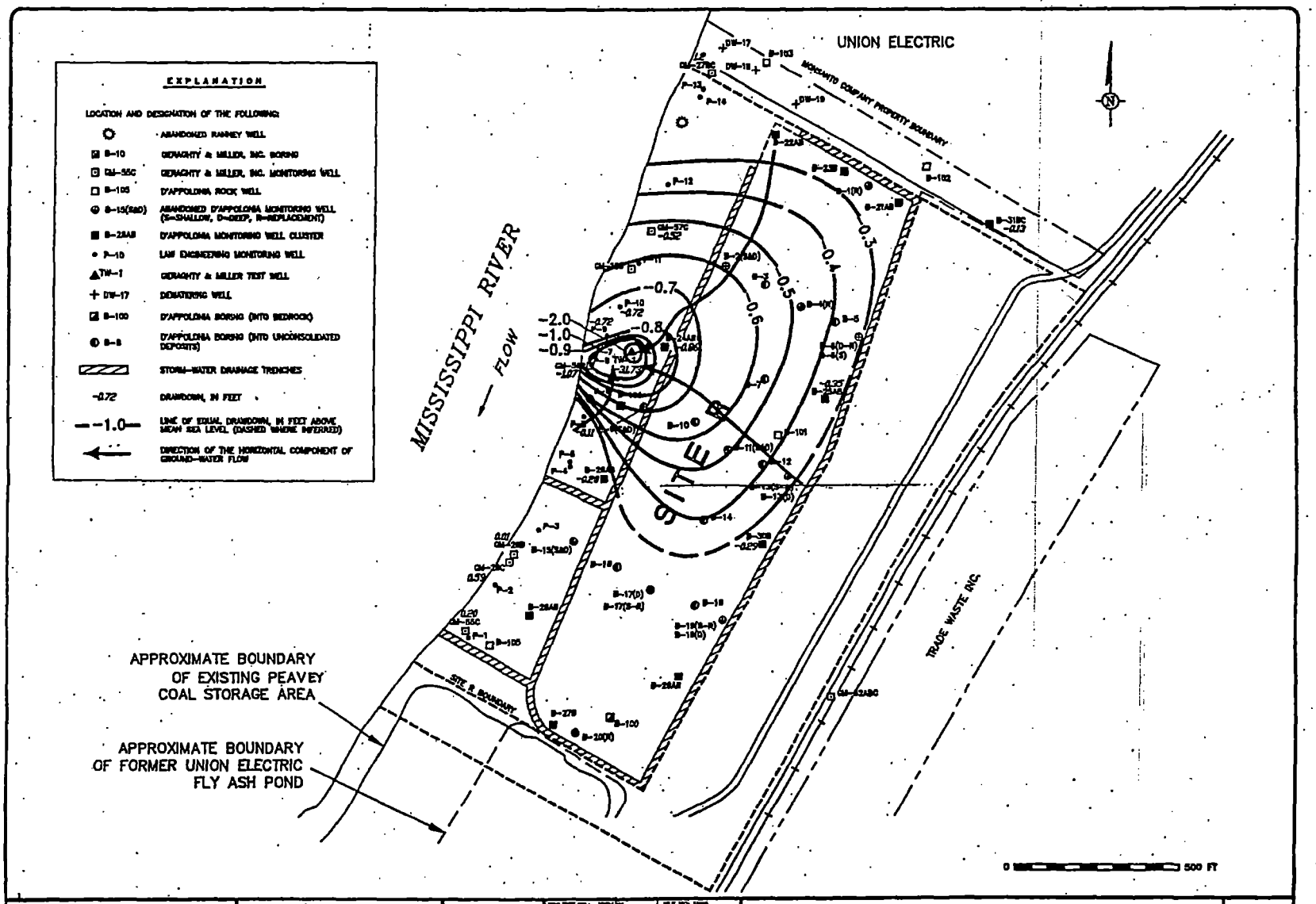
DRAWDOWN IN THE INTERMEDIATE AND DEEP AQUIFERS AFTER 550 MINUTES OF PUMPING ON JUNE 17, 1992

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 5



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CHECKED BY	DATE
APPROVED BY	DATE

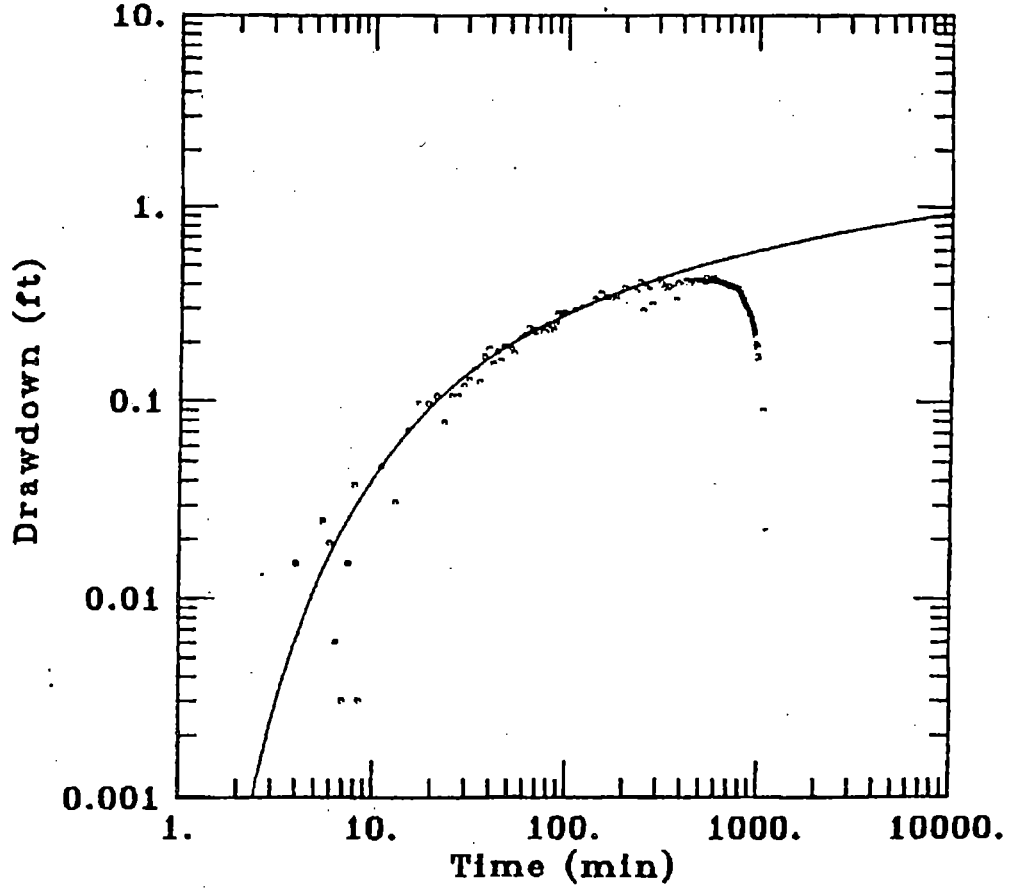
DRAWDOWN IN THE INTERMEDIATE AND DEEP AQUIFERS AFTER 1,000 MINUTES OF PUMPING ON JUNE 18, 1992

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 6

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

P-5



DATA SET:
P-5.AGT
07/08/92

AQUIFER TYPE:
Unconfined

SOLUTION METHOD:
Theis

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
P-5

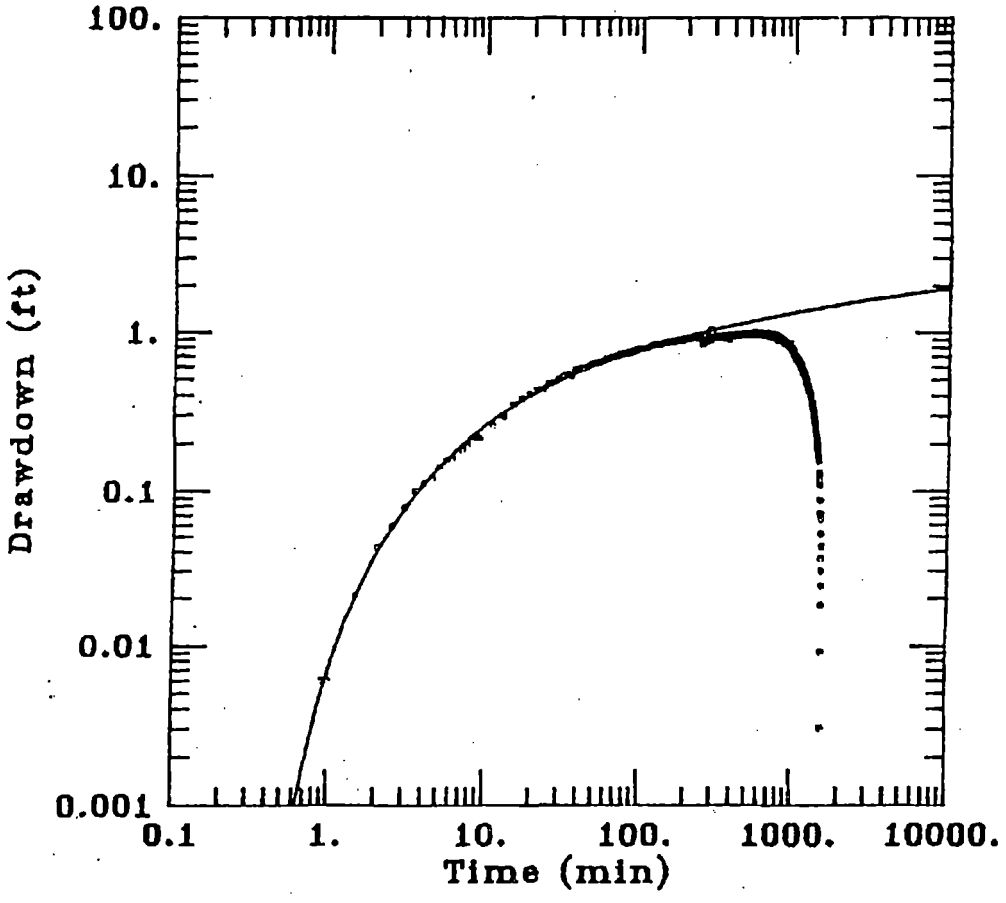
ESTIMATED PARAMETERS:
T = 26.55 ft^2/min
S = 0.01176

TEST DATA:
Q = 46.8 ft^3/min
r = 272. ft
b = 90. ft

Figure 7

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

P-9



DATA SET:
p-9.aqt
07/01/92

AQUIFER TYPE:
Unconfined

SOLUTION METHOD:
Theis

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
P-9

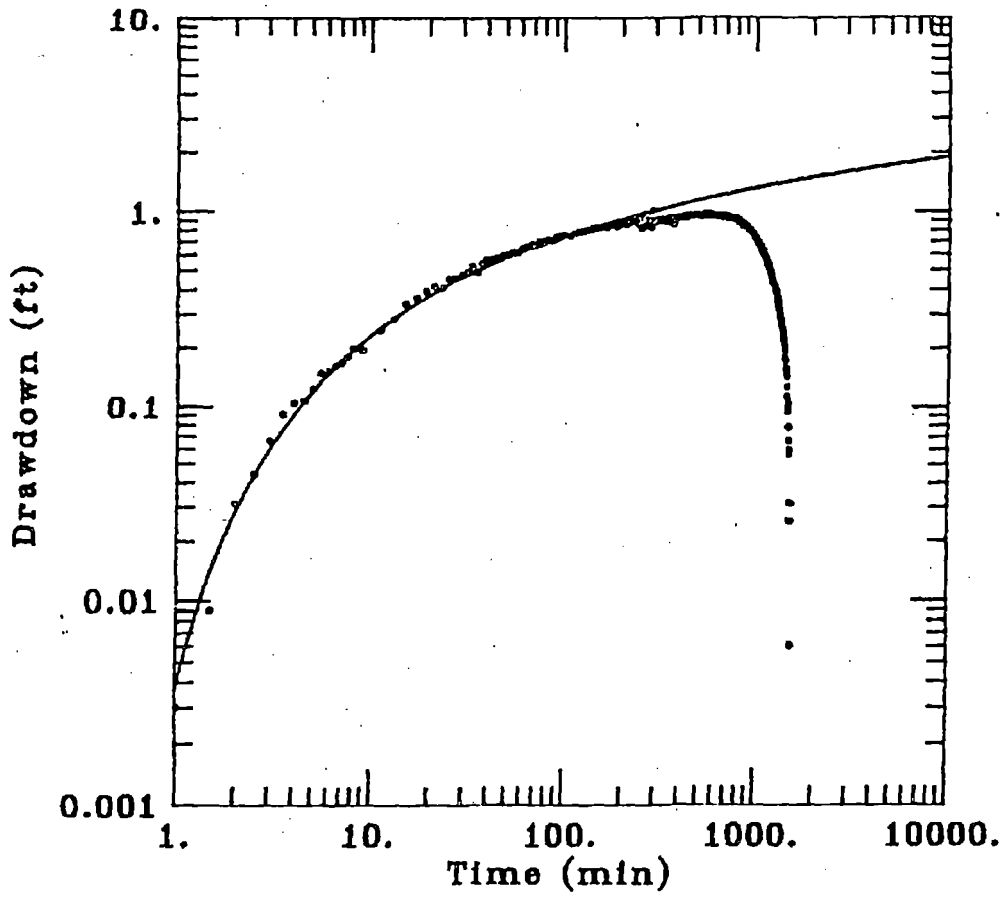
ESTIMATED PARAMETERS:
T = 15.28 ft^2/min
S = 0.01342

TEST DATA:
Q = 46.8 ft^3/min
r = 104. ft
b = 90. ft

Figure 8

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

P-10



DATA SET:
p-10.aqt
07/01/92

AQUIFER TYPE:
Unconfined

SOLUTION METHOD:
Theis

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
P-10

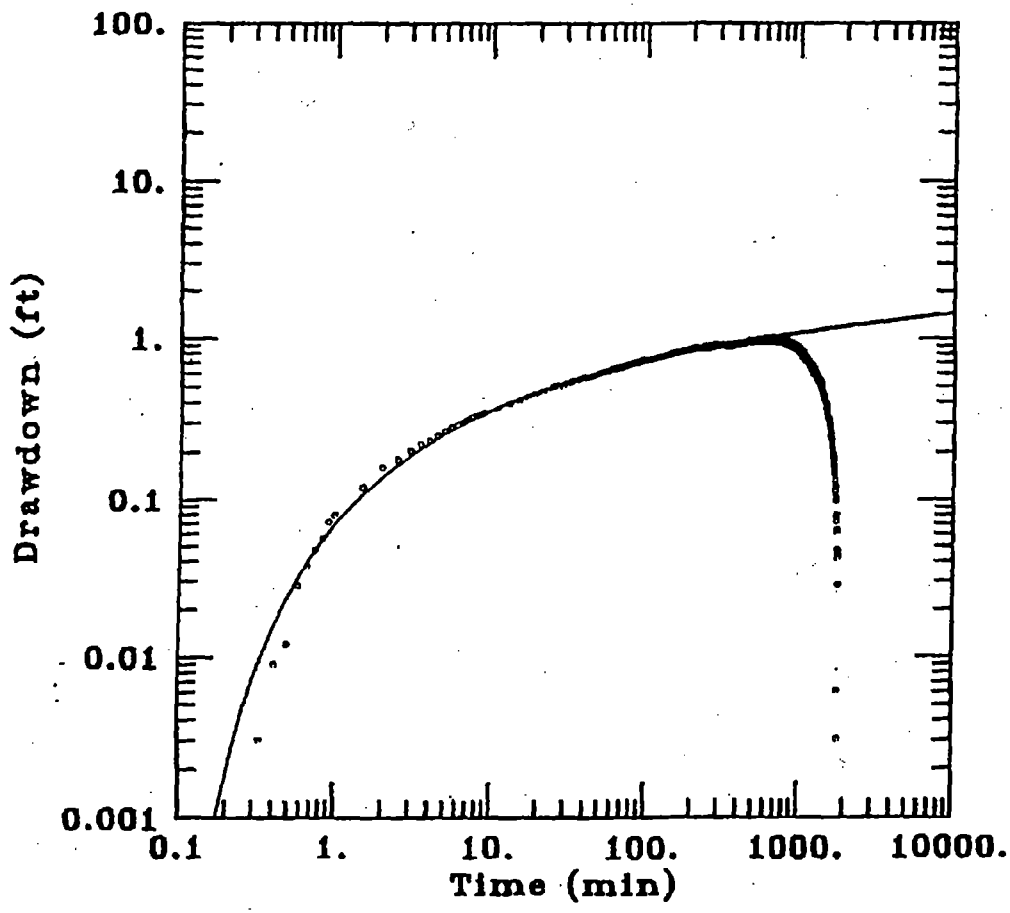
ESTIMATED PARAMETERS:
T = 15.18 ft^2/min
S = 0.008326

TEST DATA:
Q = 46.8 ft^3/min
r = 143. ft
b = 90. ft

Figure 9

G & M	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

B-24C



DATA SET:
B-24C.AQT
07/08/92

AQUIFER TYPE:
Unconfined

SOLUTION METHOD:
Theis

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
B-24C

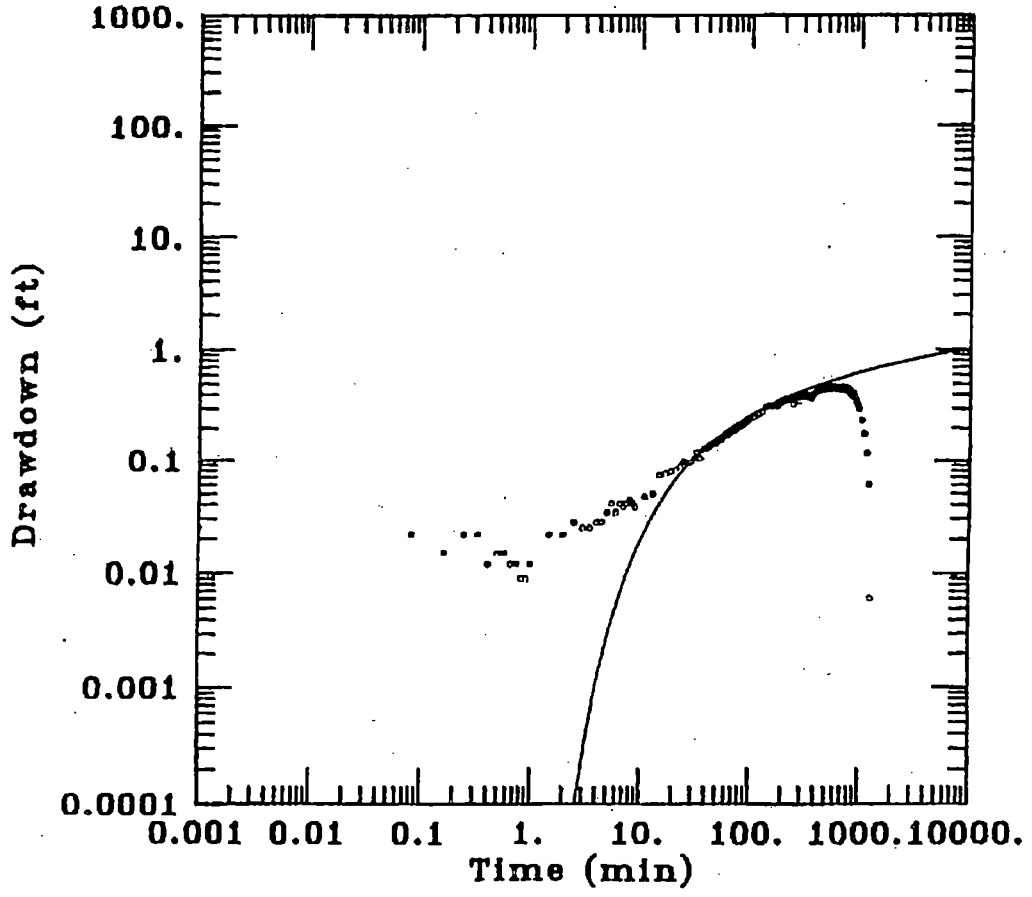
ESTIMATED PARAMETERS:
T = 23.8 ft^2/min
S = 0.004155

TEST DATA:
Q = 46.8 ft^3/min
r = 118. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft

Figure 10

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

B-26B



DATA SET:
B-26B.AQT
07/08/92

AQUIFER TYPE:
Unconfined

SOLUTION METHOD:
Theis

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
B-26B

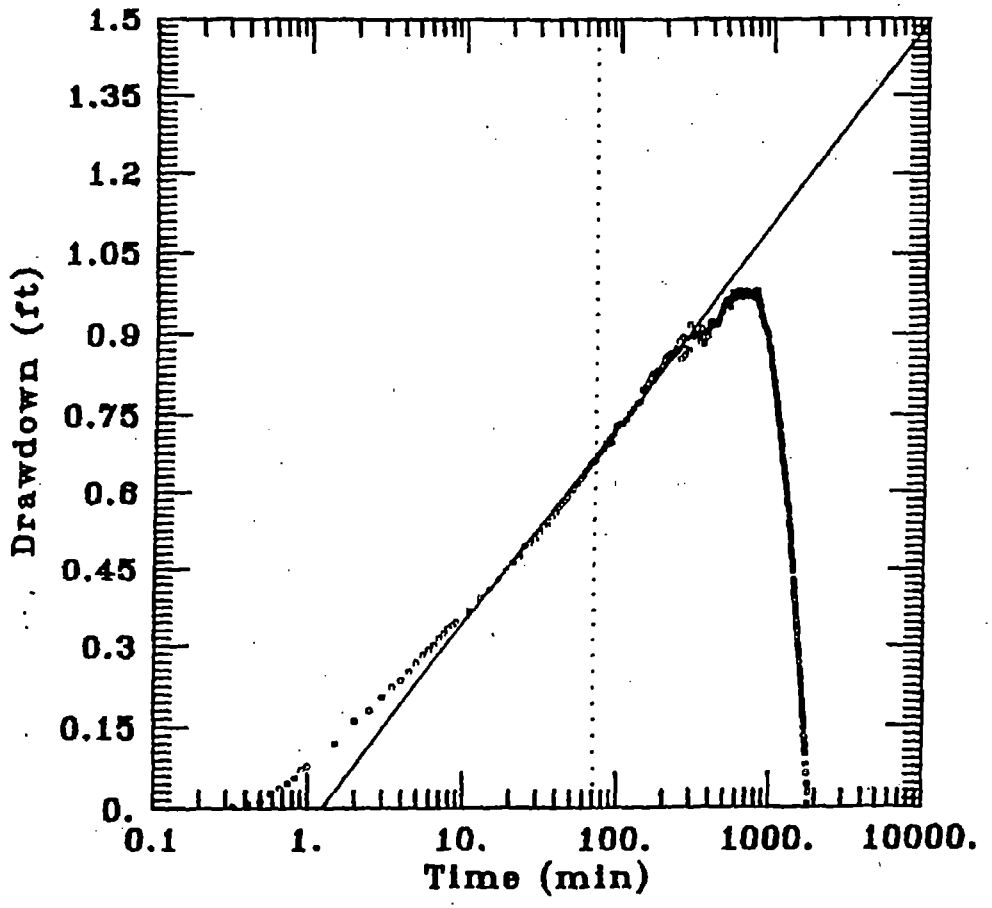
ESTIMATED PARAMETERS:
T = 22.22 ft^2/min
S = 0.006535

TEST DATA:
Q = 46.8 ft^3/min
r = 433. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft

Figure 11

G & M	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

B-24C



DATA SET:
B-24C.AGT
07/08/92

AQUIFER TYPE:
Unconfined

SOLUTION METHOD:
Cooper-Jacob

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
B-24C

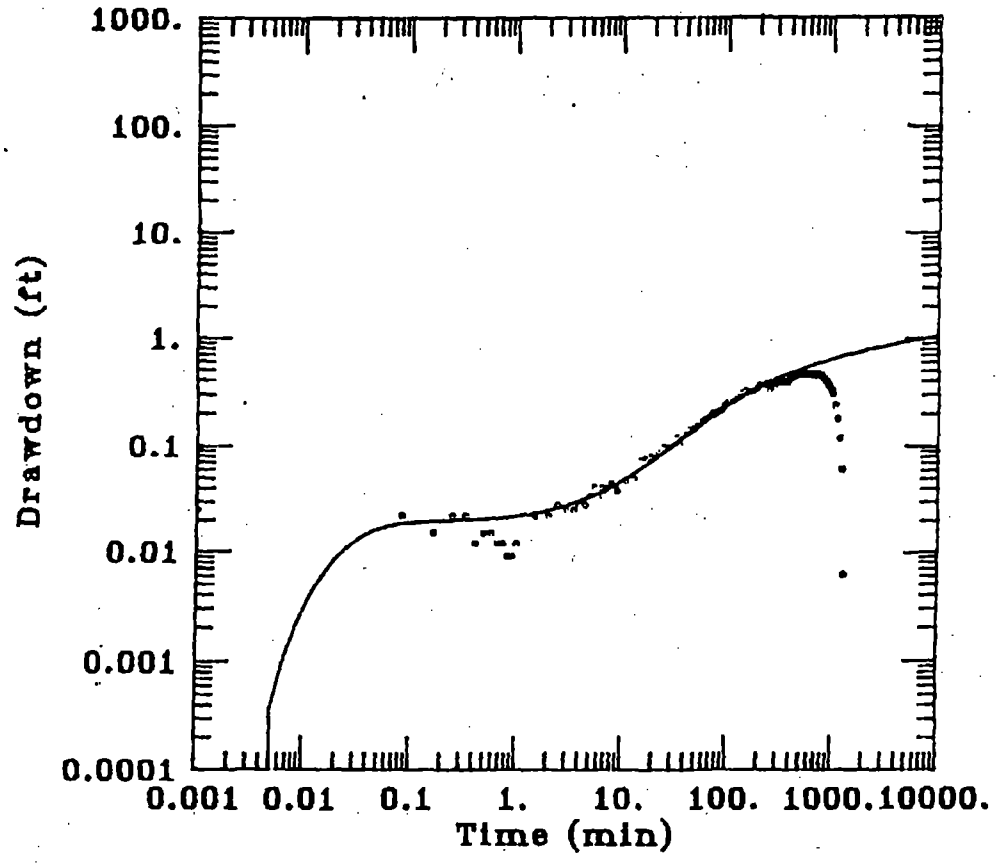
ESTIMATED PARAMETERS:
T = 22.53 ft^2/min
S = 0.004504

TEST DATA:
Q = 46.8 ft^3/min
r = 118. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft

Figure 12

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

B-26B



DATA SET:
b-26b.aqt
07/01/92

AQUIFER TYPE:
Unconfined

SOLUTION METHOD:
Neuman

TEST DATE:
JUNE 17, 1992

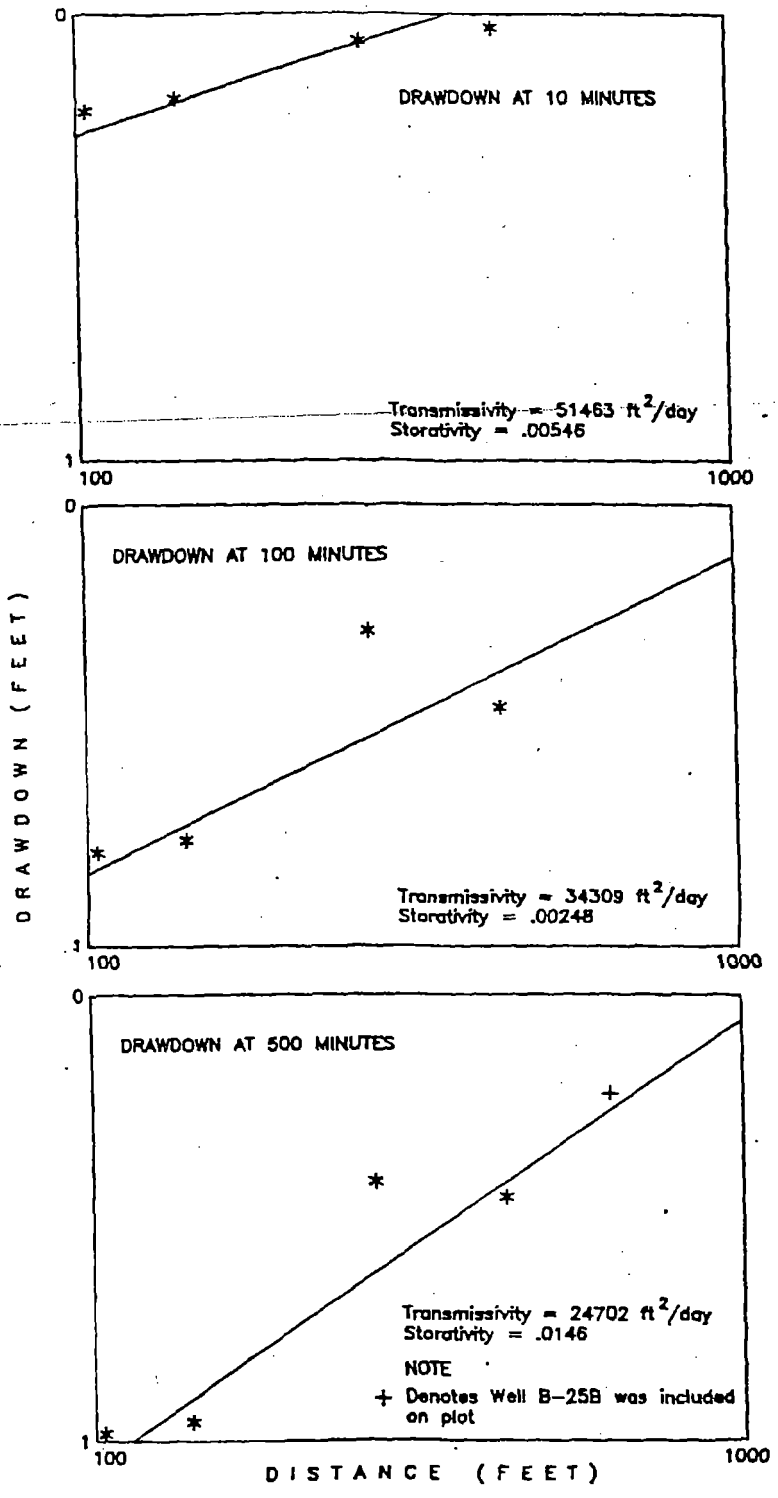
TEST WELL:
TW-1

OBS. WELL:
B-26B

ESTIMATED PARAMETERS:
T = 20.3 ft^2/min
S = 6.8199E-06
Sy = 0.007037
β = 0.8

TEST DATA:
Q = 46.8 ft^3/min
r = 433. ft
rc = 0.5 ft
rw = 0.5 ft
b = 99. ft
TEST WELL:
t.o.s. = 41.5 ft
b.o.s. = 73.5 ft
OBS. WELL:
t.o.s. = 2.4 ft
b.o.s. = 12.4 ft

Figure 13



GERAGHTY & MILLER, INC.
Environmental Services

JACOB DISTANCE-DRAWDOWN PLOTS OF FOUR INTERMEDIATE ZONE WELLS AT 10, 100, AND 500 MINUTES INTO THE AQUIFER TEST

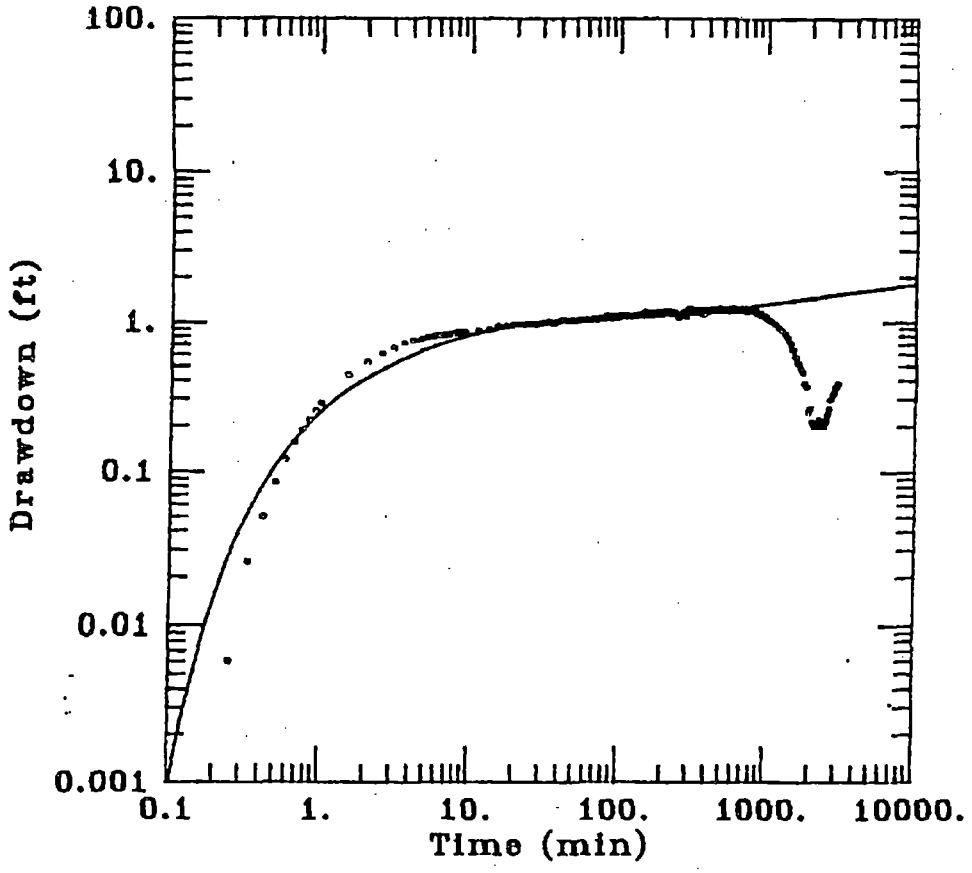
MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 14

DRAFTER: GS | APPROVED: LM | CHECKED: SF | DRAWING: IN. .DB | FILE NO.: 1558 | PRJCT NO.: NY64207 | DWG L 2-8-93

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

GM-56C



DATA SET:
GM-56C.AQT
06/29/92

AQUIFER TYPE:
Unconfined

SOLUTION METHOD:
Neuman

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
GM-56C

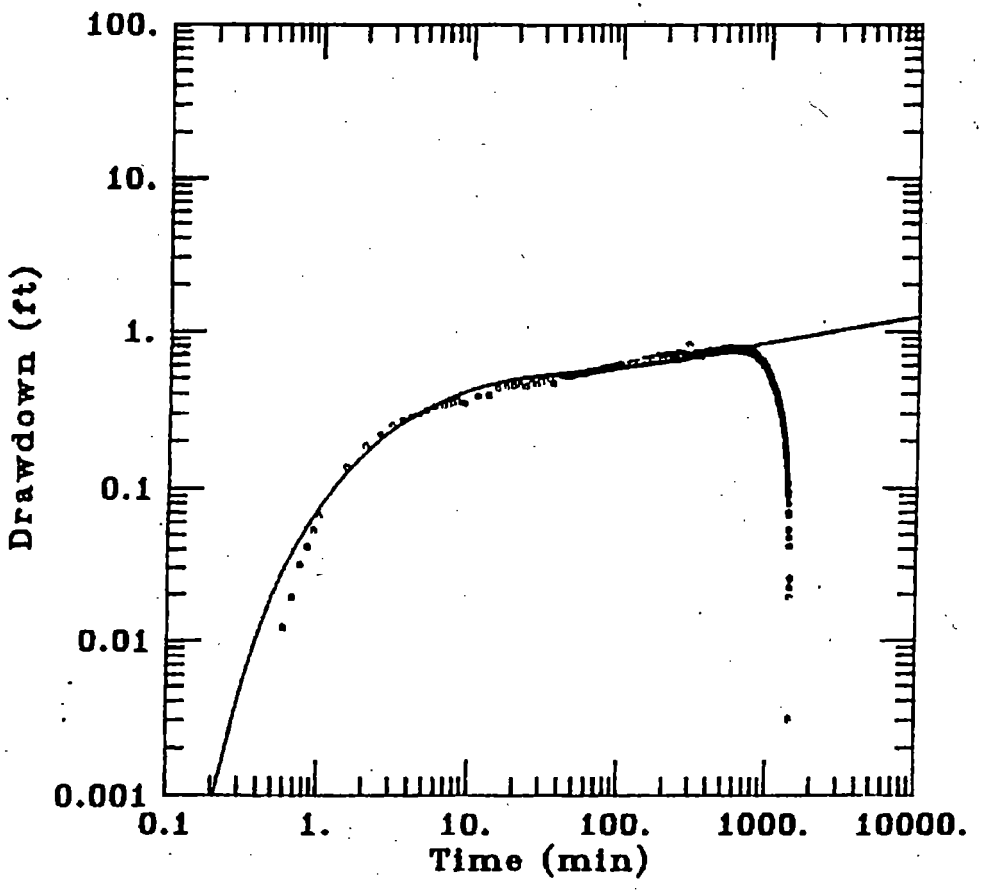
ESTIMATED PARAMETERS:
T = 16.64 ft^2/min
S = 0.001341
Sy = 0.01647
β = 0.03

TEST DATA:
Q = 46.8 ft^3/min
r = 150.5 ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft
TEST WELL:
t.o.s. = 41.5 ft
b.o.s. = 73.6 ft
OBS. WELL:
t.o.s. = 58.1 ft
b.o.s. = 78.1 ft

Figure 15

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

GM-57C



DATA SET:
gm-57c.aqt
07/01/92

AQUIFER TYPE:
Unconfined

SOLUTION METHOD:
Neuman

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
GM-57C

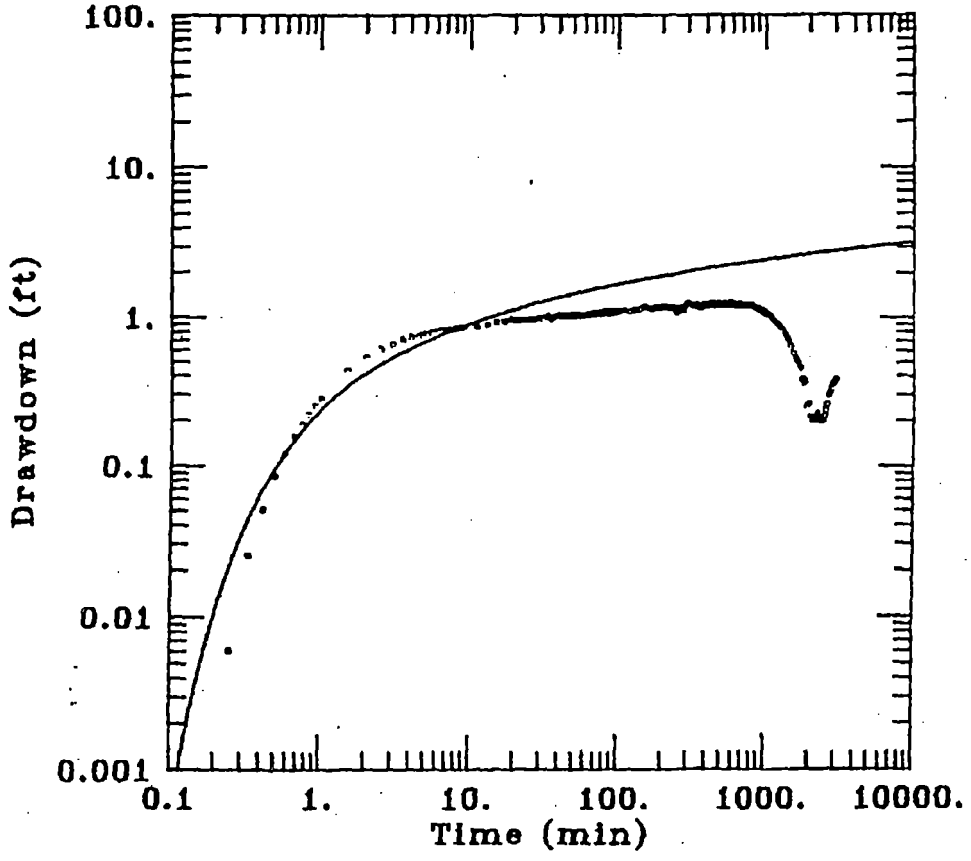
ESTIMATED PARAMETERS:
T = 20.65 ft^2/min
S = 0.0005388
Sy = 0.005517
β = 0.1

TEST DATA:
Q = 46.8 ft^3/min
r = 368. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft
TEST WELL:
t.o.s. = 41.5 ft
b.o.s. = 73.5 ft
OBS. WELL:
t.o.s. = 60. ft
b.o.s. = 80. ft

Figure 16

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

GM-56C



DATA SET:
gm-56c.aqt
06/25/92

AQUIFER TYPE:
Leaky

SOLUTION METHOD:
Hantush

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
GM-56C

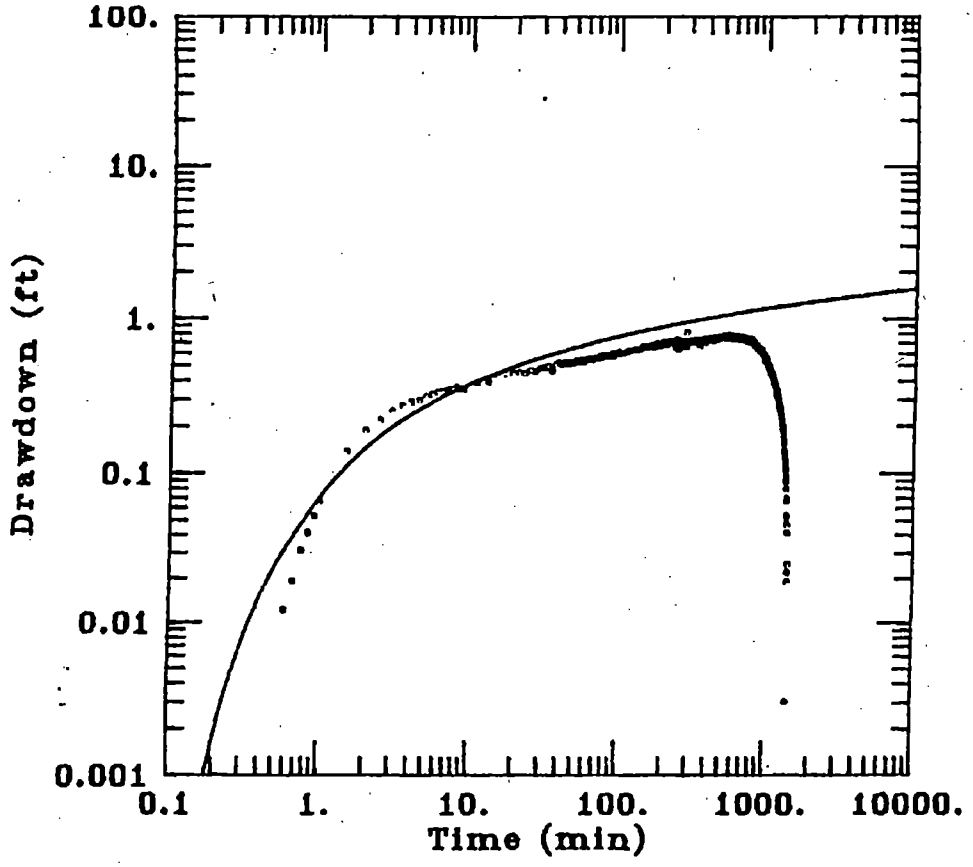
ESTIMATED PARAMETERS:
T = 10.82 ft^2/min
S = 0.001283
β = 0.0001

TEST DATA:
Q = 46.8 ft^3/min
r = 135. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft
TEST WELL:
t.o.s. = 41.5 ft
b.o.s. = 73.5 ft
OBS. WELL:
t.o.s. = 58.1 ft
b.o.s. = 78.1 ft

Figure 17

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

GM-57C



DATA SET:
gm-57c.aqt
07/01/92

AQUIFER TYPE:
Leaky

SOLUTION METHOD:
Hantush

TEST DATE:
JUNE 17, 1992

TEST WELL:
TW-1

OBS. WELL:
GM-57C

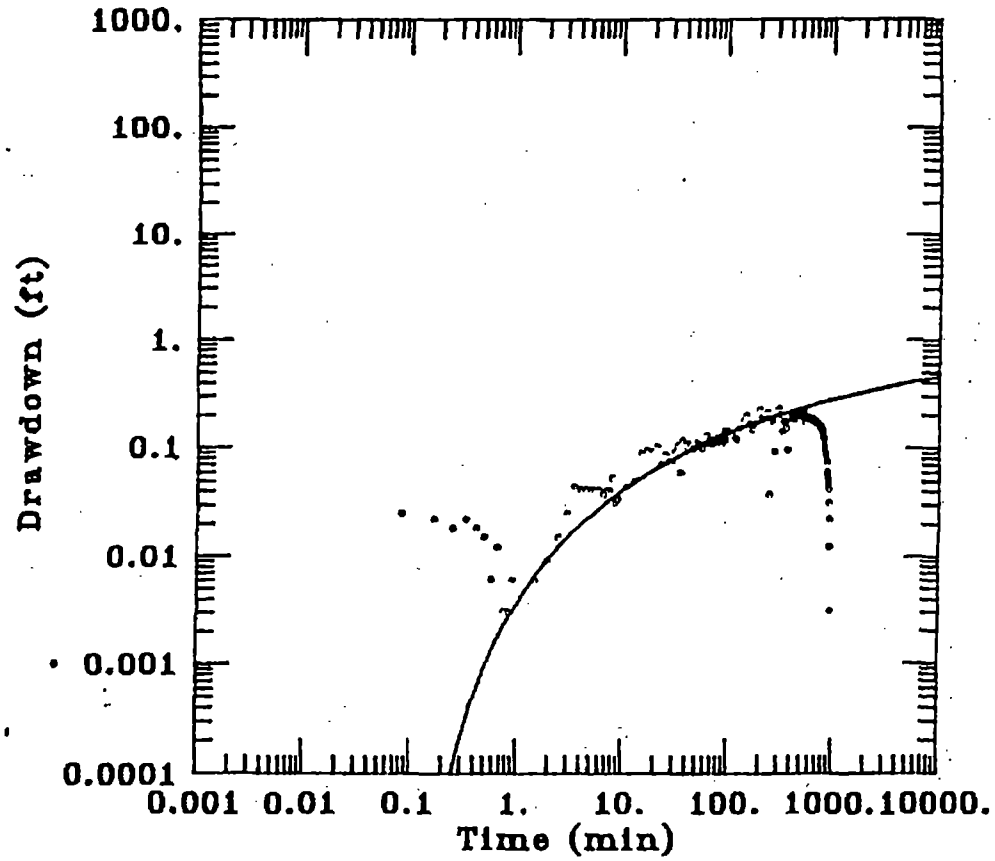
ESTIMATED PARAMETERS:
T = 21.43 ft^2/min
S = 0.0004317
β = 0.0001

TEST DATA:
Q = 46.8 ft^3/min
r = 368. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft
TEST WELL:
t.o.s. = 41.5 ft
b.o.s. = 73.5 ft
OBS. WELL:
t.o.s. = 60. ft
b.o.s. = 80. ft

Figure 18

GERAGHTY & MILLER	Client: MONSANTO
Project No.: NY64207	Location: SAUGET, ILLINOIS

GM-28C



DATA SET:
gm-28c.aqt
07/01/92

AQUIFER TYPE:
Leaky

SOLUTION METHOD:
Hantush

TEST DATE:
JUNE 17, 1992

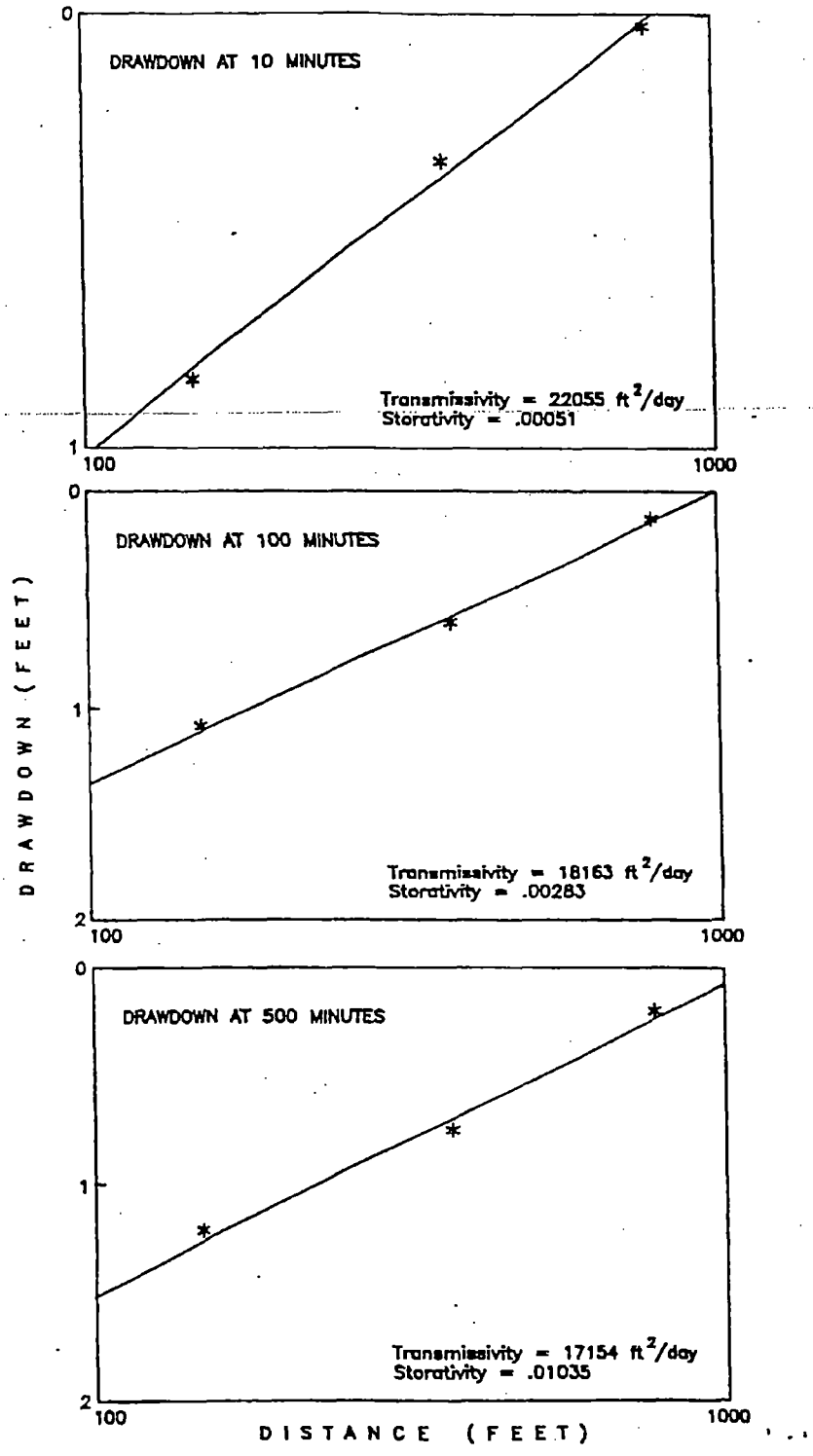
TEST WELL:
TW-1

OBS. WELL:
GM-28C

ESTIMATED PARAMETERS:
T = 21.64 ft^2/min
S = 0.0001029
β = 2.

TEST DATA:
Q = 46.8 ft^3/min
r = 772. ft
rc = 0.5 ft
rw = 0.5 ft
b = 90. ft
TEST WELL:
t.o.s. = 41.5 ft
b.o.s. = 73.5 ft
OBS. WELL:
t.o.s. = 51. ft
b.o.s. = 71. ft

Figure 19



GERAGHTY & MILLER, INC.
Environmental Services

JACOB DISTANCE-DRAWDOWN PLOTS OF THREE DEEP ZONE WELLS AT 10, 100, AND 500 MINUTES INTO THE AQUIFER TEST

MONSANTO COMPANY
SAUGET, ILLINOIS

FIGURE 20

DRAFTER: GS | APPROVED: LM | CHECKED: SF | DRAWING: DL, AC | FILE NO.: 1558 | PRJCT NO.: NY64207 | DWG ... 2-9-93

Appendix 4

Groundwater Model

(Geraghty & Miller, 1994)

DRAFT
GSI Job No. G-2561
March 31, 2002

GROUNDWATER
SERVICES, INC.

Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

ATTACHMENT 15

Circular 180

Ground-Water Levels and Pumpage in the Metro-East Area, Illinois, 1986-1990

by Richard J. Schicht and Andrew G. Buck

Illinois State Water Survey
A Division of the Illinois Department of Natural Resources

1995

Circular 180

Ground-Water Levels and Pumpage in the Metro-East Area, Illinois, 1986-1990

by

Richard J. Schicht and Andrew G. Buck

Title: Ground-Water Levels and Pumpage in the Metro-East Area, Illinois, 1986-1990

Abstract: This report discusses ground-water levels and pumpage in the Metro-East area just south of Alton, Illinois, to Dupo, Illinois, and between the Mississippi River and the river bluffs from 1986-1990. Large quantities of ground water, primarily for industrial and municipal use, are withdrawn from wells penetrating a sand-and-gravel aquifer along the valley lowlands of the Mississippi River.

Ground-water pumpage declined from 62.8 million gallons per day (mgd) in 1986 to 58.7 mgd in 1990. Of the total 1990 pumpage, 76.2 percent (or 44.7 mgd) was industrial; 20.8 percent (or 12.2 mgd) was for public water supplies; 2.0 percent (or 1.2 mgd) was for irrigation; and 1.0 percent (or 0.6 mgd) was for domestic use. Pumpage in the Metro-East area is concentrated at five major pumping centers (Alton, Wood River, Roxana, National City, and Granite City) and four minor pumping centers (Poag, Glen Carbon, Collinsville, and Venice). Pumpage in the Sauget (Monsanto) area, once considered a minor pumping center (Kohlhase, 1987), was negligible in 1990 because of declining industrial use.

Ground-water levels throughout the entire area were stable but elevated during 1986 and 1987. Water levels declined from 1988 to 1989 and increased in 1990. Factors contributing to this pattern were above-normal precipitation, the Midwestern drought of 1988-1989, changes in river stages, and the response of water levels to annual pumpage changes.

Reference: Schicht, Richard J. and Buck, Andrew G. Ground-Water Levels and Pumpage in the Metro-East Area, Illinois, 1986-1990. Illinois State Water Survey, Champaign, Circular 180. 1995.

Indexing Terms: Metro-East area, ground water, public water supplies, industrial water supplies, water levels, water-level changes, pumping, ground-water withdrawals.

STATE OF ILLINOIS
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1995

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GROUND-WATER LEVELS AND PUMPAGE IN THE METRO-EAST AREA, ILLINOIS, 1986-1990

by Richard J. Schicht and Andrew G. Buck

ABSTRACT

This report discusses ground-water levels and pumpage in the Metro-East area just south of Alton, Illinois, to Dupo, Illinois, and between the Mississippi River and the river bluffs from 1986-1990. Large quantities of ground water, primarily for industrial and municipal use, are withdrawn from wells penetrating a sand-and-gravel aquifer along the valley lowlands of the Mississippi River.

Ground-water pumpage declined from 62.8 million gallons per day (mgd) in 1986 to 58.7 mgd in 1990. Of the total 1990 pumpage, 76.2 percent (or 44.7 mgd) was industrial; 20.8 percent (or 12.2 mgd) was for public water supplies; 2.0 percent (or 1.2 mgd) was for irrigation; and 1.0 percent (or 0.6 mgd) was for domestic use. Pumpage in the Metro-East area is concentrated at five major pumping centers (Alton, Wood River, Roxana, National City, and Granite City) and four minor pumping centers (Poag, Glen Carbon, Collinsville, and Venice). Pumpage in the Sauget (Monsanto) area, once considered a minor pumping center (Kohlhase, 1987), was negligible in 1990 because of declining industrial use.

Ground-water levels throughout the entire area were stable but elevated during 1986 and 1987. Water levels declined from 1988 to 1989 and increased in 1990. Factors contributing to this pattern were above-normal precipitation, the Midwestern drought of 1988-1989, changes in river stages, and the response of water levels to annual pumpage changes.

INTRODUCTION

Previous Illinois State Water Survey (ISWS) reports have referred to the area from just south of Alton, Illinois, to Dupo, Illinois, and between the Mississippi River and the river bluffs (figure 1) as the East St. Louis area. Starting with this report, however, the area will be referred to as the Metro-East area, a more common description of the area by local and regional planning agencies. The ground-water resources of a sand-and-gravel aquifer underlying the area, one of the most heavily populated and industrialized areas in Illinois, have been developed extensively. It is estimated that during 1990, 58.7 mgd were withdrawn, primarily for industrial and municipal use.

A period of intensive data collection was initiated in 1941 after local industries observed alarming water-level recessions, culminating in ISWS Report of Investigation 51 (Schicht, 1965). The report describes in detail the ground-water resources of the area. Several previous reports have summarized water levels and pumpage in the area, which aided in the preparation of Report of Investigation 51 (Bruin and Smith, 1953; Schicht and Jones, 1962). Ground-water geology of the area had been described previously by the Illinois State Geological Survey (Bergstrom and Walker, 1956).

Report of Investigation 51 included an estimate of the aquifer yield of the sand-and-gravel aquifer based on a pumping center configuration described in the same report. This yield (188 mgd) was never realized due to the area's general economic decline and shifts in pumpage to the Mississippi River.

Data collection was originally continued to validate the predictions of Report of Investigation 51 and to delineate problem areas and now monitors the effects of rising ground-water levels and shifts in pumpage. Additional data will also be useful in calibrating and revising the digital computer ground-water model developed by Ritchey, Schicht, and Weiss (1984). In recent years, Water Survey staff have conducted studies related to ground-water contamination, and continued data collection at the scale described in this report is a valuable supplement to data collected for these contamination studies.

With the completion of the Mel Price Lock and Dam located 1.6 miles down river from the old lock and dam at Alton, it is important to continue monitoring water levels and pumpage just south of Alton to determine the effects, if any, on water levels in the area. Previous summaries of pumpage and water levels have been published (Reitz, 1968; Baker, 1972; Emmons, 1979; Collins and Richards, 1986; Kohlhase, 1987). This report summarizes water-level and pumpage data collected from 1986-1990.

Acknowledgments

This report was prepared under the direction of Adrian P. Visocky, Director of the Office of Ground-Water Resources Evaluation & Management. The authors gratefully acknowledge the assistance of Mark Sievers in providing computer-generated maps and tables, which were used in the preparation of the potentiometric maps. Kristopher Klindworth and John Blomberg provided data used to estimate ground-water withdrawals, Ellis Sanderson and Robert Olson helped interpret data from the highway dewatering sites, and Mr. Sanderson reviewed the final report and provided much useful advice. Linda Hascall prepared the illustrations, Pamela Lovett provided word processing expertise, and Eva Kingston did the final editing and layout. Special thanks go to those who participated in the water-level data collection efforts: Stuart Cravens, Ken Hlinka, Ellis Sanderson, Robert Olson, and Scott Meyer. The U.S. Army Corps of Engineers office in St. Louis provided information on Mississippi River stages and general information on dewatering at the Mel Price Lock and Dam.

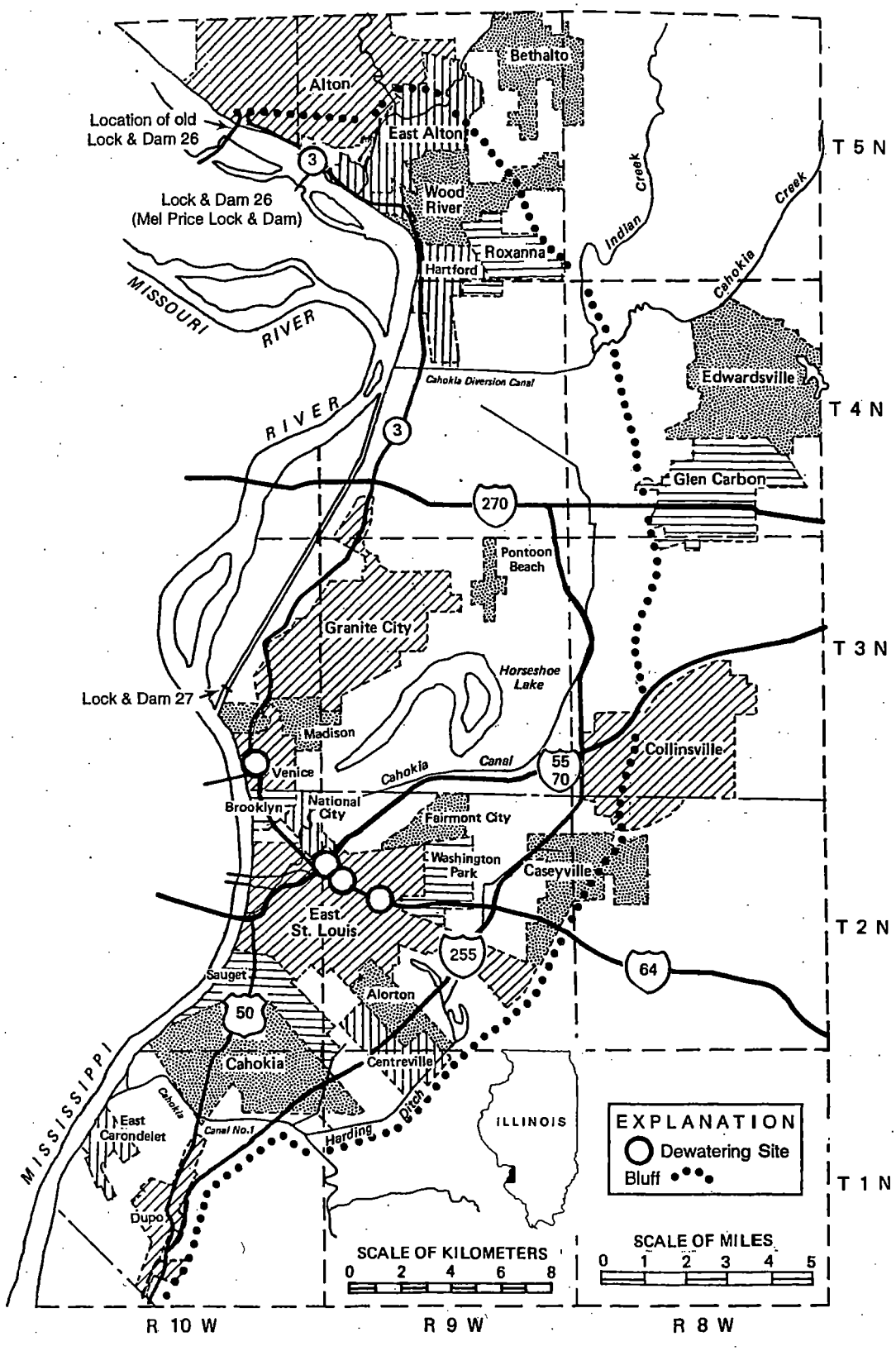


Figure 1. Location of the Metro-East area

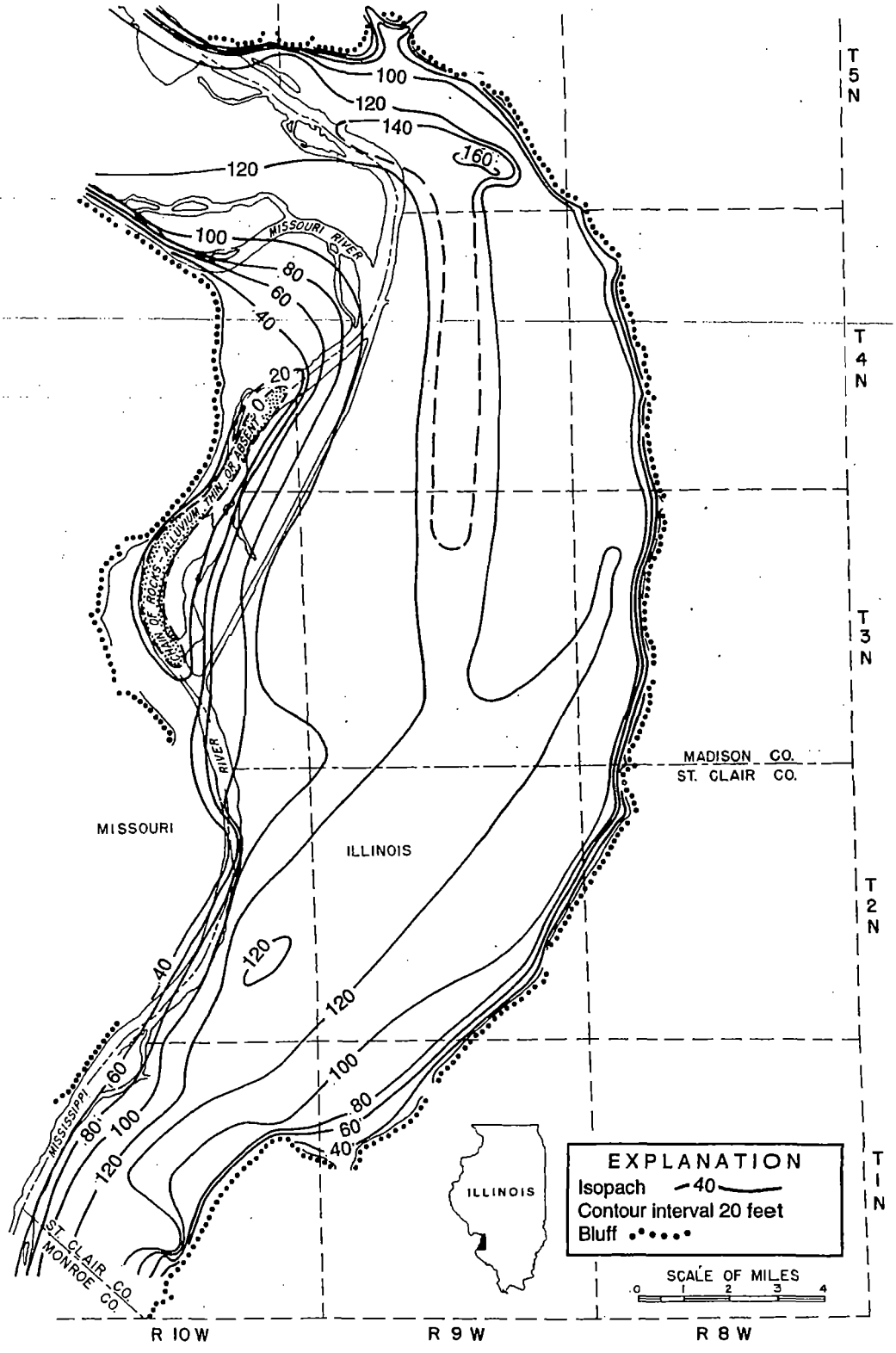
GEOLOGY AND HYDROLOGY

Large supplies of ground water, mainly for industrial use, are withdrawn from wells finished in the permeable sand-and-gravel deposits in the unconsolidated valley fill in the Metro-East area. According to Bergstrom and Walker (1956), the valley fill is composed of recent alluvium and glacial valley-train material underlain by Mississippian and Pennsylvanian rocks of low permeability. Because of the bedrock's low permeability and poor water quality with depth, it is not an important aquifer in the area.

The valley fill averages 120 feet in thickness. The thickness is greatest, 170 feet, near the city of Wood River. Near the bluffs there are bedrock outcrops. Generally, the thickness of the valley fill is greatest and exceeds the average in places near the center of a buried bedrock valley that longitudinally bisects the area, as shown in figure 2. The valley fill becomes progressively coarser with depth, and the coarsest deposits most favorable for development are commonly encountered near bedrock and often average between 30 and 40 feet in thickness.

Ground water in the valley fill occurs under leaky artesian and water-table conditions. Because ground water occurs under leaky artesian conditions in most places, the surface to which water rises in wells is referred to as the potentiometric surface in this report.

Recharge within the Metro-East area is from precipitation, infiltration of surface water from the Mississippi River and lesser water bodies in the area, and subsurface flow from the bluffs bordering the area. A fraction of the annual precipitation seeps downward through surface materials and into the valley fill material. Recharge by the river and other water bodies in the area occurs where the potentiometric surface elevation is lower than surface water elevations. Pumping centers adjacent to the river maintain ground-water levels well below the river stage, inducing large quantities of river water into these pumping centers.



(after Bergstrom & Walker, 1956)

Figure 2. Thickness of valley fill deposits

PUMPAGE FROM WELLS

The first significant withdrawal of ground water in the Metro-East area started in the late 1890s. Estimated pumpage from wells increased from 2.1 mgd in 1900 to 111.0 mgd in 1956, as shown in figure 3. Pumpage declined sharply to 92 mgd in 1958 and then increased to 110.0 mgd in 1964. After 1966, pumpage declined steadily to 54.4 mgd in 1981. By 1990, pumpage had increased slightly to 58.7 mgd. Pumpage would have been significantly less had it not been for dewatering wells maintained by the Illinois Department of Transportation (IDOT) along roadways in the area to prevent water levels from rising above the road surface. Withdrawals for dewatering began in 1963, and an estimated 11.2 mgd was pumped from dewatering wells during 1990. Figure 1 shows locations of dewatering sites, and more recent information on these dewatering sites is available (Sanderson and Olson, 1993).

Within this report pumpage data are classified according to four categories: 1) *public*, including municipal and institutional; 2) *industrial*, including dewatering; 3) *domestic*, including rural farm nonirrigation and rural nonfarm; and 4) *irrigation*, including farms, golf courses, and cemeteries. Most water-supply systems furnish water for multiple uses. A public supply commonly includes water used for drinking and other domestic uses, manufacturing processes, and lawn sprinkling. Industrial supplies may also be used in part for drinking and other domestic uses. No attempt has been made to determine the final use of water within the public and domestic categories; for example, any water pumped by a municipality is called a public supply, regardless of the use of the water. However, the final use of the water within the industrial category has been determined in part, and any water pumped by an industry and furnished to a municipality is included in the public category.

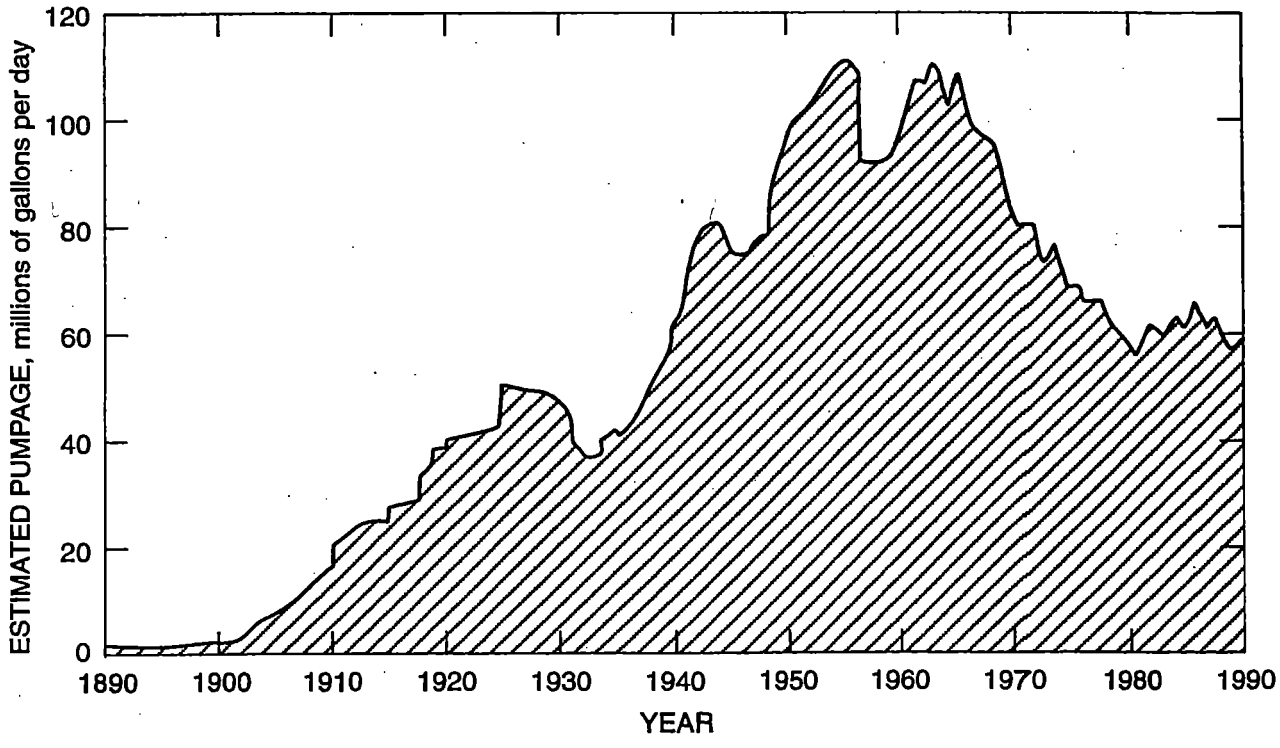


Figure 3. Estimated pumpage, 1890-1990

Pumpage, 1986-1990

Table 1 shows total pumpage, including all water use categories for the period 1986-1990. Total pumpage declined from 62.8 mgd in 1986 to 58.7 mgd in 1990. Distribution of 1990 pumpage is as follows: public supply systems (20.8 percent or 12.2 mgd), industrial pumpage (76.2 percent or 44.7 mgd), domestic pumpage (1.0 percent or 0.6 mgd), and irrigation pumpage (2.0 percent or 1.2 mgd).

Public Supplies. Municipal and institutional uses are included in public supplies. Pumpage for institutional use in the area has been negligible, however. Figure 4 shows the estimated pumpage for public supplies, which averaged 12.2 mgd for each year except 1988 when it was 13.3 mgd.

Pumpage of public supplies reflects seasonal variations to some extent. For example, municipal pumpage is generally 25 to 30 percent higher during the summer months than during the winter months because of lawn sprinkling, car washing, and other summer use of water.

Industrial Supplies. The major industrial users of ground water in the Metro-East area include oil refineries, chemical plants, ore refineries, meat packing plants, and steel plants. With its system of dewatering wells, IDOT is a major industrial user. Most industries do not meter their pumpage, and pumpage estimates are typically based on the number of hours the pump operated, on pump capacity, and in some cases on production capacity. Industrial pumpage generally is more uniform throughout the year than public pumpage unless large air-conditioning systems are used, the industry is seasonal, or a change in operation occurs as a result of strikes or vacation shutdowns. Industrial pumpage (figure 4) declined from 49.2 mgd in 1986 to 44.7 mgd in 1990.

Domestic Supplies. Estimates of domestic pumpage considered rural populations as reported by the U.S. Bureau of the Census and the per capita use of 84 gallons per day (gpd) used by Kohlhase (1987). On the basis of this per capita use, average domestic use in 1990 was estimated to be 600,000 gpd.

Irrigation Supplies. In 1989, a questionnaire was mailed to all known irrigators in the Metro-East area requesting information for 1988 on number of acres irrigated, type of crop irrigated, frequency of irrigation, and quantity of water applied. Based on the survey results, it was estimated that an average of about 0.7 mgd of ground water was withdrawn for irrigation during 1988. Respondents included 18 farmers who irrigated a total of 2000 acres. Estimated irrigation was 0.8 mgd in 1986 and 1989, 1.2 mgd in 1990, and less than 0.1 mgd in 1987, based on June-August rainfall measured at Belleville (table 2).

Table 1. Annual Pumpage (mgd), 1986-1990

Year	*Pumpage*
1986	62.8
1987	60.4
1988	61.6
1989	58.1
1990	58.7

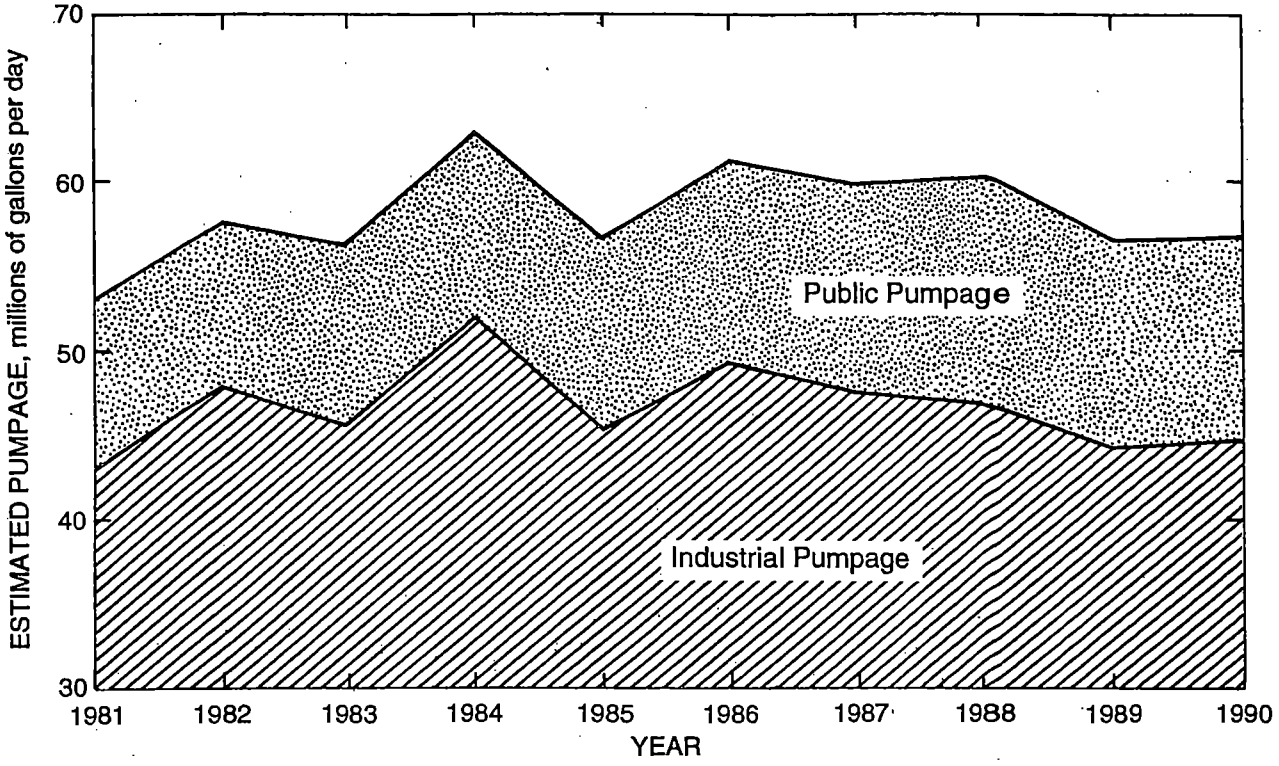


Figure 4. Estimated industrial and public pumpage, 1981-1990

Table 2. Rainfall (inches) June-August 1986-1990

Year	*Rainfall*
1986	11.45
1987	17.38
1988	10.10
1989	9.32
1990	5.82

Dewatering Pumpage during Construction of Mel Price Lock and Dam. Large quantities of ground water were withdrawn in the Alton area because of dewatering activities during construction of the Mel Price Lock and Dam. The Corps of Engineers estimated that withdrawals during the third phase of construction near the Illinois shore averaged 78,000 gallons per minute (gpm). A significant part of this pumpage was from ground water. Phase 3 began on May 31, 1990, and concluded in January 1993. Dewatering during phase 1 (1980-1984) was for a cofferdam on the Missouri side of the river. Dewatering during phase 2 (1985-1988) was for a cofferdam in the middle of the river. Based on available information, it is not possible to determine the ratio of river water to ground water pumped during dewatering operations.

Distribution of Pumpage

Figure 5 shows the distribution of the 1990 pumpage and locations of the pumping centers. Pumpage in the area is concentrated at five major pumping centers (Alton, Wood River, Roxana, National City, and Granite City) and four minor pumping centers (Poag, Glen Carbon, Collinsville, and Venice).

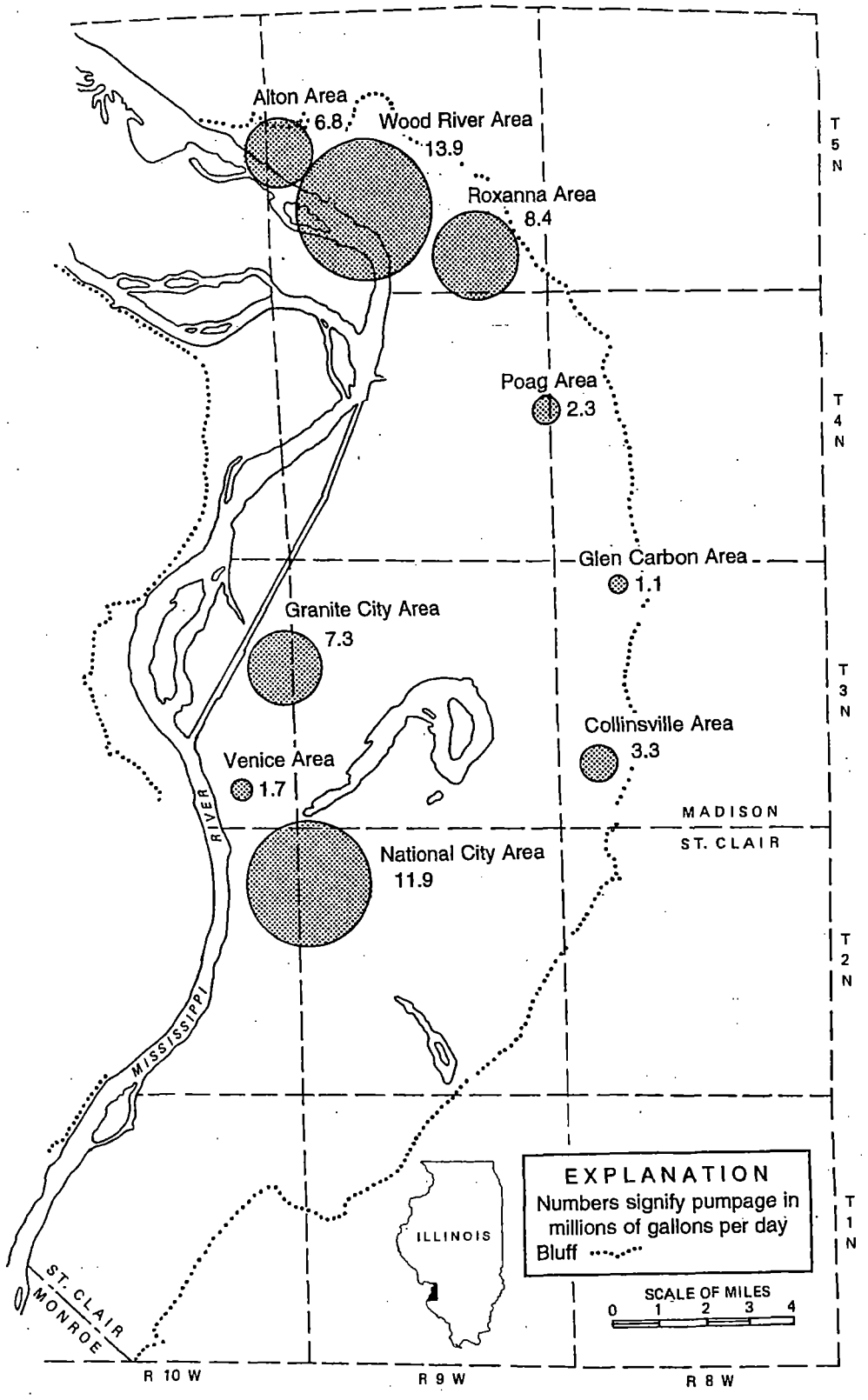


Figure 5. Distribution of estimated pumpage, 1990

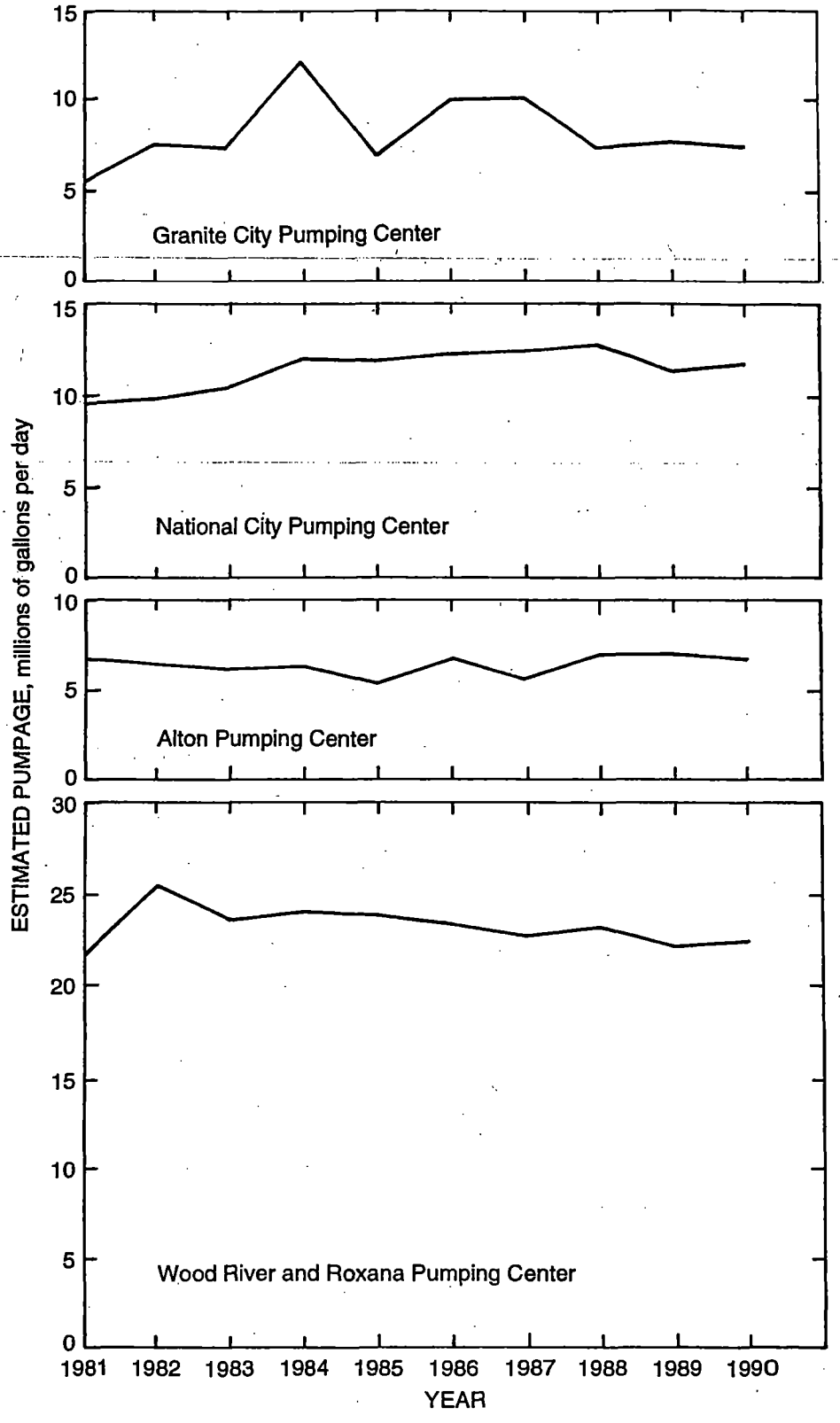


Figure 6. Estimated pumpage at major pumping centers, 1981-1990

Previous reports have included pumpage from the highway dewatering site at Venice in the total for National City. Sauget is no longer listed as a minor pumping center (Kohlhase, 1987), and pumpage there was negligible in 1990.

Figure 6 shows pumpage for 1981-1990 for each major pumping center. Ground-water withdrawals in the Alton area are primarily from wells owned by two industries and a municipality. During the 1986-1990 period pumpage at Alton varied from 6.7 mgd to 7.0 mgd, except during 1987 when pumpage was only 5.6 mgd because of reduced industrial activity.

The Wood River/Roxana area is the largest pumping center in the Metro-East area. Annual pumpage during 1986-1990 was fairly stable, varying from 22.3 mgd to 23.3 mgd. Pumpage in the Wood River/ Roxana area is mainly for oil refineries and municipalities.

Ground-water pumpage in the Granite City area was about 10 mgd in 1986 and 1987. Pumpage declined to 7.4 mgd in 1988 and was 7.3 mgd in 1990. Steel production industries are the major ground-water users in the area.

Ground-water withdrawals in the National City area are mainly from wells at the interstate dewatering sites shown in figure 1 and at a paint pigment plant. Withdrawals for the meat packing industry, formerly large users, averaged only about 0.25 mgd in 1990. Since the goal of the dewatering sites is to maintain the ground-water elevations within the pumping centers at a relatively constant elevation, pumpage from wells at the sites fluctuates in response to changes in river stages, changes in recharge from precipitation, and changes in ground-water pumpage in the vicinity of the sites. Pumpage for the 1986-1990 period was highest in 1988 (12.8 mgd) and lowest (11.5 mgd) in 1989.

Figure 7 shows combined pumpage for the minor pumping centers. Except for the dewatering site at Venice, pumpage from these centers was mainly by municipalities. Pumpage for the period was highest (9.7 mgd) in 1989 and lowest (8.6 mgd) in 1990.

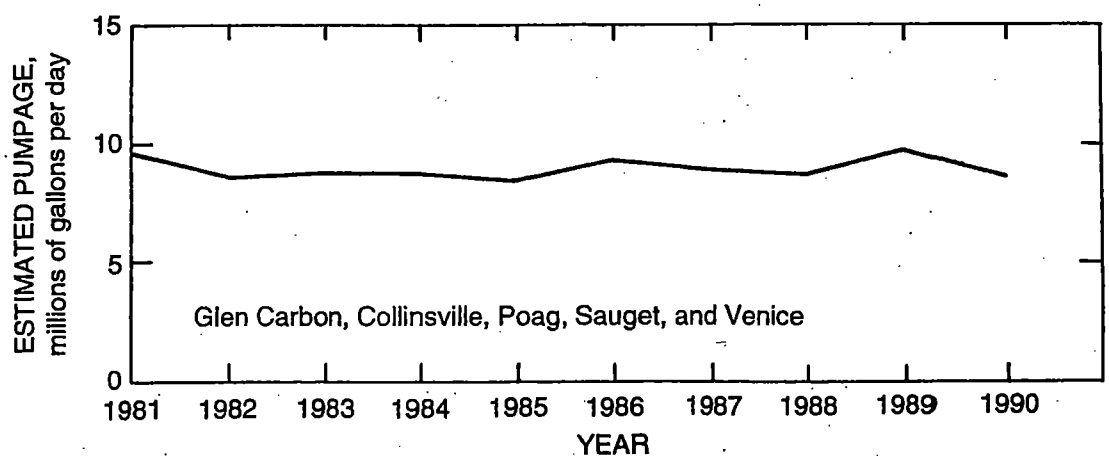


Figure 7. Estimated pumpage at minor pumping centers, 1981-1990

WATER LEVELS IN WELLS

Water levels in wells in the Metro-East area have been measured periodically for more than 50 years by the Illinois State Water Survey and others from the public and private parties. Figure 8 shows the locations of ISWS observation wells active from 1986-1990.

Water levels in wells generally recede in late spring, summer, and early fall when one, or a combination of the following, exceeds recharge from precipitation and infiltration induced from surface water bodies:

a) discharge from the ground-water reservoir by evapotranspiration
b) discharge of ground water to streams and lakes
c) discharge of ground water by pumpage

Ground-water levels generally begin to recover in early winter when conditions are favorable for recharge from precipitation. Recovery of ground-water levels is especially pronounced during the spring months when precipitation recharge exceeds evapotranspiration and discharge to streams, resulting in most of the annual recharge to the aquifer.

The water level measured in a well at a particular time reflects not only seasonal variation, but also factors such as recent climatic conditions, nearby pumpage, and the water levels of nearby surface water bodies. Figure 9(a-d) shows the average monthly high and low water levels observed during the period of record for four wells located in the Metro-East area. These graphs indicate that ground-water levels are usually highest from April to June and lowest in September, October, and November. The graphs also reveal the influence of nearby hydrologic features.

Well MAD3N9W-16.8a is located approximately 2000 feet from Horseshoe Lake. The north and eastern end of Horseshoe Lake can be considered a ground-water recharge area, whereas the southwestern tip of the lake is a ground-water discharge area. This "flow-through" hydrologic system coupled with man-made flood control systems diverting surface water into and out of the lake contribute to the lake's nearly constant water surface elevation, which in turn limits fluctuations of the surrounding ground-water levels. As a result of these factors, the annual fluctuation at Well MAD3N9W-16.8a is only about one foot (see figure 9a).

Well MAD5N9W-29.5g2, located near the Mississippi River at Alton, is influenced mainly by river stage fluctuations and pumpage (see figure 9b). It is not unusual for ground-water levels at this well to be at land surface or for ponded water to occur during high river stages. Well MAD3N10W-14.4b is located in the west-central part of the area near Chain of Rocks Canal. Water levels in this well have fluctuated on average about 7 to 8 feet annually during the last ten years (figure 9c). This is less than at Well MAD5N9W-29.5g2, probably because of less pumpage influence and because of the stabilizing effect of Lock and Dam No. 27 on river stages. In contrast, Well STC2N9W-26.8f2 is located near the bluff in the southern part of the area and is not greatly affected by pumpage or surface water influence, and the annual fluctuation is about 2 feet (figure 9d).

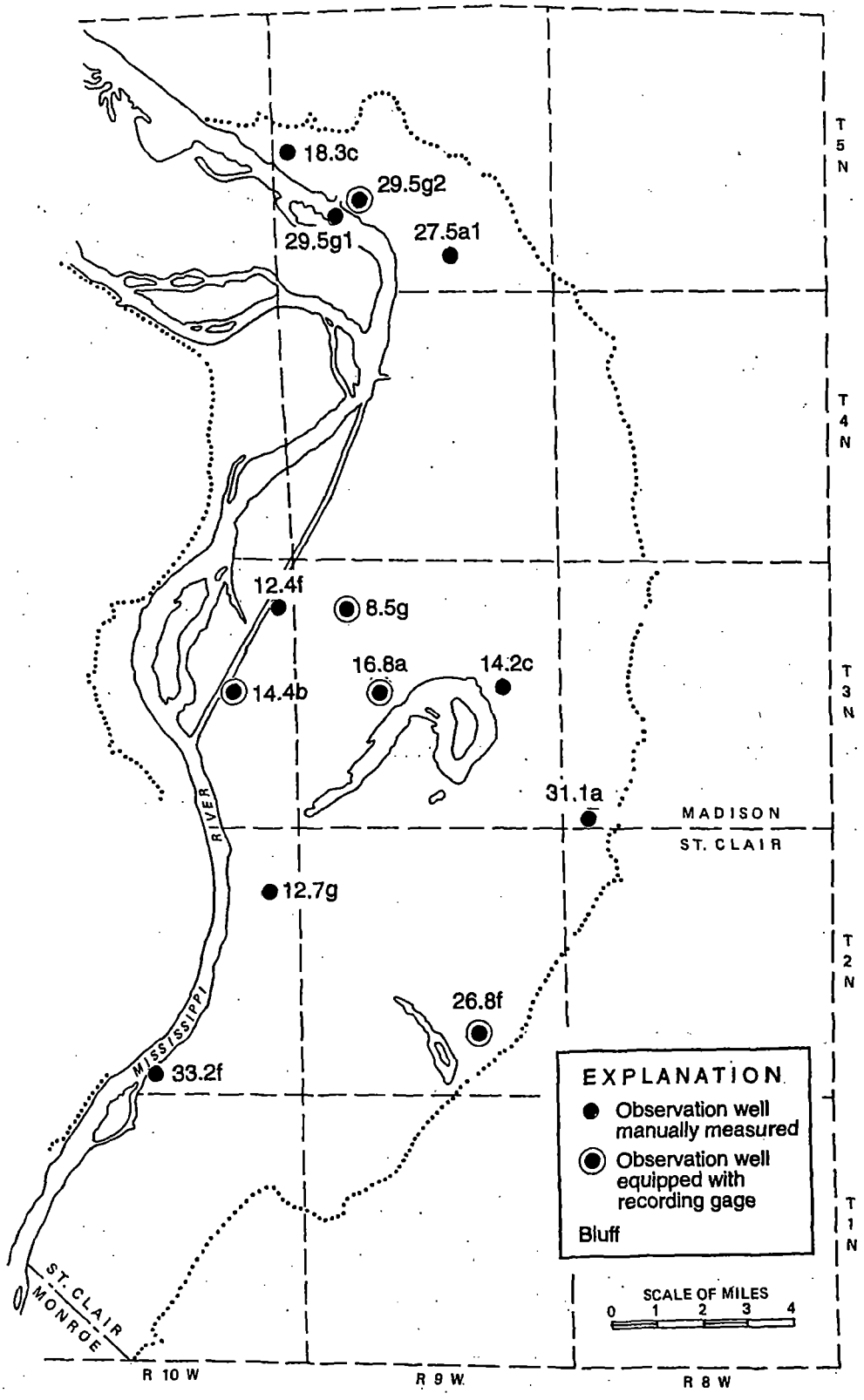


Figure 8. Locations of Illinois State Water Survey observation wells

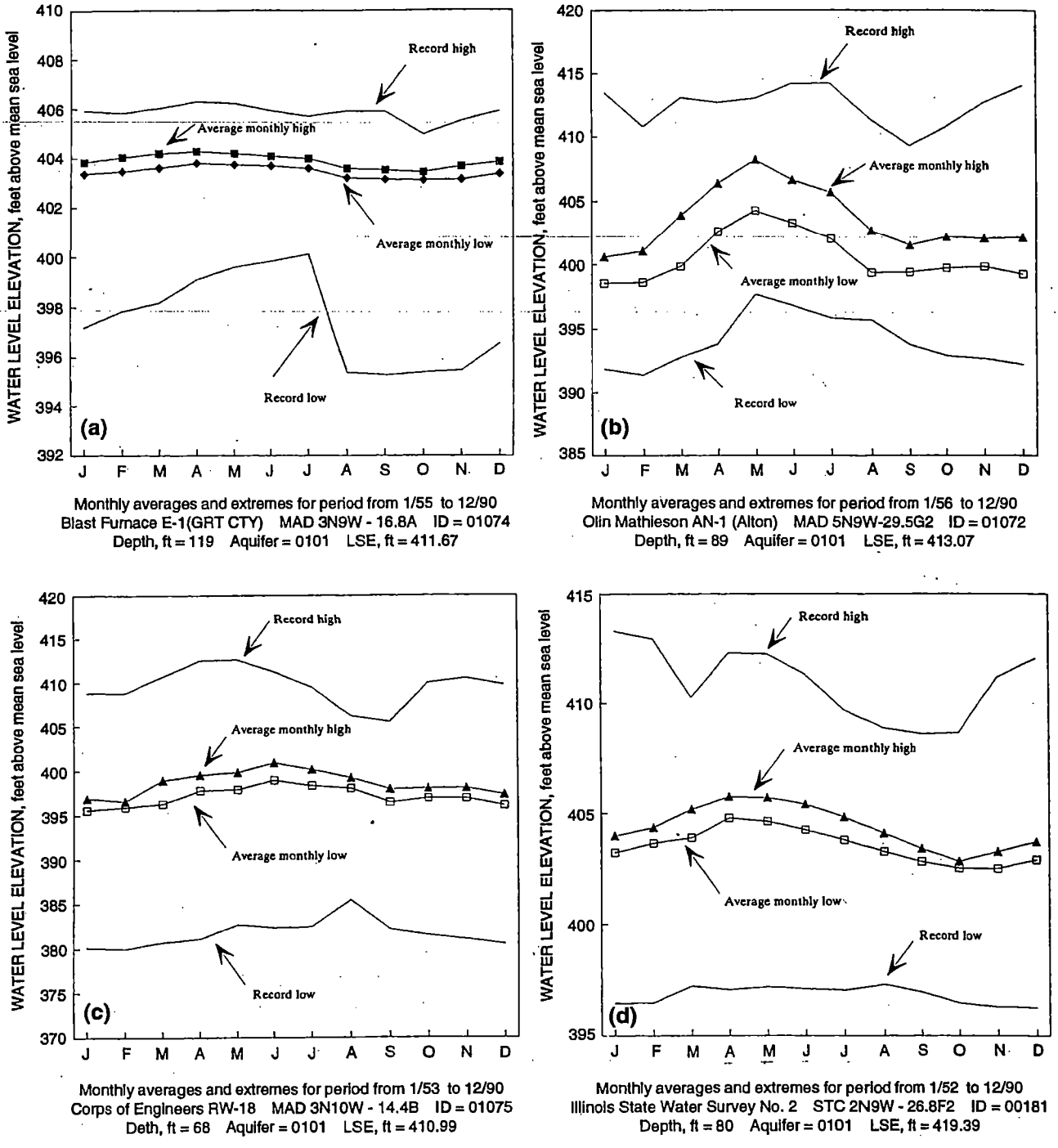


Figure 9. Average and record monthly high and low water levels at four wells in the Metro-East area: a) January 1955-December 1990, b) January 1956-December 1990, c) January 1953-December 1990, and d) January 1952-December 1990

Since 1900, ground-water levels have changed appreciably at the five major pumping centers. According to Schicht and Jones (1962), the greatest water-level declines for the period from 1900 to November 1961 occurred in major pumping: 50 feet in the Sauget area (formerly a major pumping center), 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. Part of the declines, 2 to 12 feet, was attributed to the construction of levees and drainage ditches.

Reitz (1968) and Baker (1972) described the changes in ground-water levels from 1962-1971. Ground-water levels generally continued to decline through 1964, but began to rise about 1965 as the effects of decreased pumpage and above-average precipitation and river stages became noticeable.

Ground-water levels generally continued to rise for the period from 1972-1977 (Emmons, 1979). Decreases in pumpage caused ground-water levels to rise 2 feet in the Sauget and Wood River areas and 5 feet in National City. Little change was observed in the Alton and Granite City pumping centers. In Alton, a change of observation wells to a site nearer the center of pumpage obscured the rise in ground-water levels resulting from a decrease in pumpage. Erratic pumpage in the Granite City area produced small observed changes in ground-water levels.

During the period from 1978-1980 ground-water levels outside pumping centers showed little change (Collins and Richards, 1986). Trends established between 1971 and 1977 continued near pumping centers. Decreases in water levels in areas near the Mississippi River were generally due to low river stages. Decreases in water-level elevations of more than 5 feet in the Wood River area, however, were attributed to a change in the spatial distribution of pumpage. Ground-water levels in the Granite City area generally rose in proportion to decreased pumpage. Increased pumpage in the National City area expanded the area of declining ground-water levels near the river. Ground-water levels continued to recover in the Sauget area with reduced pumpage.

The trend in ground-water levels from 1981-1985 was for increasing water levels during 1981 and 1982, with apparent stabilization within an elevated range during 1983-1985 (Kohlhase, 1987). Above-normal precipitation and river stages from 1982-1985, coupled with the response of water levels to annual pumpage changes, were the main factors contributing to this trend in water levels. From 1981-1982, ground-water level increases of as much as 17 feet were observed in the National City and Alton areas, 8 feet to 16 feet in the Granite City region, 12 feet in the Wood River area, and 7 to 14 feet in areas near the bluff. Water levels stabilized at an elevated state after this trend of increasing water levels.

Figure 10 shows the mean monthly Mississippi River stages for the period from 1981-1990, and figure 11 shows the observed annual precipitation for the same period at Belleville (the raingage lies one mile south of Scott Air Force Base). Figure 12 shows hydrographs of selected wells for this period. A single line hydrograph represent water levels for wells at which the water level is measured monthly. A double line represents water levels for wells equipped with continuous recorders; the lines represent the observed monthly high and low ground-water levels.

The hydrographs show that these wells all share a similar fluctuation pattern from 1986-1990, differing only in magnitude of fluctuation. The general trend during this period was for stable water

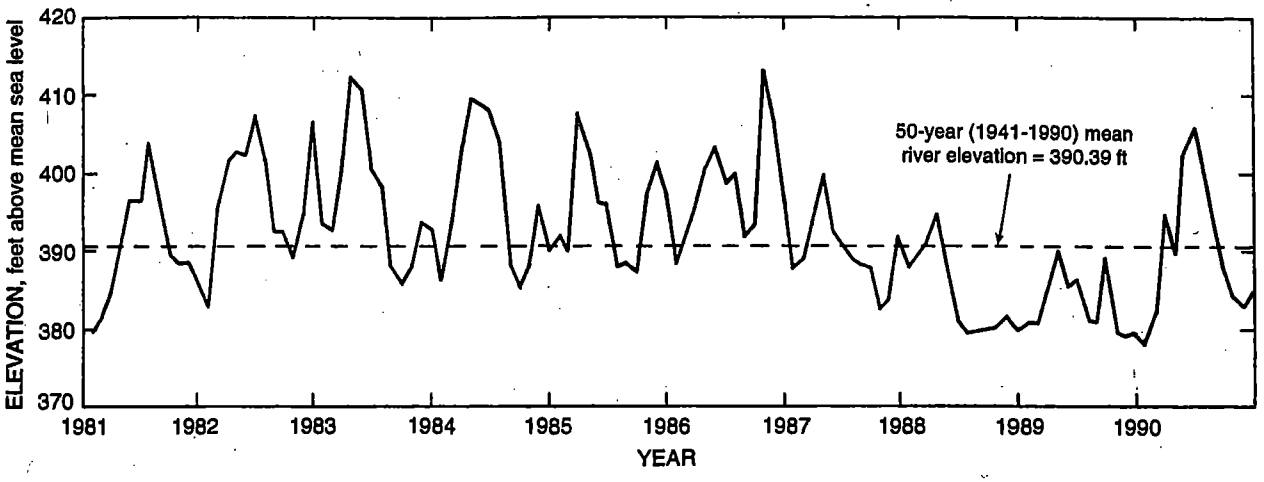


Figure 10. Mean monthly Mississippi River stages, St. Louis gaging station, 1981-1990

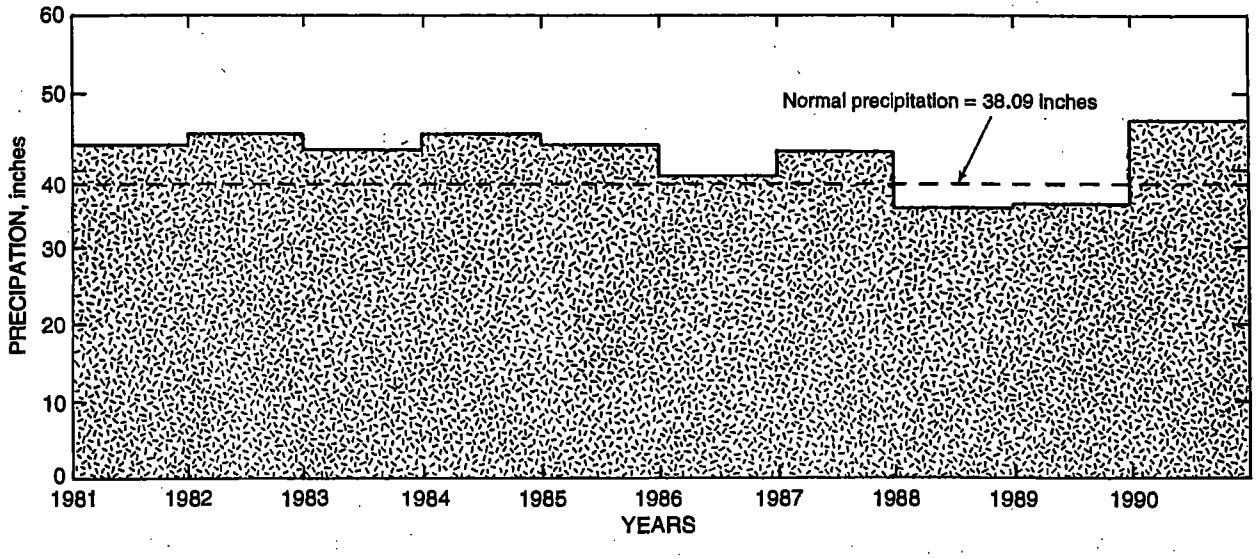


Figure 11. Annual precipitation at Belleville, 1981-1990

levels within an elevated range in 1986-1987, falling ground-water levels in 1988 and 1989, followed by increasing water levels in 1990. Factors contributing to this pattern were above-normal precipitation from 1981-1987, the Midwestern drought of 1988-1989, river stages, and water-level responses to annual pumpage changes. Annual precipitation was approximately 12 percent above normal from 1981-1987. Much of Illinois and the surrounding states experienced a substantial shortfall in precipitation during 1988-1989. Precipitation in the Belleville area was about 92 percent of normal during this period. However, the data from the climate site near Belleville are not indicative of the severity of the drought experienced by other parts of Illinois and the surrounding states. Based on the Palmer Drought Index during September 1988 and September 1989, Kunkel, Angel, and Wendland (1992) described the Metro-East area as being under mild drought conditions, whereas much of the Mississippi River valley to the north and the surrounding areas were classified as being under extreme drought conditions. During 1990, precipitation was significantly above normal. Fluctuations in the mean monthly river stages

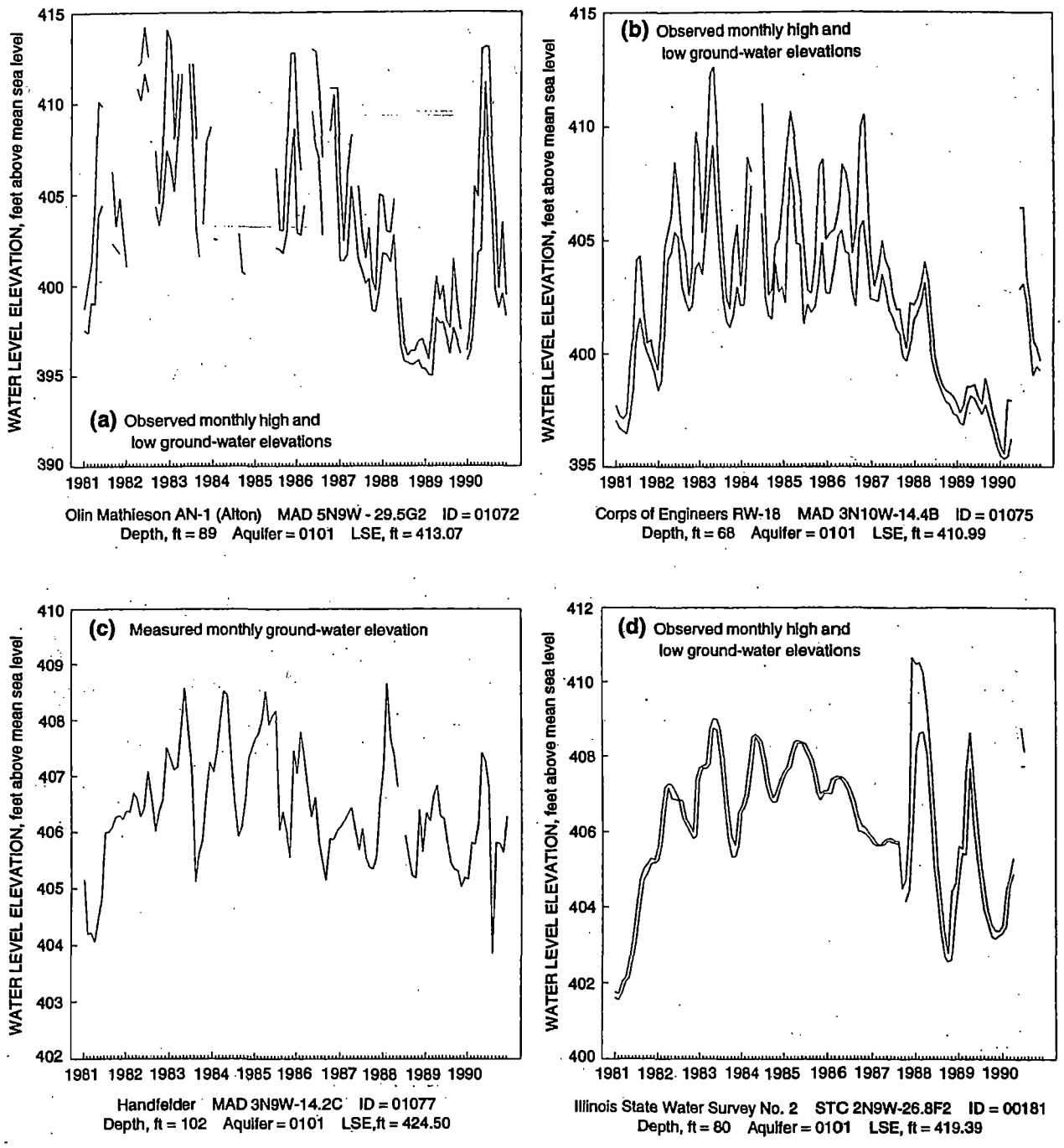


Figure 12. Hydrographs of eight selected wells, 1981-1990

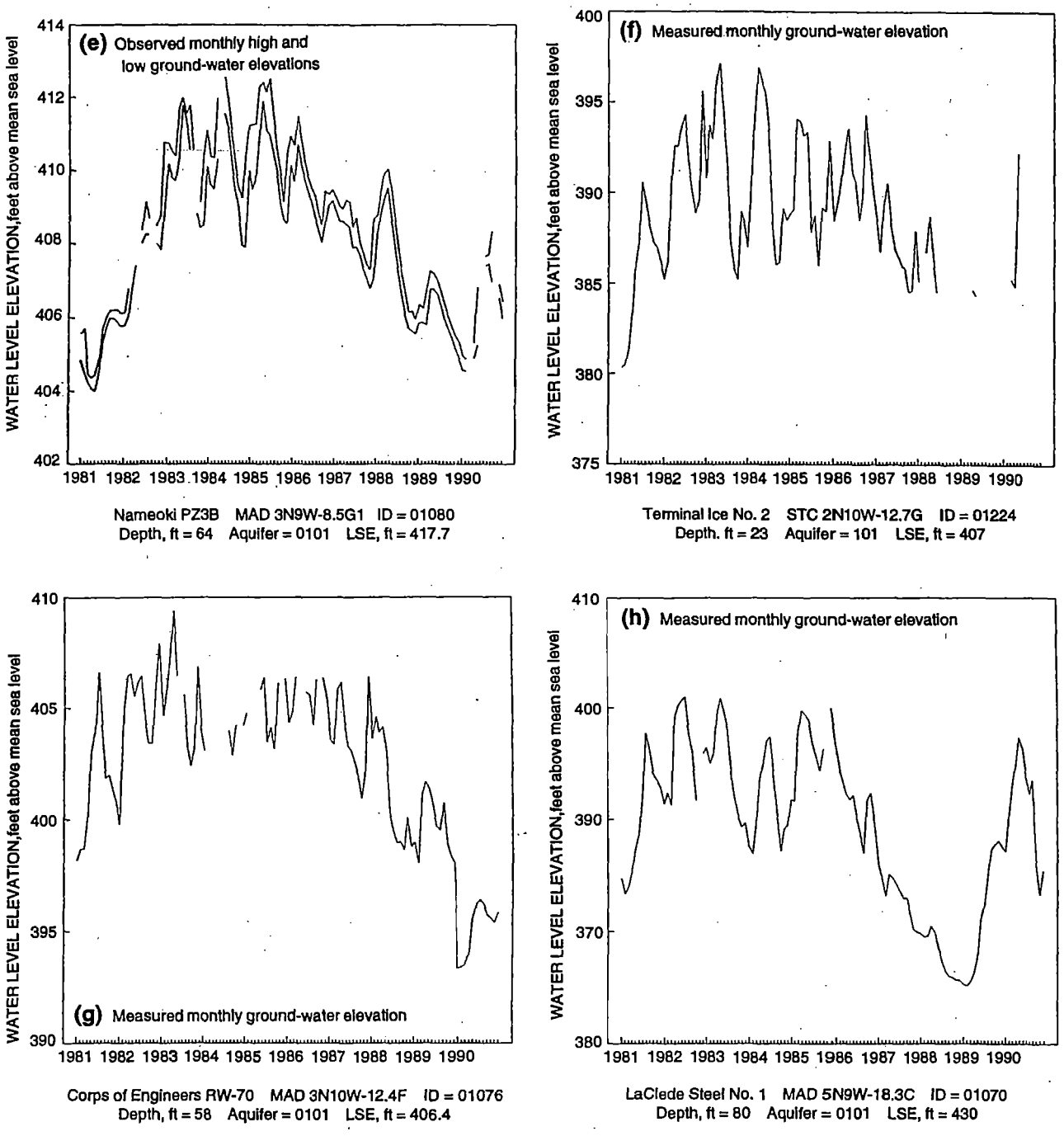


Figure 12. Concluded

correlated closely to ground-water fluctuations for the same time period. In relation to the 120-year mean river elevation, river stages during this time period had a below- and above-average pattern similar to the precipitation pattern.

From 1986-1990, ground-water levels in Well MAD5N9W-29.5g2 (figure 12a) and Well MAD3N10W-14.4b (figure 12b) generally reflect Mississippi River stages. Corresponding peaks in both ground-water hydrographs reflect high and low river stages. The effects of the drought of 1988-1989 are very evident (declining water levels) in both the mean monthly Mississippi River stage graph (figure 11) and in the hydrographs for both wells.

The magnitude of water-level change from 1986-1990 was controlled by each well's proximity to pumping centers and to the Mississippi River and other surface water bodies. Well MAD3N9W-14.2c (figure 12c) near the northeast end of Horseshoe Lake is a good example of a well that is not strongly affected by a pumping center and that has the stabilizing influence of Horseshoe Lake nearby and no drainageway in the immediate area. These conditions result in an annual fluctuation of water levels in this well of about 3 feet, more variation than in Well MAD3N9W-16.8a (figure 10a) discussed previously.The lesser fluctuation at Well MAD3N9W-16.8a is explained by the presence of the adjacent drainageway and the well's proximity to Horseshoe Lake.

Ground-water levels in Well STC2N9W-26.8f2 (figure 12d) and Well MAD3N9W-8.5g1 (figure 12e) vary in an almost identical manner, probably because both wells are in urban areas. The presence of high-density buildings and large paved areas limits the area through which vertical recharge can occur. Also, as a result of the network of storm drainage in urban areas, potential recharge from precipitation is carried away quickly, resulting in moderate water-level changes. In contrast, water levels in Well STC2N10W-12.7g (figure 12f) are impacted heavily by pumpage and by river-stage levels. The resulting impact of these influences is an annual water-level change of 5 feet. During the period 1986-1990, pumpage increased approximately 13 percent over the previous five-year period at the National City pumping center and low river stages during the drought of 1988-1989, which contributed to water levels receding below the bottom of Well STC2N10W-12.7g from July 1988 to March 1990. Rapid and dramatic water-level changes occur at Well MAD3N10W-12.4f (figure 12g) and Well MAD5N9W-18.3c (figure 12h) because of the effect of fluctuations in the Mississippi River. Declining water levels during this same period reflect below-average precipitation and river stages during 1988 and 1989 in the hydrographs for Wells MAD3N10W-12.4f and MAD5N9W-18.3c. This downward trend in ground-water levels was reversed during 1990 when precipitation and Mississippi River stages were well above normal.

Figure 13(a-d) shows hydrographs of selected wells for the entire period of record. Well MAD3N8W-31.1a (figure 13a) reflects the slight downward trend of water levels in the Collinsville area as a result of the growing pumping cone. Wells MAD3N9W-16.8a (figure 13b), MAD3N10W-12.4f (figure 13c), and MAD5N9W-27.5a1 (figure 13d) indicate that the trend of continuously rising water levels, experienced in the area since 1965 because of the overall decrease in ground-water use and shifts in the distribution of pumpage, has ceased. From 1985-1990, hydrographs for these wells have shown a stabilized to a slight downward trend. Relatively consistent pumpage from 1981-1990 has led to these pumping centers having less influence on the surrounding water levels. The controlling factors in water-level trends between 1981 and 1990 appear to be precipitation and stream levels.

When a major ground-water user began using the Mississippi River as a water-supply source after June 1957, water levels in nearby wells recovered quickly, averaging 12 feet per year through 1961. This dramatic trend is shown in the hydrograph for Well MAD3N9W-16.8a (figure 13b).

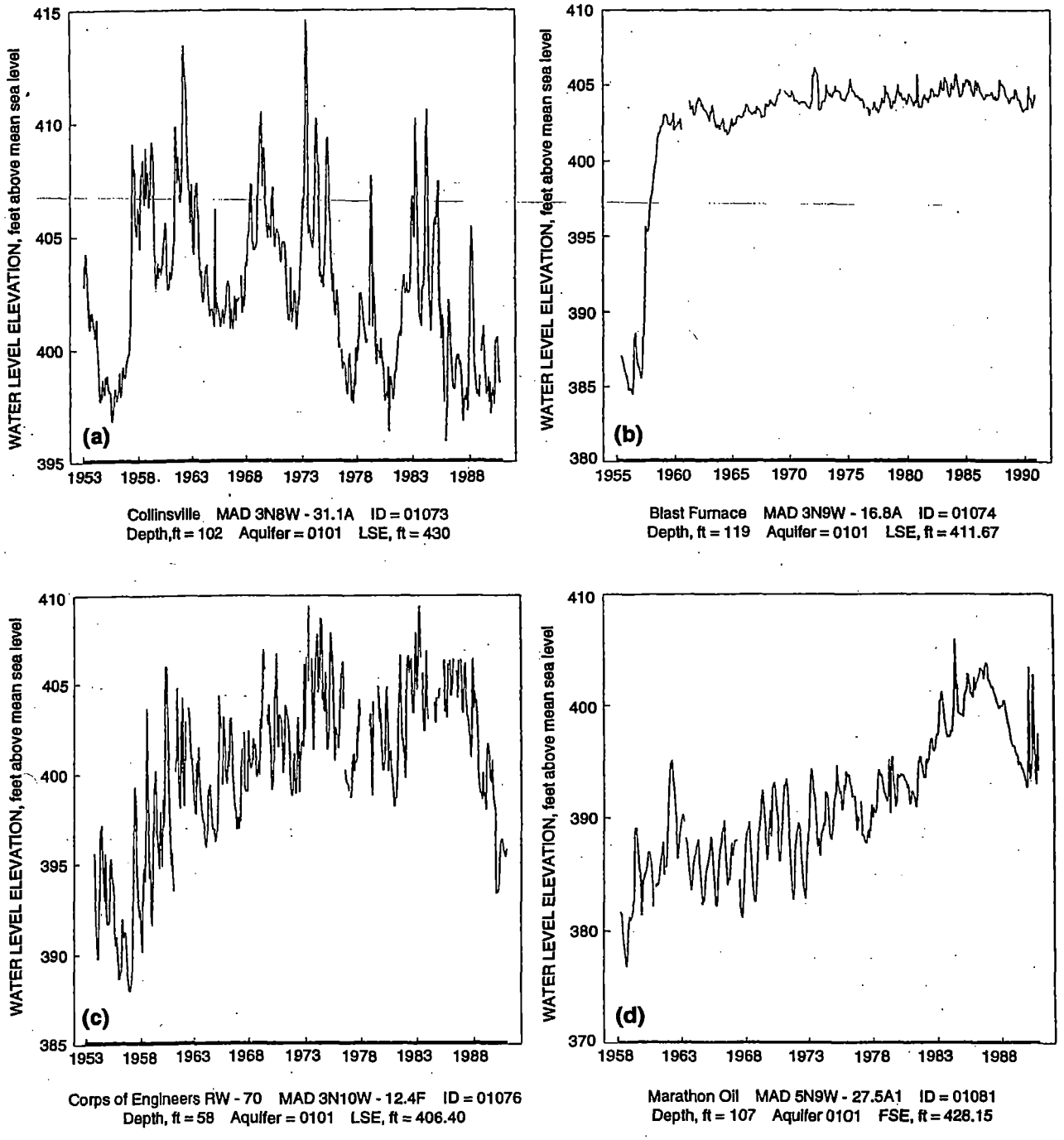


Figure 13. Hydrographs of four selected wells for entire period of record: a) 1953-1988, b) 1955-1990, c) 1953-1988, and d) 1958-1988

POTENTIOMETRIC SURFACE: NOVEMBER 1990

A potentiometric surface map (figure 14) was prepared from water levels measured in 269 wells during late October and early November 1990 when water levels are usually near minimum stages. Figure 15 provides locations of wells, and the appendix provides ground-water level data used to prepare the map. Tables 3 and 4 indicate surface water elevations used in preparing the potentiometric surface map.

Prior to development of large ground-water supplies, ground-water movement was toward the Mississippi River and other streams and lakes. During high river stages, flow was from the river. With the development of large ground-water supplies, however, the general pattern of ground-water flow has been toward the cones of depression created by pumpage or the Mississippi River and lakes and other streams. In places where cones of depression are near the river, hydraulic gradients from the river have been established and significant quantities of river water are diverted into the pumping centers.

The main features of the November 1990 potentiometric map (figure 14) are the deep cones of depression along the Mississippi River just south of Alton and near National City. The cone of depression at Alton was formed by pumping for dewatering during construction of the Mel Price Lock and Dam. The cone of depression near National City is the result of dewatering to maintain ground-water elevations below the highway surface in areas where the highway is depressed below the original land surface.

Other features include cones of depression associated mainly with industrial pumpage just south of the bluffs near Alton and at Wood River, Roxana, and Granite City. A cone of depression along the bluffs near Collinsville is the result of pumpage for municipal use. Withdrawals in the vicinity of Sauget were negligible in 1990. Consequently, the cone of depression associated with industrial pumpage at Sauget has disappeared, and ground-water movement in the vicinity was toward the river.

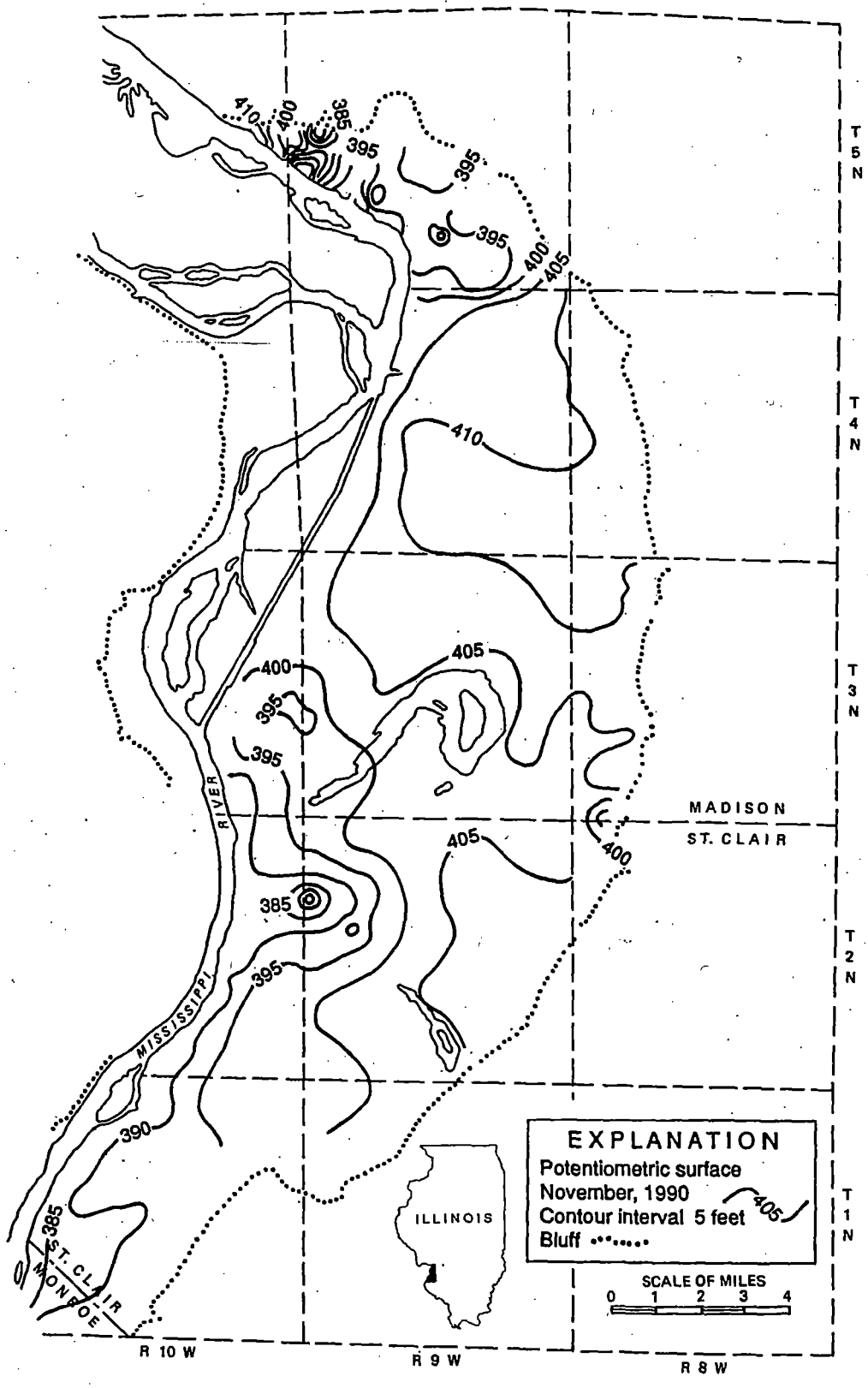


Figure 14. Approximate elevation of potentiometric surface, November 1990

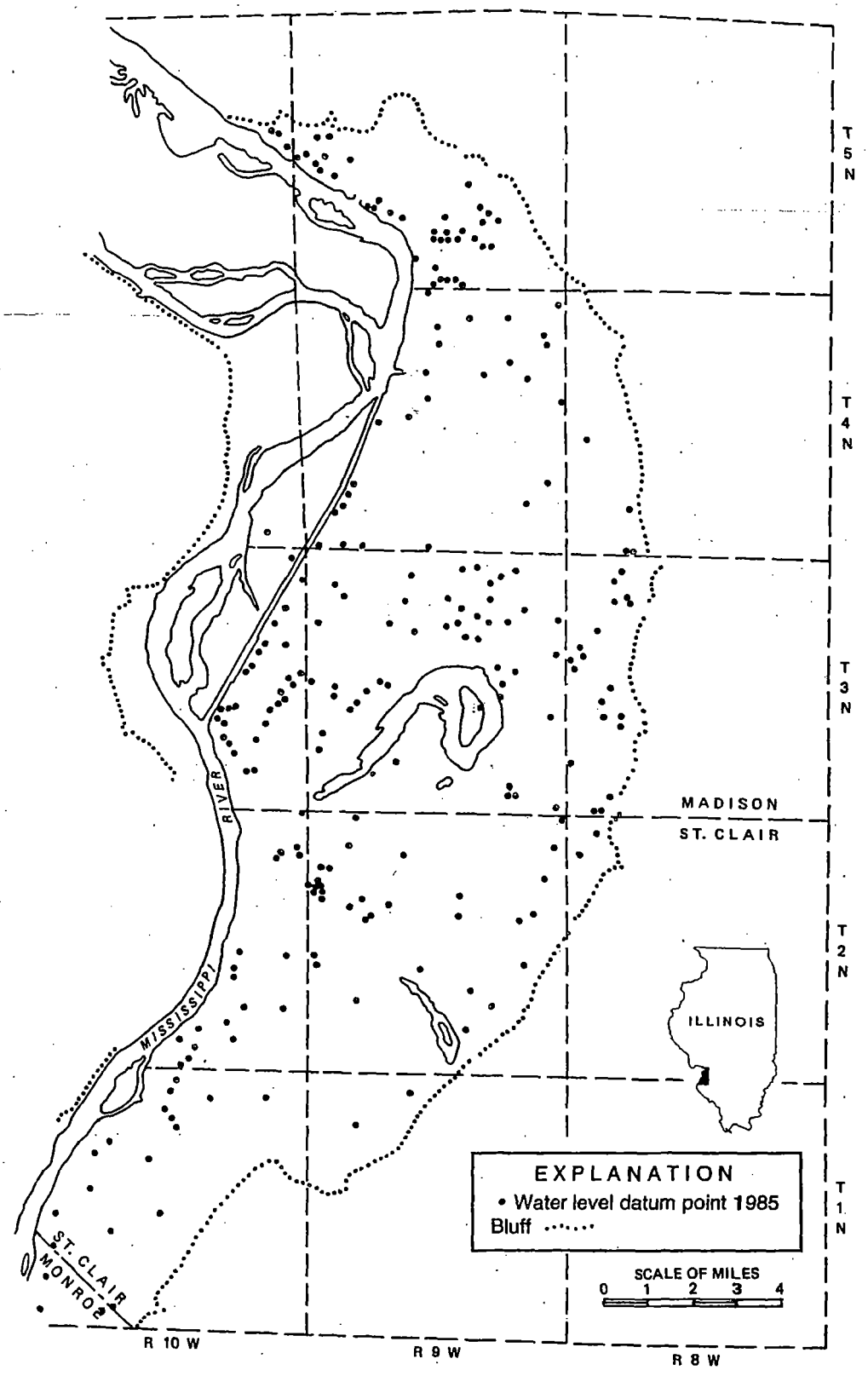


Figure 15. Location of datum points used for 1990 mass measurement

Table 3. Stream Elevations

Location of gage	*Elevation of measuring point (feet above msl)*	*Water-surface elevation (feet above msl)*	
		Nov. 1985	*Nov. 1990*
Illinois Route 203 Bridge, NW corner, Sec. 5, T2N, R9W	415.30	400.27	400.61
Black Lane Bridge, Canteen Creek, near center Sec. 36, T3N, R9W	420.80	401.55	-
Sand Prairie Road Bridge, Canteen Creek, near center Sec. 35, T3N, R9W	418.04	401.11	400.19
Sand Prairie Road Bridge, NW corner, Sec. 35, T3N, R9W	418.55	400.45	399.93
Highway Bridge, 1, NE corner, Sec. 16, T4N, R9W	444.36	414.39	414.02
Highway Bridge, 2, NW corner, Sec. 14, T4N, R9W	440.42	414.23	413.79
Highway Bridge, 3, NW corner, Sec. 13, T4N, R9W	441.38	414.26	413.88

Table 4. Mississippi River Stages

Gage description	*Mississippi River mile number*	*Water-surface elevation (feet above msl)*		
		11/12/80	*11/11/85*	*11/1/90*
Lock and Dam No. 26 Alton, IL (lower)	202.7	418.9	408.48	408.48
Mel Price (upper)	201.1	-	-	418.69
Mel Price (lower)	200.5	-	-	400.78
Hartford, IL	196.8	399.3	406.83	400.32
Lock and Dam No. 27 Granite City (upper)	185.3	-	-	399.86
Lock and Dam No. 27 Granite City (lower)	185.1	-	-	386.91
St. Louis, MO	179.6	383.4	394.34	384.84
Engineer Depot, MO	176.8	382.7	393.58	383.58

CHANGES IN GROUND-WATER LEVELS

November 1985-1990

Figure 16 shows ground-water level changes from November 1985-November 1990. Changes were estimated by comparing potentiometric surface maps for 1985 (figure 17) and 1990 (figure 14). Significant declines exceeding 25 feet occurred along the Mississippi River a few miles south of Alton adjacent to the Mel Price Lock and Dam as a result of dewatering during construction of the lock and dam. Ground-water level declines exceeded 5 feet in an area extending from Granite City to Sauget, and continuing in a narrow band south along the river to the edge of the study area. These changes were attributed to a significant change in river stage (figure 10) between November 1985 and November 1990. No changes were recorded in the vicinity of the main highway dewatering area near National City where pumpage is adjusted to maintain constant water levels. Ground-water levels were less than 5 feet below 1985 levels in the rest of the area except for a large area in the vicinity of Wood River and Roxana where declines exceeded 5 feet. These changes were attributed to below normal precipitation in 1988 and 1989 (figure 11). Although precipitation was above normal during 1990, ground-water levels had not recovered completely.

November 1966-1990

To show the impact of large declines in ground-water pumpage, a water-level change map for the period November 1966-1990 (figure 18) was estimated by comparing the potentiometric surface maps for 1966 (figure 19) and 1990 (figure 14). Ground-water pumpage was 108.1 mgd in 1966, near the peak of 111.0 mgd recorded in 1956 (Reitz, 1968). By 1990, ground-water pumpage declined to 58.7 mgd. Table 5 shows declines in pumping for each major pumping center. Pumping for dewatering during construction of the Mel Price Lock and Dam near Alton was not included in the Alton total because it is difficult to estimate and is only temporary.

Except for a narrow strip along the bluffs from Collinsville to just south of Cahokia Diversion Canal, an area in the vicinity of Alton, and a small area in the vicinity of East Carondelet along the Mississippi River, ground-water levels rose between November 1966 and November 1990, mainly because of the reduction in pumpage.

With the exception of the Alton area, ground-water levels in the vicinity of pumping centers rose during the 1966-1990 period. At Alton the impact of a large decline in estimated pumpage (7.6 mgd) was balanced by the dewatering pumpage at the Mel Price Lock and Dam and water levels not significantly different in 1990 than in previous years. The greatest recovery occurred at the Sauget pumping center where water levels rose more than 65 feet. Pumpage at Sauget for the period declined 27.3 mgd. Water-level recovery exceeded 10 feet at Wood River and exceeded 15 feet at Granite City and north of the National City pumping center. Because of the large quantities of ground water withdrawn for the highway dewatering system, ground-water level recovery was significantly less along interstate highways in the vicinity of National City as shown in figure 18. Recovery of water levels was less than 10 feet and in some areas less than 5 feet in a broad band along the interstate highway.

Ground-water levels declined in a narrow band along the bluffs from the Cahokia Diversion Channel to Collinsville. Declines also occurred along the Mississippi River south of Cahokia Canal and in small areas in Wood River and East Alton.

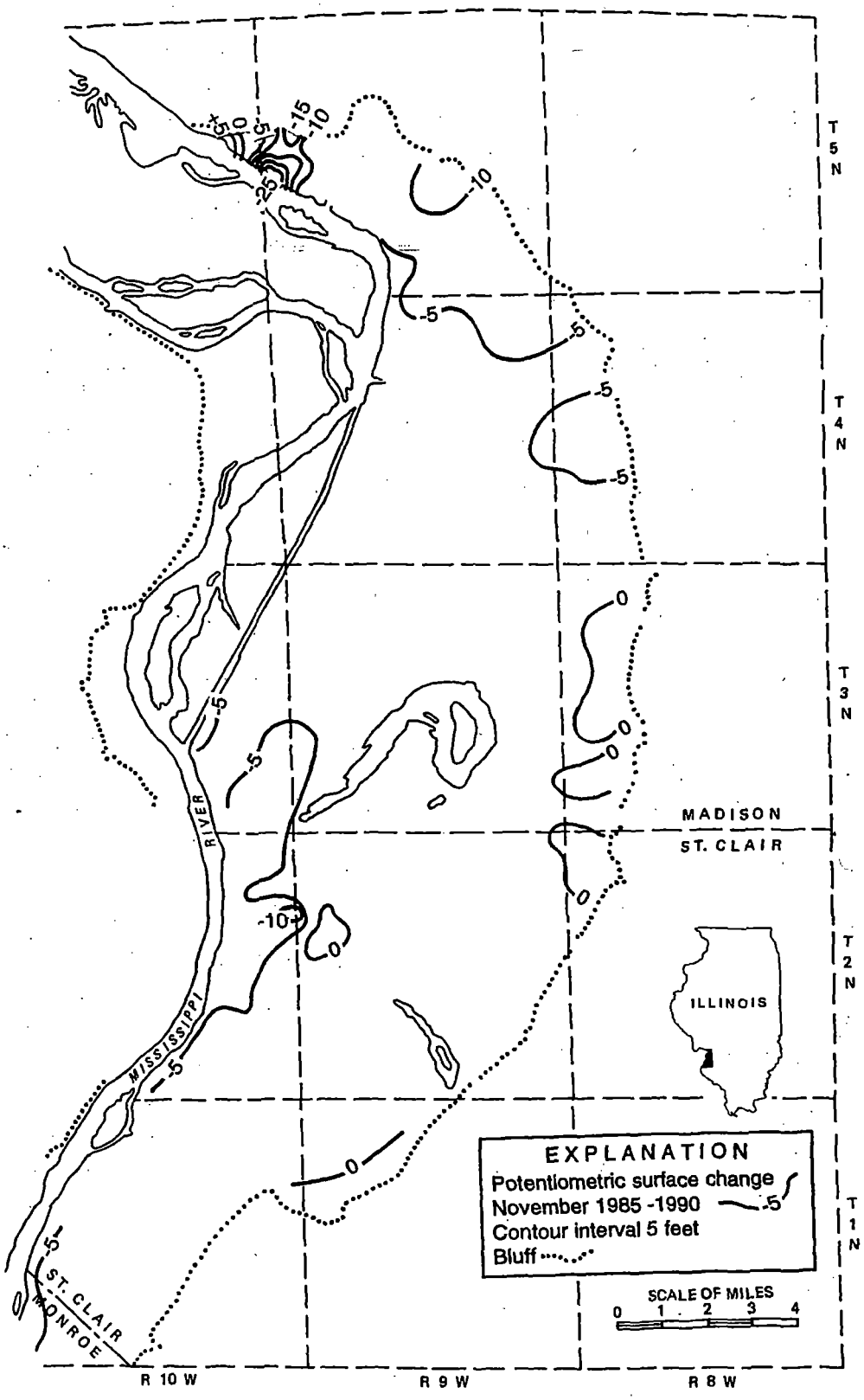
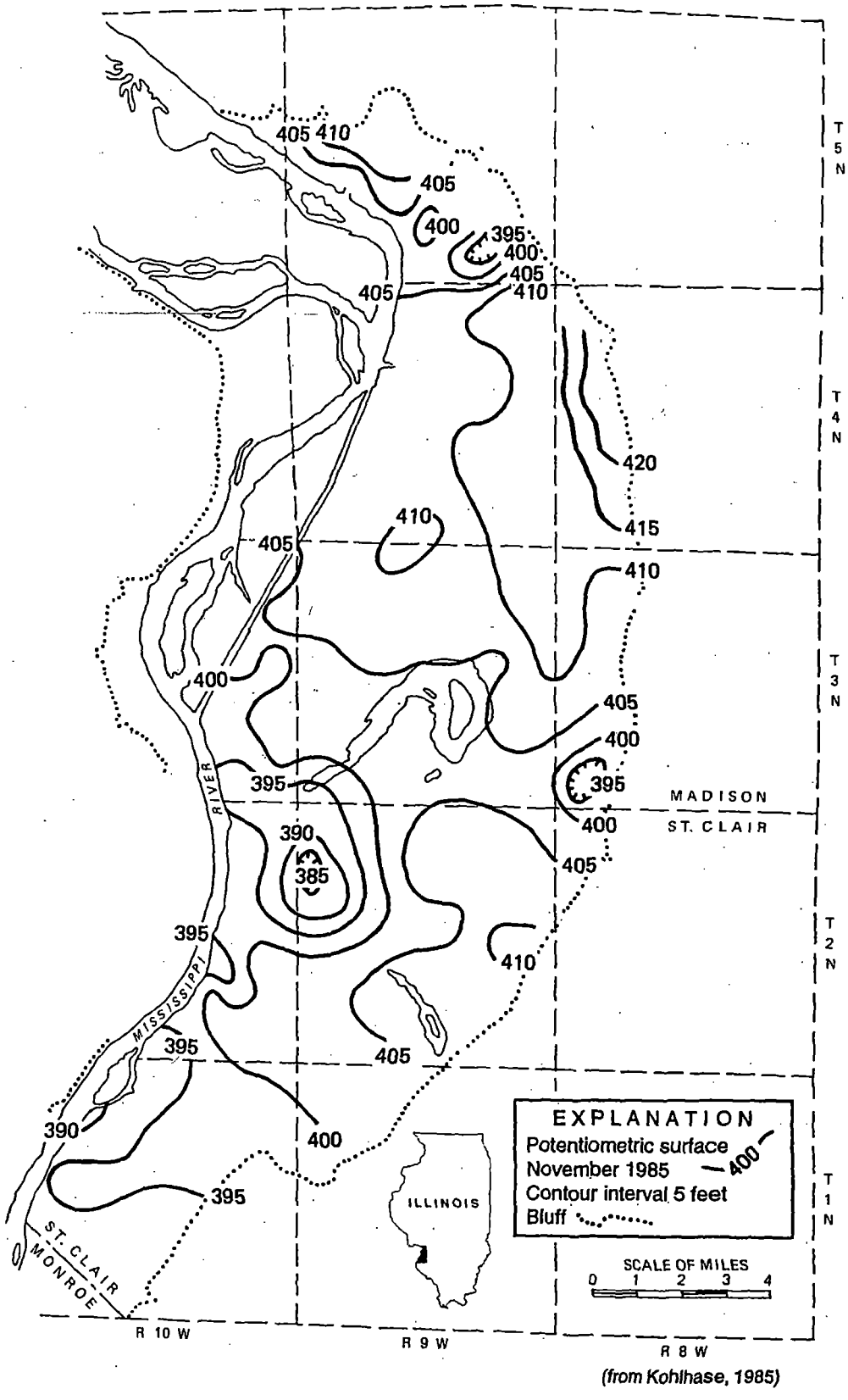


Figure 16. Changes in potentiometric surface, November 1985-November 1990



(from Kohlhase, 1985)

Figure 17. Appproximate elevation of potentiometric surface, November 1985 (from Kohlhase, 1985)

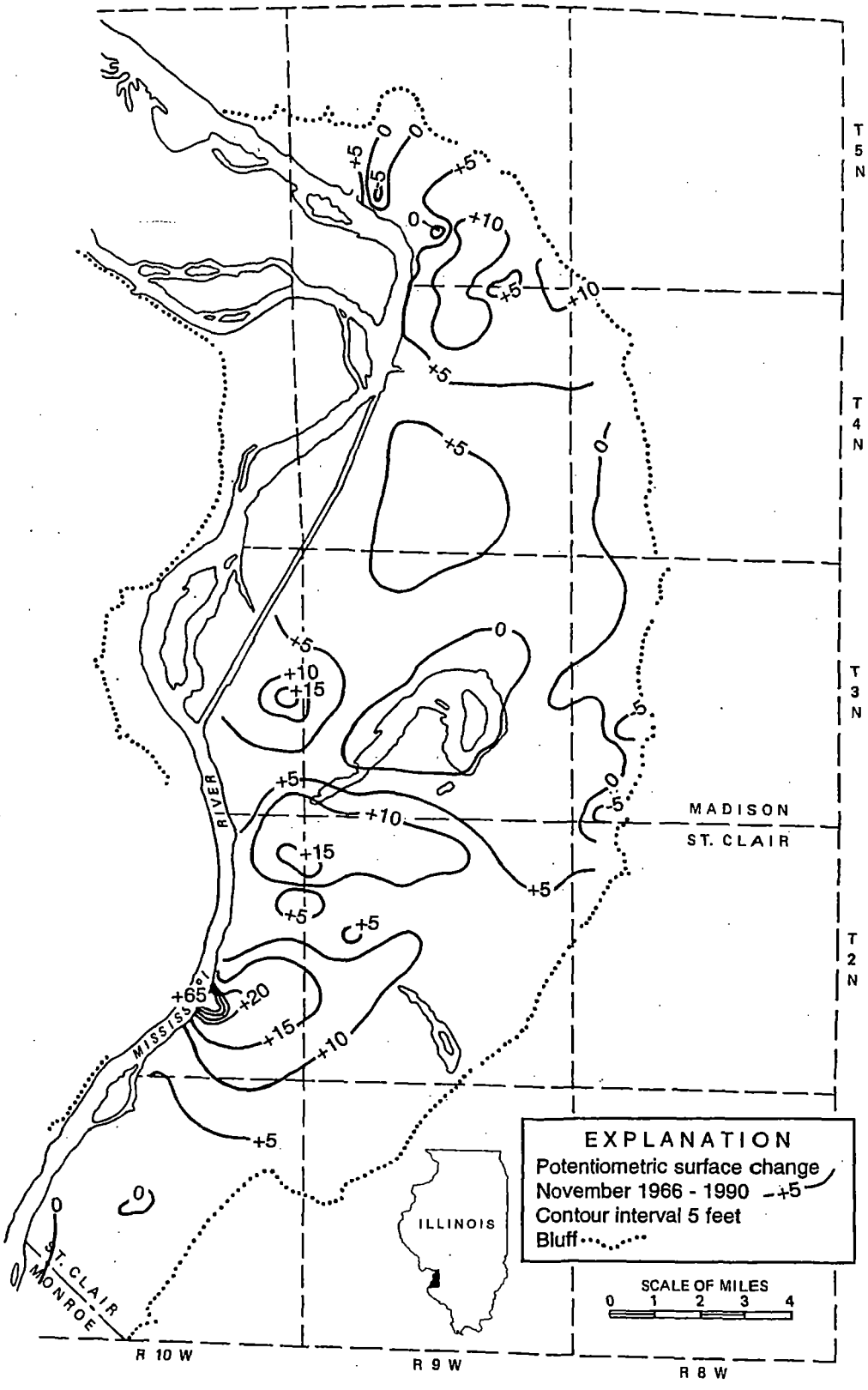


Figure 18. Changes in potentiometric surface, November 1966-November 1990

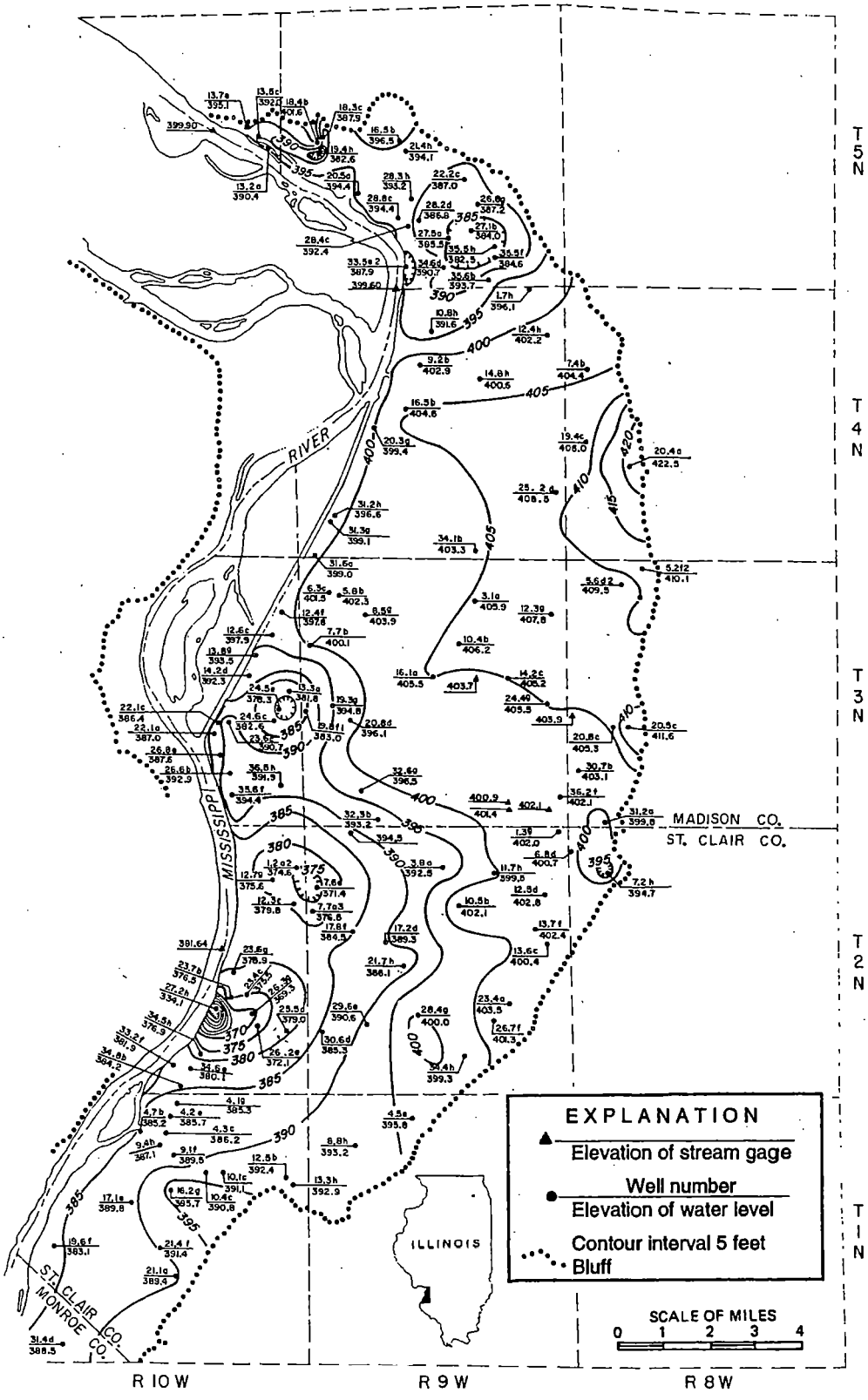


Figure 19. Approximate elevation of potentiometric surface for 1966 (from Reitz, 1968)

Table 5. Declines in Pumpage (mgd): Major Pumping Centers, 1966-1990

Pumping center	*Pumpage decline (mgd)*
Alton	7.6
Wood River	8.6
Granite City	5.3
National City	2.1
Sauget	27.3

Note: Total for Alton does not include pumpage due to dewatering for Mel Price Lock and Dam.

AREAS OF DIVERSION

Figure 20 shows boundaries of areas of diversion of pumping centers for November 1990. The boundaries delimit areas within which the general movement of ground water is toward pumping centers. In areas where ground-water levels are near the land surface, ground-water may discharge into streams, lakes, or both. It has been more difficult to determine areas of diversion of pumping centers because ground-water levels have recovered significantly in recent years. For this study only, areas of diversion that are easily recognizable on the potentiometric surface are shown.

Hydraulic gradients were established from the Mississippi River toward the pumping centers in the Alton and Wood River areas of diversion. As a result the river contributes a large part of the pumpage.

For the areas of diversion for Granite City, Venice, and National City, a ground-water divide exists between the pumping center and the river. It should be noted that the ground-water areas of diversion shown exist for only the period that water levels weremeasured. Areas of diversion may be distorted markedly by changes in river stage, particularly significant increases in stage and significant rainfall recharge events and significant changes and shifts in pumpage.

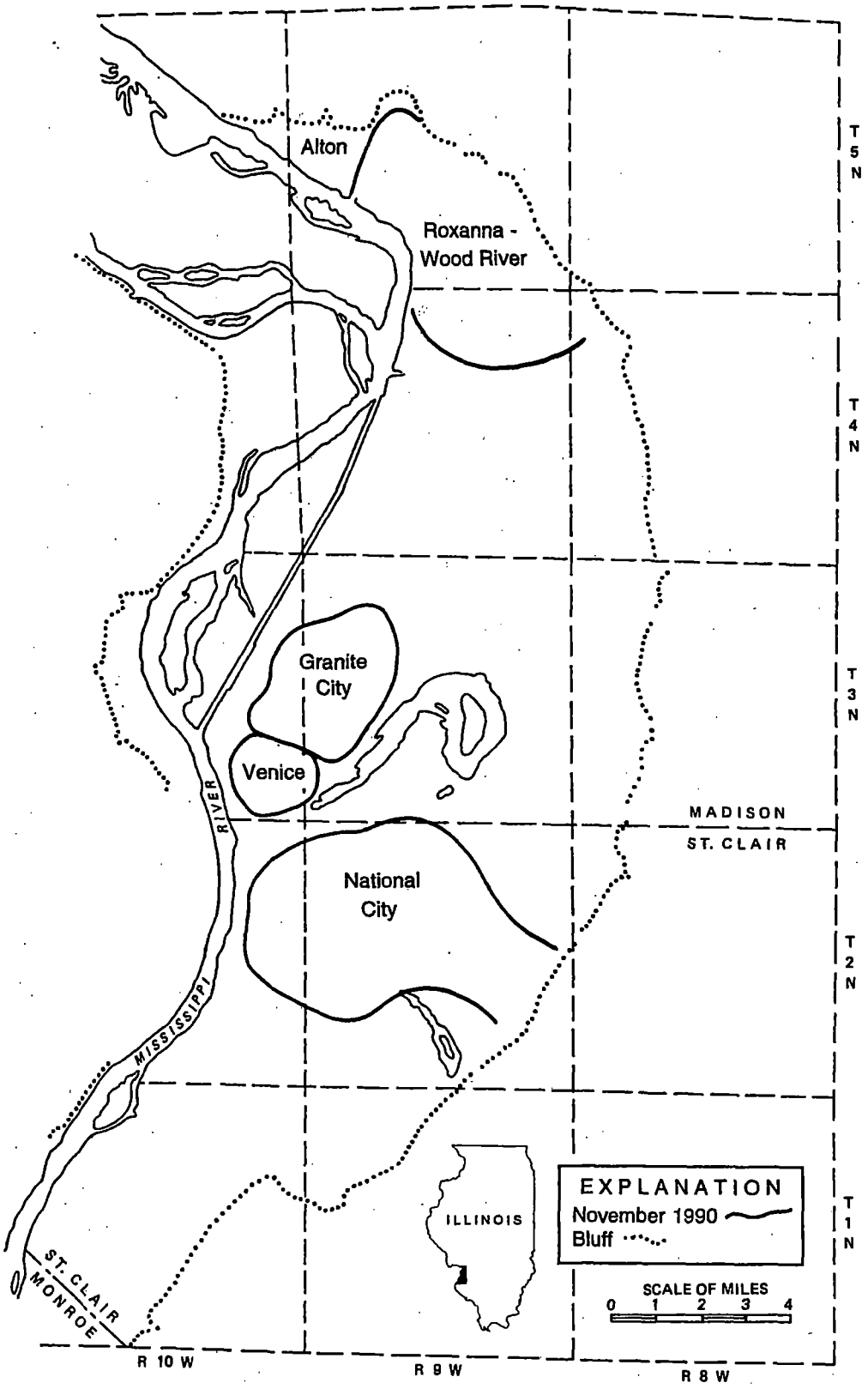


Figure 20. Approximate areas of diversion, November 1990

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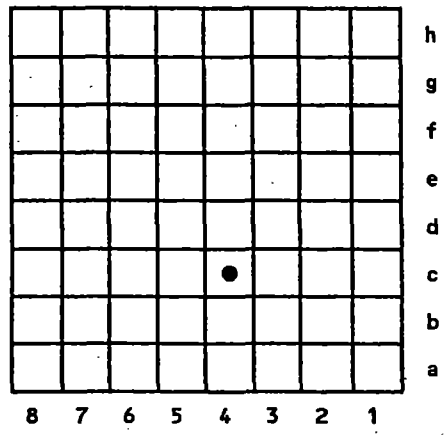
APPENDIX A. WELL NUMBERING SYSTEM

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 (subsection or 10-acre plot). 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.

Example: St. Clair County
T2N, R10W
Section 23



The location of the well shown above is STC 2N10W-23.4c. Where there is more than one well in a 10-acre square, each well is identified by arabic numbers after the lower-case letter in the well number. Any number assigned to the well by the owner is shown in parentheses after the location well number. The abbreviations for counties discussed in this report are:

Madison	MAD	Monroe	MON	St. Clair	STC

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.

APPENDIX B. WATER-LEVEL ELEVATIONS AND CHANGES IN THE METRO-EAST AREA, 1985-1990

County location	*Owner*	*Water-level elevation 1985 (ft)*	*Water-level elevation 1990 (ft)*	*Water-level change 1985-1990 (ft)*
Madison				
3N08W05.2d	SCHWARTZ		407.83	
3N08W05.2f2	V OF GLEN CARBON #2 (sealed > 1982)			
3N08W05.2f3	V OF GLEN CARBON #3 (sealed > 1982)			
3N08W05.4a1	V OF MARYVILLE #1		404.80	
3N08W05.4a2	V OF MARYVILLE #2		404.30	
3N08W05.4a3	V OF MARYVILLE #3		405.60	
3N08W05.4a4	MARYVILLE WELL FIELD - ME4		403.81	
3N08W05.4h	LOHR BROS CONST	413.23	403.52	-9.71
3N08W05.5e	V OF GLEN CARBON #6	409.99	409.12	-0.87
3N08W05.6d1	V OF GLEN CARBON #4	410.43	409.96	-0.47
3N08W05.6d2	V OF GLEN CARBON #5	411.06	410.08	-0.98
3N08W08.4g	KELLER #3			
3N08W08.6h	WILLAREDT, HARLEY	408.00	410.38	2.38
3N08W18.7e	ARLINGTON GOLF COURSE		408.95	
3N08W19.1f	FERD STRACKETJAHN	408.66	408.93	0.27
3N08W19.7e	HADLEY BRIDGE			
3N08W20.5a1	V OF TROY WELL #1	403.04	402.90	-0.14
3N08W20.5a2	V OF TROY WELL #2	401.94	402.98	1.04
3N08W20.5a3	V OF TROY WELL #3	403.88	402.31	-1.57
3N08W20.5a4	V OF TROY WELL #4		403.75	
3N08W20.5c	TED KOSTEN JR.		405.05	
3N08W20.8c	E FOURNIE	406.36	406.37	0.01
3N08W30.7b	V W ECKMANN	403.83	403.89	0.06
3N08W31.1a1	COLLINSVILLE OB WELL ID#1073	397.68	398.63	0.95
3N08W31.1a2	C OF COLLINSVILLE #7	390.25	390.05	-0.20
3N08W31.1a3	C OF COLLINSVILLE #8		388.00	
3N08W31.1a4	C OF COLLINSVILLE #11		392.43	
3N08W31.2a1	C OF COLLINSVILLE #9		391.66	
3N08W31.2a2	C OF COLLINSVILLE #10	390.44		
3N08W32.8d	WATSON		399.72	
3N09W03.1a	CARL ELLIS	408.29	406.95	-1.34
3N09W04.5e1	C OF GRANITE CITY P-2			
3N09W04.5e2	MARYVILLE SCHOOL - ME1		408.40	
3N09W06.1a	HERBERT BISCHOFF #1	408.56	406.27	-2.29
3N09W06.3c	HERBERT BISCHOFF #2	407.81	405.46	-2.35
3N09W07.6d	A O SMITH CO WELL A	407.07	403.73	-3.34
3N09W08.1d	C OF GRANITE CITY P-5	408.52	406.91	-1.61
3N09W09.4c1	MIKE GRAVES			
3N09W09.4c2	PARKVIEW SCHOOL - ME3		408.53	
3N09W10.2a	WILBERT ENGELKE (S of tracks)	406.58	406.15	-0.43
3N09W10.4b	WILBERT ENGELKE (destroyed > 1985)	408.03		
3N09W10.4g1	C OF GRANITE CITY P-4			
3N09W10.4g2	C OF GRANITE CITY P-4A	408.72	407.96	-0.76
3N09W10.5d	GOLF COURSE (THE REGENCY)		418.28	
3N09W10.6c	M ORASCO			
3N09W12.3g	CHARLES LUEHMANN	413.42	412.84	-0.58

APPENDIX B. (Continued)

County location	*Owner*	*Water-level elevation 1985 (ft)*	*Water-level elevation 1990 (ft)*	*Water-level change 1985-1990 (ft)*
Madison				
3N09W14.2c	HANDFELDER	405.05	404.29	-0.76
3N09W14.4a	L J ROSS LUMBER CO	405.04	404.69	-0.35
3N09W16.8a	BLAST FURNACE E-2 (covered 1970)			
3N09W17.2a1	LAKE SCHOOL - ME15		404.77	
3N09W17.3a	V BISCHOFF	406.48	405.37	-1.11
3N09W18.8a1	C OF GRANITE CY P-6	402.60		
3N09W18.8a2	C OF GRANITE CY P-6A	402.59	398.87	-3.72
3N09W19.3g	GRANITE CY STEEL #1	400.18	400.85	0.67
3N09W19.3h	GROVE PLUMB & HEAT	404.02	401.68	-2.34
3N09W19.8f1	GRANITE CY STEEL #3			
3N09W20.7e	GRANITE CY STEEL #4	404.14	402.57	-1.57
3N09W20.8d2	GRANITE CY STEEL #12	402.88	401.40	-1.48
3N09W20.8d3	GRANITE CY STEEL #6	402.45		
3N09W20.8d4	GRANITE CY STEEL #14 (NEW)	402.74	401.01	-1.73
3N09W23.5f	DEPT OF CONSERV #1	403.24	403.23	-0.01
3N09W23.8e1	DEPT OF CONSERV #2	404.86	404.37	-0.49
3N09W23.8e2	DOC @ WALKER'S ISLAND - ME5		404.15	
3N09W24.3c	V BRUNS		404.06	
3N09W24.4g	HOLIDAY PK MOB HOMES	410.44		
3N09W25.5f	HERBERT BISCHOFF	405.10	405.07	-0.03
3N09W25.8e	WM BRUNS #1	406.11	405.67	-0.44
3N09W28.5a	BIG BEND ROAD - ME7		402.50	
3N09W29.1a	WILLIAM STEIMAN	400.23	404.40	4.17
3N09W30.5h1	MADISON MIDDLE SCHOOL - ME6		396.77	
3N09W30.6e	MADISON HIGH SCHOOL		399.04	
3N09W32.3b	HENRY MUELLER			
3N09W32.6g	ESTELLA AUFDERHEID	400.43	398.85	-1.58
3N09W35.3d	ST OF ILLINOIS	404.04	403.53	-0.51
3N09W36.1f	VICTOR ECKMANN SR (sealed > 1977)			
3N09W36.3b	KREITNER SCHOOL - ME8		403.05	
3N10W01.1c	E ST L D&L DIS RW98	405.01	401.50	-3.51
3N10W12.4f	E ST L D&L DIS RW69	404.56	401.29	-3.27
3N10W12.6c	E ST L D&L DIS RW56	404.43	401.00	-3.43
3N10W13.1b3	NESTLES CO WELL #3	402.13	399.31	-2.82
3N10W13.1b4	NESTLES CO WELL #4			
3N10W13.2b	NESTLES CO WELL #5	398.84		
3N10W13.4a	DON PARTNEY--CS2	397.61	394.35	-3.26
3N10W13.4g1	PRATHER SCHOOL - ME2		401.70	
3N10W13.8g2	APEX OIL CO	414.21	410.90	-3.31
3N10W13.8g3	E ST L D&L DIS RW37	408.48	401.64	-6.84
3N10W14.1f	E ST L D&L DIS RW33 (SOUTH)	403.33	400.22	-3.11
3N10W14.3c	E ST L D&L DIS RW24	402.61	399.63	-2.98
3N10W14.4b	E ST L D&L DIS RW18	403.14	400.02	-3.12
3N10W22.1a1	E ST L D&L DIS RW43			
3N10W22.1a2	E ST L D&L DIS RW44	398.19	392.54	-5.65
3N10W22.1c1	E ST L D&L DIS RW33 (NORTH)			
3N10W22.1c2	E ST L D&L DIS RW32	397.57	392.33	-5.24
3N10W23.6c	E ST L D&L DIS RW7	399.87	396.25	-3.62

APPENDIX B. (Continued)

County location	Owner	Water-level elevation 1985 (ft)	Water-level elevation 1990 (ft)	Water-level change 1985-1990 (ft)
Madison				
3N10W23.7c	E ST L D&L DIS RW20	398.05	393.21	-4.84
3N10W24.1c	GRANITE CY STEEL #2		391.96	
3N10W24.3h	PRAIRE FARMS DAIRY	401.27	398.00	-3.27
3N10W24.5e	GRANITE CY STEEL #14	396.47	393.52	-2.95
3N10W24.5f	GRANITE CY STEEL #16	397.01	394.22	-2.79
3N10W24.6d	GRANITE CY STEEL #15	401.41	398.50	-2.91
3N10W24.7c	GRANITE CY STEEL #17	399.33	396.42	-2.91
3N10W25.8h	COVALCO	401.10	398.40	-2.70
3N10W26.2e1	DUNBAR SCHOOL - ME16		398.76	
3N10W26.6b	E ST L D&L DIS RW78	397.44	392.73	-4.71
3N10W26.7d	E ST L D&L DIS RW70		392.84	
3N10W26.8e	E ST L D&L DIS RW64	398.85	394.16	-4.69
3N10W26.8h	E ST L D&L DIS RW53	397.72	393.09	-4.63
3N10W35.3f	IDOT DEWATERING #4	394.46		
3N10W35.4f	IDOT DEWATERING #1	394.03		
3N10W35.6f	E ST L D&L DIS RW96			
3N10W35.6g	E ST L D&L DIS RW91	397.28		
3N10W35.6h	E ST L D&L DIS RW87	396.87		
3N10W36.5g	MAD INDUS COMPLEX#11	401.10		
3N10W36.5h	LACLEDE STEEL CO #9	400.36		
4N08W17.8b1	SIU EDWRD WELL 1	425.57		
4N08W17.8b2	SIU EDWRD WELL 2	421.13	415.03	-6.10
4N08W18.4c	BROCKMEIR WELL 2	416.36		
4N08W19.4e	I.J. HITTNER		409.85	
4N08W20.4a	BROCKMEIR WELL 1	424.78		
4N08W20.5d	SIU WELL 3	418.91		
4N08W29.4a	OTTO BAUMANN	416.32	414.06	-2.26
4N08W32.3a	VERNON KELLER WELL 1	421.09	413.55	-7.54
4N08W32.4a	VERNON KELLER WELL 2		412.31	
4N09W01.2e	LOSCH FARMS	417.87	410.22	-7.65
4N09W01.7h1	MARRIN DENTON			
4N09W02.3b	VIL OF ROXANA	413.09	407.18	-5.91
4N09W03.2b	EXPLORER PIPELINE CO	409.88	405.49	-4.39
4N09W03.2g	SHELL OIL CO	402.11		
4N09W03.6f	SHELL OIL CO			
4N09W04.2g3	VIL OF HARTFORD WELL 1		397.99	
4N09W04.2g4	VIL OF HARTFORD WELL 2	403.04	396.34	-6.70
4N09W04.2g5	VIL OF HARTFORD WELL 4		404.45	
4N09W04.3f	CITY OF HARTFD WELL 3	404.58	399.56	-5.02
4N09W04.5f	NAT MARINE SERVICE WELL 1			
4N09W04.6e	NAT MARINE SERVICE WELL 2			
4N09W04.7h	HARTFORD, IL RM196.8			
4N09W09.2b	HOEHN WELL (destroyed > 1980)			
4N09W10.8e	CONOCO PIPELINE CO		405.65	
4N09W10.8h	HARTFORD TERMINAL	408.01	404.53	-3.48
4N09W11.3b1	ROXANA DISTR SYSTEM #8			
4N09W11.3b2	ROXANA DISTR SYSTEM #9	410.36		
4N09W11.3b3	ROXANA DISTR SYSTEM #10		405.69	

APPENDIX B. (Continued)

County location	*Owner*	*Water-level elevation 1985 (ft)*	*Water-level elevation 1990 (ft)*	*Water-level change 1985-1990 (ft)*
Madison				
4N09W12.4f	LOSCH FARMS, ROCK HOUSE		409.71	
4N09W12.4h1	LOSCH FARMS IRRIGATION	417.94	411.95	-5.99
4N09W12.4h2	CHARLES LOSCH ABAND.	414.90	409.14	-5.76
4N09W12.4h3	LOSCH FARMS HOUSE	413.88		
4N09W13.1d4	CY OF EDWRDSVE WELL 4			
4N09W13.1d5	CY OF EDWRDSVE WELL 5	413.17	410.47	-2.70
4N09W13.1d7	CY OF EDWRDSVE WELL 7	415.37	410.28	-5.09
4N09W13.1d8	CY OF EDWRDSVE WELL 8	414.96	411.96	-3.00
4N09W13.1d9	CY OF EDWRDSVE WELL 9		409.66	
4N09W14.8h2	E SD LEVEE SAN DIST RW3	405.20	402.62	-2.58
4N09W16.2c1	CHEMETCO METALS CORP WELL 1	408.23	405.91	-2.32
4N09W16.2c2	CHEMETCO METALS CORP WELL 2	407.11	402.98	-4.13
4N09W20.3g	E ST L D&L DIS RW196	406.61	403.44	-3.17
4N09W21.5h	BEN KILLAM		412.86	
4N09W23.5d	SWS DRIVEN PIEZOMETER	411.01		
4N09W25.4e	EDWIN RAPP	412.48	410.69	-1.79
4N09W25.8a2	SWS DRIVEN PIEZOMETER	412.02	411.19	-0.83
4N09W25.8a1	UNKNOWN			
4N09W29.8d	E ST L D&L DIS RW161	405.52	403.08	-2.44
4N09W30.1b	E ST L D&L DIS RW155	405.53	403.07	-2.46
4N09W31.2h	E ST L D&L DIS RW150	405.40	402.92	-2.48
4N09W31.3g	E ST L D&L DIS RW145	405.43	402.69	-2.74
4N09W31.6a	E ST L D&L DIS RW126	405.32	403.09	-2.23
4N09W33.2d	TRI CITY SPEEDWAY	410.38		
4N09W33.4b	CY OF GRANITE CITY P1	410.79		
4N09W34.1b	M. THEIS			
4N10W35.3g	CHAIN OF ROCKS RM190.4			
5N09W18.3c1	ALTON BOX BRD CO WELL 10	391.88		
5N09W18.3c2	ALTON BOX BRD CO WELL 19			
5N09W18.4b	ALTON BOX BRD CO (DIESEL HOUSE)			
5N09W18.4c2	ALTON BOX BRD CO WELL 18		373.71	
5N09W18.5c1	ALTON BOX BRD CO WELL 15	406.16		
5N09W18.5c2	ALTON BOX BRD CO WELL 16	390.50		
5N09W18.6c	ALTON BOX BRD CO WELL 20	394.17	378.87	-15.30
5N09W18.7a	ALTON BOX BRD CO WELL 22	404.85	396.54	-8.31
5N09W18.8b	ALTON BOX BRD CO WELL 23			
5N09W18.8c	LACLEDE STEEL-ALTON PLANT			
5N09W19.3c	WOOD RIVER D&L DIS RW100	407.80		
5N09W19.3d	WOOD RIVER D&L DIS RW99		390.23	
5N09W19.4g	FED METALURGICAL #3			
5N09W19.4h1	FED METALURGICAL #1	399.61	381.27	-18.34
5N09W19.4h2	FED METALURGICAL #2	400.01	383.03	-16.98
5N09W19.6e1	WOOD RIVER D&L DIS RW87XX	406.39	376.94	-29.45
5N09W19.6e2	WOOD RIVER D&L DIS RW80XX	406.65	364.04	-42.61
5N09W19.8g	WOOD RIVER D&L DIS RW68X	406.40	379.22	-27.18
5N09W20.2e	OLIN MATHIESON CHEM CORP #1			
5N09W20.4h1	CY OF E ALTON #1	404.31		
5N09W20.4h2	CY OF E ALTON #2	403.98	392.38	-11.60

APPENDIX B. (Continued)

County location	Owner	Water-level elevation 1985 (ft)	Water-level elevation 1990 (ft)	Water-level change 1985-1990 (ft)
Madison				
5N09W20.4h3	CY OF E ALTON #3	404.41	392.50	-11.91
5N09W20.4h4	CY OF E ALTON #4		394.21	
5N09W20.4h5	CY OF E ALTON #5		398.00	
5N09W20.5a	WOOD RIVER D&L DIS RW105			
5N09W20.8g1	AIRCO INDUST GAS #1			
5N09W20.8g2	AIRCO INDUST GAS #2		397.80	
5N09W21.5c	DOME RAILWAY SERV #1	411.30		
5N09W21.5h1	CY OF E ALTON #15			
5N09W21.5h2	CY OF E ALTON #16			
5N09W21.5h3	CY OF E ALTON #19			
5N09W21.5h4	CY OF E ALTON #11			
5N09W22.2c1	VIL OF BETHAL #1			
5N09W22.2c2	VIL OF BETHAL #2			
5N09W22.2c3	VIL OF BETHAL #3			
5N09W22.2c6	VIL OF BETHAL #6	403.34	392.45	-10.89
5N09W22.2c7	VIL OF BETHAL #7	401.78	391.17	-10.61
5N09W22.2c8	VIL OF BETHAL #8	401.78	392.32	-9.46
5N09W22.2c9	VIL OF BETHAL #9	402.12	391.48	-10.64
5N09W22.2c10	VIL OF BETHAL #10	399.20	388.50	-10.70
5N09W22.2c11	VIL OF BETHAL #11		392.03	
5N09W22.2c12	VIL OF BETHAL #12	399.42	389.92	-9.50
5N09W22.4e	CY OF WOOD RIVER, BELK PARK			
5N09W26.7f	CY OF WOOD RIVER #17	405.89	397.60	-8.29
5N09W26.8d1	VIL OF ROXANA #6			
5N09W26.8d2	WOOD RIVER D&L DIS #136	404.38	398.68	-5.70
5N09W26.8e	VIL OF ROXANA #7			
5N09W26.8g1	CY OF WOOD RIVER #12	408.18		
5N09W26.8g2	CY OF WOOD RIVER #15	405.94		
5N09W26.8g3	CY OF WOOD RIVER #18		397.06	
5N09W27.1b2	VIL OF ROXANA #3			
5N09W27.1b4	VIL OF ROXANA #5			
5N09W27.5a1	MARATHON PLINE S WELL	400.85	395.04	-5.81
5N09W27.5a2	MARATHON OIL N WELL	400.68	393.02	-7.66
5N09W27.7a	AM OIL CO WR REF #60	395.35	390.71	-4.64
5N09W27.7b	AM OIL CO WR REF #42		389.94	
5N09W27.7e1	AM OIL CO WR REF #50	402.68	394.87	-7.81
5N09W27.7e2	AM OIL CO WR REF #51	395.64		
5N09W27.7e3	AM OIL CO WR REF #53	402.91	397.06	-5.85
5N09W27.8a1	AM OIL CO WR REF #58	396.38	388.86	-7.52
5N09W27.8a2	AM OIL CO WR REF #61	396.53	392.16	-4.37
5N09W27.8b1	AM OIL CO WR REF #56	397.38		
5N09W27.8b2	AM OIL CO WR REF #55	411.62		
5N09W27.8b3	AM OIL CO WR REF #65			
5N09W27.8c	AM OIL CO WR REF #33	398.64		
5N09W27.8d1	AM OIL CO WR REF #30			
5N09W27.8d2	AM OIL CO WR REF #52			
5N09W28.1a1	AM OIL CO WR REF #59	398.96	393.09	-5.87
5N09W28.1a2	AM OIL CO WR REF #62	398.37		

APPENDIX B. (Continued)

County location	*Owner*	*Water-level elevation 1985 (ft)*	*Water-level elevation 1990 (ft)*	*Water-level change 1985-1990 (ft)*
Madison				
5N09W28.1b1	AM OIL CO WR REF #46			
5N09W28.1b2	AM OIL CO WR REF #57	399.50	393.47	-6.03
5N09W28.2d	AM OIL CO WR REF TEST			
5N09W28.4c	WOOD R D&L DIS RW146	407.75		
5N09W28.5b1	AMOCO - RIVER WELL #1		398.45	
5N09W28.5b2	AMOCO - RIVER WELL #2		386.03	
5N09W28.5b3	AMOCO - RIVER WELL #3		390.78	
5N09W28.5b4	AMOCO - RIVER WELL #4		389.28	
5N09W28.7e1	CY OF WOOD RIVER #3	410.77		
5N09W28.7e3	CY OF WOOD RIVER #6	405.90		
5N09W28.7e4	WOOD R D&L DIS RW140		398.32	
5N09W28.8e1	CY OF WOOD RIVER #1	402.46		
5N09W28.8e2	CY OF WOOD RIVER #2			
5N09W28.8e5	CY OF WOOD RIVER #5			
5N09W28.8e6	WOOD R D&L DIS RW138	404.11		
5N09W29.1e	WOOD R D&L DIS RW135	405.15	398.80	-6.35
5N09W29.3h1	O MATHIESON CH CO #3			
5N09W29.3h2	O MATHIESON CH CO #4		385.16	
5N09W29.3h3	O MATHIESON CH CO #5	393.87	383.92	-9.95
5N09W29.4g3	O MATHIESON CH CO #2	395.56		
5N09W29.4g1	WOOD R D&L DIS RW114	409.53	402.01	-7.52
5N09W29.4g2	WOOD R D&L DIS RW121			
5N09W29.5f	OLIN CORP		400.99	
5N09W29.5g1	OLIN CHEM - AE1		400.49	
5N09W29.5g2	OLIN CHEM - AN1		399.34	
5N09W33.1a	CLARK OIL & REF - B3-W		398.07	
5N09W33.1d	CLARK OIL & REF - B34-W		397.25	
5N09W33.5e1	SHELL OIL REF N TEST			
5N09W33.5e2	SHELL OIL REF S TEST			
5N09W33.5f	SHELL OIL MIS RIV #4		390.95	
5N09W34.3e1	ANLIN CO #1 (aka ANLIN EAST)			
5N09W34.3e2	ANLIN CO #2 (aka ANLIN WEST)			
5N09W34.4a1	CLARK OIL CO REF #5			
5N09W34.4a2	CLARK OIL CO REF - B25-E		392.44	
5N09W34.5a1	CLARK OIL CO REF #3			
5N09W34.5b	CLARK OIL CO REF - B9-E		396.13	
5N09W34.6a1	CLARK OIL CO REF #1			
5N09W34.6a2	CLARK OIL CO REF #2	400.76		
5N09W34.6b	CLARK OIL CO REF #4			
5N09W34.7b	CLARK OIL CO REF - B6-E		395.49	
5N09W34.7d1	INT'L SHOE CO - WEST WELL	402.86		
5N09W34.7d2	INT'L SHOE CO - EAST WELL	401.35		
5N09W34.8b	CLARK OIL CO REF - B38-W		398.00	
5N09W35.5f	SHELL OIL REF W #52	400.40		
5N09W35.5h	SHELL OIL REF W #41	400.36	391.11	-9.25
5N09W35.6b	SHELL OIL REF W #60			
5N09W35.8h	SHELL OIL CO TW#1	402.40	393.77	-8.63
5N09W36.4c	SHELL OIL CO K H WELL			

APPENDIX B. (Continued)

County location	Owner	Water-level elevation 1985 (ft)	Water-level elevation 1990 (ft)	Water-level change 1985-1990 (ft)
Madison				
5N10W13.1a1	LACLEDE STL CO (ALTON) #1		394.80	
5N10W13.1a2	LACLEDE STL CO (ALTON) #3	407.10		
5N10W13.1b	LACLEDE STL CO (ALTON) #2	401.25		
5N10W13.2a1	WOOD RIVER D&R DIS RW41X		403.92	
5N10W13.2a2	WOOD RIVER D&R DIS RW42X	407.09	403.44	-3.65
5N10W13.4c1	OWENS IL GLASS CO #1			
5N10W13.4c3	OWENS IL GLASS CO #3	402.03		
5N10W13.4c6	OWENS IL GLASS CO #6	402.03		
5N10W13.4c7	OWENS IL GLASS CO #7	401.18		
5N10W13.4c8	OWENS IL GLASS CO - COE WELL		404.80	
5N10W13.5c	WOOD RIV D&L DIS RW20	408.84	411.17	2.33
5N10W13.5d1	WOOD RIV D&L DIS RW16			
5N10W13.5d2	WOOD RIV D&L DIS RW18	410.50		
5N10W14.4e	LOCK & DAM #26			
5N10W24.1h	WOOD RIV D&L DIS RW51	407.50	397.69	-9.81
St. Clair				
1N09W04.5e	E WESTERHEIDE			
1N09W04.6f1	LaLUMIER SCHOOL - ME22		402.50	
1N09W06.1e	SWS PIEZOMETER			
1N09W08.8h	VA RISTER	401.28	400.11	-1.17
1N10W01.8d1	CAHOKIA HIGH SCHOOL - ME13		397.33	
1N10W02.8e	SWS PIEZOMETER			
1N10W03.3c1	HUFFMAN SCHOOL - ME14		395.12	
1N10W04.1g	E ST L D&L DIS RW196	393.90	391.05	-2.85
1N10W04.2e	E ST L D&L DIS RW207	393.36	389.46	-3.90
1N10W04.3b	E ST L D&L DIS RW237		389.86	
1N10W04.3c	E ST L D&L DIS RW223			
1N10W04.7b	PRAIR DUP D&L RW23	390.81	386.95	-3.86
1N10W08.2h	PRAIR DUP D&L RW28	390.33	388.41	-1.92
1N10W08.5c	PRAIR DUP D&L RW34	390.84	389.45	-1.39
1N10W08.7a	PRAIR DUP D&L RW45	390.36	389.02	-1.34
1N10W09.1f	E ST L D&L DIS RW262	394.21	391.99	-2.22
1N10W09.2h	E ST L D&L DIS RW251	395.36	392.09	-3.27
1N10W09.4h	PRAIR DUP D&L RW15	392.92	389.01	-3.91
1N10W10.1c	E ST L D&L DIS RW273			
1N10W10.4c	E ST L D&L DIS RW263			
1N10W12.5b	E ST L D&L DIS RW278	398.61	398.16	-0.45
1N10W13.3h	E ST L D&L DIS RW286	397.74	398.21	0.47
1N10W16.2g	WALTER DRESCHER	397.63		
1N10W16.6h	OSCAR KELLING	395.60	394.74	-0.86
1N10W17.1e	OSCAR KELLING			
1N10W17.5g	D CHARTRAND			
1N10W17.8b	D CHARTRAND	399.64	397.23	-2.41
1N10W19.6f	PRAIR DUP D&L RW46	390.40	386.77	-3.63
1N10W20.4c	C LINDHORST	390.91	389.22	-1.69
1N10W20.5f	D CHARTRAND			
1N10W20.6a	D CHARTRAND			

APPENDIX B. (Continued)

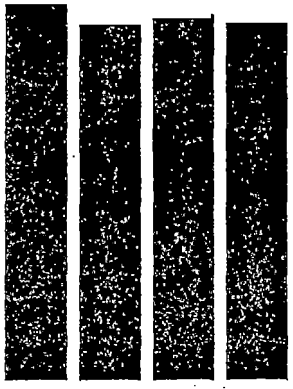
County location	*Owner*	*Water-level elevation 1985 (ft)*	*Water-level elevation 1990 (ft)*	*Water-level change 1985-1990 (ft)*
St. Clair				
1N10W21.1a	BUCK RANGE			
1N10W21.4f	MO PAC RR (PIEZ #3)	391.65	390.29	-1.36
1N10W30.6h	PRAIR DUP D&L RW55	390.50	386.13	-4.37
1N10W30.8b	PRAIR DUP D&L RW69	391.51	384.91	-6.60
1N10W31.4d	LLOYD PULCHER	393.28		
1N10W31.7c	LLOYD PULCHER			
1N10W31.8d	PRAIR DUP D&L RW80	397.04	392.83	-4.21
1N10W32.3e	L W BIELLER	395.30	393.51	-1.79
1N10W32.5d	CLIFFORD CATES	393.24	391.46	-1.78
2N08W06.1e	KELLER BROS #2	399.76	400.38	0.62
2N08W06.5a	C WEISSERT #2	401.33	400.64	-0.69
2N08W06.5h	KELLER BROS #1			
2N08W07.2h2	AUTO WH INC #2			
2N08W07.2h3	ATLAS LEATHER #3			
2N09W01.1h	MOUND PUB WAT DIST #3	401.56	404.16	2.60
2N09W01.3b	A WEISSERT #2	403.80	402.17	-1.63
2N09W01.3g	MOUND PUB WAT DIS #2	402.36		
2N09W02.4e	CAHOKIA MOUNDS ST PK			
2N09W03.2g	FS SERVICE INC #1			
2N09W04.1a	MOBIL CHEM CO			
2N09W04.7a1	ROSELAKE SCHOOL - ME18		401.08	
2N09W06.1b	I70/I55-RTE203 - ME17		396.33	
2N09W07.5e1	CIRCLE PACKING CO #1	388.56		
2N09W07.5e2	CIRCLE PACKING CO #2	389.02	390.18	1.16
2N09W07.5e3	CIRCLE PACKING CO #3			
2N09W07.6a1	IDOT DEWAT I64 #1	380.95	374.35	-6.60
2N09W07.6a2	IDOT DEWAT I64 #2	382.94	377.04	-5.90
2N09W07.6a3	IDOT DEWAT I64 #3	385.48	378.92	-6.56
2N09W07.6a4	IDOT DEWAT I64 #4	385.47	380.27	-5.20
2N09W07.6b1	IDOT DEWAT I70 #10	371.74	370.45	-1.29
2N09W07.6b2	IDOT DEWAT I70 #11	380.43	371.36	-9.07
2N09W07.6b3	IDOT DEWAT I70 #12	385.01		
2N09W07.6c	BOWMAN PUMP STA - ME19		379.49	
2N09W07.6e1	HUNTER PACKING CO #1			
2N09W07.6e3	HUNTER PACKING CO #3	388.91	389.38	0.47
2N09W07.7a1	IDOT DEWAT I64 #11	374.24	376.62	2.38
2N09W07.7a2	IDOT DEWAT I64 #12	383.53		
2N09W07.7b1	IDOT DEWAT I70 #2	381.21		
2N09W07.7b2	IDOT DEWAT I70 #3	369.09	372.14	3.05
2N09W07.7b3	IDOT DEWAT I70 #4	378.77	370.02	-8.75
2N09W07.7b4	IDOT DEWAT I70 #7A	382.25		
2N09W07.7b5	IDOT DEWAT I70 #8A	380.86		
2N09W07.7b6	IDOT DEWAT I70 #9A	379.63		
2N09W07.8b1	IDOT DEWAT I70 #1	375.78	376.30	0.52
2N09W07.8b2	IDOT DEWAT I70 #5	375.76		
2N09W07.8b3	IDOT DEWAT I70 #6	381.87	373.48	-8.39
2N09W10.5a	J E JOUGTARD	410.20	409.00	-1.20
2N09W11.4c	HYTLA			

APPENDIX B. (Continued)

County location	Owner	Water-level elevation 1985 (ft)	Water-level elevation 1990 (ft)	Water-level change 1985-1990 (ft)
St. Clair				
2N09W12.5d1	BILL HENSON (ex VERNON STAFFORD)	407.57	406.34	-1.23
2N09W12.5d2	BILL HENSON #2		407.75	
2N09W13.7f	J COURTNEY	408.14	406.69	-1.45
2N09W14.2e	BLUFFVIEW PARK - ME21		408.68	
2N09W14.3d	NAGLE			
2N09W14.3f	C WEISSERT #3	405.47		
2N09W14.6h	FRANK TOJO			
2N09W15.5e1	C WEISSERT #1	402.10	401.39	-0.71
2N09W15.5e2	A WEISSERT #1	408.63	406.69	-1.94
2N09W16.7a	ESL CASTINGS CO			
2N09W17.2g	CY OF E ST L JONES P	397.63	397.02	-0.61
2N09W17.7h1	CHAS PFIZER INC #12	386.69	384.95	-1.74
2N09W17.7h2	CHAS PFIZER INC #14	389.39	389.67	0.28
2N09W18.1g	ATHLETIC FIELD - ME9		391.53	
2N09W18.6h1	IDOT DEWAT I64 #5	386.37	382.09	-4.28
2N09W18.6h2	IDOT DEWAT I64 #13	385.26	379.70	-5.56
2N09W18.6h3	IDOT DEWAT I64 #14	386.96		
2N09W18.6h4	IDOT DEWAT I64 #15	387.09	383.77	-3.32
2N09W19.7d1	OBER NESTOR GLASS CO (SE WELL)	400.53	399.23	-1.30
2N09W19.7d2	OBER NESTOR GLASS CO (NW WELL)	399.65	398.24	-1.41
2N09W19.8f1	CERTAIN-TEED PROD #1		394.40	
2N09W19.8f2	CERTAIN-TEED PROD #2	396.28	394.85	-1.43
2N09W21.4d	ESL HIGH SCHOOL - ME20		404.37	
2N09W23.1e	RICHARD POPP	410.88	409.98	-0.90
2N09W24.6e	MITCHELLS	410.42		
2N09W26.7e	SWS #2	408.70	407.81	-0.89
2N09W27.3g2	KENNEDY-KING SCHOOL - ME11		406.93	
2N09W27.8g	HOLTEN ST PK (GRAND MARIOS)			
2N09W28.3a	De MANGE			
2N09W28.4g	HOLTEN ST PK (GRAND MARIOS)	408.91	408.32	-0.59
2N09W29.8f1	CHEMTEK PRODS INC #14			
2N09W29.8f2	CHEMTEK PRODS INC #3	405.01	403.97	-1.04
2N09W29.8f3	CHEMTEK PRODS INC #7	405.57		
2N09W29.8f4	CHEMTEK PRODS INC #10			
2N09W29.8f5	CHEMTEK PRODS INC #12			
2N09W29.8f6	CHEMTEK PRODS INC #16		403.66	
2N09W33.1e	VINCE DEMANGE			
2N09W34.4h	H W THOMAS	407.57	406.07	-1.50
2N10W01.2h	USS AG CHEMICALS	390.44	388.32	-2.12
2N10W01.3a	ARMOUR AND CO WELL #2	392.19	390.73	-1.46
2N10W11.4e1	E ST L D&L DIS RW105			
2N10W11.4e2	E ST L D&L DIS RW108			
2N10W12.2h3	NATIONAL CY COLD STRG #6			
2N10W12.3g	SWIFT AND CO #17			
2N10W12.3h1	ARMOR AND CO WELL #4	392.87	392.85	-0.02
2N10W12.3h2	SWIFT AND CO #18	390.88		
2N10W12.6h1	ROYAL PACKING CO #1	395.69		
2N10W12.6h2	ROYAL PACKING CO #2	396.02	389.58	-6.44

APPENDIX B. (Concluded)

County location	*Owner*	*Water-level elevation 1985 (ft)*	*Water-level elevation 1990 (ft)*	*Water-level change 1985-1990 (ft)*
St. Clair				
2N10W12.7g	TERMINAL ICE PLANT	396.96	393.43	-3.53
2N10W14.7e	MISSISSIPPI RM 179.6			
2N10W23.3a3	AM ZINC CO #9	401.46		
2N10W23.4c	MISSISSIPPI AVE WH			
2N10W23.6f	E ST L D&L DIS RW118	394.62	387.19	-7.43
2N10W23.6g	E ST L D&L DIS RW111	395.10		
2N10W23.7a	E ST L D&L DIS RW136	392.72		
2N10W23.7b1	E ST L D&L DIS RW126			
2N10W23.7b2	E ST L D&L DIS RW135		389.35	
2N10W23.7c	E ST L D&L DIS RW127	400.60	393.80	-6.80
2N10W24.4f1	ROBINSON SCHOOL - ME10		395.30	
2N10W25.5d1	MOBIL OIL CO - FIRE HOUSE WELL	398.57	398.11	-0.46
2N10W25.5d2	MOBIL OIL CO #21		398.89	
2N10W25.6e	MOBIL OIL CO #6			
2N10W25.7b	LEFTON IRON & MET #2			
2N10W26.1g1	MONSANTO CHEM CO #13A			
2N10W26.1g2	MONSANTO CHEM CO #8A			
2N10W26.2e	MONSANTO CHEM CO #SR-2 (TEST WELL)			
2N10W26.3g	MONSANTO CHEM CO #14			
2N10W26.4f	MONSANTO CHEM CO #20			
2N10W26.5d2	CERRO COPPER&BRASS #5			
2N10W26.5d3	CERRO COPPER&BRASS #6	394.92		
2N10W26.5d4	CERRO COPPER&BRASS WCD#3		395.51	
2N10W26.5d5	CERRO COPPER&BRASS WCD#8		395.67	
2N10W26.6g	MONSANTO CHEM CO #R-2			
2N10W26.8a2	MIDWEST RUBBER RECLAIM #8			
2N10W26.8a3	MIDWEST RUBBER RECLAIM #10	395.68	393.15	-2.53
2N10W26.8a5	MIDWEST RUBBER RECLAIM SOUTH RES		394.83	
2N10W26.8g	SAUGET WASTE TREAT PLANT			
2N10W27.2h1	MONSANTO CHEM CO #XS-1 TEST WELL			
2N10W27.2h2	MONSANTO CHEM CO #S-1 TEST WELL			
2N10W27.3g	MONSANTO CHEM CO #21			
2N10W27.3h	MONSANTO CHEM CO RANNEY WELL			
2N10W33.1f	FOX TERMINAL	395.17	387.62	-7.55
2N10W33.2h	ENGINEER DEPOT RM 176.8			
2N10W34.5g	E ST L D&L DIS RW138	397.10		
2N10W34.5h	E ST L D&L DIS RW137		390.83	
2N10W34.6e	E ST L D&L DIS RW159	393.92	389.82	-4.10
2N10W34.7c	E ST L D&L DIS RW169	394.09	391.14	-2.95
2N10W34.8b	E ST L D&L DIS RW180	394.74	391.39	-3.35
2N10W35.3e	SWS DRIVEN PIEZOMETER			
2N10W35.7f1	PITZMAN SCHOOL - ME12		396.30	



DRAFT
GSI Job No. G-2561
March 31, 2002

GROUNDWATER
SERVICES, INC.

Interim Groundwater Remedy Design Basis

Sauget Area 2
Sauget, Illinois

ATTACHMENT 16

CHAPTER IV

Strategies for Design of Capture and Containment Remedial Systems

A. Introduction

The first three chapters reviewed the principles of groundwater contamination and summarized the types of field data needed for a proper analysis of a contamination event. We are now ready to turn to the central focus, that of design of pumping systems for capture and containment of groundwater contamination. In this chapter we look at several alternative strategies for the design of such systems. Chapter V presents a detailed design methodology, and Chapter VI provides qualitative guidelines for design in complex settings.

1. Modes of Occurrence of Groundwater Contamination

The remedial strategy to be employed at a site will depend on the location and mode of occurrence of groundwater contamination. Foremost, it is important to know whether contamination is located in unconsolidated surficial materials or in fractured bedrock. It is also important to know whether the contamination occurs in the unsaturated zone above the water table or in the saturated zone below the water table, or both. Finally, it is important to know whether the contaminants are miscible or immiscible, and if immiscible, whether they are LNAPLs or DNAPLs. Summarizing these issues, it is apparent that contamination at a site may occur in one or more of the four following modes: (1) as residual immiscible contamination in the unsaturated zone, (2) as pools of immiscible LNAPL contamination floating on the water table, (3) as pools of immiscible DNAPL contamination at depth in the saturated zone, and (4) as a dissolved-solute plume of miscible contamination in the saturated zone.

Because the emphasis of this book is on capture and containment systems, it should be noted that such systems are best-suited for the remediation of dissolved-solute plumes of miscible contamination in the saturated zone of an unconsolidated aquifer. The complexities discussed in Chapter VI are primarily those introduced by the presence of frac-

tured rock, NAPL pools, and unsaturated-zone contamination. The existence of any of these complexities, especially if they occur in combination, may be sufficiently adverse to call into question our ability to successfully remediate a site.

2. *Remedial Strategies and Remedial Technologies*

It is important that capture and containment technology be seen within the context of a wider suite of available remedial technologies. First, let us differentiate, as the EPA does, between individual *remedial technologies*, such as capping or pumping, and a *remedial strategy*, which involves the grouping of one or more of these technologies into an overall remedial plan.

Within this framework, three broad remedial strategies could be developed for any particular site: (1) no action, (2) monitoring alone, and (3) monitoring and remedial action.

Given the availability of these three alternative strategies, a five-step process must be carried out for any site: (1) selection of the appropriate remedial strategy; (2) defense of the selected remedial strategy to the appropriate regulatory agencies involved; (3) design of the monitoring network for those sites requiring monitoring; (4) design of the component technologies of the remedial strategy for those sites requiring remedial action; and (5) construction and operation of the remedial systems.

The process used to determine whether remedial action is required at a site involves four major steps: (1) determining if contaminants are present in the groundwater, (2) identifying potential receptors of contaminated groundwater, (3) calculating potential future contaminant concentrations at the receptor points, and (4) determining if the concentrations at the receptor points are acceptable. In the highly regulated environment that has developed in the United States in the past decade, receptor points usually take the form of regulatory compliance points, and acceptable concentrations usually take the form of maximum concentration limits established by legislation and enforced by State or Federal regulatory agencies.

If remedial action is required, a large number of remedial technologies should be considered. Table 4 provides a list of some of the most commonly considered options. Remedial activities at "Superfund" sites, authorized under CERCLA over the past few years, provide a growing body of knowledge about the feasibility of these various technologies in different hydrogeological environments.

Table 4 shows that remedial action at most Superfund sites has included a requirement for some soil excavation. Capping has been widely

Table 4. Remedial Technologies

Objective	Remedial Technology
Source Removal	Excavation of Soils
Source Containment	Capping Cutoff Walls
Contaminant Removal: Unsaturated Zone	Soil-Vapor Extraction
Contaminant Removal Saturated Zone (cleanup)	LNAPL Bailing or Skimming Enhanced Recovery Flushing Steam Displacement Surfactants Enhanced Biodegradation Extraction
Contaminant Containment: Saturated Zone (migration control)	Extraction and Injection Wells Slurry Walls Sheet Piling

used at facilities where sources are of limited areal extent and source boundaries are well established. Cutoff walls have also been used at a limited number of sites. Soil-vapor extraction has been widely used but usually in combination with one or more of the other technologies. Removal of LNAPL from the water table by bailing or skimming has become relatively common. Methods based on enhanced biodegradation, or enhanced recovery through flushing, steam displacement, or the introduction of surfactants, have been investigated in the laboratory and, in some cases, at pilot-plant scale but are not yet sufficiently proven to constitute common alternatives for commercial remediation.

With this brief summary complete, the capture and containment technology can be placed in context. It has been the most common remedial technology to date, and it is likely to remain so. In some cases it may be coupled with excavation, capping, or soil-gas collection, but in most cases it will be the primary component of the remedial strategy, especially at sites where dissolved-contaminant plumes have developed in the saturated zone in unconsolidated surficial aquifers. We again note that capture and containment systems alone will usually not result in complete aquifer clean up. If clean up is achieved, it will often take decades or longer.

3. Objectives of Capture and Containment Systems

As indicated in Table 4 (and discussed earlier in Chapter I.B.4) there are two very different reasons why one might use pumping wells as a

component of a remedial strategy. On the one hand, the goal may be contaminant removal from the plume; the purpose is often *hot-spot cleanup*. On the other hand, the goal may be to eliminate contaminant-plume migration; in this case, the purpose is *containment through migration control*.

For the *hot-spot cleanup* option, the primary component of a capture and containment technology will involve extraction wells located within the plume in the zones of highest concentration. The contaminant-removal option is most effective for high-solubility contaminants forming a solution with a density close to that of water and little predilection for sorption or matrix diffusion. Unfortunately, few contaminants meet these specifications. Extraction wells cannot be expected to be efficient in removing contaminants that are sorbed on aquifer materials, contaminants that have diffused into the matrix of low-permeability materials, or contaminants that occur as LNAPL or DNAPL pools. The desorption and matrix-diffusion processes are likely to be slow, and pumping of many aquifer volumes, more or less in perpetuity, would be required to attain complete cleanup. LNAPL or DNAPL pools (or even residual globules, fingers, and ganglia) are equally hard to remove. After all, petroleum engineers only achieve partial recovery of oil and gas from their reservoirs, even with secondary-recovery waterflooding techniques. It is unlikely that remedial action DNAPL recovery efforts will be complete. The usual observation in pump-and-treat extraction systems is that contaminant concentrations decline over time to some low non-zero value. At that point, large volumes of water are being treated to remove small quantities of contaminants. When pumping is stopped, concentrations in the groundwater often rise again. We are forced to conclude that once a subsurface volume of aquifer has been contaminated it is difficult, if not impossible, to return the aquifer to its pristine condition. Even after significant contaminant removal, it is likely that many groundwater samples taken on the site will still fail to meet maximum concentration limits.

If we accept this discouraging scenario with respect to contaminant removal, we must turn to the other potential objective, that of migration control. Here, the potential for meeting remedial objectives is much more encouraging. For the migration-control option, it would be common to use both extraction wells and injection wells. Their purpose is to control hydraulic gradients in such a way that the advective plume-front velocity is reduced to zero, and the contaminant plume is contained within the volume of aquifer material already contaminated prior to the instigation of remedial action. Some contaminant-removal wells might also be included in the design, but with the recognition that complete

cleanup cannot be expected. It is noteworthy that by providing optimal containment, enhanced remediation methods such as surfactant injection or bioremediation may be used.

Our emphasis will be on pump-and-treat systems that are designed with the objective of hydraulic containment of dissolved contaminant plumes through migration control.

4. *Components of Capture and Containment Systems*

Capture and containment systems usually involve several components, including (1) extraction wells, (2) injection wells, (3) pipeline networks, and (4) treatment facilities. Extracted water is treated to remove contaminants to a level that meets regulatory standards. Depending on the situation, the treated water may be reinjected into the aquifer by means of injection wells, made available for water-supply use, or released to surface water. The simplest injection/extraction systems to design are those that do not involve water-supply or surface-water release, in which case total injection rates can be set equal to total extraction rates, and local hydraulic gradient control can be achieved without regional water-level declines. However, it is often found that design injection rates are hard to sustain due to the clogging of well screens and well-pack materials.

Potential treatment systems may be based on physical separation through carbon adsorption or air-stripping, chemical treatment such as oxidation, or biological treatment involving activated sludge. Carbon adsorption systems and air-stripping towers have been widely used in the Superfund program. Potential system designs span the range from individual treatment facilities associated with each extraction-injection well pair to large central treatment facilities connected to the wells by complex pipeline networks. In urbanized areas, the addition of a pipeline network to an already-complex network of roads, sewers, and service lines may not be a simple matter. There may be trade-offs between the costs of complex pipeline networks and the benefits of large central treatment facilities. Our mandate is limited to the design issues associated with extraction and injection well networks.

Information on costs of all components of a pump-and-treat system can be obtained from the CORA software developed by the EPA and is available through their contractor: CH_2M–Hill, Mid-Atlantic Office, P.O. Box 4400, Reston, VA 22090.

B. A Framework for Design

In this subsection we begin to address the question of how to design a network of extraction wells and injection wells for the purpose of plume-migration control.

In a more general context, the process of engineering design involves a sequence of decisions between *alternatives*. Alternatives are established so that they meet the technical *objectives* of the project. In most projects it is necessary to meet these objectives within a set of technical, legal, political, or economic *constraints*. Engineering alternatives are differentiated from one another on the basis of their technical components. The variables that can be used to define and differentiate alternatives are known as *decision variables*. Decision variables may take on discrete values, giving rise to discrete alternatives; or they may be continuous functions, giving rise to a continuous range of alternatives.

Designers base their decisions on an economic analysis of the alternatives. The design framework must provide a link between the economic milieu in which decisions are made and the results of the technical analyses on which decisions are based. A discussion of the various methods that can be used to determine which alternative is "best" is postponed until Chapter IV.C. Let us first clarify the concepts associated with alternatives, objectives, constraints, and decision variables for the case at hand.

1. *Decision Variables, State Variables, and Hydrogeological Parameters*

The alternatives that can be identified for a capture and containment system revolve around the well-network geometry. The *decision variables* include (1) the number and location of extraction wells, (2) the number and location of injection wells, (3) the pumping and/or injection rates for each well, and (4) the pumping and/or injection schedules for each well. These are the variables that can be specified, managed, or controlled by the design engineer. The purpose of the design process is to identify the best combination of these decision variables. If we specify the pumping rate, Q, as a function of space and time (positive for extraction, negative for injection), then all four of the decision variables identified above are essentially collected into a single decision variable, $Q(x,y,t)$.

All the design approaches described in this chapter involve the use of a simulation model of groundwater flow and transport. In its most general form, the simulation model has a hydraulic component based on

the flow equation and a contaminant component based on the transport equation. The *state variables* in this context are the hydraulic head, which is the dependent variable in the flow equation, and the concentration, which is the dependent variable in the transport equation. In steady-state simulations, the state variables are functions of position; in transient simulations, they are functions of position and time. Many contaminant simulations couple a steady-state hydraulic model with a transient transport model.

Hydrogeological parameters include all media properties such as porosity, hydraulic conductivity, transmissivity, storativity, dispersivity, and the like. In heterogeneous media these parameters may vary through space, but they usually do not vary with time.

The input to a transient transport simulation must include information on the spatial distribution of the hydrogeological parameters, together with information on the initial conditions and boundary conditions. Output from the simulation takes the form of predicted changes in the state variables through space and time. In some cases, output includes calculations of *auxiliary variables*, which are additional, conveniently defined output quantities such as velocities, gradients, travel times, or capture zone dimensions.

2. Objective Functions

For this discussion, assume that the goal of our pumping scheme is the containment of a contaminant plume such that no further downgradient migration of the plume front occurs. There may be a number of alternative well networks that can meet this goal within the types of constraints discussed in the following subsection. Such feasible alternatives are compared with one another on the basis of an economic objective function.

From the perspective of the engineer (or the owner-operator he or she represents), we can define an objective function as the net present value of the expected stream of remedial costs, taken over an engineering planning horizon, and discounted at the market interest rate. If an objective function, Z_j, is defined for each $j = 1 \ldots N$ alternatives, then the goal is to minimize Z_j, where

$$Z_j = \sum_{t=0}^{T} \left[\frac{1}{(1+i)^t} \right] [C_j(t)] \tag{51}$$

and Z_j = objective function for alternative j [\$],
T = planning horizon [years],
i = annual discount rate [decimal fraction], and
$C_j(t)$ = costs of alternative j in year t [\$].

For a remedial pump-and-treat scheme, the costs include the *capital costs* associated with site investigation, well installation, and treatment-facility construction; and the *operational costs* associated with pumping and treatment over the life of the project. If the alternatives all have similar total pumping rates, then the costs associated with treatment will be more-or-less common for all alternatives and they can be removed from the comparative analysis. Furthermore, the differences in capital costs between alternatives are often small, and the relative merits of the alternative pump-and-treat network designs are decided on the basis of their relative pumping costs (where the term *pumping* includes both *extraction* and *injection*).

In Chapter V, the optimization procedures are carried out with an objective function that emphasizes total pumping. In fact, if treatment costs are the dominant expense and are assumed to depend only on pumping rates, then the total pumping rate is an economic surrogate for the total cost in the objective function. As shown in Chapter V.D.3, this leads to a linear objective function that can be solved with a linear-programming algorithm. If, on the other hand, pumping costs are the dominant expense and are assumed to be a function of both the pumping rate and the total lift to bring water from the well bore to the surface, a quadratic objective function results and a quadratic-programming approach is needed.

There are many other technical objectives, other than those associated with minimizing pumping, that could be formulated in an objective function. For example, the objective function might reflect an attempt to minimize the maximum drawdown, maximize the minimum hydraulic head, or minimize the sum of squared deviations from target heads, drawdowns, gradients, or velocities. There are also alternative economic criteria to that of maximizing net present value. There are criteria based on minimizing maximum regret, where regret is defined as the opportunity loss suffered by making a non-optimal decision. There are criteria that give greater weight to alternatives that exhibit robustness over a wide range of potential technical or economic conditions. It is also possible to formulate multiobjective problems in the same framework as the one we have described for a single objective, but of course solution methodologies are more complex. Having drawn attention to some of the complexities, we proceed now along the simpler and more conven-

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tional track based on a single-objective, cost-minimization objective function.

3. Constraints

The objective must be met, and alternatives compared, within a set of constraints derived from technical, economic, legal, or political conditions associated with the remedial project. There may be constraints on decision variables, state variables, or auxiliary variables. They may take the form of either equalities (e.g., drawdown must equal 10 meters) or inequalities (e.g., drawdown must not exceed 10 meters).

Constraints on decision variables may involve the number of wells or their pumping rates. With respect to pumping rates, it may be necessary to meet a certain demand or, on the contrary, it may be necessary not to exceed a certain capacity. There may be limitations on rates for individual wells or on total well field pumping. There may be limitations on the rates themselves or possibly on the changes in rates that are acceptable. In extraction/injection scenarios, there may be a requirement that the two be balanced or that a particular imbalance be maintained.

Constraints on the state variables might include requirements that hydraulic heads be maintained above a certain level or below a certain level or that contaminant concentrations not exceed regulatory standards at a compliance point.

Constraints on auxiliary variables could include limitations on the magnitudes of drawdowns, gradients, or velocities; or the restrictions of gradients or velocities to certain directions. In the design of pump-and-treat systems for migration control, the overall technical objective of attaining containment of a contaminant plume may often be replaced by a series of gradient-control constraints.

The linear- and quadratic-programming optimization techniques described in Chapter V are capable of identifying the alternative that minimizes cost while simultaneously satisfying all of the constraints.

4. Program Integration

The design process for remediation of contaminated groundwater involves a sequence of at least three steps: (1) design of a site investigation program, (2) design of the remedial well network, and (3) design of a monitoring network. Each step involves a decision among alternatives. How many holes will be drilled during site investigation? How many

wells are needed in the pump-and-treat network? What monitoring-well spacing is required?

In this book, the design framework is limited to the design of the remedial well network itself. However, it is important to emphasize that a successful remedial action is dependent on (1) a proper site investigation prior to design of the remedial action and (2) a thorough performance assessment program during and after construction of the remedial system. We have discussed data needs in Chapter III, and we address performance assessment in Chapter VII. The point to be made here is that the same design framework used for the remedial network, with decision variables, objectives, and constraints, can also be used for the design of site investigation programs and monitoring networks.

Better yet, the framework might be expanded to allow an integrated design process that allows the engineer to assess economic trade-offs between the various steps. Would it be better, for example, to use minimal site investigation and conservative design; or would it be better to carry out a detailed site investigation in the hopes of buying reduced construction costs. The owner-operator would like to know how to partition his or her resources among the competing requirements of site investigation, remedial action, and monitoring. While this type of expanded and integrated design process is desirable, it has not yet been developed into on-the-shelf technology and, therefore, a detailed methodology is not presented here.

5. Deterministic and Stochastic Analysis

The process of engineering design involves making decisions under conditions of uncertainty. This is particularly so in engineering projects that require a knowledge of the hydrogeological environment, where uncertainty as to the system's properties and expected conditions is far greater than in most traditional engineering practice. There is uncertainty associated with the parameter values needed for design calculations and with the very geometry of the system being analyzed. The uncertainties of lithology, stratigraphy, and structure introduce a level of complexity to hydrogeological analysis that is completely unknown in most other engineering disciplines.

Recognition of these uncertainties has led hydrogeological researchers to adapt geostatistical techniques, first developed in the mineral exploration field, to a hydrogeological context. Geostatistical interpretations of field data can be used to generate probabilistic interpretations, whereby

our uncertainty as to the geometry of the geologic system and the values of hydrogeological parameters within the geologic system can be placed in a quantitative framework.

This leads to two possible approaches to the simulation component of the design framework: deterministic analysis and stochastic analysis. In a *deterministic analysis*, all initial conditions, boundary conditions, and hydrogeological parameter values are assumed to be known with certainty. In a *stochastic analysis*, one or more of these features is represented as having a distribution in probability.

The classical approach to groundwater modelling is deterministic. The modeller estimates the most likely parameter values and then makes a single simulation to estimate the most likely output values. Deterministic modelling is often carried out in conjunction with sensitivity analysis, whereby a set of simulations are run to investigate the influence of changes in input parameters on output variables. This provides a quantitative assessment of the impact of changes in parameter values across the range of uncertainty, but it does not associate a probability with each of the possible outcomes.

In one type of *stochastic analysis*, geostatistical methodology is invoked to generate a set of equally likely realizations of the hydrogeological environment at a site. The simulation model is applied to each realization.

With this approach, the final answer is not merely a set of single-valued output variables. Instead, each variable at each location at each point in time has a probability density function associated with it. The mean value can be interpreted as the most likely value; it should be equal to the value calculated using a deterministic simulation. The variance can be interpreted as a measure of the uncertainty in the output variable generated by the uncertainty in input parameters.

There are a variety of possible methods of stochastic analysis, but the most commonly used one is that of Monte Carlo analysis as described above. This technique involves multiple runs with the exact same simulation model that would require only one run in a deterministic analysis. Stochastic analysis is thus much more computer-intensive than deterministic analysis.

We introduce these stochastic concepts in preparation for the next part of this chapter, wherein the differentiation between decision analysis and optimization analysis rests in part on an understanding of these concepts. However, when it comes to our recommended design procedures, both the simulation-analysis approach in Chapter IV.D and the optimization approach in Chapter V are carried out in a deterministic rather than a stochastic framework.

C. Alternative Approaches to Design

Figure 4 summarizes the various options that have been considered in our discussion of a design framework for pump-and-treat remedial systems. The asterisks identify the decisions we have reached with respect to the various options, at least insofar as they apply to the quantitative design methods presented in this chapter and the next. These design methods are limited to *saturated* conditions and are best suited to *unconsolidated* aquifer materials or bedrock aquifers that are porous and/or sufficiently fractured to be treated as an equivalent porous medium. They can be applied to both uniform and non-uniform *steady flow* and under transient flow conditions. It is assumed that the objective of the remedial system is migration control of a *dissolved-solute plume.* Both analytical and numerical simulation methods are utilized, but they are limited to a *hydraulic treatment* of the problem that assumes the primacy of *advective transport.* We assume a *deterministic framework* that does not take uncertainty into account, except possibly by means of a sensitivity analysis. Our objective function is one that *minimizes cost.*

With these conditions in mind, four alternative approaches to design are identified at the bottom of Figure 4: (1) simulation, (2) simulation plus optimization, (3) simulation plus decision analysis, and (4) qualitative guidelines. These alternatives are more clearly laid out on Figure 5, where we differentiate between simple systems, optimizable systems, and non-optimizable systems. For simple systems, application of a simulation model alone may suffice as an approach to design. Optimizable systems make use of a simulation model coupled to an optimization procedure or a decision-analysis procedure. Non-optimizable systems are too complex to satisfy the assumptions required by the simulation model, the optimization procedure, or the decision-analysis procedure; in such cases, we must revert to a design process based on qualitative guidelines.

Figure 5 shows three approaches for optimizable systems. These require further discussion. They differ from one another in three ways: (1) the upper approach uses a deterministic simulation model; the middle and lower approaches are stochastic; (2) the upper and middle approaches use an optimization model; the lower uses decision analysis; and (3) the upper optimization model uses linear and quadratic programming; the middle one uses nonlinear programming. The difference between deterministic and stochastic analysis has been discussed above. The difference between linear and nonlinear programming will be described in Chapter V. The difference between optimization and decision analysis is discussed next.

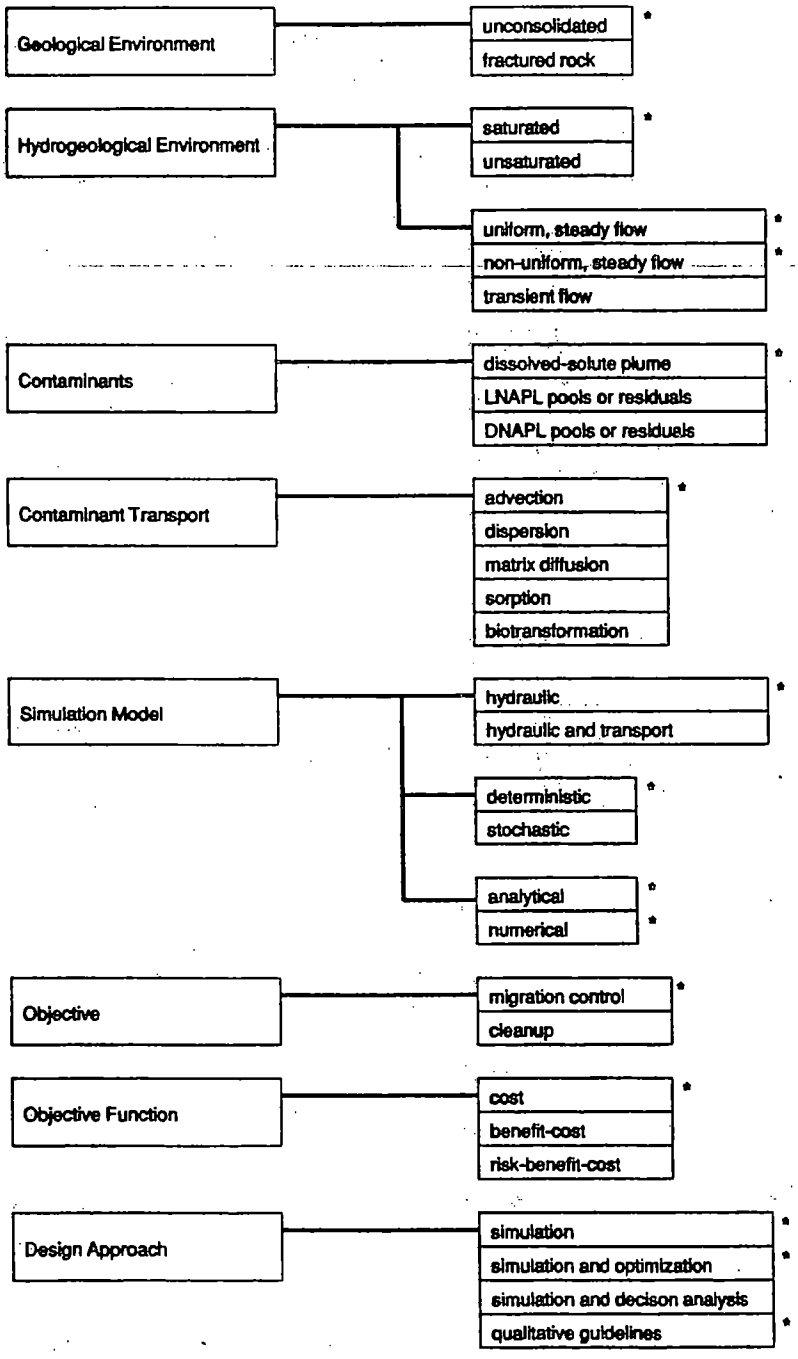

Figure 4. Summary of design options for pump-and-treat remedial systems; asterisks indicate conditions best suited to pump-and-treat remediation.

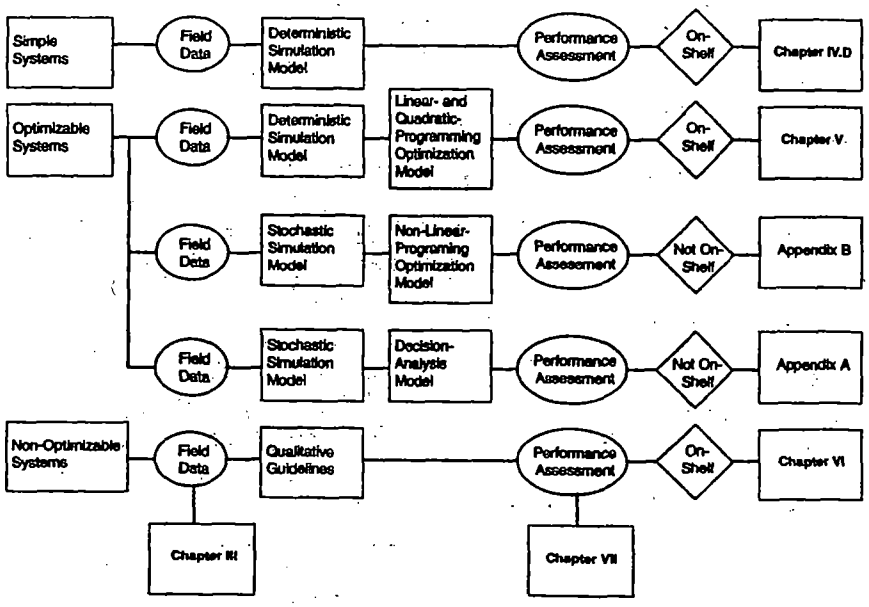

Figure 5. Differentiation of alternative remedial approaches.

All of the alternative approaches to design in Figure 5 are based on field data collection as discussed in Chapter III, and all require a performance-assessment module as discussed in Chapter VII.

1. Simulation

Simulation analysis alone can be used for simple systems as suggested in Figure 5, or it can be used for scoping and screening alternatives prior to the application of optimization procedures in more complex systems. The methods are "on-the-shelf" and widely used. A more detailed discussion of their application to the design of well networks for pump-and-treat remedial systems appears in Chapter IV.D.

2. Simulation Plus Optimization

Optimization involves the determination of optimal values for a set of decision variables in an engineering system. Optimality is defined with respect to a specified objective function and is subject to a set of constraints.

Of the many available optimization techniques, the one that has proven most popular and tractable for coupling with groundwater simulation models is linear programming. It requires a linear objective func-

tion, linear constraints, and linear flow equations in the simulation model. If the latter two linearities are retained, it is possible to move from a linear to a quadratic objective function and remain in the same optimization framework. Gorelick (1983) provides a review of linear and quadratic optimization techniques applied to groundwater problems. Lefkoff and Gorelick (1987) provide a user's manual for their linear and quadratic optimization program, AQMAN, which is the program described and recommended in Chapter V. This program is "on-the-shelf" and can be applied directly to the design of extraction-well/injection-well networks. Gorelick and Wagner (1986) and Lefkoff and Gorelick (1986) report applications to aquifer remediation.

The requirement for linear flow equations in the simulation model limits applications to treating systems as confined aquifers. The methods presented in Chapter V are applicable to unconfined aquifers only if the drawdowns and buildups in hydraulic head created by the extraction and injection wells are small in comparison with the total saturated thickness of the aquifer. Under these conditions, the confined-aquifer equations can be applied to an unconfined aquifer with little loss in accuracy. There are also simple iterative methods whereby the nonlinear equations can be linearized (cf. Danskin and Gorelick, 1985). This technique is also demonstrated in Chapter V.

At the research level, restrictions with respect to nonlinearity have been fully removed (Ahlfeld et al., 1988a, 1988b). Gorelick et al. (1984) discuss aquifer reclamation design with a simulation/optimization methodology that allows either linear or nonlinear objective functions, constraints, and flow and transport equations. Unfortunately, documented manuals for the programs used in that study are not yet available.

It has been traditional to apply simulation/optimization techniques in a deterministic framework, and this is the most common framework utilized in "on-the-shelf" optimization packages. Researchers have investigated the effects of uncertainty on optimization problems through stochastic analysis (cf. Gorelick, 1987; Ward and Peralta, 1990). There are two ways that a linear program can be applied in a stochastic framework. *Stochastic linear programming* treats the coefficients of the objective function and/or the constraints as random variables, but the constraints hold with probability equal to one. *Chance-constrained programming* states the constraints probabilistically. The coefficients are treated deterministically but the constraints are only satisfied on an expected value basis. Wagner and Gorelick (1987) present a chance-constrained nonlinear optimization solution to the plume-capture problem. Tung (1986) describes a chance-constrained model in a groundwater management context. Ward and Peralta (1990) present an

"on-the-shelf" code that is capable of solving chance-constrained or deterministic optimization problems for short-term emergency plume containment.

3. Simulation Plus Decision Analysis

Decision analysis involves the determination of the best alternative (that is, the best values for a set of decision variables) from a discrete set of specific alternatives. For example, we might wish to decide between a particular three-well extraction system and a particular five-well extraction system. Decision analysis is less general than optimization in that optimization provides the optimal alternative from the set of all possible alternatives, whereas decision analysis provides only the best alternative from a specified set of alternatives. On the other hand, it is less limited with respect to linearity than are linear-programming optimization techniques.

There is a fundamental difference in the treatment of objectives and constraints between optimization and decision analysis. In an optimization framework, the objective function involves only the costs [Equation (51)], or in some other applications, the costs and benefits. The performance requirements on the engineered system appear as constraints. In a decision-analysis framework, the potential failure to meet performance requirements produces risks, and the risks are given dollar value and included with the benefits and costs in the objective function. The goal becomes to maximize Z_j over $j = 1 \ldots N$ alternatives, where

$$Z_j = \sum_{t=0}^{T} \left[\frac{1}{(1+i)^t}\right] [B_j(t) - C_j(t) - R_j(t)] \tag{52}$$

and $B_j(t)$ = benefits of alternative j in year t [\$],
$C_j(t)$ = costs of alternative j in year t [\$], and
$R_j(t)$ = risks of alternative j in year t [\$].

The risks, $R_j(t)$, associated with alternative j in year t are defined as

$$R_j(t) = [Pf_j(t)]\ [Cf_j(t)] \tag{53}$$

where $Pf_j(t)$ = probability of failure of alternative j in year t [decimal fraction], and
$Cf_j(t)$ = cost associated with a failure of alternative j in year t [\$].

For a remedial pump-and-treat scheme designed to provide migration control for a contaminant plume, failure would be defined by the spread of contamination into previously uncontaminated areas of the aquifer. This could occur by contaminants slipping downstream through the well network or by lateral migration of contaminants across the presumed capture-zone boundaries. A failure of the system would be associated with a failure to meet constraints on the magnitude and/or direction of drawdowns, gradients, or velocities. Whatever the reasons, there will be expected costs associated with potential failures. These could take the form of regulatory penalties, loss of goodwill in the community, possible facility closure, and/or the costs of further remedial action.

In many risk-analysis textbooks (cf. Crouch and Wilson, 1982), Equation (53) has a third factor on the right-hand side. It is a term that allows one to take into account the risk-averse nature of some decision makers. We will not address this issue.

The primary point to be made here is that decision analysis with a risk-cost-benefit objective function requires a stochastic analysis. The probability-of-failure term in the objective function can be determined with a simulation model operating in Monte Carlo mode. The Monte Carlo simulations are carried out on a set of geostatistically generated realizations of the hydrogeological regime that reflects our uncertainty as to the geological system, hydrogeological parameter values, and/or initial plume distribution. It is this input uncertainty that creates output uncertainty and the output uncertainty that creates risk. There is risk associated even with respect to the "best" design geometry we can produce for the network of remedial wells.

Risk can be reduced by a commitment to additional costs, either for additional site investigation, which reduces input uncertainty, or for an increased number of pumping wells, which provides a more conservative remedial design. The "best" design from the owner-operator's perspective is the one that maximizes Z_j, not one that satisfies any predetermined acceptable level of risk. One might contrast this approach with a chance-constrained optimization scheme, which is also a stochastic approach. There, the probability of failure to meet a constraint is not usually coupled with the cost of failure; and the optimal solution is usually determined for a specified probability of failure, Pf, (or reliability, 1 – Pf), a process that is similar in principle to setting an *a priori* acceptable risk.

In summary, decision analysis is less general than optimization, and it is well-suited to a risk-based philosophy of engineering design. Its greatest weakness lies in the difficulties associated with quantifying the anticipated cost of failure. The coupling of a stochastic simulation model and

a decision-analysis model is one of the possible design approaches for an optimizable system. This approach has recently been espoused by Massmann and Freeze (1987), but thus far their applications have been directed toward the design of new waste-management facilities rather than remedial action, and documented computer programs are not yet "on-the-shelf." We cannot recommend the approach as an alternative to optimization at this time. Nevertheless, future developments in this area are worth watching for, and a more detailed outline of the philosophy and methodology is included in Appendix A of this book.

D. Simulation Analysis

In very simple hydrogeologic settings or for scoping and initial screening of alternatives, simulation analysis applied alone, without subsequent optimization, can be an instructive design tool. It requires, as a start, an understanding of capture zones.

1. Capture Zones

The design framework for contaminant-plume migration control with a pump-and-treat remedial system is based on the concept of capture zones.

Capture zones are best explained for steady-state flow conditions in a horizontal, confined aquifer. Consider a small portion of such an aquifer, with a regional hydraulic gradient as shown in Figure 6a, and orthogonal regional flow directions as shown in Figure 6b. Now assume that it has been pumped for a sufficiently long time to attain steady-state conditions. A drawdown cone will have developed in the hydraulic head field as shown in Figure 6c, and the associated flow lines will be as shown in Figure 6d. Water will be drawn into the well from the stippled capture zone. It is located primarily on the (preoperational) upgradient side of the well but includes a small region of the downgradient side as well. Point B is a stagnation point.

The *capture zone* associated with an extraction well is defined as that portion of the aquifer that contains groundwater that will eventually be captured and discharged by the well. It does not include the entire area of perturbed heads, unless the velocity of the preoperational, natural flow system is zero.

Use of the capture-zone concept in remedial design should be clear from Figure 6d. If a contaminant plume exists within the stippled region, it will not migrate outside the capture-zone boundaries of Well A.

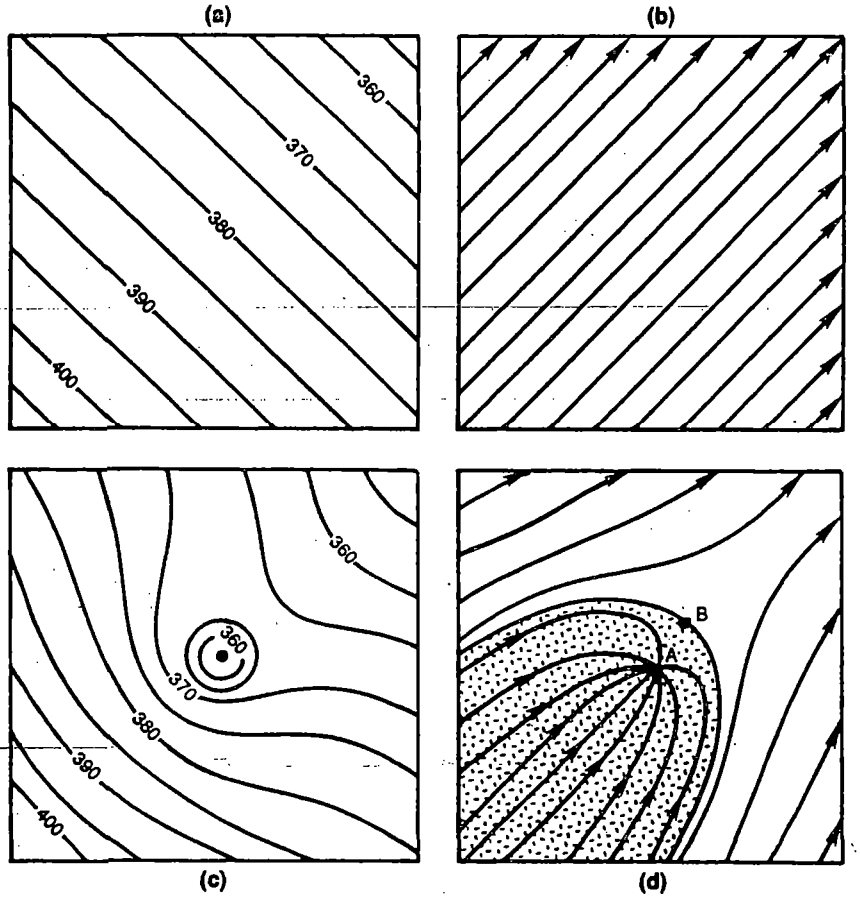

Figure 6. Capture-zone concept.

Given a contaminant-plume geometry, the design goal for a migration-control system is to establish a well network that will create a capture zone that encompasses the entire plume.

It must be emphasized that capture zones are a purely hydraulic concept. They therefore address only the advective component of contaminant transport. For this component, one can define time lines as shown in Figure 7. A plug of water inside the 1-year time line will be captured by Well A within 1 year. If a plume were totally encompassed by the 20-year time line as shown in Figure 7, one might be tempted to think that all contamination would be drawn into the well in 20 years. However, as noted earlier in our discussion of the limitations to total contaminant removal, this is not likely to be the case due to the influences of sorp-

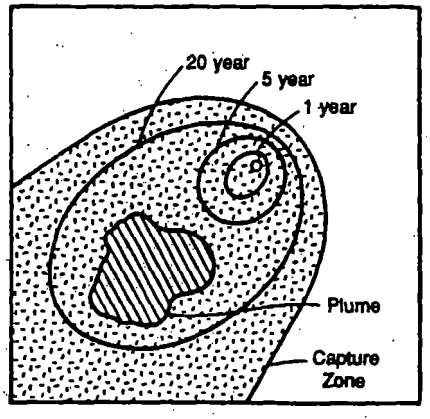

Figure 7. Time lines associated with the advective components of contaminant transport.

tion, matrix diffusion, and the possible presence of non-aqueous phase liquids.

Figure 8 shows a single injection well in a uniform flow field. By reversing the frame of reference, one can define a *rejection zone* associated with an injection well as that portion of the aquifer that will eventually contain only injected water. All regional flow lines are diverted around it. As with a capture zone, it does not include the entire area of the buildup cone generated by the injection well, unless the velocity of the preoperational, natural flow system is zero. Here the stagnation point is just upstream from the well.

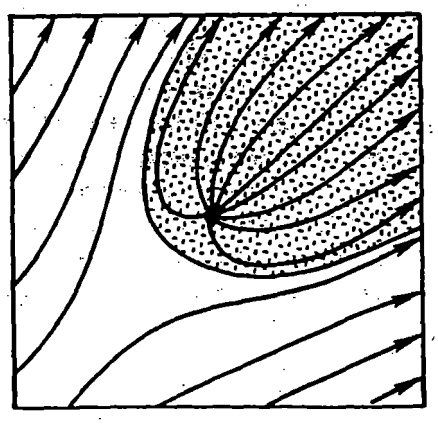

Figure 8. Single injection well in a uniform flow field.

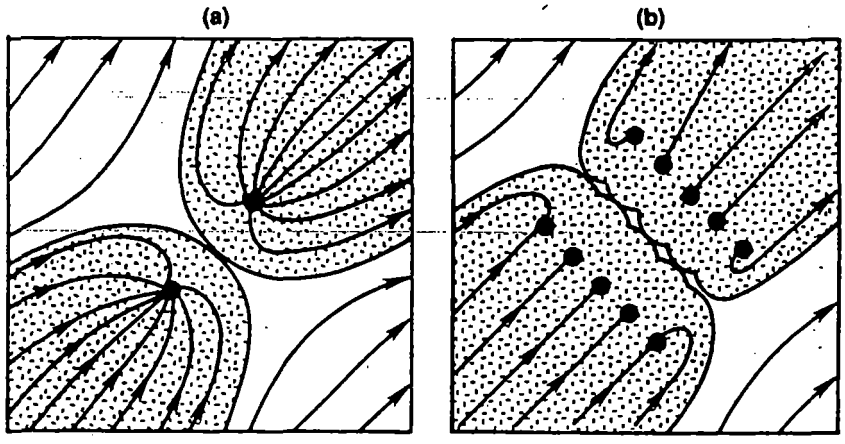

Figure 9. Flow regimes for (a) extraction/injection well pair and (b) paired line of extraction and injection wells.

Figure 9a shows the flow regime for an extraction/injection well pair, and Figure 9b shows a paired line of extraction and injection wells. In the latter case, the stagnant point becomes a stagnant zone. In both cases, migration control is achieved with little impact on regional flow.

The concept of a capture zone was introduced into the groundwater literature by Keely and Tsang (1983) and has been popularized through the widely used monograph of Javandel et al. (1984) and the paper based on it by Javandel and Tsang (1986). These latter papers provide general analytical solutions for capture-zone geometry for a two-dimensional representation of a homogeneous, isotropic confined aquifer under uniform, steady flow.

Under such conditions, capture-zone width is directly proportional to the pumping rate, Q [L^3/T], and inversely proportional to the product of aquifer thickness, b [L], and the regional specific discharge, q [L/T]. Javandel et al. (1984) provide capture-zone type curves for single- and multiple-well extraction systems. Figure 10 reproduces a set of their type curves showing capture zones for a four-well extraction system for several values of the parameter Q/bq. Note that because $q = KI$ and $T = Kb$, the parameter of Q/bq can also be expressed as Q/TI where T is aquifer transmissivity and I is the magnitude of the regional flow gradient.

The limitations on the analytical expressions for capture-zone geometry are quite severe, but most of them can be removed by using a numerical model to calculate the postoperational flow net. With numerical models, one can treat heterogeneous and anisotropic systems, uncon-

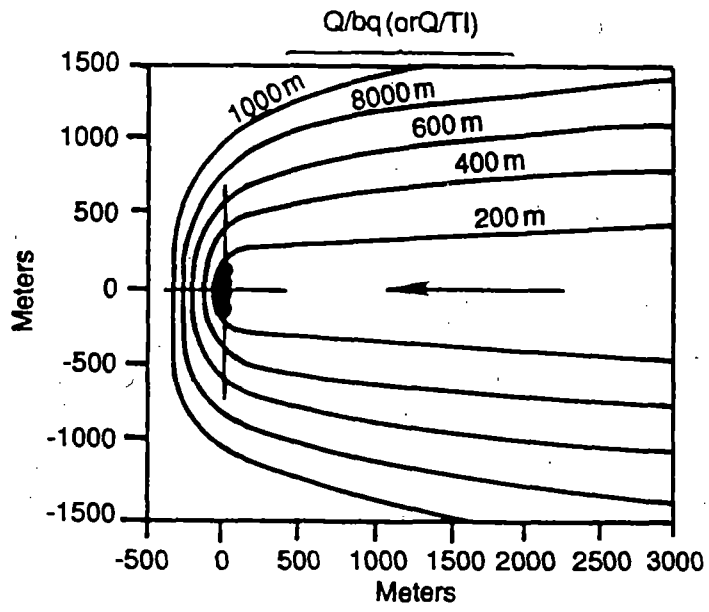

Figure 10. Capture zones for a four-well extraction system.

fined aquifers, nonuniform preoperational flow, and complex extraction/injection well networks. Shafer (1987a, 1987b) provides examples of such an approach. Figure 11a shows the hydraulic-conductivity pattern for one of his examples. Figure 11b shows the hydraulic-head distribution that results from the placement of a single pumping well in such an aquifer, and Figure 11c shows the 20-year capture zone for this well.

With the concepts associated with capture zones clearly in hand, we can proceed to show how they can be used in a simulation-based design procedure.

We will describe two approaches, one that applies to uniform-flow capture-zone analysis and one that applies to nonuniform-flow capture-zone analysis. The first uses analytical solutions and the programs RESSQ (Javandel et al., 1984) or DREAM (Rounds and Bonn, 1989). The second uses numerical solutions based on MODFLOW (McDonald and Harbaugh, 1984) and the programs GWPATH (Shafer, 1987b) or MODPATH (Pollock, 1989).

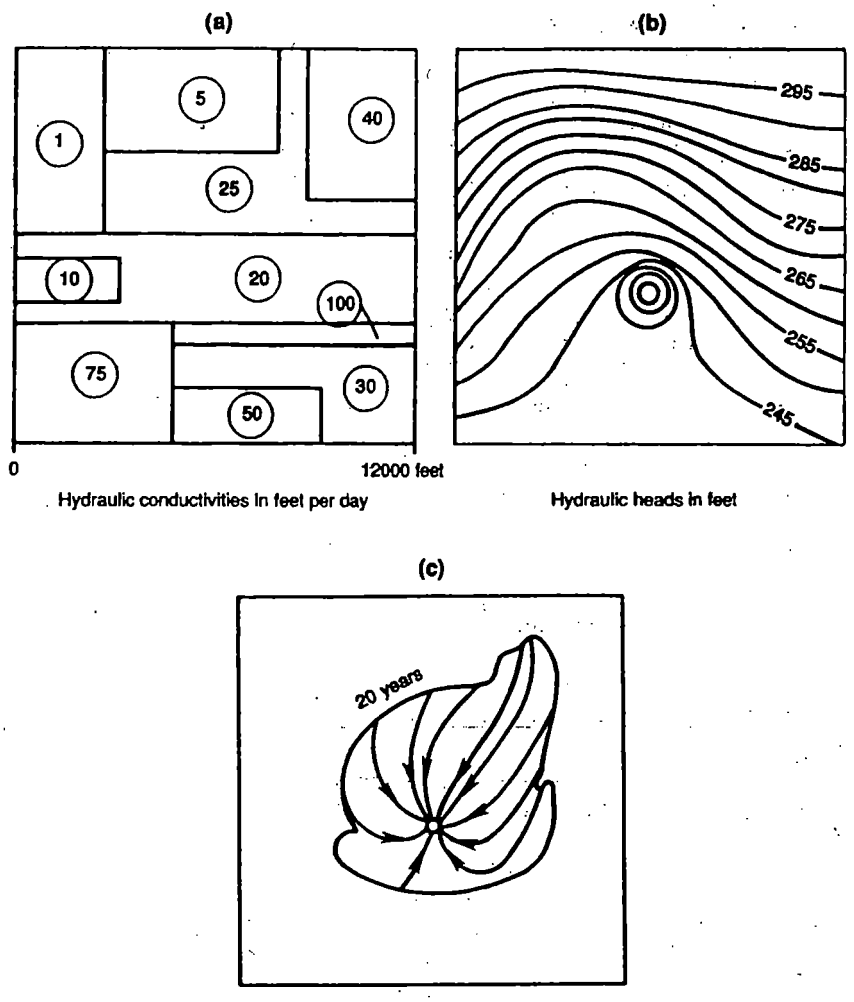

Figure 11. Capture-zone geometry using numerical models.

2. *Uniform-Flow Capture-Zone Analysis*

This subsection will address the design of a pumping-well network in a homogeneous, isotropic, horizontal confined aquifer, in which the preoperational, natural flow field is one of uniform, steady flow, like that shown earlier in this chapter in Figures 6a and 6b. In this simple system, the transient drawdown from an extraction well, or the transient buildup from an injection well, can be calculated using the Theis equa-

tion, and the final steady-state drawdown or buildup is given by the Thiem equation, as summarized in Chapter II.G of this book.

The theory on which the method is based is usually carried out on two-dimensional, planar, steady-state flow fields. It is fully developed in the monograph by Javandel et al. (1984). It relies on the concept of a complex velocity potential, Φ, which is defined as

$$\Phi(x,y) = \phi(x,y) + i\psi(x,y), \tag{54}$$

where $\Phi(x,y)$ = complex velocity potential $[L^2/T]$,
$\phi(x,y)$ = potential function $[L^2/T]$,
$i = \sqrt{-1}$, and
$\psi(x,y)$ = stream function $[L^2/T]$.

The velocity potential Φ, is related to the hydraulic head, h, by

$$\Phi = Kh. \tag{55}$$

Because K is a constant for the homogeneous media to which this analytical development applies, there is no reason not to divide the three terms of Equation (54) by K to produce a complex *head* potential:

$$P(x,y) = h(x,y) + iw(x,y) \tag{56}$$

where $P(x,y) = \Phi/K$ = complex head potential [L],
$h(x,y) = \phi/K$ = hydraulic head [L], and
$w(x,y) = \psi/K$ = modified stream function [L].

The stream function of a flow system with a known potential function is obtained from the relationships:

$$\left(\frac{\partial w}{\partial x}\right) = -\left(\frac{\partial h}{\partial y}\right) \qquad \left(\frac{\partial w}{\partial y}\right) = \left(\frac{\partial h}{\partial x}\right) \tag{57}$$

which simply state that h and w are everywhere orthogonal.

The components of specific discharge are given directly by Darcy's law:

$$q_x = -K\left(\frac{\partial h}{\partial x}\right) \qquad q_y = -K\left(\frac{\partial h}{\partial y}\right). \tag{58}$$

For a flow system that is influenced both by uniform regional gradients, and a number of extraction or injection wells, the equations that define h and w can be developed on the basis of the superposition

principle. They are well-known from classical potential theory (Javandel et al., 1984):

$$h(x,y) = -I\,(x\cos\alpha + y\sin\alpha) - \sum_{i=1}^{n} \left(\frac{Q_i}{4\pi T}\right) \ln\left[(x - x_i)^2 + (y - y_i)^2\right] \qquad (59)$$

$$w(x,y) = -I\,(y\cos\alpha - x\sin\alpha) - \sum_{i=1}^{n} \left(\frac{Q_i}{2\pi T}\right) \tan^{-1}\left(\frac{y - y_i}{x - x_i}\right) \qquad (60)$$

where I = preoperational, regional hydraulic gradient [decimal fraction]; note that I is always positive;
α = angle between regional flow direction and positive x axis [degrees or radians];
(x,y) = coordinates of point at which h and w are being evaluated [L];
Q_i = pumping rate of ith well [L^3/T], positive for injection, negative for extraction;
n = number of wells;
(x_i,y_i) = coordinates of ith well [L]; and
T = transmissivity of aquifer [L^2/T].

The first term in Equations (59) and (60) is due to the regional gradient. The second term is due to the pumping wells; the similarities in (59) to the Thiem equation (34) should be evident.

With Equations (59) and (60) it is possible to calculate h(x,y) and w(x,y) for a large number of locations, (x,y), and hence to map the two functions for any particular set of n pumping wells at locations, (x_i, y_i), with pumping rates, Q_i. Figure 6c, presented earlier in this chapter, is an example of an h(x,y) plot determined from Equation (59). Figures 6d, 8, and 9b are examples of w(x,y) plots determined from Equation (60).

Differentiating (59) with respect to x and y and multiplying through by –K, as indicated by Equation (58), provides analytical expressions for the components of specific discharge, q_x and q_y:

$$q_x(x,y) = KI\cos\alpha + \sum_{i=1}^{n} \left\{\frac{Q_i}{2\pi b}\,\frac{(x - x_i)}{[(x - x_i)^2 + (y - y_i)^2]}\right\} \qquad (61)$$

$$q_y(x,y) = KI \sin\alpha + \sum_{i=1}^{n} \left\{ \frac{Q_i}{2\pi b} \frac{(y - y_i)}{[(x - x_i)^2 + (y - y_i)^2]} \right\} \tag{62}$$

The components of the average linear velocity, V_x and V_y, are then given by

$$V_x = q_x/n_e \qquad V_y = q_y/n_e \tag{63}$$

where n_e is the effective porosity of the aquifer. This is the velocity at which contaminants move through the aquifer toward an extraction well during remediation. Calculations of V(x,y) can be used to develop plots of remedial time lines like those shown earlier in Figure 7.

For contaminants that are retarded, the velocity components are given by

$$V_x = q_x/n_e R_f \qquad V_y = q_y/n_e R_f \tag{64}$$

where R_f is the retardation factor, as defined in Chapter II.F. No comparably simple method is available to take into account dispersion or matrix diffusion in an analytic, hydraulics-based model.

The analytical methodology presented in this chapter can also be used to determine capture-zone geometry. Javandel and Tsang (1986) use Equation (60) to develop an equation for the dividing streamlines that separate the capture zone of a single well, pumping at Q, from the rest of the aquifer (Figure 12). For $\alpha = 0$, it is given by

$$y = \pm \left(\frac{Q}{2TI}\right) - \left(\frac{Q}{2\pi TI}\right)\tan^{-1}\left(\frac{y}{x}\right). \tag{65}$$

Solving this equation for $x = 0$ and $x = \infty$ allows one to calculate the distance between the dividing streamlines at the line of wells and far upstream from the wells. One can also calculate the downstream distance from the well to the stagnation point by solving for x at $y = 0$. For a single extraction well, these distances are given by $Q/2TI$, Q/TI, and $Q/2\pi TI$. Javandel and Tsang (1986) calculate these values for one-, two- and three-well extraction systems; Table 5 records their results. They also provide capture-zone type curves of the kind shown earlier in Figure 10 for one-, two-, three-, and four-well extraction systems.

Coming finally to the point of well-network design, Javandel and Tsang (1986) use their analysis to calculate the maximum distance that can exist between multiple wells such that capture zones are continuous

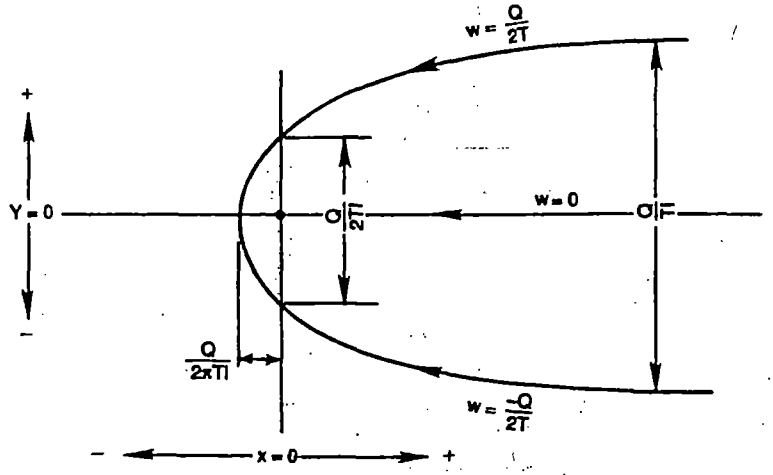

Figure 12. Equation for the dividing streamlines separating the capture zone of a single well from the rest of an aquifer.

and no flow tubes (or contaminants) can slip between the extraction wells. For two or three equally spaced wells, located along a line perpendicular to the regional gradient, and all pumping at the same rate, Javandel and Tsang provide the recommended spacings listed in the right-hand column of Table 5.

The design methodology for a one-, two-, or three-well extraction system using Table 5 involves a trial-and-error procedure with a set of alternative well networks. One tries to identify the lowest cost network that will meet the following specifications, given measured values for aquifer transmissivity, T, and regional hydraulic gradient, I:

1. The capture-zone geometry, as indicated by the values given in Table 5 for the distance between dividing streamlines, must be adequate to encompass the known boundaries of the contaminant plume.
2. The pumping rate, Q, to be applied at each of the wells, must not create drawdowns in excess of any constraints on the available drawdown at the wells.
3. The distances between the wells must be equal to or less than the recommended distances given in Table 5.

It must be emphasized that use of Table 5 to design remedial well networks will *not* lead to an optimal design. The limitations on the analytical solutions on which the table is based are too severe. It will provide a design that works for a pre-specified number of wells, all on a

Table 5. Parameters for Design of Remedial Well Fields Based on Javandel and Tsang (1986) Capture-Zone Theory. For multiple-well systems, Q is the constant pumping rate applied to each well.

Number of Wells	Distance Between Dividing Stream-lines at Line of Wells	Distance Between Dividing Stream-lines Far Upstream From Wells	Downstream Distance to Stagnation Point at Center Point of Capture Zone	Recommended Distance Between Each Pair of Extraction Wells
1	$\frac{Q}{2TI}$	$\frac{Q}{TI}$	$\frac{Q}{2\pi TI}$	—
2	$\frac{Q}{TI}$	$\frac{2Q}{TI}$	$\frac{Q}{2\pi TI}$	$\frac{Q}{\pi TI}$
3	$\frac{3Q}{2TI}$	$\frac{3Q}{TI}$	$\frac{3Q}{4\pi TI}$	$\frac{\sqrt[3]{2}Q}{\pi TI}$

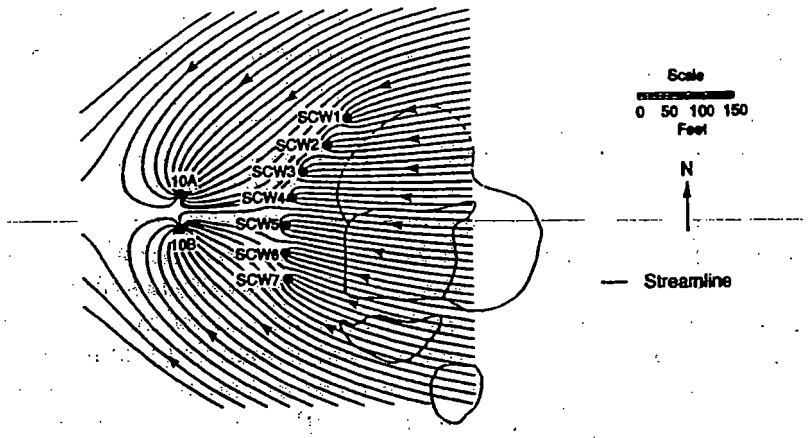

Figure 13. Predicted streamlines for the design network.

line, and all pumping at the same rate. The optimal solution might involve irregular spacings and/or pumping rates.

The concepts and methodology of this chapter have been embodied in a computer code named RESSQ. It is fully documented by Javandel et al. (1984). Given a homogeneous, isotropic, confined aquifer and a system of regional flow, which together with a set of extraction and/or injection wells, creates a steady-state flow field, RESSQ calculates and plots the streamline pattern in the aquifer. With a trial-and-error approach, one can examine the capture-zone geometry of any set of irregularly spaced wells, pumping at any desired rates. RESSQ has been included as the simulation component of a nonlinear optimization model (Greenwald and Gorelick, 1989).

RESSQ is not the only available code for producing streamline patterns. Rounds and Bonn (1989) describe a similar program called DREAM that is a user-friendly, menu-driven program designed specifically for personal computer application. They describe in more detail than Javandel et al. (1984) the complications that arise in the programming and plotting procedures due to the multivalued $\tan^{-1}$ function that appears in Equation (60). RESSQ is available through the International Ground Water Modelling Center at the Colorado School of Mines, Golden, CO 80401. DREAM is available from Lewis Publishers, 121 South Main Street, P.O. Box 519, Chelsea, MI 48118.

Cosgrave et al. (1989) have used a RESSQ-type code in the remedial design of a gradient-control well network at a coal-tar refinery in Illinois. Figure 13 shows the predicted streamlines for the design network. Routine monthly measurements of water-level elevations in a monitor-

ing network during the year following installation of the remedial program showed that the system quickly reached steady state and is performing as designed.

3. Nonuniform-Flow Capture-Zone Analysis

In aquifers that are heterogeneous and anisotropic, the preoperational steady-state regional flow system is not likely to be uniform. Rather, it will show spatial variations in direction and gradient induced by the pattern of high- and low-permeability layers, lenses, and trends. Similarly, steady-state drawdown and buildup cones induced by extraction and injection wells are unlikely to be symmetric; they too will exhibit more complex patterns under the influence of permeability distributions. In these circumstances, RESSQ and DREAM are no longer suitable. The assumptions underlying their analytical basis are no longer satisfied.

It is, however, still possible to utilize deterministic simulation analysis to aid in the design of remedial well networks. It will be necessary to use a finite-difference or finite-element model that is capable of producing steady head distributions in heterogeneous media. There are many such flow-net simulators available. Perhaps the most widely used is MODFLOW, the USGS finite-difference model. This program is capable of transient simulation in three dimensions, and it has many options that allow consideration of wells, drains, streams, recharge, and evapotranspiration. However, it is written in a modular style that is well-suited to invoking the simpler option package we require: steady-state flow in two dimensions with wells. McDonald and Harbaugh (1984) provide easy-to-follow user instructions in their program book. MODFLOW can be run either on a mainframe computer or on a personal computer.

The output from MODFLOW is in the form of hydraulic-head values at finite-difference nodal points; the program does not provide pathline output. To obtain such output, one must use a post-processor program that constructs pathlines from potential-field output. Shafer (1987b) has developed such a program in his GWPATH code. It is an interactive software package for calculating pathlines and travel times in a two-dimensional planar flow field. It allows either forward or reverse pathline tracking, capture-zone analysis, and multiple-pathline capture detection mechanisms. The USGS program, MODPATH (Pollock, 1988; 1989), uses a particle-tracking scheme to develop pathlines for three-dimensional outflow from MODFLOW.

MODFLOW and GWPATH or MODPATH can be used in a trial-and-error format for well-network design in nonuniform flow fields in

heterogeneous, anisotropic aquifers in the same way that RESSQ or DREAM is used in uniform flow fields in homogeneous, isotropic aquifers. Given the patterns of aquifer transmissivity, T(x,y), and regional gradients, I(x,y), the well locations, (x_i, y_i), and pumping rates, Q_i, are established such that drawdowns are acceptable and the plume is completely contained within the capture-zone geometry.

Once again, our caveat must be stated. Simulation alone using MODFLOW and GWPATH or MODPATH does not produce an optimized design, only one that is designed to work. In order to produce a design that minimizes a cost-based objective function, while satisfying all constraints, the simulation model must be coupled to an optimization procedure, as described in Chapter V.

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APPENDIX M

Groundwater Modeling Technical Memorandum

GROUNDWATER SERVICES, INC.

MEMORANDUM

TO: Mr. Chris Fitzgerald, URS Corporation.

FROM: Charles Newell and Shahla Farhat

RE: Sauget Area 2 Focused Feasibility Study

BACKGROUND

As requested by URS Corporation (URS), Groundwater Services, Inc. (GSI) has completed a study of pumping rates and remediation timeframe for five sites at Sauget Area 2 (Sites O, P, Q, R, and S). Five alternatives, ranging from no action to hydraulic control of affected groundwater in Sauget Area 2 were evaluated.

The MODFLOW groundwater flow model developed for the *Interim Groundwater Remedy Design Basis* (GSI, 2002) was modified to obtain groundwater pumping flowrates for each alternative. A simple source decay model was used to estimate remediation timeframes for each of the five sites. The source decay model was originally presented in the *Source Evaluation Study* for Sauget Area 1 (GSI, 2001).

MODFLOW MODEL CALIBRATION

The MODFLOW groundwater model developed for the *Interim Groundwater Remedy Design Basis* and described in detail therein, was refined and calibrated for the entire Sauget Area 2.

- The finite-difference grid with 60 ft by 60 ft cells in the vicinity of Site R was extended to include all of Sauget Area 2 with cell size gradually increasing with distance from Sauget Area 2. Adjacent model cell column and row widths were not altered more than a factor of 2.0 compared to adjacent columns (Zheng and Bennett, 1995, and Spitz and Moreno, 1996). The grid aspect ratio (ratio of column width to row width) was limited between 10 and 0.1.

- Flow calibration against water levels measured on June 9, 2003 was performed by adjusting the river level to 390.31 ft amsl (the average river level for the 24 hours preceding the midpoint of the groundwater level monitoring period) (Table 1b) and comparing the predicted values to the actual measured values (Table 1a) for nine piezometers (URS, 2003) each screened in the shallow, middle, and deep hydrogeologic unit.

- The Mississippi River stage value of 390.31 ft amsl is an average of the hourly river stage values between 12:00 pm on June 8, 2003 and 12:00 pm on June 9, 2003 (Table 1b).

- An overall root mean square (rms) error of 2.00 ft was obtained for the middle horizon, while an rms error of 1.93 ft was obtained for the lower horizon. Overall, the MODFLOW groundwater flow model was considered to yield a reasonable simulation of the aquifer system and all parameters for the initial *Interim Groundwater Remedy Design Basis* model were retained.

- For a discussion of the limitations of the MODFLOW model, see the *Interim Groundwater Remedy Design Basis* report.

FLOWRATE ESTIMATION

The MODFLOW groundwater model was used to develop flow estimates for the five alternatives described below for each of the five sites.

- Groundwater Alternative 1: No Action

- Groundwater Alternative 2: Institutional Controls and Monitoring

- Groundwater Alternative 3: Physical Barrier Adjacent to Site R Only

- Groundwater Alternative 4: Physical Barrier Adjacent to Western Side of Sauget Area 2

- Groundwater Alternative 5: Hydraulic Control Adjacent to the Western Side of Sauget Area 2

For Alternatives 1 and 2, no pumping is involved. Therefore, the groundwater extraction rate is zero and no wells are required.

For Alternative 3, the pumping rate described in the *Interim Groundwater Remedy Design Basis*, a total of 535 gpm from three wells, was used for this study.

For Alternative 4 (Physical Barrier), the Zone Budget feature in MODFLOW was used to calculate the groundwater discharge to the river along the entire length of Sauget Area 2 (approximately 12,500 ft) under natural conditions. This flowrate, 3000 gpm, was assumed to be equal to the extraction rate of a pumping system associated with intercepting water flowing into a 12,500 foot, fully-penetrating, "U"-shaped physical barrier wall extending along the downgradient portion of Sauget Area 2, with side walls extending upgradient towards Site P on the north and along the edge of Site Q in the south. This was the same approach used for the physical barrier for Site R in the *Interim Groundwater Remedy Design Basis*.

It was assumed that 24 pumping wells at well spacings of 531 ft (the same spacing used for the Site R physical barrier) would represent a reasonable extraction system.

For Alternative 5, it was assumed that each well would pump at a high rate to greatly increase the flushing of groundwater through the Sauget Area 2 sites and therefore increase the mass removal rate of site constituents. As described in the *Source Evaluation Study*, typical flowrates for large municipal wells in the area ranged from 1000 to 1500 gpm. For this study, it was assumed that 24 wells, each pumping 1100 gpm, would be representative of a high-flowrate pumping system for Alternative 5.

Summary

The number of wells, flowrates, and total system flowrate for each alternative are shown below. All results are shown for average river level conditions.

	Number of Wells	Pumping Rate per Well (gpm)	Total System Flowrate (gpm)
Alternative 1 – No Action	0	0	0
Alternative 2 – Institutional Controls	0	0	0
Alternative 3 – Physical Barrier Site R Only	3	178	535
Alternative 4 – Physical Barrier Sauget Area 2	24	127	3,000
Alternative 5 – Hydraulic Control	24	1,100	26,400

REMEDIATION TIMEFRAME

The evaluation of cleanup times was based on the methodology presented in the *Source Evaluation Study*. The available groundwater data were evaluated to develop planning-level constituent mass estimates and mass flux estimates for Site O, Site P, Site Q, Site R, and Site S. For the purpose of this study, Site Q was divided into four separate sub-sites with separate mass and mass flux estimates. Source concentration was used to estimate both the source mass and mass flux.

Source Concentration

The average total VOC + SVOC concentrations in the middle and lower horizons were evaluated from the transect well closest to the source area for each site (see Figure 1). For this planning level calculation, it was assumed that these concentrations extended throughout the entire width of the site under consideration.

For Sites O, P, Q, and S, groundwater monitoring data collected from June 18, 2002, through August 13, 2002, were used in the analyses. Groundwater monitoring data from well AA-O-1 for Site O, well AA-P-1 for Site P, and well AA-S-1 (AA-S-1B shown in Figure 1) for Site S were used. Site Q is divided into the North, Central, South, and Ponds areas, therefore, four transect wells (AA-Q-1 for Q North, AA-Q-4 for Q Central, AA-Q-6 for Q South, and AA-Q-8 for Q Ponds), each closest to the area under study, were used for this site. Groundwater monitoring data for Sites O, P, Q, and S are

presented in Table 2.

For Sites O, P, Q, and S, sampling depths were assigned to the middle or deep horizons based on the modeled surface elevations at the wells of interest. Source concentrations for each hydrogeologic unit at a site were then calculated by obtaining the average concentration of all the sampling depths in that unit. For duplicate measurements, the maximum observed values were retained. Samples designated as rejected or "R" by URS quality control review were not considered. For each site, VOC concentrations were determined independently of the SVOC data. The sum of the VOC and SVOC concentrations yielded the total source concentration for that site.

For Site R, average VOC + SVOC concentrations were determined from 1) concentration maps generated by Roux Associates, Inc. (Solutia, 2002. Figures 2-8, 2-9, 2-11, and 2-12), for the middle and lower horizons; and 2) groundwater monitoring data collected for wells GW-1 and GW-2 between July 27, 2001, and August 6, 2001. The following method was used for this site:

1) The length of the site was divided into 10 zones.
2) The concentration in each zone was calculated by taking the average of all the concentration points (monitoring wells, contour levels, and zone boundaries) that lay in that zone.
3) The concentrations at the zone boundaries were determined by interpolating from closest wells and contour levels.
4) For each zone, VOC concentrations were determined independently of the SVOC data.
5) The sum of the VOC and SVOC concentrations yielded the total concentration for that zone.
6) The concentration assigned to Site R was then determined by taking the average of the total concentrations for all the 10 zones.

Resulting concentrations for Site R are shown in Table 3.

Estimated Source Mass

Source mass is a function of source volume, total porosity, residual saturation of DNAPL in the source zone, and the fraction of source volume containing DNAPL. Source thickness was assumed to be the thickness of the horizon at the transect of interest. Source length and width were obtained from Figure 1. A porosity of 0.35, DNAPL saturation of 5%, and DNAPL density of 1.25 kg/L were assumed for all sites.

The percent of the aquifer containing DNAPL was assumed to be proportional to the total dissolved constituent concentration, with the values used in the *Interim Groundwater Remedy Design Basis* as the reference point (i.e., 1% of the aquifer mass containing DNAPL was associated with the ~ 20 mg/L dissolved constituent concentration at Site I in Sauget Area 1). The following relationship was used to

determine the percent of aquifer containing DNAPL for all of the Sauget Area 2 sites (Table 4):

(fraction of aquifer containing DNAPL) = (observed total dissolved phase concentration) x 0.005

Note that there is considerable uncertainty in the source mass estimates, and the results of the following calculations are more appropriate for relative comparisons rather than absolute predictions of remediation timeframe.

Mass Flux

The mass flux (also called the mass removal rate) under natural conditions was estimated by multiplying average VOC + SVOC concentrations for each horizon at each site by the flow for that horizon, and converting to a mass rate of kg/yr leaving the source zone (3.78 L/gal; 1440 min/day; 365 day/yr; 10^{-6} kg/mg). For each site except Site R, the Darcy groundwater velocity was taken from values used in the MODFLOW groundwater model. Site dimensions were taken from Figure 1. Table 5 shows the resulting estimated source mass for each site. Mass flux estimates for Site R were developed separately (See Table 3).

Source Decay Model

The source decay coefficient represents how quickly the source is being depleted and can be derived using estimates of the source mass and the rate at which contaminants leave the source (Newell et al, 1996):

$$k_s = \frac{Q.C_{so}}{M_o}$$

where:

k_s = Source decay coefficient (1/year)
Q = Groundwater flowrate through source zone (L/year)
C_{so} = Observed source concentration at time = 0 (mg/L)
M_o = Dissolvable mass in source at time = 0 (mg)

Estimations of the source decay coefficients are presented in Table 6. For first order decay (see *Source Evaluation Study*), the time required to reach any concentration is given by

$$t = -\frac{1}{k_s}\ln\left(\frac{C_t}{C_{so}}\right)$$

where:

t = Time (years)
k_s = Source decay coefficient (1/year)

C_t = Source concentration goal (mg/L) (assumed to be 0.005 mg/L)
C_{so} = Observed source concentration at time = 0 (mg/L)

Using the estimated source mass and mass flux (equal to the product of the flowrate and the initial concentration), planning level estimations of the source lifetimes under natural attenuation for Sites O, P, Q, R, and S were developed (Table 6).

As described in the *Source Evaluation Study*, source zones include a mixture of pools and fingers that have lower mass transfer rates from DNAPL to groundwater when the groundwater velocity through the source zone is increased. Therefore, it was assumed in this study that increasing the groundwater flowrate through the source zone by a factor of 8.9 (by pumping at 26,400 gpm rather than 3000 gpm) would increase the mass transfer by a factor of 2.5 (the mid-point of finger value of 2.1 and pool values of 3.0) when pumping was started (for details see *Source Evaluation Study*, p. 13). Therefore, the source decay coefficient for Alternative 5 was obtained by multiplying the natural conditions source decay coefficient by 2.5.

The planning level estimations of the source lifetimes under intensive pumping for Sites O, P, Q, R, and S are presented in Table 6.

Summary

Planning level source lifetime calculations indicate that groundwater remediation timeframes for sites with elevated concentrations (sites that now exceed 0.005 mg/L) range from 8 years (Site S) to 351 years (Site R) (Table 6). Intensive groundwater pumping under Alternative 5 generally shortens the remediation timeframes by 60%. Site R is expected to have the longest remediation timeframe, with 351 years predicted for Alternatives 1-4 and 140 years for Alternative 5.

There is considerable uncertainty in the remediation timeframe calculations, making the results more appropriate for relative comparisons rather than absolute predictions.

C. Newell and S. Farhat

GROUNDWATER SERVICES, INC.

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TABLE 1a
COMPARISON OF OBSERVED AND CALCULATED PIEZOMETER WATER LEVELS

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Piezometer	Observed Head[1] (ft AMSL)	Calculated Head[2] (ft AMSL)
Layer 2		
Piez-1	389.72	391.17
Piez-2	390.98	392.13
Piez-3	391.90	393.46
Piez-4	389.86	391.28
Piez-5	392.70	394.14
Piez-6	393.25	395.54
Piez-7	389.06	391.17
Piez-8	393.97	396.31
Piez-9	394.68	398.00
	Layer 2 RMS error (ft)	**2.00**
Layer 3		
Piez-1	389.75	391.17
Piez-2	391.05	392.13
Piez-3	391.83	393.46
Piez-4	390.16	391.28
Piez-5	392.71	394.14
Piez-6	393.32	395.54
Piez-7	389.43	391.17
Piez-8	393.98	396.31
Piez-9	394.69	398.00
	Layer 3 RMS error (ft)	**1.93**

Notes:
1. Observed values measured on June 9, 2003.
2. Modeled groundwater elevations obtained from MODFLOW using a river elevation of 390.31 ft amsl.

ft = feet
AMSL = American mean sea level
RMS = root mean square

GROUNDWATER SERVICES, INC.

TABLE 1b
RIVER STAGE ESTIMATE FOR MODFLOW CALIBRATION

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Date and Time	Gage Height (ft)	River Stage (ft AMSL)
June 8 2003 12:00	11.25	391.19
June 8 2003 13:00	11.24	391.18
June 8 2003 14:00	11.13	391.07
June 8 2003 15:00	11.13	391.07
June 8 2003 16:00	11.19	391.13
June 8 2003 17:00	11.02	390.96
June 8 2003 18:00	10.96	390.90
June 8 2003 19:00	10.87	390.81
June 8 2003 20:00	10.76	390.70
June 8 2003 21:00	10.67	390.61
June 8 2003 22:00	10.57	390.51
June 8 2003 23:00	10.48	390.42
June 8 2003 24:00	10.38	390.32
June 9 2003 1.00	10.25	390.19
June 9 2003 2:00	10.15	390.09
June 9 2003 3:00	10.03	389.97
June 9 2003 4:00	9.97	389.91
June 9 2003 5:00	9.9	389.84
June 9 2003 6:00	9.86	389.80
June 9 2003 7:00	9.72	389.66
June 9 2003 8:00	9.64	389.58
June 9 2003 9:00	9.58	389.52
June 9 2003 10:00	9.54	389.48
June 9 2003 11:00	9.46	389.40
June 9 2003 12:00	9.42	389.36
	Mean of Stage	390.31

Notes:
1. Gage Zero = 379.94 ft: obtained from http://mvs-wc.mvs.usace.army.mil/MISS/MISL.html; gage number 0179A Mississippi River at St. Louis, Missouri).
2. Hourly gage data for station USGS 07010000 (Mississippi River at St. Louis, Missouri) obtained from U.S. Geological Survey - WRD, Rolla, Missouri.
   ft = feet
   AMSL = American mean sea level

GROUNDWATER SERVICES, INC.

TABLE 2
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)											
	Layer 1	Layer 2			Layer 3							
	16 ft bgs	26 ft bgs	36 ft bgs	46 ft bgs	56 ft bgs	66 ft bgs	76 ft bgs	86 ft bgs	96 ft bgs	106 ft bgs	116 ft bgs	120 ft bgs
SITE O												
VOCs												
1,1,1-Trichloroethane	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
1,1,2,2-Tetrachloroethane	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
1,1,2-Trichloroethane	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
1,1-Dichloroethane	2.4	2.6	2.2	2.5	2.8	1.2	1	<5	<20	<10	<10	<2.5
1,1-Dichloroethylene	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	1.1 J
1,2-Dichloroethane	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
1,2-Dichloroethene (total)	3	5.7	0.62 J	0.57 J	2	<2	0.77 J	<10	<40	<20	<20	<5
1,2-Dichloropropane	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
2-Butanone (MEK)	<10	<10	<10	<10	<10	<10	<10	<50	<200	<100	<100	<25
4-Methyl-2-pentanone (MIBK)	<10	<10	<10	<10	<10	<10	<10	<50	<200	<100	<100	<25
Acetone	-	-	-	-	-	<25	<25	<120	<500	<250	<250	<62
Benzene	0.1 J	0.13 J	<1	<1	<1	<1	<1	<5	<20	9.6 J	<10	<2.5
Bromodichloromethane	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
Bromoform	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
Bromomethane	<1	<1	<1	<1	<1	<1	<1	-	<20	<10	<10	<2.5
Carbon Disulfide	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
Carbon Tetrachloride	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
Chlorobenzene	7.8	5.4	32	7.1	5.6	7.8	180	620	1500	1300	900	400
Chlorodibromomethane	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
Chloroethane	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
Chloroform	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
Chloromethane	<1	<1	<1	<1	<1	0.45 J	<1	<5	<20	<10	<10	<2.5
cis-1,3-Dichloropropene	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
Dichloromethane	<5	<5	<5	<5	<5	<5	0.72 J	<25	<100	<50	<50	<12
Ethylbenzene	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
Methyl N-Butyl Ketone	<10	<10	<10	<10	<10	<10	<10	<50	<200	<100	<100	<25
Styrene (Monomer)	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
Tetrachloroethene	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
Toluene	<1	<1	<1	<1	<1	<1	0.52 J	<5	<20	<10	<10	1 J
trans-1,3-Dichloropropene	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5
Trichloroethylene	<1	<1	<1	<1	<1	<1	<1	<5	<20	<10	<10	<2.5

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)											
	Layer 1	Layer 2			Layer 3							
	16 ft bgs	26 ft bgs	36 ft bgs	46 ft bgs	56 ft bgs	66 ft bgs	76 ft bgs	86 ft bgs	96 ft bgs	106 ft bgs	116 ft bgs	120 ft bgs
VOCs cont'd												
Vinyl chloride	0.35 J	0.76 J	0.47 J	1.6	2.5	2	5.4	3.8 J	<20	<10	<10	2.2 J
Xylenes, Total	<2	<2	<2	<2	<2	<2	<2	<10	<40	<20	<20	<5
SVOCs												
1,2,4-Trichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
1,2-Dichlorobenzene	<10	<10	7.4 J	2.2 J	<10	1.2 J	1.6 J	1.1 J	3 J	2.7 J	2.3 J	1.2 J
1,3-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	1.5 J	<10	<10
1,4-Dichlorobenzene	<10	<10	3.5 J	1.4 J	<10	<10	1.4 J	3.7 J	51	80	66	21
2,2'-Oxybis(1-Chloropropane)	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4,5-Trichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4,6-Trichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4-Dichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	3.4 J	3.5 J	1.2 J
2,4-Dimethylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	1.4 J	2.1 J	<10
2,4-Dinitrophenol	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
2,4-Dinitrotoluene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,6-Dinitrotoluene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Chloronaphthalene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Chlorophenol	<10	<10	<10	<10	<10	<10	4.8 J	11	18	19	17	6.5 J
2-Methylnaphthalene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Methylphenol (o-Cresol)	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
2-Nitrophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
3,3'-Dichlorobenzidine	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20
3-Methylphenol/4-Methylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
3-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
4,6-Dinitro-2-methylphenol	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
4-Bromophenyl Phenyl Ether	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Chloro-3-methylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Chlorophenyl Phenyl Ether	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Nitrophenol	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
Acenaphthene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Acenaphthylene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Anthracene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(a)anthracene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(a)pyrene	1.6 J	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(b)fluoranthene	1.1 J	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)											
	Layer 1	Layer 2			Layer 3							
	16 ft bgs	26 ft bgs	36 ft bgs	46 ft bgs	56 ft bgs	66 ft bgs	76 ft bgs	86 ft bgs	96 ft bgs	106 ft bgs	116 ft bgs	120 ft bgs
SVOCs cont'd												
Benzo(g,h,i)perylene	2.8 J	<10	<10	<10	1.6 J	<10	<10	<10	<10	<10	<10	<10
Benzo(k)fluoranthene	1.2 J	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzyl Butyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
bis(2-Chloroethoxy)methane	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
bis(2-Chloroethyl)ether	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
bis(2-Ethylhexyl)phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Carbazole	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Chrysene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Dibenzo(a,h)anthracene	2.7 J	<10	<10	<10	1.3 J	<10	<10	<10	<10	<10	<10	<10
Dibenzofuran	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Diethyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Dimethyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Di-n-butylphthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Di-n-octylphthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Fluoranthene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Fluorene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorobutadiene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorocyclopentadiene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachloroethane	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Indeno(1,2,3-cd)pyrene	3 J	<10	<10	<10	1.8 J	<10	<10	<10	<10	<10	<10	<10
Isophorone	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Naphthalene	<10	<10	<10	<10	<10	<10	2.1 J	5.7 J	8.7 J	10	8 J	3 J
Nitrobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
N-Nitroso-di-n-propylamine	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
N-Nitrosodiphenylamine	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
P-Chloroaniline	<20	<20	<20	<20	<20	<20	<20	1.1 J	3.4 J	4.1 J	2.8 J	<20
Pentachlorophenol	<1	<50	<50	<50	<1	<50	<50	<50	<1	<50	<50	<1
Phenanthrene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Phenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
P-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
Pyrene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)										
	Layer 2			Layer 3							
	24 ft bgs	34 ft bgs	44 ft bgs	54 ft bgs	64 ft bgs	74 ft bgs	84 ft bgs	94 ft bgs	104 ft bgs	114 ft bgs	120 ft bgs
SITE P											
VOCs											
1,1,1-Trichloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
1,1,2,2-Tetrachloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
1,1,2-Trichloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
1,1-Dichloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
1,1-Dichloroethylene	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
1,2-Dichloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
1,2-Dichloroethene (total)	<2	<2	<2	<2	<2	<2	<2	<2	<50	<50	<20
1,2-Dichloropropane	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
2-Butanone (MEK)	7.5 J	<10	<10	<10	<10	<10	<10	<10	<250	<250	<100
4-Methyl-2-pentanone (MIBK)	<10	<10	<10	<10	<10	<10	<10	<10	<250	<250	<100
Acetone	39	<25	<25	<25	<25	<25	<25	<25	<620	<620	<250
Benzene	<1	<1	<1	<1	<1	<1	<1	<1	190	60	77
Bromodichloromethane	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
Bromoform	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
Bromomethane	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
Carbon Disulfide	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	6.4 J
Carbon Tetrachloride	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
Chlorobenzene	<1	<1	<1	<1	1	5.8	12	4.8	4900	4600	2300
Chlorodibromomethane	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
Chloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
Chloroform	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
Chloromethane	0.33 J	<1	<1	<1	<1	<1	<1	0.53 J	<25	<25	<10
cis-1,3-Dichloropropene	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
Dichloromethane	<5	<5	<5	<5	<5	<5	<5	<5	<120	<120	<50
Ethylbenzene	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
Methyl N-Butyl Ketone	<10	<10	<10	<10	<10	<10	<10	<10	<250	<250	<100
Styrene (Monomer)	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
Tetrachloroethene	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
Toluene	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
trans-1,3-Dichloropropene	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)										
	Layer 2			Layer 3							
	24 ft bgs	34 ft bgs	44 ft bgs	54 ft bgs	64 ft bgs	74 ft bgs	84 ft bgs	94 ft bgs	104 ft bgs	114 ft bgs	120 ft bgs
VOCs cont'd											
Trichloroethylene	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	16
Vinyl chloride	<1	<1	<1	<1	<1	<1	<1	<1	<25	<25	<10
Xylenes, Total	<2	<2	<2	<2	<2	<2	<2	<2	<50	<50	<20
SVOCs											
1,2,4-Trichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
1,2-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
1,3-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
1,4-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	10 J	9.8 J	3.1 J
2,2'-Oxybis(1-Chloropropane)	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
2,4,5-Trichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
2,4,6-Trichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
2,4-Dichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
2,4-Dimethylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
2,4-Dinitrophenol	<50	<50	<50	<50	<50	<50	<50	<50	<100	<50	<50
2,4-Dinitrotoluene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
2,6-Dinitrotoluene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
2-Chloronaphthalene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
2-Chlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	78	46	22
2-Methylnaphthalene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
2-Methylphenol (o-Cresol)	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
2-Nitroaniline	<50	<50	<50	<50	1.2 J	<50	<50	<50	<100	<50	<50
2-Nitrophenol	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
3,3'-Dichlorobenzidine	<20	<20	-	<20	<20	<20	<20	<20	<40	<20	<20
3-Methylphenol/4-Methylphenol	<10	<10	<10	<10	1.4 J	<10	<10	<10	<20	<10	<10
3-Nitroaniline	<50	<50	-	<50	<50	<50	<50	<50	<100	<50	<50
4,6-Dinitro-2-methylphenol	<50	<50	<50	<50	5.2 J	<50	<50	<50	<100	<50	<50
4-Bromophenyl Phenyl Ether	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
4-Chloro-3-methylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
4-Chlorophenyl Phenyl Ether	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
4-Nitrophenol	<50	<50	<50	<50	<50	<50	<50	<50	<100	<50	<50
Acenaphthene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Acenaphthylene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Anthracene	<10	<10	<10	<10	0.8 J	<10	<10	<10	<20	<10	<10
Benzo(a)anthracene	<10	<10	<10	<10	1 J	<10	<10	<10	<20	<10	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)										
	Layer 2			Layer 3							
	24 ft bgs	34 ft bgs	44 ft bgs	54 ft bgs	64 ft bgs	74 ft bgs	84 ft bgs	94 ft bgs	104 ft bgs	114 ft bgs	120 ft bgs
SVOCs cont'd											
Benzo(a)pyrene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Benzo(b)fluoranthene	<10	<10	<10	<10	4 J	<10	<10	<10	<20	<10	<10
Benzo(g,h,i)perylene	<10	<10	<10	<10	1.1 J	<10	<10	<10	<20	<10	<10
Benzo(k)fluoranthene	<10	<10	<10	<10	0.89 J	<10	<10	<10	<20	<10	<10
Benzyl Butyl Phthalate	<10	<10	<10	<10	0.87 J	<10	<10	<10	<20	<10	<10
bis(2-Chloroethoxy)methane	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
bis(2-Chloroethyl)ether	<10	<10	<10	<10	1.1 J	<10	<10	<10	<20	<10	<10
bis(2-Ethylhexyl)phthalate	<10	<10	<10	<10	1.2 J	<10	<10	<10	<20	<10	<10
Carbazole	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Chrysene	<10	<10	<10	<10	0.95 J	<10	<10	<10	<20	<10	<10
Dibenzo(a,h)anthracene	<10	<10	<10	<10	2.9 J	<10	<10	<10	4.2 J	<10	<10
Dibenzofuran	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Diethyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Dimethyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Di-n-butylphthalate	<10	<10	<10	<10	1.1 J	<10	<10	<10	<20	<10	<10
Di-n-octylphthalate	<10	<10	<10	<10	0.72 J	<10	<10	<10	<20	<10	<10
Fluoranthene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Fluorene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Hexachlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Hexachlorobutadiene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Hexachlorocyclopentadiene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Hexachloroethane	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Indeno(1,2,3-cd)pyrene	<10	<10	<10	<10	0.89 J	<10	<10	<10	<20	<10	<10
Isophorone	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
Naphthalene	<10	<10	<10	<10	<10	<10	<10	<10	39	21	11
Nitrobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
N-Nitroso-di-n-propylamine	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
N-Nitrosodiphenylamine	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10
P-Chloroaniline	<20	<20	-	<20	<20	<20	<20	<20	24 J	7.3 J	2.8 J
Pentachlorophenol	0.21 J	<50	<50	<50	<1	6.4 J	5.9 J	<50	4.7 J	<50	3.9 J
Phenanthrene	<10	<10	<10	<10	0.98 J	<10	<10	<10	<20	<10	<10
Phenol	<10	<10	<10	<10	<10	<10	<10	<10	14 J	3.4 J	2.8 J
P-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<100	<50	<50
Pyrene	<10	<10	<10	<10	<10	<10	<10	<10	<20	<10	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L) Layer 3								
	50 ft bgs	60 ft bgs	70 ft bgs	80 ft bgs	90 ft bgs	100 ft bgs	110 ft bgs	120 ft bgs	130 ft bgs
SITE Q: AA-Q-1									
VOCs									
1,1,1-Trichloroethane	<1	<1	<1	<2	<1	<1	<1	<2	<5
1,1,2,2-Tetrachloroethane	<1	<1	<1	<2	<1	<1	<1	<2	<5
1,1,2-Trichloroethane	<1	<1	<1	<2	<1	<1	<1	<2	<5
1,1-Dichloroethane	5.6	4.9	1.1	2.2	2.9	<1	<1	<2	<5
1,1-Dichloroethylene	<1	<1	<1	<2	<1	<1	<1	<2	<5
1,2-Dichloroethane	<1	<1	<1	<2	<1	<1	<1	<2	<5
1,2-Dichloroethene (total)	0.96 J	<2	<2	<4	<2	<2	<2	<4	<10
1,2-Dichloropropane	<1	<1	<1	<2	<1	<1	<1	<2	<5
2-Butanone (MEK)	<10	<10	<10	<20	<10	<10	<10	<20	<50
4-Methyl-2-pentanone (MIBK)	<10	<10	<10	<20	<10	<10	<10	<20	<50
Acetone	11 J	16 J	<25	<50	<25	<25	<25	<50	<120
Benzene	2.4	<1	<1	<2	1.4	<1	10	28	33
Bromodichloromethane	<1	<1	<1	<2	<1	<1	<1	<2	<5
Bromoform	<1	<1	<1	<2	<1	<1	<1	<2	<5
Bromomethane	<1	<1	<1	<2	<1	<1	<1	<2	<5
Carbon Disulfide	0.37 J	2.5 J	<1	<2	<1	<1	<1	<2	<5
Carbon Tetrachloride	<1	<1	<1	<2	<1	<1	<1	<2	<5
Chlorobenzene	130	240	100	130	220	200	130	310	1100
Chlorodibromomethane	<1	<1	<1	<2	<1	<1	<1	<2	<5
Chloroethane	1.3	0.89 J	<1	0.69 J	0.84 J	<1	<1	<2	<5
Chloroform	<1	<1	<1	<2	<1	<1	<1	<2	<5
Chloromethane	0.96 J	<1	<1	<2	<1	<1	0.21 J	<2	2.7 J
cis-1,3-Dichloropropene	<1	<1	<1	<2	<1	<1	<1	<2	<5
Dichloromethane	<5	<5	<5	<10	<5	<5	<5	<10	<25
Ethylbenzene	1	<1	<1	<2	<1	<1	0.89 J	2.1	6.9
Methyl N-Butyl Ketone	<10	<10	<10	<20	<10	<10	<10	<20	<50
Styrene (Monomer)	<1	<1	<1	<2	<1	<1	<1	<2	<5
Tetrachloroethene	<1	<1	<1	<2	<1	<1	<1	<2	<5
Toluene	<1	<1	<1	<2	<1	<1	0.39 J	<2	1.5 J
trans-1,3-Dichloropropene	<1	<1	<1	<2	<1	<1	<1	<2	<5
Trichloroethylene	<1	<1	<1	<2	<1	<1	<1	<2	<5
Vinyl chloride	0.72 J	<1	<1	<2	<1	<1	3.2	2.9	4.6 J
Xylenes, Total	<2	<2	<2	<4	<2	<2	2	<4	22

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L) Layer 3											
	50 ft bgs	Filtered 50 ft bgs	60 ft bgs	70 ft bgs	80 ft bgs	90 ft bgs	100 ft bgs	110 ft bgs	120 ft bgs	Filtered 120 ft bgs	128 ft bgs	Filtered 128 ft bgs
SVOCs												
1,2,4-Trichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	6.1 J
1,2-Dichlorobenzene	11	9.3 J	8 J	4 J	5.6 J	<10	<10	3 J	7.4 J	3.4 J	440	490
1,3-Dichlorobenzene	2.5 J	1.6 J	1.5 J	<10	<10	<10	<10	<10	<10	<10	<100	2.5 J
1,4-Dichlorobenzene	30	15	21	9.6 J	12	12	7.8 J	3.2 J	6.1 J	<10	19 J	22
2,2'-Oxybis(1-Chloropropane)	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
2,4,5-Trichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
2,4,6-Trichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
2,4-Dichlorophenol	1 J	<10	<10	<10	1 J	<10	<10	<10	<10	<10	<100	12
2,4-Dimethylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
2,4-Dinitrophenol	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<500	<50
2,4-Dinitrotoluene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
2,6-Dinitrotoluene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
2-Chloronaphthalene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
2-Chlorophenol	3.2 J	2.8 J	3 J	1.6 J	2.4 J	3.7 J	4.6 J	3.6 J	10	4 J	<100	5.6 J
2-Methylnaphthalene	<10	-	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
2-Methylphenol (o-Cresol)	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
2-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<500	<50
2-Nitrophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
3,3'-Dichlorobenzidine	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<200	<20
3-Methylphenol/4-Methylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
3-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<500	<50
4,6-Dinitro-2-methylphenol	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<500	<50
4-Bromophenyl Phenyl Ether	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
4-Chloro-3-methylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
4-Chlorophenyl Phenyl Ether	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
4-Nitrophenol	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<500	<50
Acenaphthene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Acenaphthylene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Anthracene	<10	<10	<10	<10	0.62 J	<10	<10	<10	<10	<10	<100	<10
Benzo(a)anthracene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Benzo(a)pyrene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Benzo(b)fluoranthene	<10	<10	<10	<10	3.7 J	<10	<10	<10	<10	<10	<100	<10
Benzo(g,h,i)perylene	<10	<10	<10	<10	1.2 J	<10	<10	<10	1 J	<10	<100	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L) Layer 3											
	50 ft bgs	Filtered 50 ft bgs	60 ft bgs	70 ft bgs	80 ft bgs	90 ft bgs	100 ft bgs	110 ft bgs	120 ft bgs	Filtered 120 ft bgs	128 ft bgs	Filtered 128 ft bgs
SVOCs cont'd												
Benzo(k)fluoranthene	<10	<10	<10	<10	0.84 J	<10	<10	<10	<10	<10	<100	<10
Benzyl Butyl Phthalate	<10	<10	<10	<10	0.66 J	<10	<10	<10	<10	<10	<100	<10
bis(2-Chloroethoxy)methane	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
bis(2-Chloroethyl)ether	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
bis(2-Ethylhexyl)phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Carbazole	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Chrysene	<10	<10	<10	<10	0.58 J	<10	<10	<10	<10	<10	<100	<10
Dibenzo(a,h)anthracene	<10	<10	<10	<10	2.8 J	<10	<10	<10	1 J	<10	<100	<10
Dibenzofuran	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Diethyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Dimethyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Di-n-butylphthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Di-n-octylphthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Fluoranthene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Fluorene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Hexachlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Hexachlorobutadiene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Hexachlorocyclopentadiene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Hexachloroethane	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Indeno(1,2,3-cd)pyrene	<10	<10	<10	<10	0.88 J	<10	<10	<10	1 J	<10	<100	<10
Isophorone	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Naphthalene	<10	<10	<10	<10	<10	<10	<10	2 J	5.9 J	2.1 J	<100	3.1 J
Nitrobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
N-Nitroso-di-n-propylamine	<10	<10	<10	<10	1.5 J	<10	<10	<10	<10	<10	<100	<10
N-Nitrosodiphenylamine	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
P-Chloroaniline	130	120	80	14 J	55	110	5.2 J	1.6 J	27	36	3200	3500
Pentachlorophenol	<1	<50	<50	<50	<1	<50	<50	<50	<1	<50	<1	<50
Phenanthrene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10
Phenol	<10	<10	<10	<10	<10	<10	<10	<10	3.4 J	2.3 J	<100	2 J
P-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<500	<50
Pyrene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<100	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L) Layer 3						
	50 ft bgs	60 ft bgs	70 ft bgs	80 ft bgs	90 ft bgs	100 ft bgs	110 ft bgs
SITE Q: AA-Q-4							
VOCs							
1,1,1-Trichloroethane	<1	<1	<1	<1	<1	<1	<1
1,1,2,2-Tetrachloroethane	<1	<1	<1	<1	<1	<1	<1
1,1,2-Trichloroethane	<1	<1	<1	<1	<1	<1	<1
1,1-Dichloroethane	<1	<1	<1	<1	<1	<1	<1
1,1-Dichloroethylene	<1	<1	<1	<1	<1	<1	<1
1,2-Dichloroethane	<1	<1	<1	<1	<1	<1	<1
1,2-Dichloroethene (total)	<2	<2	<2	<2	<2	<2	<2
1,2-Dichloropropane	<1	<1	<1	<1	<1	<1	<1
2-Butanone (MEK)	<10	<10	<10	<10	1.8 J	<10	<10
4-Methyl-2-pentanone (MIBK)	<10	<10	<10	<10	<10	<10	<10
Acetone	-	<25	<25	<25	<25	<25	<25
Benzene	<1	<1	<1	0.38 J	<1	0.18 J	<1
Bromodichloromethane	<1	<1	<1	<1	<1	<1	<1
Bromoform	<1	<1	<1	<1	<1	<1	<1
Bromomethane	<1	<1	<1	<1	<1	<1	<1
Carbon Disulfide	<1	0.65 J	0.32 J	<1	<1	2.5	0.33 J
Carbon Tetrachloride	<1	<1	<1	<1	<1	<1	<1
Chlorobenzene	59	9.8	5.6	45	52	7.7	1.3
Chlorodibromomethane	<1	<1	<1	<1	<1	<1	<1
Chloroethane	<1	<1	<1	<1	<1	<1	<1
Chloroform	<1	<1	<1	<1	<1	<1	<1
Chloromethane	<1	<1	<1	<1	<1	<1	<1
cis-1,3-Dichloropropene	<1	<1	<1	<1	<1	<1	<1
Dichloromethane	<5	<5	<5	<5	<5	<5	<5
Ethylbenzene	<1	0.82 J	<1	0.9 J	<1	<1	<1
Methyl N-Butyl Ketone	<10	<10	<10	<10	<10	<10	<10
Styrene (Monomer)	<1	<1	<1	<1	<1	<1	<1
Tetrachloroethene	<1	<1	<1	<1	<1	<1	<1
Toluene	0.36 J	0.34 J	0.38 J	0.32 J	<1	<1	<1
trans-1,3-Dichloropropene	<1	<1	<1	<1	<1	<1	<1
Trichloroethylene	<1	<1	<1	<1	<1	<1	<1
Vinyl chloride	<1	<1	<1	<1	<1	<1	<1
Xylenes, Total	<2	<2	<2	<2	<2	<2	<2

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L) Layer 3								
	50 ft bgs	Filtered 50 ft bgs	60 ft bgs	70 ft bgs	80 ft bgs	90 ft bgs	100 ft bgs	110 ft bgs	Filtered 110 ft bgs
SVOCs									
1,2,4-Trichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10
1,2-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10
1,3-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10
1,4-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,2'-Oxybis(1-Chloropropane)	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4,5-Trichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4,6-Trichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4-Dichlorophenol	<10	<10	2.2 J	2 J	2.1 J	2.4 J	<10	<10	<10
2,4-Dimethylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4-Dinitrophenol	<50	<50	<50	<50	<50	<50	<50	<50	<50
2,4-Dinitrotoluene	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,6-Dinitrotoluene	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Chloronaphthalene	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Chlorophenol	7.6 J	4 J	<10	<10	<10	1.8 J	<10	<10	<10
2-Methylnaphthalene	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Methylphenol (o-Cresol)	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50
2-Nitrophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10
3,3'-Dichlorobenzidine	<20	<20	-	-	-	-	<20	<20	<20
3-Methylphenol/4-Methylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10
3-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50
4,6-Dinitro-2-methylphenol	<50	<50	<50	<50	<50	<50	<50	<50	<50
4-Bromophenyl Phenyl Ether	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Chloro-3-methylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Chlorophenyl Phenyl Ether	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Nitrophenol	<50	<50	<50	<50	<50	<50	<50	<50	<50
Acenaphthene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Acenaphthylene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Anthracene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(a)anthracene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(a)pyrene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(b)fluoranthene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(g,h,i)perylene	<10	<10	<10	<10	<10	<10	<10	<10	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L) Layer 3								
	50 ft bgs	Filtered 50 ft bgs	60 ft bgs	70 ft bgs	80 ft bgs	90 ft bgs	100 ft bgs	110 ft bgs	Filtered 110 ft bgs
SVOCs cont'd									
Benzo(k)fluoranthene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzyl Butyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10
bis(2-Chloroethoxy)methane	<10	<10	-	-	-	-	<10	<10	<10
bis(2-Chloroethyl)ether	3.8 J	4.4 J	<10	<10	<10	<10	<10	<10	<10
bis(2-Ethylhexyl)phthalate	<10	<10	<10	<10	0.87 J	<10	<10	<10	<10
Carbazole	<10	<10	-	-	-	-	<10	<10	<10
Chrysene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Dibenzo(a,h)anthracene	<10	-	<10	<10	<10	<10	<10	<10	-
Dibenzofuran	<10	<10	<10	<10	<10	<10	<10	<10	<10
Diethyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10
Dimethyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10
Di-n-butylphthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10
Di-n-octylphthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10
Fluoranthene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Fluorene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorobutadiene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorocyclopentadiene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachloroethane	<10	<10	<10	<10	<10	<10	<10	<10	<10
Indeno(1,2,3-cd)pyrene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Isophorone	<10	<10	<10	<10	<10	<10	<10	<10	<10
Naphthalene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Nitrobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10
N-Nitroso-di-n-propylamine	<10	<10	<10	<10	<10	<10	<10	<10	<10
N-Nitrosodiphenylamine	<10	<10	<10	<10	<10	<10	<10	<10	<10
P-Chloroaniline	<20	<20	-	-	-	-	<20	<20	<20
Pentachlorophenol	<1	<50	<50	<50	0.41 J	<50	<50	0.49 J	<50
Phenanthrene	<10	<10	<10	<10	<10	<10	<10	<10	<10
Phenol	<10	<10	<10	<10	<10	<10	<10	<10	<10
P-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50
Pyrene	<10	<10	<10	<10	<10	<10	<10	<10	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)									
	Layer 2		Layer 3							
	24 ft bgs	34 ft bgs	44 ft bgs	54 ft bgs	64 ft bgs	74 ft bgs	84 ft bgs	94 ft bgs	104 ft bgs	110 ft bgs
SITE Q: AA-Q-6										
VOCs										
1,1,1-Trichloroethane	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
1,1,2,2-Tetrachloroethane	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
1,1,2-Trichloroethane	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
1,1-Dichloroethane	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
1,1-Dichloroethylene	<5	<100	<5	<1	<1	<1	<5	0.37 J	<1	<1
1,2-Dichloroethane	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
1,2-Dichloroethene (total)	<10	<200	<10	<2	<2	<2	<10	0.37 J	<2	<2
1,2-Dichloropropane	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
2-Butanone (MEK)	<50	<1000	<50	1.4 J	10	<10	<50	3.1 J	2.3 J	1.4 J
4-Methyl-2-pentanone (MIBK)	<50	<1000	<50	<10	<10	<10	<50	<10	<10	<10
Acetone	61 J	<2500	<120	<25	24 J	14 J	<120	<25	<25	<25
Benzene	490	12000	520	1100	72	56	580	84	110	39
Bromodichloromethane	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
Bromoform	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
Bromomethane	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
Carbon Disulfide	<5	<100	<5	2.2	<1	0.57 J	<5	0.67 J	0.33 J	0.38 J
Carbon Tetrachloride	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
Chlorobenzene	<5	<100	<5	1.5	1.4	1.2	<5	0.58 J	<1	0.55 J
Chlorodibromomethane	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
Chloroethane	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
Chloroform	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
Chloromethane	<5	<100	<5	<1	<1	<1	1.5 J	<1	<1	<1
cis-1,3-Dichloropropene	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
Dichloromethane	<25	<500	<25	<5	<5	<5	<25	1.2 J	<5	<5
Ethylbenzene	64	<100	<5.7	2.3	4.2	1.6	9.1	1.4	1.4	1.1
Methyl N-Butyl Ketone	<50	<1000	<50	<10	<10	<10	<50	<10	<10	<10
Styrene (Monomer)	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
Tetrachloroethene	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
Toluene	70	52 J	4.7 J	3.8	0.88 J	0.69 J	9.4	1.1	0.66 J	0.48 J
trans-1,3-Dichloropropene	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
Trichloroethylene	<5	<100	<5	<1	<1	<1	<5	0.76 J	<1	<1
Vinyl chloride	<5	<100	<5	<1	<1	<1	<5	<1	<1	<1
Xylenes, Total	26	<200	10	4	3.8	<2	11	2.8	0.9 J	<2

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)													
	Layer 2			Layer 3										
	24 ft bgs	Filtered 24 ft bgs	34 ft bgs	44 ft bgs	54 ft bgs	64 ft bgs	Filtered 64 ft bgs	74 ft bgs	84 ft bgs	94 ft bgs	104 ft bgs	Filtered 104 ft bgs	110 ft bgs	Filtered 110 ft bgs
SVOCs														
1,2,4-Trichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
1,2-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
1,3-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
1,4-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,2'-Oxybis(1-Chloropropane)	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4,5-Trichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4,6-Trichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4-Dichlorophenol	9.4 J	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4-Dimethylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4-Dinitrophenol	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
2,4-Dinitrotoluene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,6-Dinitrotoluene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Chloronaphthalene	<10	<10	<10	<10	<10	<10	<10	<10	-	-	<10	<10	<10	<10
2-Chlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Methylnaphthalene	500 J	-	150	16	7.9 J	32	14	10	20	2 J	2.6 J	<10	2 J	-
2-Methylphenol (o-Cresol)	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
2-Nitrophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
3,3'-Dichlorobenzidine	<20	<20	<20	<20	-	-	-	-	<20	<20	-	<20	-	<20
3-Methylphenol/4-Methylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
3-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
4,6-Dinitro-2-methylphenol	<50	<50	2.2 J	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
4-Bromophenyl Phenyl Ether	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Chloro-3-methylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Chlorophenyl Phenyl Ether	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Nitrophenol	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
Acenaphthene	8 J	6.9 J	2.7 J	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Acenaphthylene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Anthracene	0.65 J	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(a)anthracene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(a)pyrene	1.4 J	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(b)fluoranthene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(g,h,i)perylene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)													
	Layer 2			Layer 3										
	24 ft bgs	Filtered 24 ft bgs	34 ft bgs	44 ft bgs	54 ft bgs	64 ft bgs	Filtered 64 ft bgs	74 ft bgs	84 ft bgs	94 ft bgs	104 ft bgs	Filtered 104 ft bgs	110 ft bgs	Filtered 110 ft bgs
SVOCs cont'd														
Benzo(k)fluoranthene	1.4 J	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzyl Butyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
bis(2-Chloroethoxy)methane	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
bis(2-Chloroethyl)ether	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
bis(2-Ethylhexyl)phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Carbazole	12	8.3 J	4.8 J	<10	<10	<10	<10	<10	1.2 J	<10	<10	<10	<10	<10
Chrysene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Dibenzo(a,h)anthracene	3.2 J	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Dibenzofuran	5.2 J	2.9 J	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Diethyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Dimethyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Di-n-butylphthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Di-n-octylphthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Fluoranthene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Fluorene	9.8 J	6.2 J	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorobutadiene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorocyclopentadiene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachloroethane	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Indeno(1,2,3-cd)pyrene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Isophorone	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Naphthalene	120 J	91 J	<10	<10	<10	15	10	3.2 J	2.2 J	<10	<10	<10	<10	<10
Nitrobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
N-Nitroso-di-n-propylamine	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
N-Nitrosodiphenylamine	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
P-Chloroaniline	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20
Pentachlorophenol	<1	<50	<50	<50	<50	<1	<50	<50	<50	<50	<1	<50	<1	<50
Phenanthrene	18	7.2 J	<10	<10	<10	5 J	<10	2.4 J	<10	<10	<10	<10	<10	<10
Phenol	-	14	320	46	38	4 J	4.7 J	3.4 J	24	3.2 J	7 J	9.5 J	3 J	4.4 J
P-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
Pyrene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)									
	Layer 2		Layer 3							
	24 ft bgs	34 ft bgs	44 ft bgs	54 ft bgs	64 ft bgs	74 ft bgs	84 ft bgs	94 ft bgs	104 ft bgs	111 ft bgs
SITE Q: AA-Q-8										
VOCs										
1,1,1-Trichloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
1,1,2,2-Tetrachloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
1,1,2-Trichloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
1,1-Dichloroethane	<1	<1	0.52 J	0.57 J	<1	0.22 J	0.32 J	0.43 J	0.35 J	0.23 J
1,1-Dichloroethylene	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
1,2-Dichloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
1,2-Dichloroethene (total)	2.8	0.66 J	16	15	7.4	1.2 J	1.4 J	1.1 J	0.54 J	5.7
1,2-Dichloropropane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
2-Butanone (MEK)	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Methyl-2-pentanone (MIBK)	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Acetone	-	-	-	-	-	<25	<25	<25	<25	<25
Benzene	<1	0.11 J	<1	0.24 J	<1	<1	<1	<1	<1	<1
Bromodichloromethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Bromoform	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Bromomethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Carbon Disulfide	<1	<1	<1	0.88 J	0.36 J	<1	<1	<1	<1	0.42 J
Carbon Tetrachloride	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Chlorobenzene	<1	0.74 J	<1	<1	<1	<1	<1	<1	<1	<1
Chlorodibromomethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Chloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Chloroform	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Chloromethane	<1	<1	<1	<1	<1	<1	<1	<1	0.24 J	0.22 J
cis-1,3-Dichloropropene	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Dichloromethane	<5	<5	<5	<5	<5	<5	<5	<5	<5	<5
Ethylbenzene	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Methyl N-Butyl Ketone	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Styrene (Monomer)	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Tetrachloroethene	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Toluene	<1	<1	0.29 J	0.38 J	<1	<1	<1	<1	<1	<1
trans-1,3-Dichloropropene	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Trichloroethylene	0.17 J	0.22 J	<1	0.18 J	<1	<1	0.41 J	<1	<1	<1
Vinyl chloride	<1	<1	3.8	2.6	0.98 J	<1	<1	<1	<1	1.7
Xylenes, Total	<2	<2	<2	<2	<2	<2	<2	<2	<2	<2

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)													
	Layer 2			Layer 3										
	24 ft bgs	Filtered 24 ft bgs	34 ft bgs	44 ft bgs	54 ft bgs	64 ft bgs	Filtered 64 ft bgs	74 ft bgs	84 ft bgs	94 ft bgs	104 ft bgs	Filtered 104 ft bgs	110 ft bgs	Filtered 110 ft bgs
SVOCs														
1,2,4-Trichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
1,2-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
1,3-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
1,4-Dichlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,2'-Oxybis(1-Chloropropane)	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4,5-Trichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4,6-Trichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4-Dichlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4-Dimethylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,4-Dinitrophenol	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
2,4-Dinitrotoluene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2,6-Dinitrotoluene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Chloronaphthalene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Chlorophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Methylnaphthalene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Methylphenol (o-Cresol)	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
2-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
2-Nitrophenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
3,3'-Dichlorobenzidine	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20
3-Methylphenol/4-Methylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
3-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
4,6-Dinitro-2-methylphenol	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
4-Bromophenyl Phenyl Ether	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Chloro-3-methylphenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Chlorophenyl Phenyl Ether	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
4-Nitrophenol	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
Acenaphthene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Acenaphthylene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Anthracene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(a)anthracene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(a)pyrene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(b)fluoranthene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(g,h,i)perylene	<10	<10	<10	<10	0.84 J	0.77 J	<10	<10	<10	<10	<10	<10	0.81 J	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)													
	Layer 2			Layer 3										
	24 ft bgs	Filtered 24 ft bgs	34 ft bgs	44 ft bgs	54 ft bgs	64 ft bgs	Filtered 64 ft bgs	74 ft bgs	84 ft bgs	94 ft bgs	104 ft bgs	Filtered 104 ft bgs	110 ft bgs	Filtered 110 ft bgs
SVOCs cont'd														
Benzo(k)fluoranthene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Benzyl Butyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
bis(2-Chloroethoxy)methane	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
bis(2-Chloroethyl)ether	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
bis(2-Ethylhexyl)phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Carbazole	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Chrysene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Dibenzo(a,h)anthracene	<10	0.62 J	<10	<10	0.64 J	0.58 J	<10	<10	<10	<10	<10	<10	0.64 J	<10
Dibenzofuran	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Diethyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Dimethyl Phthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Di-n-butylphthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Di-n-octylphthalate	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Fluoranthene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Fluorene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorobutadiene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorocyclopentadiene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Hexachloroethane	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Indeno(1,2,3-cd)pyrene	<10	<10	<10	<10	0.77 J	0.68 J	<10	<10	<10	<10	<10	<10	0.94 J	<10
Isophorone	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Naphthalene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Nitrobenzene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
N-Nitroso-di-n-propylamine	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
N-Nitrosodiphenylamine	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
P-Chloroaniline	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20	<20
Pentachlorophenol	<1	<50	<50	<50	<50	<1	<50	<50	<50	<50	<1	<50	<1	<50
Phenanthrene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Phenol	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
P-Nitroaniline	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50	<50
Pyrene	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)										
	Layer 2			Layer 3							
	24 ft bgs	34 ft bgs	44 ft bgs	54 ft bgs	64 ft bgs	74 ft bgs	84 ft bgs	94 ft bgs	104 ft bgs	114 ft bgs	124 ft bgs
SITE S											
VOCs											
1,1,1-Trichloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
1,1,2,2-Tetrachloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
1,1,2-Trichloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
1,1-Dichloroethane	<1	<1	<1	<1	<1	<1	0.62 J	0.53 J	<1	<1	<1
1,1-Dichloroethylene	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
1,2-Dichloroethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
1,2-Dichloroethene (total)	0.3 J	<2	<2	<2	<1	0.53 J	0.84 J	0.74 J	1.5 J	0.57 J	<2
1,2-Dichloropropane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
2-Butanone (MEK)	<10	<10	<10	11	<10	<10	8.4 J	<10	<10	<10	15
4-Methyl-2-pentanone (MIBK)	<10	<10	<10	<10	<10	<10	<10	<10	<10	1 J	<10
Acetone	<25	<25	<25	<25	-	<25	<25	<25	-	-	-
Benzene	3	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Bromodichloromethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Bromoform	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Bromomethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Carbon Disulfide	<1	<1	<1	<1	<1	<1	<1	<1	0.94 J	<1	0.88 J
Carbon Tetrachloride	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Chlorobenzene	<1	<1.7	<1	0.44 J	<1	4.8	22	27	70	410	78
Chlorodibromomethane	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Chloroethane	<1	3.3	<1	<1	<1	<1	<1	<1	<1	<1	<1
Chloroform	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Chloromethane	<1	0.34 J	<1	0.32 J	<1	<1	<1	<1	<1	<1	<1
cis-1,3-Dichloropropene	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Dichloromethane	<5	<5	<5	<5	0.42 J	<5	<5	<5	<5	<5	<5
Ethylbenzene	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Methyl N-Butyl Ketone	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10	<10
Styrene (Monomer)	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Tetrachloroethene	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Toluene	0.41 J	<1	<1	<1	<1	<1	0.46 J	<1	<1	<1	0.79 J
trans-1,3-Dichloropropene	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1	<1
Trichloroethylene	<1	<1	<1	<1	<1	<1	0.68 J	<1	<1	<1	<1
Vinyl chloride	0.32 J	<1	<1	<1	<1	<1	<1	<1	<1	<1	2.5
Xylenes, Total	<2	<2	<2	<2	<2	<2	<2	<2	<2	<2	<2

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)										
	Layer 2			Layer 3							
	24 ft bgs	34 ft bgs	44 ft bgs	54 ft bgs	64 ft bgs	74 ft bgs	84 ft bgs	94 ft bgs	104 ft bgs	114 ft bgs	124 ft bgs
SVOCs											
1,2,4-Trichlorobenzene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
1,2-Dichlorobenzene	1.5 J	1.4 J	<20	<10	<10	<10	<10	<10	<10	<10	<10
1,3-Dichlorobenzene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
1,4-Dichlorobenzene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
2,2'-Oxybis(1-Chloropropane)	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
2,4,5-Trichlorophenol	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
2,4,6-Trichlorophenol	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
2,4-Dichlorophenol	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
2,4-Dimethylphenol	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
2,4-Dinitrophenol	<50	<50	<100	<50	<50	<50	<50	<50	<50	<50	<50
2,4-Dinitrotoluene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
2,6-Dinitrotoluene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
2-Chloronaphthalene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
2-Chlorophenol	<10	<10	<20	<10	<10	<10	<10	<10	<10	7.1 J	<10
2-Methylnaphthalene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
2-Methylphenol (o-Cresol)	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
2-Nitroaniline	<50	<50	<100	<50	<50	<50	<50	<50	<50	<50	<50
2-Nitrophenol	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
3,3'-Dichlorobenzidine	<20	<20	<40	<20	<20	<20	<20	<20	<20	<20	<20
3-Methylphenol/4-Methylphenol	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
3-Nitroaniline	<50	<50	<100	<50	<50	<50	<50	<50	<50	<50	<50
4,6-Dinitro-2-methylphenol	<50	<50	<100	<50	<50	<50	<50	<50	<50	<50	<50
4-Bromophenyl Phenyl Ether	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
4-Chloro-3-methylphenol	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
4-Chlorophenyl Phenyl Ether	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
4-Nitrophenol	<50	<50	<100	<50	<50	<50	<50	<50	<50	<50	<50
Acenaphthene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Acenaphthylene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Anthracene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(a)anthracene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(a)pyrene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(b)fluoranthene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(g,h,i)perylene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Benzo(k)fluoranthene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Benzyl Butyl Phthalate	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10

GROUNDWATER SERVICES, INC.

TABLE 2 CONT'D
SUMMARY OF SITE GROUNDWATER CONCENTRATIONS BY DEPTH

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Chemical Name	Concentration (ug/L)										
	Layer 2			Layer 3							
	24 ft bgs	34 ft bgs	44 ft bgs	54 ft bgs	64 ft bgs	74 ft bgs	84 ft bgs	94 ft bgs	104 ft bgs	114 ft bgs	124 ft bgs
SVOCs cont'd											
bis(2-Chloroethoxy)methane	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
bis(2-Chloroethyl)ether	1.8 J	1.4 J	<20	<10	<10	<10	<10	<10	<10	<10	<10
bis(2-Ethylhexyl)phthalate	<10	<10	<20	3.5 J	<10	<10	<10	<10	<10	<10	<10
Carbazole	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Chrysene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Dibenzo(a,h)anthracene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Dibenzofuran	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Diethyl Phthalate	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Dimethyl Phthalate	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Di-n-butylphthalate	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Di-n-octylphthalate	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Fluoranthene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Fluorene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorobenzene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorobutadiene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Hexachlorocyclopentadiene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Hexachloroethane	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Indeno(1,2,3-cd)pyrene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Isophorone	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Naphthalene	<10	<10	<20	<10	<10	<10	<10	<10	<10	3.3 J	<10
Nitrobenzene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
N-Nitroso-di-n-propylamine	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
N-Nitrosodiphenylamine	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
P-Chloroaniline	<20	<20	<40	<20	<20	<20	<20	<20	<20	<20	<20
Pentachlorophenol	<1	<50	<100	<50	<1	<50	<50	<50	<1	<50	<1
Phenanthrene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
Phenol	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10
P-Nitroaniline	<50	<50	<100	<50	<50	<50	<50	<50	<50	<50	<50
Pyrene	<10	<10	<20	<10	<10	<10	<10	<10	<10	<10	<10

Notes:
1. Layer is the MODFLOW model layer. Layer 2 corresponds to the Middle Hydrogeologic Unit while Layer 3 corresponds to the Lower Hydrogeologic Unit.
ft bgs = feet below ground surface
ug/L = microgram per liter
VOC = volatile organic compound
SVOC = semivolatile organic compound
J = estimated value
< = constituent analyzed, but not detected at the detection limit shown
- = constituent not analyzed

GSI Job No. G-2839
Issued: 11/20/03

PRELIMINARY

GROUNDWATER SERVICES, INC.

TABLE 3
CALCULATION OF SITE R CONCENTRATION AND MASS FLUX

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

	Zone									
	1	2	3	4	5	6	7	8	9	10
Middle Hydrogeologic Unit										
VOC Concentration (mg/L)	12	17	22	26	26	22	16	8	5	2
SVOC Concentration (mg/L)	135	317	354	489	728	461	325	361	479	271
VOC + SVOC Concentration (mg/L)	147	334	376	515	754	483	341	369	484	273
Flowrate in Zone (gpm)	17	18	18	21	17	19	18	17	21	25
Mass Flux (kg/yr)	4,936	11,665	13,572	21,362	25,905	18,636	11,938	12,305	20,625	13,315
									Mass Flux for Middle Hydrogeologic Unit (kg/yr)	**154,260**
									Average Concentration (VOC+SVOC) for Middle Hydrogeologic Unit (mg/L)	**408**
Lower Hydrogeologic Unit										
VOC Concentration (mg/L)	2	1	1	1	1	1	1	0	0	0
SVOC Concentration (mg/L)	47	128	209	196	88	46	69	84	52	14
VOC + SVOC Concentration (mg/L)	49	129	210	197	89	47	70	84	52	14
Flowrate in Zone (gpm)	30	31	32	31	34	31	30	38	43	38
Mass Flux (kg/yr)	2,906	7,925	13,429	11,920	6,018	2,920	4,132	6,397	4,489	1,071
									Mass Flux for Lower Hydrogeologic Unit (kg/yr)	**61,207**
									Average Concentration (VOC+SVOC) for Lower Hydrogeologic Unit (mg/L)	**94**
									TOTAL MASS FLUX MIDDLE AND LOWER HYDROGEOLOGIC UNITS (kg/yr)	**215,466**

NOTES:
1. Flow obtained from MODFLOW Zone Budget.
2. See text for method of concentration calculation.
3. Average river stage of 391 ft.

mg/L = milligram per liter
kg/yr = kilogram per year
gpm = gallons per minute
VOC = volatile organic compound
SVOC = semivolatile organic compound

GROUNDWATER SERVICES, INC.

TABLE 4
ESTIMATION OF SOURCE MASS

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Site		Average Concentration (mg/L)		Thickness (ft)		Width (ft)		Length (ft)		Residual Saturation (-)		Fraction Aquifer With NAPL (-)		Porosity (-)		NAPL Density (kg/L)		Mass COCs (kg)	
		MHU	LHU	MHU	LHU	MHU	LHU	MHU	LHU	MHU	LHU	MHU	LHU	MHU	LHU	MHU	LHU	MHU	LHU
Site O		0.004	0.086	30	71	1491	1491	887	887	0.05	0.05	2.2E-6	4.3E-5	0.35	0.35	1.25	1.25	56	2,494
Site P		0.012	0.250	30	72	3934	3934	528	528	0.05	0.05	5.9E-6	1.3E-4	0.35	0.35	1.25	1.25	227	11,550
Site Q	AA-Q-1	-	0.099	30	62	1453	1453	1170	1170	0.05	0.05	-	4.9E-5	0.35	0.35	1.25	1.25	-	3,205
	AA-Q-4	-	0.007	30	59	1868	1868	1302	1302	0.05	0.05	-	3.5E-6	0.35	0.35	1.25	1.25	-	309
	AA-Q-6	0.453	0.038	30	65	1443	1443	698	698	0.05	0.05	2.3E-4	1.9E-5	0.35	0.35	1.25	1.25	4,188	780
	AA-Q-8	0.001	0.002	28	58	1925	1925	1981	1981	0.05	0.05	3.8E-7	9.3E-7	0.35	0.35	1.25	1.25	25	127
Site R		408	94	30	69	2000	2000	540	540	0.05	0.05	2.0E-1	4.7E-2	0.35	0.35	1.25	1.25	4,129,341	2,180,580
Site S		0.001	0.023	30	69	349	349	89	89	0.05	0.05	6.9E-7	1.2E-5	0.35	0.35	1.25	1.25	0	15

Notes:
1. Concentrations based on location AA-O-1 for Ssite O, AA-P-1 for Site P, and AA-S-1 for Site S.
2. Hydrogeologic unit thickness and hydraulic gradient based on MODFLOW model.

mg/L = Milligram per liter
ft = Feet
cm/sec = Centimeter per second
ft/yr = Feet per year
kg/yr = Kilograms per year
gpm = Gallons per minute
MHU = Middle hydrogeologic unit
LHU = Lower hydrogeologic unit

GROUNDWATER SERVICES, INC.

TABLE 5
CALCULATION OF MASS FLUX TO RIVER

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Site		Average Concentration (mg/L)		Thickness (ft)		Width (ft)		K (cm/sec)		I (ft/ft)		Darcy Velocity (ft/yr)		Flow (gpm)		Mass Flux (kg/yr)	
		MHU	LHU	MHU	LHU	MHU	LHU	MHU	LHU	MHU	LHU	MHU	LHU	MHU	LHU	MHU	LHU
Site O		0.004	0.086	30	71	1491	1491	0.15	0.15	0.0010	0.0010	158	158	102	238	9.1E-1	4.1E+1
Site P		0.012	0.250	30	72	3934	3934	0.14	0.14	0.0010	0.0010	144	144	244	581	5.7E+0	2.9E+2
Site Q	AA-Q-1	-	0.099	30	62	1453	1453	0.14	0.14	0.0015	0.0015	211	211	132	270	-	5.3E+1
	AA-Q-4	-	0.007	30	59	1868	1868	0.11	0.11	0.0015	0.0015	168	168	134	266	-	3.7E+0
	AA-Q-6	0.453	0.038	30	65	1443	1443	0.11	0.11	0.0015	0.0015	175	175	107	235	9.6E+1	1.8E+1
	AA-Q-8	0.001	0.002	28	58	1925	1925	0.06	0.06	0.0017	0.0017	102	102	80	163	1.2E-1	6.0E-1
Site S		0.001	0.023	30	69	349	349	0.15	0.15	0.0012	0.0012	187	187	28	65	7.8E-2	3.0E+0

Notes:
1. Concentrations based on location AA-O-1 for Site O, AA-P-1 for Site P, and AA-S-1 for Site S.
2. Hydrogeologic unit thickness and hydraulic gradient based on MODFLOW model.
3. Assuming uniform hydraulic conductivity over entire site.
4. See Table 3 for Site R mass flux data.

mg/L = Milligram per liter
ft = Feet
cm/sec = Centimeter per second
ft/yr = Feet per year
kg/yr = Kilograms per year
gpm = Gallons per minute
MHU = Middle hydrogeologic unit
LHU = Lower hydrogeologic unit

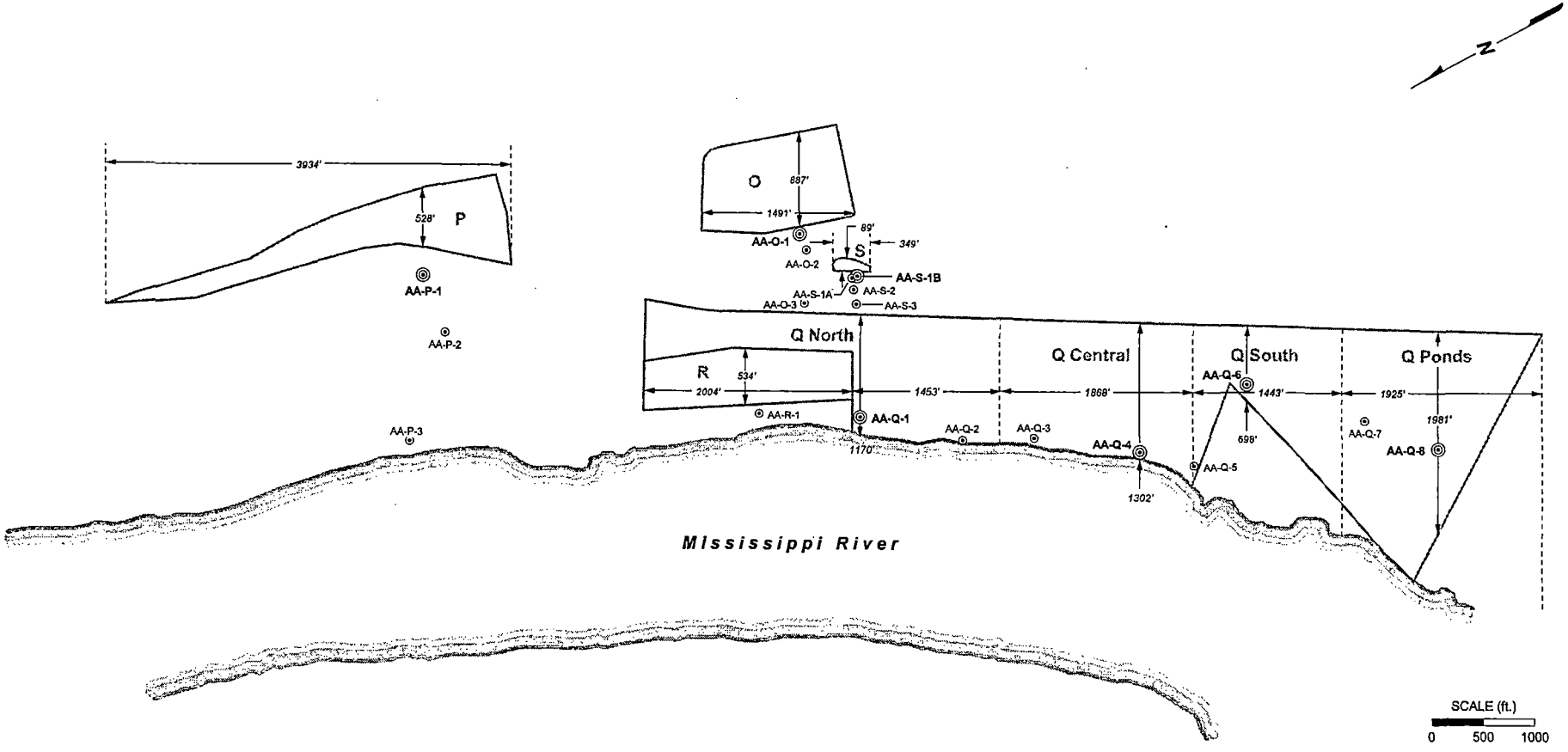
GROUNDWATER SERVICES, INC.

TABLE 6
REMEDIATION TIMEFRAME FOR FIVE ALTERNATIVES

Sauget Area 2 Focused Feasibility Study
Sauget Area 2, Sauget, Illinois

Site		Hydrogeologic Unit	Initial Concentration (mg/L)	Source Mass (kg)	Mass Flux (kg/yr)	k_s Natural (per year)	Remediation Timeframe for Alternatives 1-4 (years)	k_s Pumping (per year)	Remediation Timeframe for Alternative 5 (years)
Site O		MHU	0.004	5.6E+1	9.1E-1	1.6E-2	0	4.1E-2	0
		LHU	0.086	2.5E+3	4.1E+1	1.6E-2	174	4.1E-2	70
Site P		MHU	0.012	2.3E+2	5.7E+0	2.5E-2	34	6.3E-2	14
		LHU	0.250	1.2E+4	2.9E+2	2.5E-2	156	6.3E-2	63
Site Q	AA-Q-1	MHU	-	-	-	-	-	-	-
		LHU	0.099	3.2E+3	5.3E+1	1.7E-2	180	4.1E-2	72
	AA-Q-4	MHU	-	-	-	-	-	-	-
		LHU	0.007	3.1E+2	3.7E+0	1.2E-2	28	3.0E-2	11
	AA-Q-6	MHU	0.453	4.2E+3	9.6E+1	2.3E-2	196	5.7E-2	78
		LHU	0.038	7.8E+2	1.8E+1	2.3E-2	89	5.7E-2	35
	AA-Q-8	MHU	0.001	2.5E+1	1.2E-1	4.7E-3	0	1.2E-2	0
		LHU	0.002	1.3E+2	6.0E-1	4.7E-3	0	1.2E-2	0
Site R		MHU	407.671	4.1E+6	1.5E+5	3.7E-2	303	9.3E-2	121
		LHU	94.140	2.2E+6	6.1E+4	2.8E-2	351	7.0E-2	140
Site S		MHU	0.001	4.0E-1	7.8E-2	1.9E-1	0	4.9E-1	0
		LHU	0.023	1.5E+1	3.0E+0	1.9E-1	8	4.9E-1	3

NOTES:
1. Alternative 1 represents no action; Alternative 2 represents institutional controls and monitoring; Alternative 3 represents a physical barrier to Site R only; Alternative 4 represents a physical barrier adjacent to the western side of Sauget Area 2; and Alternative 5 represents a hydraulic control adjacent to the western side of Sauget Area 2.
   mg/L = Milligram per liter
   kg/yr = Kilogram per year
   MHU = Middle hydrogeologic unit
   LHU = Lower hydrogeologic unit



LEGEND

- Site-related groundwater sampling locations
- Sampling location used for remediation timeframe estimate
- Site boundaries

Notes:

1) Data from Area 1 background locations will also be considered as background for Area 2.
2) Map data taken from URS Base Map, 3/27/03, Job No. 2320010024.02

GROUNDWATER SERVICES, INC.

SITE LOCATION MAP

Sauget Area 2 FFS
Sauget Area 2, Sauget, Illinois

GSI Job No:	G-2839	Drawn By:	DLB
Issued:	11/20/03	Ch'k'd By:	SKF
Revised:	———	App'v'd By:	SKF
Scale:	As Shown		FIGURE 1

APPENDIX N

City Ordinances

ORDINANCE NO. 99-5

AN ORDINANCE PROHIBITING THE USE OF GROUNDWATER AS A POTABLE WATER SUPPLY BY THE INSTALLATION OR USE OF POTABLE WATER SUPPLY WELLS OR BY ANY OTHER METHOD

WHEREAS, certain properties in the Village of Sauget, Illinois, have been used over a period of time for commercial/industrial purposes; and

WHEREAS, because of said use, concentrations of certain chemical constituents in the groundwater beneath the Village may exceed Class I groundwater quality standards for potable resource groundwater, as set forth in 35 Illinois Administrative Code Part 620, or Tier 1 residential remediation objectives, as set forth in 35 Ill. Adm. Code Part 742; and

WHEREAS, the Village of Sauget desires to limit potential threats to human health from groundwater contamination while facilitating the redevelopment and productive use of properties that are the source of said chemical constituents;

NOW, THEREFORE, BE IT ORDAINED BY THE VILLAGE COUNCIL IN THE VILLAGE OF SAUGET, ILLINOIS:

Section One: Use of groundwater as a potable water supply prohibited.

The use or attempted use of groundwater from within the corporate limits of the Village as a potable water supply by the installation or drilling of wells or by any other method is hereby prohibited.

Section Two: Penalties

Any person violating the provisions of this ordinance shall be subject to a fine of up to $100.00 for each violation.

Section Three: Definitions.

"Person" is any individual, partnership, co-partnership, firm, company, limited liability company, corporation, association, joint stock company, trust, estate, political subdivision, or any other legal entity, or their representatives, agents or assigns.

"Potable water" is any water used for human or domestic consumption, including, but not limited to, water used for drinking, bathing, swimming, washing dishes, or preparing foods.

Section Four: Repealer.

All ordinances or parts of ordinances in conflict with this ordinance are hereby repealed insofar as they are in conflict with this ordinance.

Section Five: Severablity.

If any provision of this ordinance or its application to any person or under any circumstances is adjudged invalid, such adjudication shall not affect the validity of the ordinance as a whole or of any portion not adjudged invalid.

Section Six: Effective Date.

This ordinance shall be in full force and effect from and after its passage, approval and publication, as required by law.

INTRODUCED AND READ FOR THE FIRST TIME: October 12, 1999

READ FOR THE SECOND TIME:
(under suspension of rules): October 12, 1999

READ FOR THE THIRD TIME:
(under suspension of rules): October 12, 1999

ADOPTED AND ENACTED: October 12, 1999

ROLL CALL VOTE:
Ayes: Adele, McDaniel, Rich, Cates, Thornton, Sengel
Nays: NONE
Absent: NONE
Unfilled Vacancy:

APPROVED: October 12, 1999

APPROVED:

President (mayor) Pro Tempore

ATTEST:

Village Clerk

ORDINACE No. 981

AN ORDINANCE PROHIBITING THE USE OF GROUNDWATER AS A POTABLE WATER SUPPLY BY THE INSTALLATION OR USE OF POTABLE WATER SUPPLY WELLS OR BY ANY OTHER METHOD

WHEREAS, certain properties in the Village of Cahokia, Illinois, have been used over a period of time for commercial/industrial uses; and

WHEREAS, because of said use, concentrations of certain chemical constituents in the groundwater beneath the Village may exceed Class I groundwater quality standards for potable resource groundwater, as set forth in 35 Administrative Code Part 620, or Tier 1 residential remediation objectives, as set forth in 35 Ill. Admin. Code Part 742; and

WHEREAS, the Village of Cahokia desires to limit potential threats to human health from groundwater contamination while facilitating the redevelopment and productive use of properties that are the source of said chemical constituents;

NOW, THEREFORE, BE IT ORDAINED BY THE VILLAGE BOARD IN THE VILLAGE OF CAHOKIA, ILLINOIS:

Section One: Use of groundwater as a potable water supply prohibited.

The use or attempted use of groundwater from within the corporate limits of the Village as a potable water supply by the installation or drilling of wells or by any other method is hereby prohibited.

Section Two: Penalties.

Any person violating the provisions of this ordinance shall be subject to a fine of up to $1,000.00 for each violation.

Section Three: Definitions.

"Person" is any individual, partnership, co-partnership, firm, company, limited liability company, corporation, association, joint stock company, trust, estate, political subdivision, or any other legal entity, or their representatives, agents or assigns.

"Potable water" is any water used for human or domestic consumption, including, but not limited to, water used for drinking, bathing, swimming, washing dishes, garden or lawn watering, or preparing foods..

Section Four: Repealer.

All ordinances or parts of ordinances in conflict with this ordinance are hereby repealed insofar as they are in conflict with this ordinance.

Section Five: Severability.

If any provision of this ordinance or its application to any person or under any circumstances is adjudged invalid, such adjudication shall not affect the validity of the ordinance as a whole or of any portion not adjudged invalid.

Section six: Effective Date.

This ordinance shall be in full force and effect from and after its passage, approval and publication, as required by law.

ADOPTED: 6-06-2000
(Date)

Xavier Brown
(Village Clerk)

ADOPTED: 6-06-2000
(Date)

Michael King
(Mayor)

Officially published this 21st day of June, 2000.

STATE OF ILLINOIS)
)
COUNTY OF ST. CLAIR) SS
)
VILLAGE OF CAHOKIA)

CERTIFICATE OF VILLAGE CLERK

I, Jessie Brown, Clerk of the Village of Cahokia, St. Clair County, Illinois, do hereby certify that as such Village Clerk of the Village of Cahokia, Illinois, I am legal custodian and keeper of the journal of the proceedings of the Village of Cahokia Board of Trustees of said Village, and I do certify that the attached documents are true and faithful copies of said documents. I do further certify that the original of said documents are now remaining on file and of record in my said office.

IN WITNESS WHEREOF, I have hereunto set my hand and affixed the seal of the Village of Cahokia, Illinois, the 6 day of June, A.D. ~~29~~ 2000.

Jessie Brown
Jessie Brown, Village Clerk
Village of Cahokia, Illinois

(SEAL)