Hydrogeology of the Scioto River Valley Near Piketon, South-Central Ohio

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1872

Prepared in cooperation with the Ohio Department of Natural Resources, Division of Water, and the U.S. Atomic Energy Commission

HYDROGEOLOGY OF

THE SCIOTO RIVER VALLEY

NEAR PIKETON, SOUTH-CENTRAL OHIO

Aerial photographic view of Piketon and aquifer test-site area.

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By STANLEY E. NORRIS *and* RICHARD E. FIDLER

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A quantitative study of ground-water yield and induced infiltration in a glacial outwash aquifer

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HYDROGEOLOGY OF THE SCIOTO RIVER VALLEY NEAR PIKETON, SOUTH-CENTRAL OHIO

By STANLEY E. NORRIS and RICHARD E. FIDLER

ABSTRACT

A systematic study was made of one of Ohio's principal aquifers, a sand and gravel outwash in the Scioto River Valley, to determine the feasibility of developing a ground-water supply of 20 million gallons per day at a site near Piketon.

The first part of the study was spent in determining the thickness and physical properties of the sand and gravel aquifer and in drilling test wells to determine the best site for the supply wells.

The second part of the investigation was an aquifer infiltration test to determine the hyraulic properties of the aquifer and the conditions of stream recharge. A well 83 feet deep was drilled on the flood plain and was pumped for 9 days at the rate of 1,000 gallons per minute. The effect on the hydrologic system during and after the pumping was determined by measuring the water levels in an array of deep and shallow observation wells and in 8 drive-point wells installed in the bed of the river. Seldom have more comprehensive data been collected showing the effects of pumping on a natural, unconfined, hydrologic system. From these data were calculated the coefficient of transmissibility (215,000 gallons per day per foot) and the rate of streambed infiltration (0.235 million gallons per day per acre per foot).

The aquifer was tested near the end of a long drought; so the ground-water levels and the river stage were very nearly following a level trend. Because the ground-water levels were essentially unaffected by extraneous influences, the test data are probably as precise and uncomplicated as is practical to obtain in the field. These data proved to be valid for use as design criteria for the location, spacing, and construction of four supply wells.

The third part of the investigation was the testing and quantitative evaluation of the four supply wells before they were put into service. The wells were found to perform about as predicted, indicating that the hydraulic properties of the aquifer, as determined by standard methods, are fairly representative.

INTRODUCTION

PURPOSE AND SCOPE

In 1963–65 the U.S. Geological Survey, in collaboration with the Division of Water of the Ohio Department of Natural Resources, investigated on behalf of the U.S. Atomic Energy Commission the feasibility of developing a ground-water supply of 20 mgd (million gallons per day) at a site near Piketon, in the Scioto River valley in southern Ohio (fig. 1). The water was required for use at AEC's

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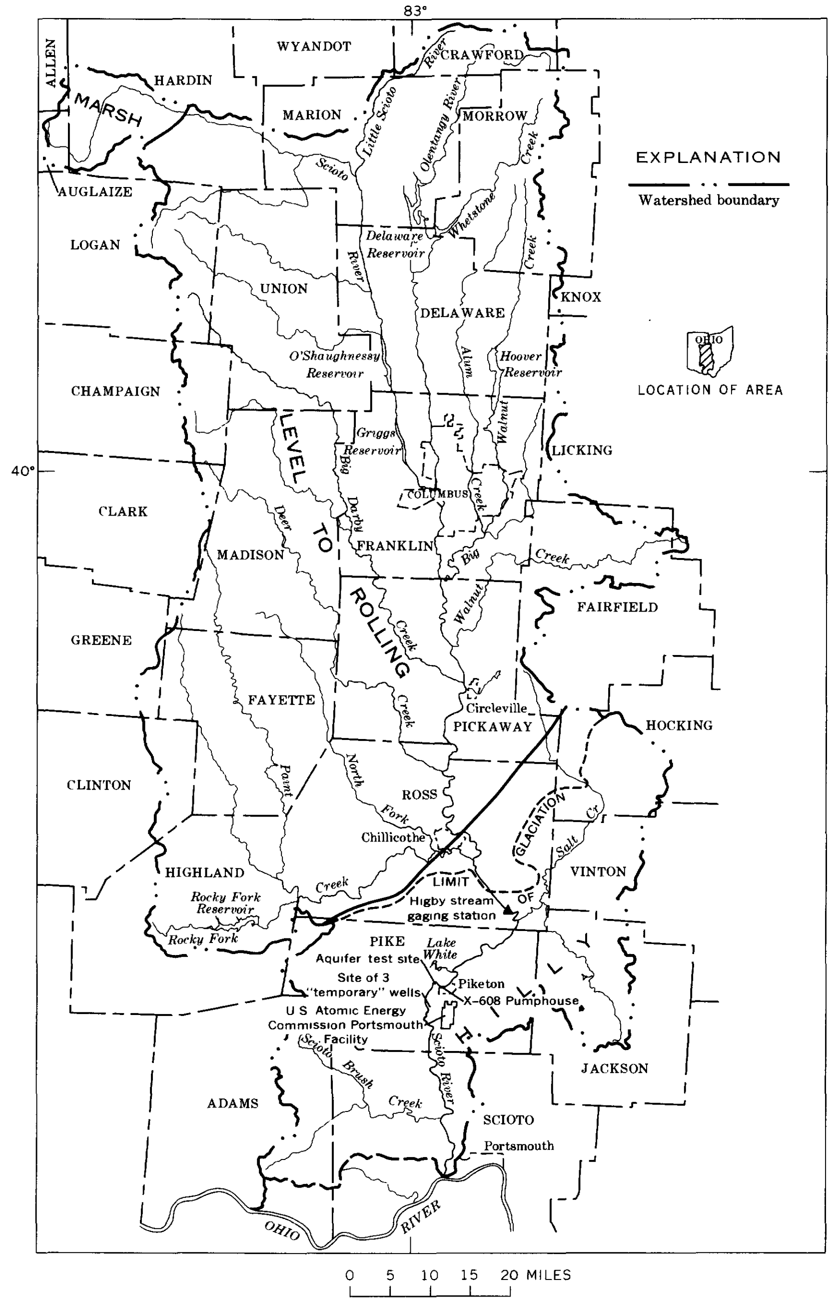

FIGURE 1.—Scioto River basin and pertinent features in the Piketon area.

Portsmouth facility, one of three gaseous diffusion plants in the United States which enrich uranium with the uranium-235 isotope. The plant is operated under contract by the Goodyear Atomic Corp.

The investigation was in three parts. The first part was test drilling in the spring and summer of 1963 to determine the thickness and the physical properties of the sand and gravel aquifer and to select the best site for drilling the supply wells. The second, and most significant part of the investigation, was an aquifer-infiltration test of 9-days duration, made in October 1963, to determine the hydraulic properties of the aquifer and conditions of stream recharge. The results of the aquifer-infiltration test were used to develop the design criteria which became the basis for the location, spacing, and construction of the four supply wells. The third part of the investigation, in 1965, involved the testing and evaluation of the supply wells before they were placed in service.

The chief purpose of this report is to summarize the findings of the investigation, for they relate to the water-yielding properties of one of the State's most important watercourse aquifer systems. These findings, made at a fairly typical site, have important transfer value and should be of aid in the planning and development of water-supply projects elsewhere in the Scioto River valley and in other parts of the State.

Sand and gravel deposits similar to those in the Scioto River valley occur in nearly all the major valleys in Ohio as well as in major valleys throughout the glaciated region. Approximately three-fourths of the ground water used in Ohio is pumped from these watercourse aquifer systems. Because of the importance of these aquifers and the growing need for evaluating them quantitatively, the analytical techniques used in determining the aquifer constants and the rate of stream infiltration at Piketon are described in detail.

Three short papers describing special technical aspects of the investigation were prepared for professional journals. Two of the papers (Norris and Fidler, 1965, 1966a) describe results of particle-size analyses; the third (Norris and Fidler, 1966b) describes application of a method for determining the vertical permeability of the aquifer. Only the significant findings reported in these journal articles are repeated here. The reader is referred to the respective papers for more complete data and discussion of the analytical methods.

The basic data collected during the aquifer test are available in tabular form as an open-file release of the Geological Survey (Norris and Fidler, 1967). It is expected that these data will be of value as training material to colleges and universities offering courses in hydrology and may also prove useful to hydrologists in developing and testing new methods of analysis.

The aquifer test was unusually well instrumented and was made near the end of a long drought in which ground-water levels and the river stage were following very nearly a level trend. Essentially unaffected by extraneous influences, such as atmospheric pressure changes or uncontrolled pumping, the test data probably are as precise and uncomplicated as are practical to obtain in the field. The time-drawdown and distance-drawdown plots classically illustrate the effects of pumping in an essentially unconfined aquifer hydraulically connected to a surface stream. Seldom have more comprehensive data been collected showing the effects of pumping on a natural, unconfined, hydrologic system.

THE PROBLEM

From 1953, when the Atomic Energy Commission plant was built, until a ground-water supply was developed in 1965, water for industrial processing—chiefly makeup water used for cooling—was taken directly from the Scioto River through an intake facility on Piketon's northwest side, referred to as the X–608 pumphouse. (See frontispiece). From the pumphouse, water (which now comes from the wells) is piped to the plant, 4 miles southeast of Piketon, where it is treated and used in cooling towers to replace water lost by evaporation.

Before the plant was built, water requirements were estimated at 40 mgd. Use of river water was originally decided upon because of the anticipated high cost of the large number of wells thought necessary. Economies in water-use practices, however, reduced the process water requirements to about half the original estimate, or to about 20 mgd. In 1963, use of river water ranged between 15 and 18 mgd. A subsequent reduction in plant output further reduced water requirements, and water use in 1965 was about 10 mgd.

Although river water was plentiful, the deterioration in quality of the water in the years preceding the investigation was a matter of increasing concern to plant engineers because of rising costs of treatment. The problem of quality was the result of increased population and industrial growth in upstream cities; the cities were contributing more sewage and industrial pollution to the stream, and each major drought was intensifying such pollution. Furthermore, the concentration of suspended solids, as well as minerals and organic pollutants, depends upon the degree of dilution afforded by the streamflow and varies within a wide range; thus, the engineers began to seek a source of water less affected by surface conditions.

Because of the relatively good quality of the ground water available at Piketon, water treatment engineers concluded that a substantial amount in treatment cost could be saved by changing to a ground-water supply. The engineers were aware that ground water derived from the induced infiltration of streamflow would gradually approach the surface water in certain of its chemical properties; however, the cost of treating the ground water would remain relatively low, primarily because ground water is free of suspended solids and is not subject to the rapid change and extreme variability in quality that characterize the river water at Piketon.

In March 1963, Mr. R. V. Anderson, Manager, Portsmouth Area, Atomic Energy Commission, requested the aid of the Geological Survey in determining the feasibility of developing a 20 mgd ground-water supply at the Portsmouth facility. A review of existing hydrologic data indicated the probability that the required quantity of ground water could be obtained, but also that selection of drilling sites and development of well spacing and design criteria would require additional test drilling and an aquifer test. These conclusions were based on data from an aquifer test of 2½ days duration made in 1953 by the Geological Survey,[1] on the records of 10 test borings made in 1953 by the U.S. Army Corps of Engineers in the vicinity of the X–608 pumphouse, and on records of scattered wells and test holes in the files of the Ohio Division of Water.

THE VARIABLES

It was recognized that design of a ground-water system at Piketon, using the Scioto River as a source of recharge, would require collection of additional hydrogeologic data, including more information on the depth and width of the bedrock valley in which the sand and gravel deposits occur. It was important to determine whether the character of the deposits varied significantly from place to place and whether the sand and gravel deposits were interbedded with extensive beds of clay or silt which might reduce the transmissibility of the aquifer or make it difficult to construct wells at a given site.

A well-discharge test to determine the transmission characteristics of the aquifer and to ascertain whether there is a good hydraulic connection between the river and the aquifer also was deemed essential. The objective was to test the full thickness of the aquifer at a site where hydrologic conditions were representative.

Implicit in the investigation was the assumption that the Scioto River, normally an effluent stream, would be the principal source of recharge to the aquifer under pumping conditions. The unit rate of streambed infiltration per foot of head in periods of low flow, together with data on the width and depth of the stream, was required, as this unit rate, together with the other data, would determine the area of streambed required for development.

[1] The 1953 aquifer test was made using three wells drilled on the flood plain at a site 3 miles down the valley from the X–608 pumphouse. These "temporary" wells, drilled to supply water for construction purposes, are still in use, supplying about 2 mgd for sanitary and domestic purposes at the plant.

ACKNOWLEDGMENTS

The authors are indebted to R. W. Stallman, research engineer of the U.S. Geological Survey, who advised on all phases of the aquifer test, including layout of the aquifer test-well pattern, techniques of analysis, and interpretation of results. Our appreciation is extended also to the late Edward J. Schaefer [2], engineer of the Ohio Division of Water, for advice and suggestions relative to analysis of the test data, and especially for the development, with the help of Paul Kaser, of graphs which are presented herein for rapid solution of certain equations used in the analysis.

The authors acknowledge with thanks the assistance of Norman Bailey, Alfred C. Walker, and James J. Schmidt, geologists of the Ohio Division of Water, who augered test holes and installed drive-point wells in the area of investigation. Appreciated also is the help of Robert E. Shepherd, project engineer of the Goodyear Atomic Corp., who participated in all phases of the test drilling and aquifer-test programs. The thorough work of the subcontractor, Everett G. Hamm, Columbus, Ohio, in striving to collect the maximum of hydrologic data when drilling the aquifer-test pattern also is appreciated. For assistance in collecting field data, the help of personnel of G. M. Baker and Son, Inc., the contractor who installed the four production wells, is also appreciated. Finally, the authors are indebted to John C. Krolczyk and Jack N. Pennell of the Ohio Division of Water for drafting many of the illustrations in this report.

THE HYDROLOGIC SYSTEM

STREAM DISCHARGE CHARACTERISTICS

The Scioto River rises in north-central Ohio, flows south through Columbus, and enters the Ohio River at Portsmouth, about 25 miles south of Piketon. There are 6,510 square miles in the drainage basin, which is the third largest in the State. (See fig. 1.)

The U.S. Geological Survey operates a gaging station at Higby, 13 miles above Piketon. The drainage area above the gage is 5,129 square miles. Pertinent discharge characteristics at the Higby station are listed as follows (Cross and Hedges, 1959, p. 128):

Minimum daily discharge—October 1930 to July 1936, March 1937 to September 1955: 244 cfs (cubic feet per second) (154 mgd), October 23, 1930.

Mean discharge (23 years)—1931–35, 1938–55: 4,189 cfs (2,700 mgd); 0.817 cfs per sq mi; 11.09 in.

[2] Formerly district engineer, Ground Water Branch, U.S. Geological Survey, Columbus, Ohio.

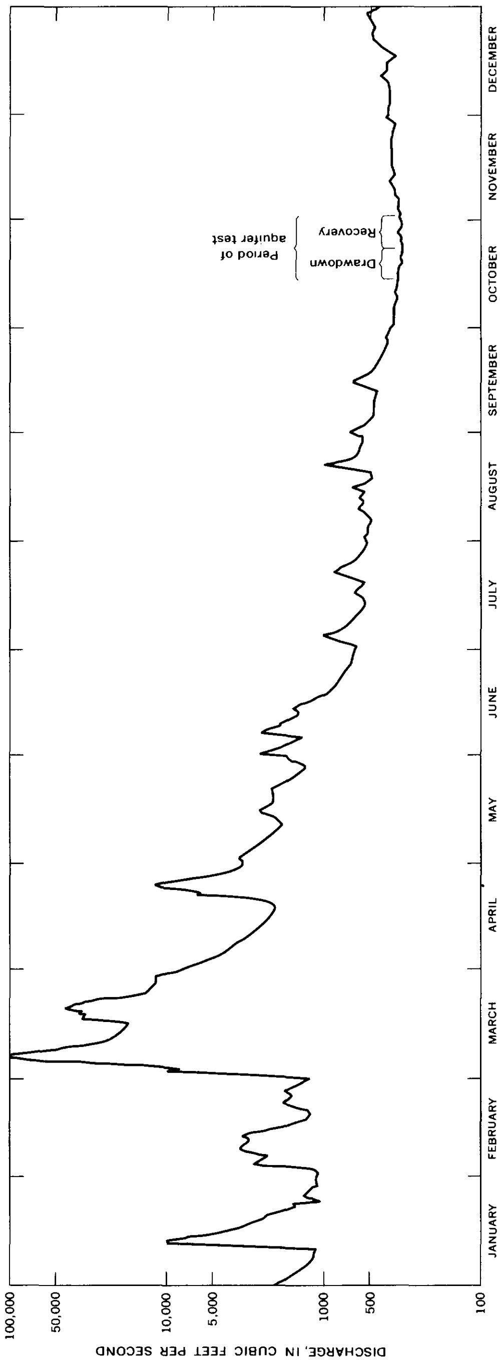

FIGURE 2.—Mean daily discharge of Scioto River at Higby, 1963.

Maximum recorded discharge: 112,000 cfs (72,200 mgd), April 21, 1940.

Discharge equaled or exceeded—1931–35, 1938–55:

95 percent of time, 333 cfs (215 mgd);

90 percent of time, 390 cfs (252 mgd);

75 percent of time, 610 cfs (393 mgd);

50 percent of time, 1,460 cfs (940 mgd).

Figure 2 is a hydrograph showing mean daily discharge at the Higby station in 1963.

GEOMORPHIC DEVELOPMENT

Piketon is about 20 miles beyond the southern limit of glaciation in a region of rugged, hilly terrain. The area is underlain by a thick sequence of sedimentary rocks of Devonian and Mississippian age, consisting predominately of shale interbedded with thinly bedded to massive sandstone units. The Scioto River winds through this scenic, sparsely settled region in a wide, flat-bottomed, and steep-sided valley, 400–500 feet below the general level of the higher adjacent hills. At Piketon the valley is about 1½ miles wide.

The Scioto River valley predates the modern river, having been developed by streams of former drainage systems. The valley was established in preglacial time by a short, north-flowing tributary of the Teays River, which joined the main stem at a point near Waverly, 5 miles north of Piketon at altitude 640 feet (Stout and others, 1943, p. 51–53). With disruption of the Teays River system by the advance of an early glacier, the former tributary valley became part of a major south-flowing stream called the Newark River (fig. 3). The Newark River widened the valley and had deepened it to about 500 feet altitude when a change in regional drainage conditions caused the stream to increase its rate of downcutting. The rejuvenated stream cut a relatively straight, narrow channel, about half a mile wide and as much as 35 feet deeper into the bedrock, to a minimum altitude of about 465 feet.

The Newark River system ended when its valleys and those of other major south-draining streams were partly filled with sand and gravel outwash. The valley at Piketon was filled to altitudes of 560–580 feet. The modern Scioto River, which came into existence during the final retreat of the glacier, has removed 20–30 feet of the outwash; the terrace-like remnants left on the sides of the valley mark the original level of the glacial deposits. The village of Piketon is built above the present flood plain on one of these sand and gravel terraces.

The Scioto River meanders in irregular sweeping curves across the broad valley. The average radius of curvature of the meanders of the modern river is very much less than the average radius of curvature

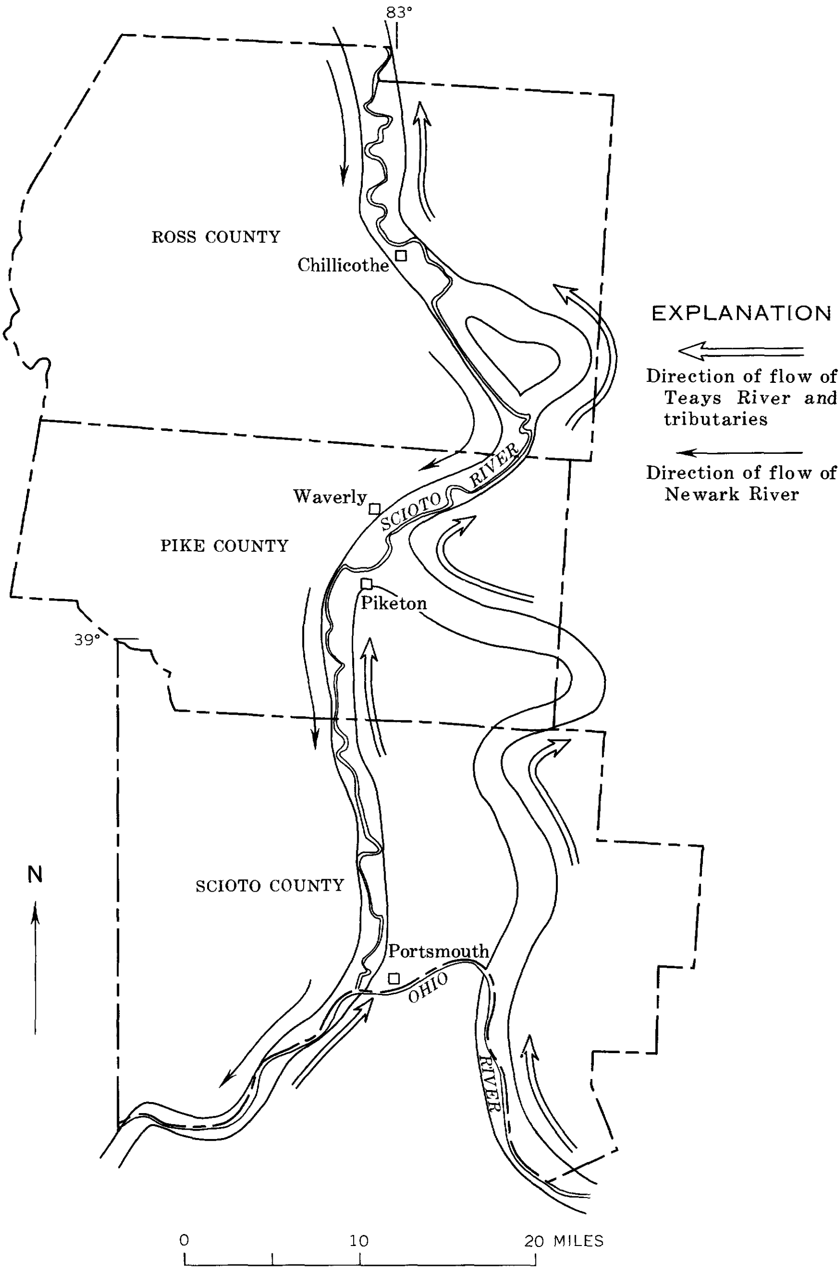

FIGURE 3.—Teays stage and post-Teays stage (Newark River) valleys in southern Ohio (from Stout and others, 1943).

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of the meanders of the bedrock valley, a condition characteristic of an underfit stream (Thornbury, 1954, p. 156).

In times of flood the river strives to lengthen its meander loops and once in 1–2 years, on the average, overflows its banks and inundates the flood plain. A severe flood in March 1963 produced a peak discharge of 105,000 cfs (Cross, 1964, table 5).

OCCURRENCE OF GROUND WATER

Water moves from the thinly covered bedrock uplands into the Scioto River valley chiefly as surface runoff; the ground-water contribution from the consolidated rocks is relatively small. Ground water in the outwash deposits, received both from upland runoff and the deep percolation of direct precipitation, moves under low gradients to discharge into the river. Figure 4 is a map of the Piketon area showing the surface of the water table, based on depth to water measured in wells on July 12, 1963. Natural ground-water gradients are estimated to range between 5 and 10 feet per mile, most of the time, and the velocity of ground-water flow is on the order of 1–2 feet per day.

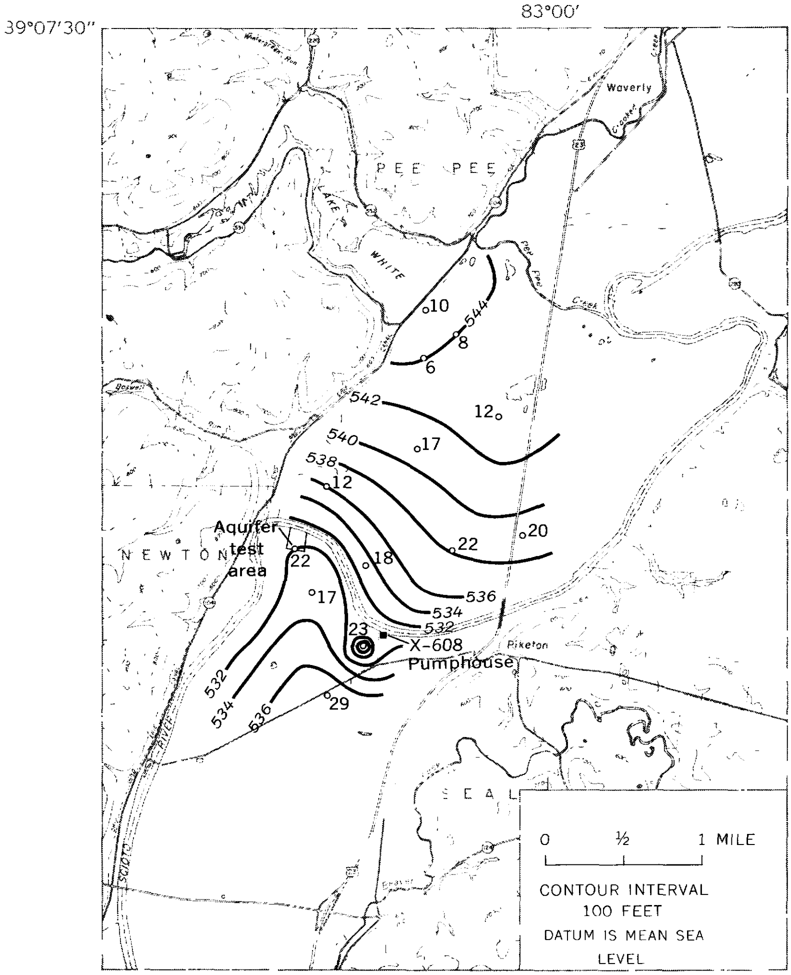
Two weeks before the start of the aquifer test, at a time of extremely low flow, the discharge of ground water into the river was at the rate of approximately 1.8 mgd per mile of valley length, or about 1 mgd per mile of stream. The discharge at the Higby station on October 3, 1963, was 342 cfs, a flow equaled or exceeded nearly 95 percent of the time. On the same day the discharge, measured by current meter, was 378 cfs at the Piketon highway bridge. This was a pickup of 36 cfs, or about 23 mgd. The distance between the Higby station and the Piketon bridge is 13 miles if measured along the axis of the valley; it is about 21 miles following the curves and bends of the river.

THE AQUIFER

THICKNESS OF THE UNCONSOLIDATED DEPOSITS

The depth to bedrock and general character of the unconsolidated deposits were determined by augering a line of approximately 25 test holes, spaced across the valley at intervals of a few hundred feet. Data from these augered test holes, supplemented by results of test drilling done in the vicinity of the X–608 pumphouse in 1953 by the U.S. Corps of Engineers, were used to construct the bedrock surface map shown in figure 5.

The bedrock contour map reveals the axis of a relatively deep channel in the bedrock on the west side of Scioto River valley, about 4,200 feet west of the X–608 pumphouse. In the deeper part of this incised channel, the sand and gravel outwash is 70–85 feet thick, averaging about 15–20 feet thicker than the unconsolidated deposits



EXPLANATION

——540——

Water-table contour

Shows altitude of water table. Contour interval 2 feet. Datum is mean sea level

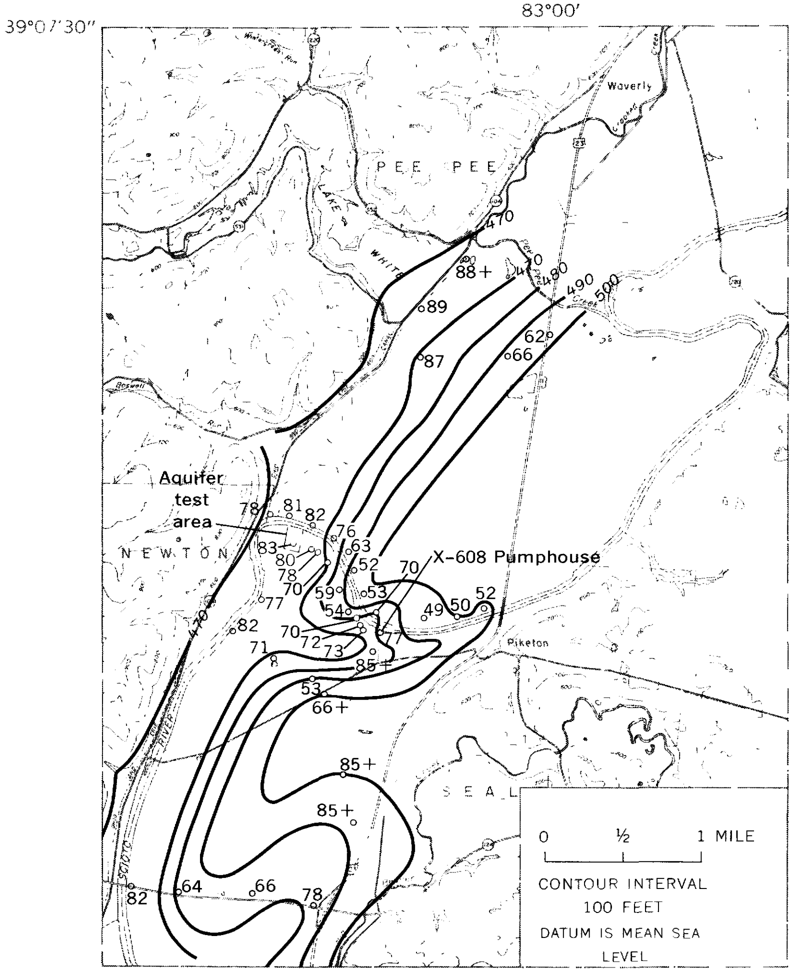
o18

Location of observation well

Number is depth to water, in feet below land surface

Note: Altitude of river at X-608 pumphouse, 531.7 feet

FIGURE 4.—Part of the water table surface, based on depth to water in wells measured July 12, 1963.



EXPLANATION

———480———	₀70
Bedrock contour	Location of well or test hole
Shows altitude of bedrock surface. Contour interval 10 feet. Datum is mean sea level	*Number is depth to bedrock, in feet; plus sign (+) indicates that bedrock was not reached*

Note: Bedrock contours not shown for altitudes above 470 feet on the west side of the valley, and above 500 feet on the east side

FIGURE 5.—Part of the bedrock surface.

which lie in the shallower part of the bedrock valley between the incised channel, which is about 2,500 feet wide, and the east side of the valley. The saturated thickness of the deposits is about 20 feet less than their total thickness and generally corresponds to the depth of the water table.

CHARACTER OF THE SEDIMENTS

On the basis of the preliminary test drilling, a site for an aquifer test was selected near the center of the incised channel. The Goodyear Atomic Corp. contracted for drilling, developing, and pumping a 12-inch diameter well and drilling and developing ten 6-inch diameter observation wells for an aquifer test.

Designated "PW," a 12-inch diameter well for pumping, 83 feet deep, was drilled 450 feet from the left (locally south) bank of the Scioto River at lat 39°04′12″ N., long 83°01′47″ W. Ten 6-inch diameter observation wells were drilled at intervals along two lines, one extended from the pumped well toward the river and the other extended through the pumped well approximately parallel to the river. Wells on the river line are prefixed "N" or "S" and those on the parallel line "E" or "W," depending on their direction with respect to the pumped well. The observation wells are numbered in order of their distance from the pumped well; for example, observation well W–1 is the well closest to the pumped well on the west side of the parallel line. The layout of the aquifer-test pattern is shown in figure 6, and the distances between wells are as shown on following table. Drilling was done by the cable-tool method, and all wells were "bailed in," that is, they were deepened by alternately bailing the hole and driving the casing.

In the vicinity of the test site, the glacial outwash is overlain by 5–10 feet of soil and modern river alluvium. The aquifer, 80–85 feet thick, consists in large part of coarse sand and medium gravel and of lesser amounts of medium to coarse sand. As shown on the fence diagram, plate 1, the finer grained material constitutes two discrete zones, a thin zone that lies a few feet above the bedrock and a 10–20 feet thick zone that separates the coarser-grained material into lower and upper zones of about equal thicknesses. Areally the two zones of finer grained material comprise about a third of the saturated thickness, but locally they comprise as much as half.

Goodyear Atomic Corp. personnel made sieve analyses of samples collected at 5-foot depth intervals in the test wells. The accumulative particle-size distribution curves fall into two distinct families, corresponding to the coarser grained and finer grained material, as shown, for example, by the curves for observation well W–1 (pl. 1, inset).

Distance from pumped well

	Feet		Feet		Feet
W-1(W-1A)	100	S-1(S-1A)	50	R-1	967
W-2(W-2A)	250	S-2A	208	R-2	852
W-3(W-3A)	350	N-1	10	R-3	731
E-1B	33	N-2(N-2A)	100	R-4	585
E-2(E-2A)	50	N-3(N-3A)	216	R-5	652
E-2B	50	N-4(N-4A)	340	R-6	589
E-3B	75	N-5A, B, C	462	R-7	711
E-4(E-4A)	150	N-6A	790	R-8	1,135

The ranges of data for the distribution curves of well W-1 are typical for those of all the other wells drilled at the aquifer test site. The distribution curves for the sand layers not only lie farther to the left—in the direction of finer grain size—but also are steeper than the curves representing the coarser grained material.

The brownish-gray sand zones were of remarkably uniform appearance from well to well and contrasted markedly in both color and texture with the yellowish-brown, more heterogeneous-appearing sand and gravel that composes the coarser grained part of the aquifer. (See photographs, inset pl. 1.)

The driller could tell by the response of the water level as the wells were being drilled that the "gray sand" layers were of relatively low permeability. He described this material as "tight" or "dirty" and the coarser grained material as loosely compacted and "clean." Although the sand zone in the middle of the aquifer functioned in slight degree as a confining bed, water-level data obtained from both deep and shallow observation wells during the aquifer test indicated that with respect to lateral flow the aquifer responded hydraulically as a unit.

Samples from the 12-inch-diameter drilled well and, for comparison, from the augered test hole DW-46 were sent to the U.S. Geological Survey, Hydrologic Laboratory, Denver, Colo., for permeability determinations. Test hole DW-46 had been put down within an estimated 3 feet of the site of the drilled well. For both sets of samples, transmissibility values computed from laboratory results were far lower than those determined from the aquifer test; the samples from the augered hole showed the greatest disparity. From a study of the logs and laboratory results, the authors concluded that the samples from the drilled well, obtained by bailing, lacked a large fraction of the smaller size particles and that the samples from test hole DW-46, obtained by augering, lacked a large fraction of the larger size particles (Norris and Fidler, 1966a). Part of the difference in field and laboratory permeability values was attributed to the difference in direction of flow with respect to the bedding or to the preferred orientation of the

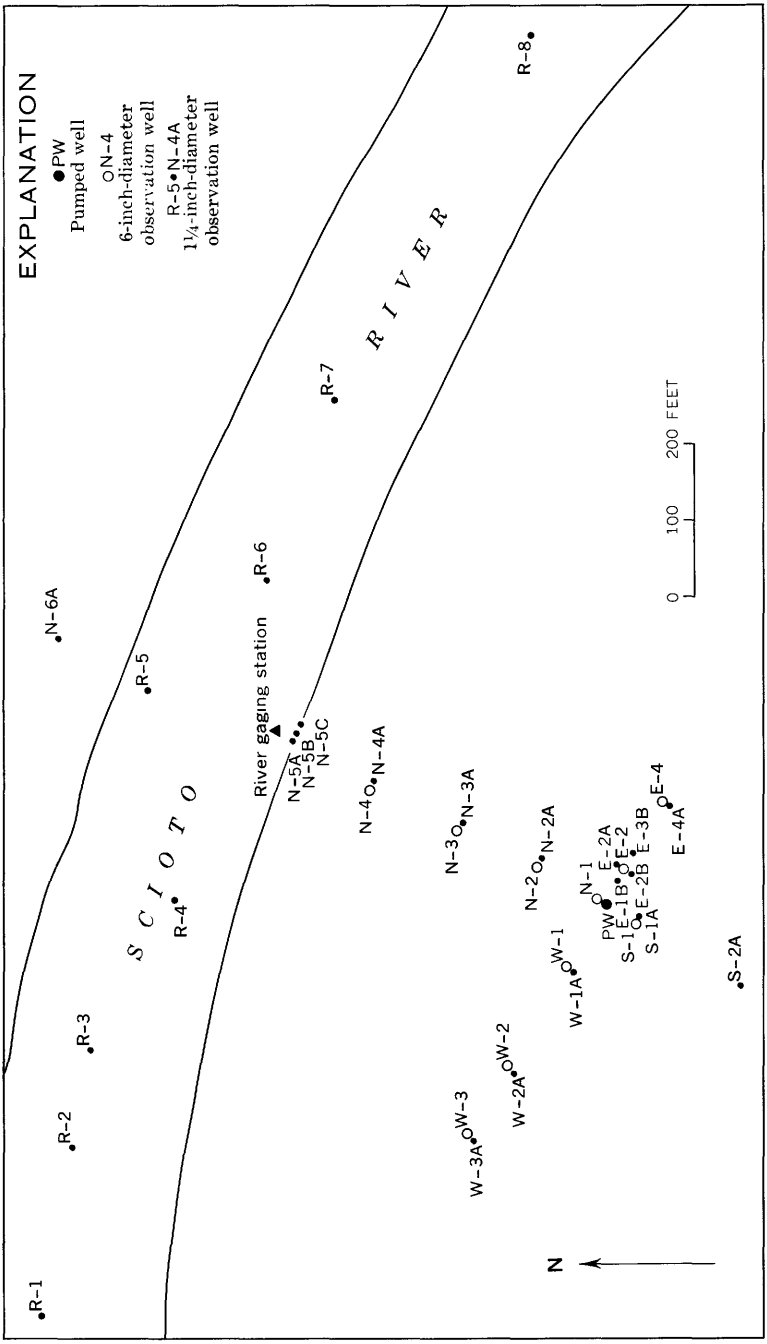


FIGURE 6.—Location of wells used in the aquifer test.

sand and gravel particles. In the field test the flow of water through the aquifer was largely horizontal, that is, parallel to the bedding, whereas it was vertical through the recompacted sample in the laboratory permeameter.

CHARACTER OF THE STREAM CHANNEL

Nine cross sections of the river were made in the test-site area; they were spaced approximately 200 feet apart. (See pl. 4.) In the measured reach, the river ranged in width from about 160 to 285 feet and averaged 260 feet. The average depth was 3.6 feet; the depth at the thalweg, which lies near the far bank in the test-site area, ranged from about 3½ to 11 feet.

The streambed was composed mostly of coarse sand and gravel thinly strewn with larger stones and cobbles. Except in a few places near sand bars, the bed was strong enough to support readily the weight of a man wading. In places along the shore, the bed was covered with several inches of mud and organic debris. The river carried so much waste in suspension that the water was nearly black. Only near the shore, where the water was less than a few inches deep, was the stream bottom visible.

AQUIFER-INFILTRATION TEST

CONSTRUCTION AND TESTING OF THE PUMPED WELL

In constructing the 12-inch-diameter well for pumping, the driller bailed a large hole through a poorly permeable zone, between depths of about 28 and 48 feet, and allowed coarse gravel above that zone to move down and to fill the annular space around the well casing. The coarse sand and gravel fill thus constituted a permeable annulus extending through the zone of finer grained material and locally "tied together" the more permeable upper and lower sections of the aquifer.

The pumped well was screened between depths of 63 and 83 feet with No. 100 slot (0.10-inch openings) wire-wound steel screen. The 6-inch-diameter observation wells were screened in the lower 5 feet, immediately above the bedrock at a depth of about 85 feet. The wells were developed by bailing and by surging with the bailer until the water was clear and the response of the water level to bailing was rapid. The rate of response was tested by rapidly pouring into each well 5 gallons of water and making measurements every few seconds as the water returned to its former level. Response of all wells was rapid, and the water level in each returned to essentially its original position in 1–2 minutes. Screen-loss corrections, computed by a method described by Rorabaugh (1956, p. 138–139), proved to be negligible and were ignored in plotting the time-drawdown data.

On October 9, 1963, the 12-inch-diameter well was pumped for 1 hour at each of five rates—300, 400, 600, 800, and 1,000 gpm (gallons per minute). Periodic measurements were made of the water levels in the well, and the drawdown at each pumping rate was determined. The purpose of the step-drawdown test was twofold: to evaluate the well loss in the pumped well and hence the efficiency of development and to permit separation of well loss from other components of drawdown. By identifying the drawdown components, losses due to partial penetration effects could be determined, and these determinations could be used to evaluate aquifer anisotropy and to estimate vertical permeability.

The drawdown in a pumped well has two major components; formation loss (includes dewatering and partial penetration effects), which is directly proportional to the discharge, and well loss, which varies approximately as the square of the discharge. Well loss is due to turbulence in the well bore, in the screen, or in the formation very close to the well face. Using Jacob's equation (Jacob, 1947, p. 1048),

$$s_w = BQ + CQ^2,$$

where

s_w=drawdown in the pumped well,
B=formation factor,
C=well constant, or screen-loss coefficient, and
Q=discharge of the pumped well.

The well-loss coefficient was determined by a graphical method described by Bruin and Hudson (1955, p. 29). The drawdown observed at the end of each hour-long pumping period was divided by the pumping rate for that period (s_w/Q), and then plotted against the pumping rate (Q) on plain coordinate paper (fig. 7). The intercept of a straight line drawn through the plotted points with the line of zero pumping rate yields the value of B, the formation factor. The slope of the line is the well-loss coefficient. The data used in constructing the graph, figure 7, are as follows:

Pumping rate Q (gpm)	*Drawdown s_w (ft)*	*s_w/Q (ft/gpm)*
300	6. 13	0. 0204
400	8. 20	. 0205
600	12. 53	. 0209
800	17. 27	. 0216
1, 000	21. 86	. 0219

The formation factor B, determined from the graph, is 1.96×10^{-2} ft per gpm; the well-loss coefficient C is 2.3×10^{-6} ft per (gpm)^2.[3]

[3] The well-loss coefficient is commonly expressed as sec^2 per ft^5, where the discharge is in cubic feet per second. By conversion, 2.3×10^{-6} ft per (gpm)^2 $x \left(\frac{\text{gpm}}{0.00223\text{ cfs}}\right)^2 = 0.46$ sec^2 per ft^5.

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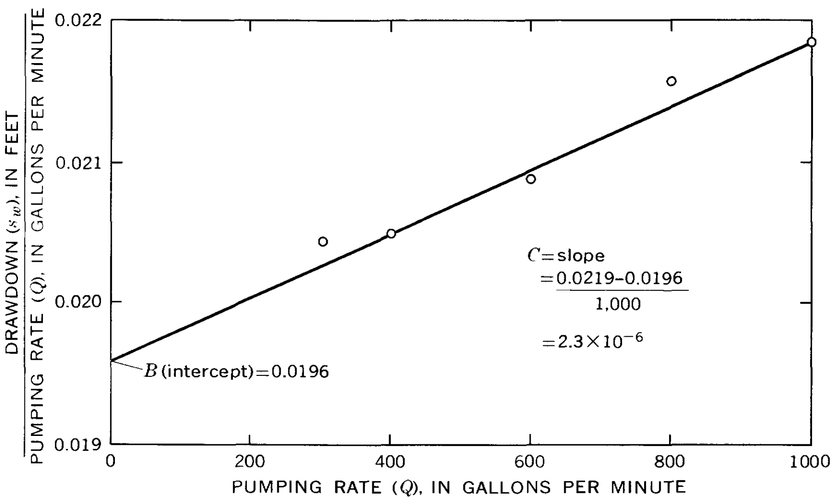

FIGURE 7.—Plot of s_w/Q versus Q to solve for values of B and C.

Thus, of the drawdown of 21.9 ft observed in the 12-inch-diameter pumped well at the end of the step-drawdown test, the component due to well loss was 2.3 ft, or

$$\begin{aligned} s_w &= BQ + CQ^2, \\ &= 0.0196\ (1{,}000) + 0.0000023\ (1{,}000)^2, \\ &= 19.6 + 2.3 = 21.9 \text{ ft.} \end{aligned}$$

The well-loss coefficient for the 12-inch well, 2.3×10^{-6} ft per (gpm)^2, is low compared with the values obtained by the Geological Survey in 1953 for the plant's three supply wells: 3.0×10^{-6}, 6.0×10^{-6}, and 1.0×10^{-5} ft per (gpm)^2, respectively. The relatively low well-loss coefficient indicates that the well drilled for the aquifer test was highly efficient. Moreover, the construction technique used by the driller, who by excessive bailing brought down relatively coarse material to "wall off" fine material from proximity to the well screen, proved to be an effective aid to development. Similar methods subsequently were used in constructing two of the four wells drilled for the process water supply.

INSTALLATION OF DRIVE-POINT WELLS

Approximately twenty-five 1¼-inch-diameter drive-point wells were installed in the aquifer-test area (fig. 6). The drive-point wells are numbered in sequence with the 6-inch diameter observation wells and are further designated by the suffixes A, B, or C to indicate their depths. The suffix A denotes drive-point wells open in the upper part of the aquifer; B refers to wells open in the middle of the aquifer; and C refers to wells open near the base of the aquifer. A drive-point well, open near the top of the aquifer, was installed alongside each of the 6-inch-diameter observation wells, except at the site of well N–1. Drive-point wells at the sites of the 6-inch-diameter wells are numbered the same as their companion wells and, in addition, are given an appropriate suffix corresponding to their depth. Four drive-point wells, E–1B, E–2A, E–2B, and E–3B, in conjunction with 6-inch-diameter observation well E–2, constituted a special array designed to determine the vertical permeability of the aquifer. (See Stallman, 1963, fig. 57 and eq 9, p. 210). Drive-point wells N–5A, N–5B, and N–5C were installed in a closely spaced group at the near edge of the river to observe the drawdown at different depths in the aquifer.

All drive-point wells were developed with a pitcher pump prior to the aquifer test. Development was considered adequate when the wells yielded clear or reasonably clear water, and when the water level quickly returned to its original position after water had been poured into the wells.

To measure head changes in the aquifer immediately beneath the river, eight drive-point wells were installed in the streambed. Efforts were made initially to have these wells open in the top 1 foot of the aquifer, immediately below the streambed. To give each well stability and to hold it in place in the river, a pointed blank section of pipe about 3 feet long was to be used as the lower part below the 1-foot section containing the perforations. Above the perforated section, additional blank pipe was to extend above the water surface. The plan was to drive the well to such depth that the top of the 1-foot perforated section would be about 1 foot beneath the streambed. Unfortunately, the holes in the perforated section were so large that the pipes filled with sand as the wells were being driven, and the wells could not be cleared and developed. When this plan to install the wells failed, regular commercial-type well points, 1¼ inches in diameter, were driven to a depth such that the tops of the screens were 5 feet beneath the streambed.

A difference between the water level inside the pipe and stream level was taken to indicate that the pipe was effectively sealed where it penetrated the bed materials. Adequate development was assumed when the water pumped from the well was clear (in contrast to the muddy river water) and when the temperature of the water was lower than that of the river water. Water pumped from the drive-point wells ranged from 2°F to 11°F colder than the river.

PREPUMPING CONDITIONS

Prior to the aquifer test, the water level in the 6-inch-diameter observation wells, open near the base of the aquifer, averaged about

0.04 foot higher than the water level in the respective drive-point wells at the same sites, screened in the top of the aquifer. Because the water in the lower part of the aquifer was under slight pressure, being partially confined by the poorly permeable zone near the middle of the aquifer, there was a small component of upward flow in the test-site area.

Ground-water gradients on the south bank of the river in the test-site area were very low, ranging from about 1 to 2 feet per mile (fig. 8). The surface of the water table prior to pumping was as follows (fig. 8):

Altitude of water surface

[River gage 530.57]

	Feet		*Feet*		*Feet*
R–2	530. 99	N–2	530. 92	S–2	530. 87
R–3	530. 98	N–3	530. 94	E–2	530. 92
R–4	530. 82	N–4	530. 89	E–4	530. 94
R–5	530. 95	N–5	530. 96	W–1	530. 90
R–6	530. 90	N–6	531. 45	W–2	530. 88
PW	530. 91	S–1	530. 95	W–3	530. 85

Except for the riverbed wells, the altitudes represent the average altitude in deep and shallow wells; the average altitude is estimated for wells PW, S–2, and N–6. The sinuosity of the contours in figure 8 is due in part to slight inaccuracies in measurements or in altitude of measuring points. The arrows in figure 8 indicate general direction of ground-water movement. The direction of movement on the south bank was generally northwest, and there was practically no component of flow towards the river parallel to the river line of wells.

Ground-water gradients beneath the north bank of the river were much steeper than those beneath the south bank. (See figs. 8 and 4.) The general direction of ground-water flow beneath the north bank was southward, and the water was discharged into the river uniformly along the entire reach in the test-site area. Flow from the north locally constituted the principal component of ground-water discharge to the river. The local steepening of the gradient near the river improved, in effect, the hydraulic connection between the aquifer and the partially penetrating stream, thus accentuating its response to water-level changes caused by pumping.

Prior to pumping, the water level in the drive-point wells in the bed of the river stood approximately 0.2–0.3 foot above river level. This head difference, which represented the force required to move water from the aquifer into the stream, was caused chiefly by a zone of low permeability at the interface between the stream and the aquifer.

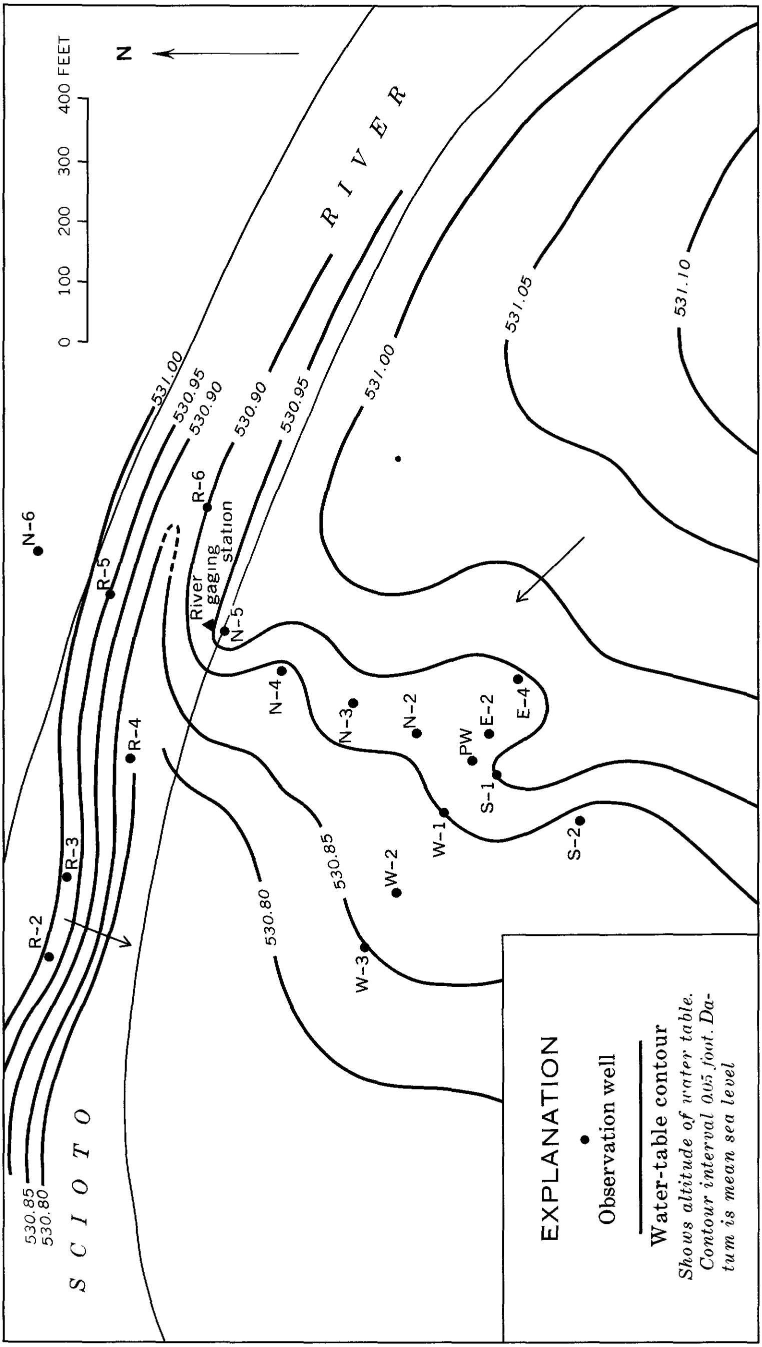

FIGURE 8.—Aquifer test-site area showing surface of the water table on October 8, 1963, prior to pumping.

NINE-DAY CONSTANT-RATE TEST

On October 14, 1963, after the water table had recovered from the step-drawdown test made 5 days previously, and water levels were following a level trend, pumping was started at the rate of 1,000 gpm and maintained with little variation in rate for 9 days. When the pump was turned on at 1018 hours e.s.t., the natural ground-water regimen was abruptly altered. Ground-water gradients became radically steeper, and eventually the water table was lowered below stream level. As the cone of depression developed, water pumped from storage was replaced partly by ground water diverted to the well, and thereby prevented from entering the stream, and partly, though in lesser quantity, by river water entering the aquifer by induced infiltration.

During the 9-day pumping period, and for an additional 9 days after the pump was turned off, recorders were operated on the ten 6-inch-diameter observation wells, a river stilling well, and drive-point well N–5A. In the early minutes of both the drawdown and recovery periods, a series of rapid measurements were made in each of the 6-inch-diameter wells. Throughout the test the water level was measured periodically in the pumped well and in the drive-point wells, including those wells in the bed of the river, which were accessible by boat.

The decline of the water level in the observation wells, especially in the deeper wells, was rapid at first, because of slow vertical drainage in the aquifer and consequent rapid depletion of artesian-type storage below the water table. The rate of drawdown was slowed within a few minutes, however, by the delayed vertical flow, including leakage through the semiconfining, poorly permeable zone. With slowing of the initial rapid drawdown, the response became increasingly like that of a homogeneous, unconfined aquifer. As the pumped well was screened in the lower one-third of the aquifer, drawdowns in the 6-inch-diameter observation wells, open near the bottom of the aquifer, were slightly greater than drawdowns in their companion drive-point wells, open near the top of the aquifer, except in those wells at considerable distance from the pumped well.

About 20 hours after pumping started, the river boundary started to sensibly affect the development of the cone of depression. This was indicated by a relatively rapid increase in the apparent value of the storage coefficient computed for the river line of wells for progressively longer time periods compared with values computed for the parallel line of wells for similar time periods. (See fig. 16.)

The plan was to pump for 4 days; at the end of this time it was believed that water levels would be stabilized and approaching equilibrium. By coincidence, however, the caretakers at Lake White, an artificial impoundment of 337 acres at the head of Pee Pee Creek, chose the second day of the test to lower the lake level for maintenance purposes. Control gates at the dam were opened and a "slug" of water was released into the river via Pee Pee Creek; the "slug" ultimately raised the stage 0.34 foot at the test site. During recovery, following the 9 days of pumping, a 0.37-inch rain on October 28 raised the river stage about 0.05 foot, but this amount of water was not enough to seriously affect the trend in the wells.

The water released from Lake White began to raise the stage at the aquifer test site approximately 32 hours after pumping started. As the stream stage rose, water levels also rose in the wells nearest the river, and the decline of the water table in wells farther from the river was slowed. The river stage returned to "normal" in 2–3 days; the slow decline of the water table continued, and by the ninth day of the aquifer test the rate of drawdown in the wells had become slight because the hydraulic system was approaching equilibrium.

Hydrographs showing the fluctuation of water levels in the wells during the aquifer test are shown on plate 2. The hydrographs of the 6-inch-diameter wells are shown in solid lines; those of the corresponding drive-point wells at each site, designated by the suffix A or B, are shown in dashed lines. Plate 3 shows graphs of water levels in the riverbed drive-point wells compared with the river stage. The effect of the release of water from Lake White shows up plainly on these graphs.

During the pumping period, samples of water were collected nearly every day from the pumping well, the 6-inch-diameter observation wells, and the Scioto River. The 6-inch wells were sampled by removing the recorder float and by lowering a "thief-type" water sampler into the well; the sampler was triggered shut so as to take water about at the level of the well screen. The samples were analyzed in the Goodyear Atomic Corp. water treatment laboratory for hardness, alkalinity (as $CaCO_3$), chloride, and sulfate (table 2). Water temperature was measured nearly every day in the wells and the river with a thermistor-type electric thermometer which could be read to approximately 0.2°F.

Besides the analyses made by the Goodyear Corp., the U.S. Geological Survey made analyses of water from the pumped well (the samples were collected before and near the end of the 9-day discharge test) and of samples from the Scioto River, Lake White, well E–1B, and the four production wells. Results of these analyses are given in table 1.

DISTANCE TO LINE SOURCE AND COEFFICIENT OF TRANSMISSIBILITY

The line-source distance represents the distance from the pumped well to a line along which recharge is assumed to originate. As stated by Rorabaugh (1956, p. 119):

> "Infiltration into the river bed will take place over an area, but for simplification in computations the area is replaced by a 'line source'; that is, the assumption is made that, so far as effects in observation wells are concerned, the water levels will behave the same whether the water is entering over an area or at a line which is located at the effective or weighted-average distance to the area. * * * The distance from the pumped well to the line source is designated a. For further simplification, the line source may be replaced by a recharging 'image well' placed at a distance a beyond the line source. The problem now is to evaluate the effect of a well discharging at a rate Q and an image well a distance $2a$ from the discharging well, recharging at the same rate."

Usually, the line-source distance will be greater than the physical distance from the well to the near bank of the river and may even exceed the distance to the far bank. This is true because the stream only partly penetrates the aquifer; the two only make an imperfect hydraulic connection. The line-source distance also is increased when the permeability of the streambed is relatively low.

The average line-source distance at the end of the 9-day aquifer test was computed to be about 510 feet, which may be compared with the physical distance of 450 feet between the pumped well and the river. The computed distance probably would have been greater had it not been for the steep ground-water gradients prevailing near and beneath the river that were associated with the discharge of ground water along the opposite (north) bank. Ground water beneath the north bank was already moving in the direction of the well. When this water was diverted by pumping, the extra head, formerly required to move water into the stream, became available to augment the discharge of the well. In effect, the recharging image well was thus moved closer to the pumping well, offsetting to some extent the partial penetration of the stream.

The coefficient of transmissibility, a term originally defined by Theis (1935), is the product of the coefficient of permeability of an aquifer and its saturated thickness. The field coefficient of permeability is defined (Ferris and others, 1962, p. 72) as the rate of flow, at prevailing temperature, in gallons per day, through a cross-sectional area of 1 square foot under a hydraulic gradient of unity. From the 9-day test, the coefficient of transmissibility of the sand and gravel aquifer was computed to be approximately 215,000 gpd per ft (gallons per day per foot). The coefficient of permeability at the test site thus is about 3,300 gpd per sq ft, which may be compared with the value of 4,100 gpd per sq ft obtained by the Geological Survey in 1953 from an aquifer test made 3 miles farther downstream at the wells that supply water for domestic purposes to the Portsmouth facility.

The distance to the line source and the coefficient of transmissibility were determined by distance-drawdown methods developed by Rorabaugh (1956, p. 120–122). To correct for error resulting from partial penetration of the pumped well, the drawdowns observed in the deep and in the shallow observation wells were averaged (Jacob, 1945). The average values were corrected for dewatering effects (Jacob, 1944) using the correction factor $s^2/2m$, where s is the drawdown and m is the original saturated thickness of the aquifer. Values of $s^2/2m$ were subtracted from the drawdown and added to the recovery values before they were substituted in Rorabaugh's equations. Calculations were simplified by use of a graphical method developed by Schaefer and Kaser (1965); its use eliminated much tedious computation.

The equations presented by Rorabaugh (1956) for distance to line source are given below. If observation wells are placed on a line perpendicular to the river, then the equation (using common logarithms) for the river side of the pumped well is

$$\frac{s_1}{s_2}=\frac{\log\left(\frac{2a-r_1}{r_1}\right)}{\log\left(\frac{2a-r_2}{r_2}\right)}.$$

If the wells are placed on a line through the pumped well and parallel with the river, then

$$\frac{s_1}{s_2}=\frac{\log\frac{\sqrt{4a^2+r_1^2}}{r_1}}{\log\frac{\sqrt{4a^2+r_2^2}}{r_2}},$$

where

s_1 and s_2=drawdowns in observation wells, in feet;
r_1 and r_2=respective distances from the observation wells to the pumped well, in feet;
a=distance from pumped well to line source, in feet.

Once the line-source distance is known, the coefficient of transmissibility can be calculated using a modification of the Thiem equilibrium formula. The Thiem formula, in nondimensional form (Wenzel, 1942, p. 81), is

$$T=\frac{Q\log_e(r_2/r_1)}{2\pi(s_1-s_2)},$$

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where the subscript e in the log term indicates the natural logarithm. For a well on the river line, the equation (Rorabaugh, 1956), using common logarithms, is

$$T=\frac{527.7Q\log\frac{2a-r}{r}}{s}.$$

For a well on the parallel line,

$$T=\frac{527.7Q\log\frac{\sqrt{4a^2+r^2}}{r}}{s},$$

where

T=coefficient of transmissibility, gallons per day per foot;
s=drawdown in observation well, in feet;
r=distance from observation well to the pumped well, in feet;
a=distance from pumped well to line source, in feet;
Q=pumping rate, in gallons per minute.

In practice, Rorabaugh's equations for determining the distance to the line source are usually solved by substituting values for a in the log term and finding agreement with the left-hand side of the equation by trial and error. Schaefer and Kaser reduced the equations to graphical form and, for convenience, designated the left-hand side of the equation as the abscissa and the line-source distance a as the ordinate. The graph was constructed for distances of r_1=10 feet, and r_2=100 feet. To use the graph, values of drawdown are plotted against distance, preferably on semilogarithmic paper, as shown in figures 9 and 10. The values s_1 and s_2, corresponding to distances r_1 and r_2, are read off the semilogarithmic plot and the ratio s_1/s_2 is determined. If this value is entered into Schaefer and Kaser's graph, the line-source distance is easily found. A somewhat similar procedure can be used on a comparable graph to determine the coefficient of transmissibility, if the line-source distance a is known. The method is illustrated by examples shown in figures 11, 12, 13, and 14, respectively.

The line-source distance was calculated for selected times during the pumping and recovery periods and the results are shown graphically in figure 15. Note that values computed from data collected early in the test are less than the real distance from the pumped well to the river. As the computation involves only a ratio of drawdowns in a pair of observation wells, a "line-source distance" can be determined for any period of pumping long enough to produce drawdowns in the wells. Whether or not a computed value does, in fact, represent the effect of an infiltrating stream can be determined only from

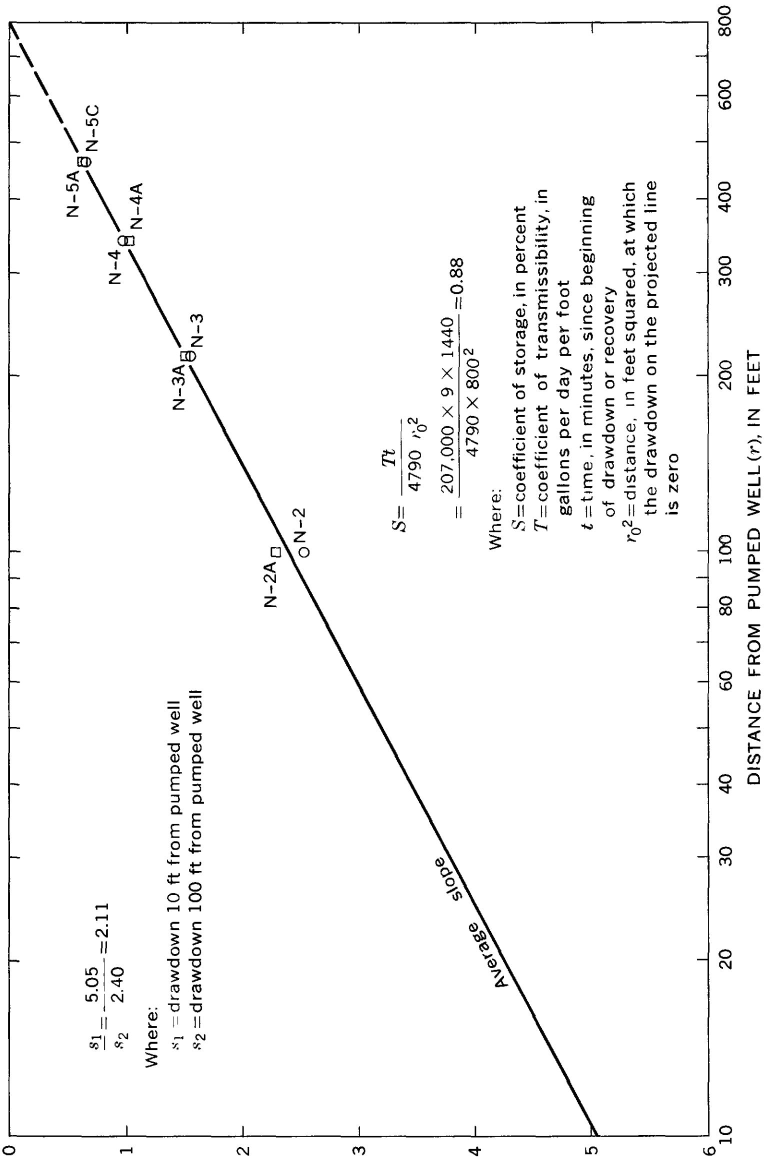


FIGURE 9.—Drawdown versus distance in river line of wells after 9 days of pumping at rate of 1,000 gpm.

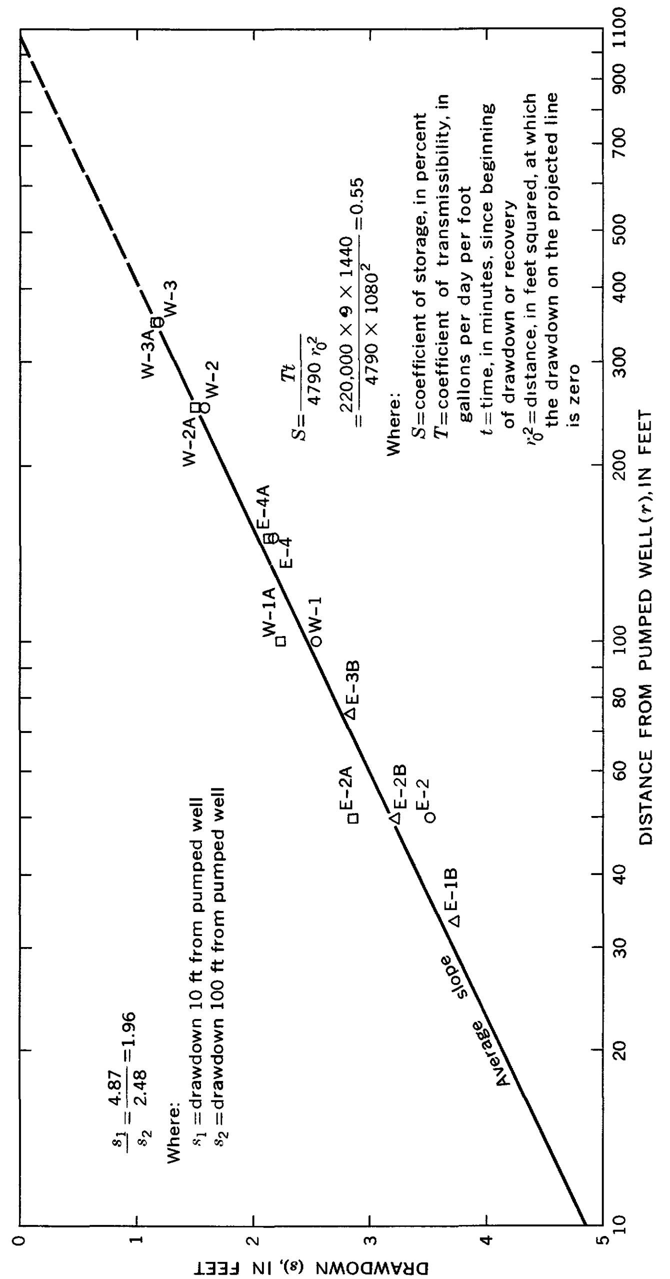


FIGURE 10.—Drawdown versus distance in parallel line of wells after 9 days of pumping at rate of 1,000 gpm.

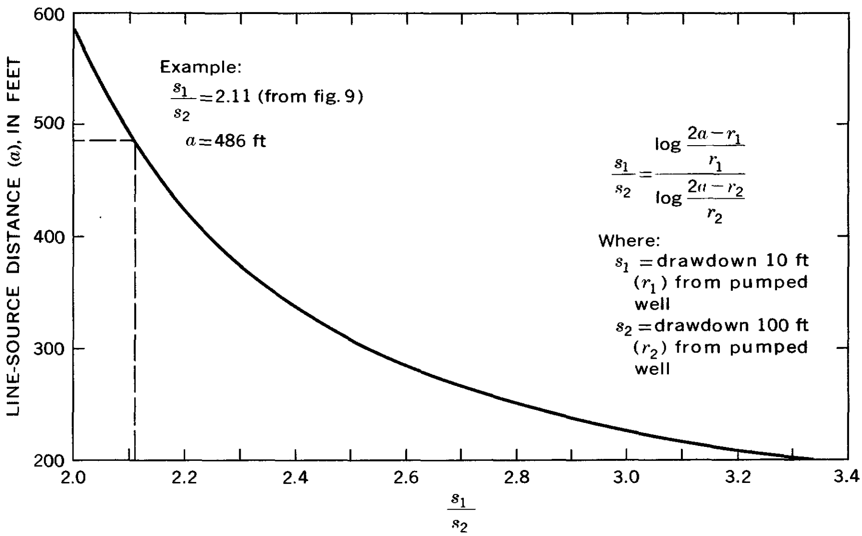


FIGURE 11.—Determination of line-source distance (a) from ratio s_1/s_2 on river line of wells. Range of graph, 200 feet to approximately 600 feet (after Schaefer and Kaser, 1965).

consideration of known hydrogeologic controls operating at the test site. At the Piketon site computed values of the line-source distance, representing progressively longer time periods, become essentially constant at a distance slightly exceeding that between the pumped well and the river. This stabilizing of the cone of depression is compelling evidence, though in itself does not constitute proof, of river infiltration.

The influence of the slug of water released from Lake White is clearly evident on the graphs showing distance to line source (fig. 15). The temporary increase in river stage had the effect of moving the line source—and associated recharging image well—closer to the pumped well. As the river stage subsequently declined, the line-source distance became greater. Effects of the slug of water do not, of course, show up in the calculations based on data from the recovery period. The data from the recovery period show that not before the lapse of about 1½ days for the river line, and about 2 days for the parallel line, does the computed line-source distance equal the distance from the pumped well to the near bank of the river. Several days of pumping or recovery were required for the line-source distance to become nearly constant on the graphs.

The coefficient of transmissibility was calculated for the same time periods that were used in determining the line-source distance. The values are given below:

Coefficient of transmissibility

[Thousands of gallons per day per foot]

Time (days)	*Parallel line*		*River line*	
	Drawdown	*Recovery*	*Drawdown*	*Recovery*
1	246	237	263	240
2	230	228	228	223
3	228	228	219	218
4	225	225	221	216
5	223	226	216	209
6	224	221	212	205
7	222	220	210	200
8	220	221	209	207
9	220	220	207	207

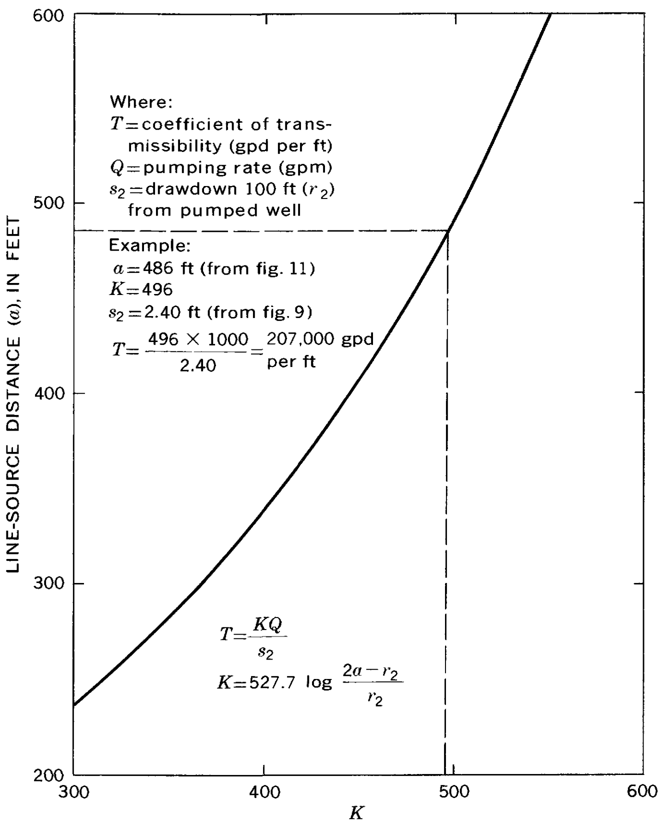


FIGURE 12.—Determination of coefficient of transmissibility from K for river line of wells (after Schaefer and Kaser, 1965).

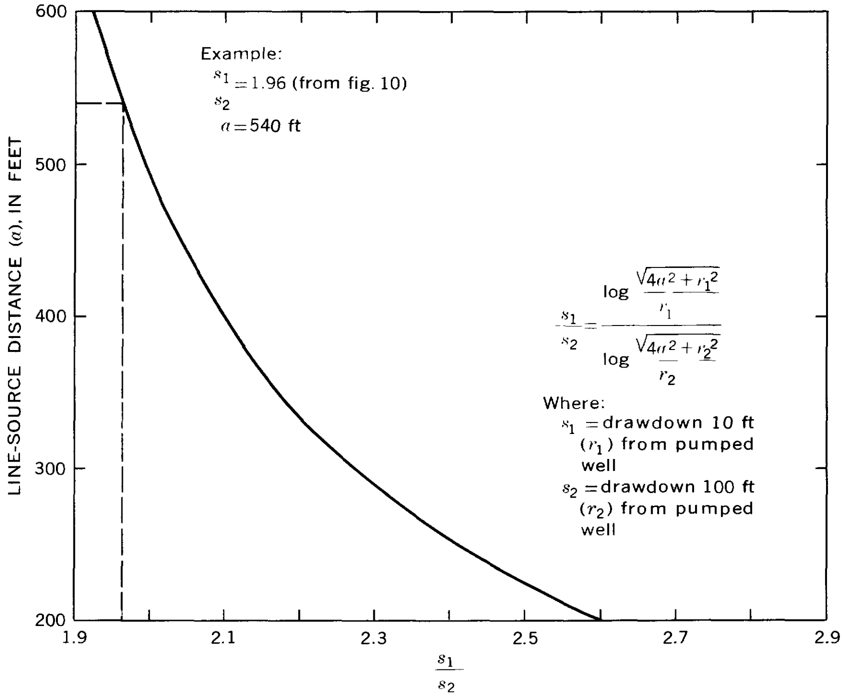


FIGURE 13.—Determination of line-source distance (a) from ratio s_1/s_2 for parallel line of wells. Range of graph, 200 feet to 600 feet (after Schaefer and Kaser, 1965).

EVIDENCE OF INFILTRATION

River control of the hydrologic system at Piketon is shown by the eccentricity of the cone of depression (figs. 9, 10) and by the excessively large and continually increasing value of the coefficient of storage computed for progressively longer time periods.

The coefficient of storage of an aquifer, expressed as a percentage, is defined (Ferris and others, 1962, p. 74) as the volume of water released from or taken into storage per unit surface area of the aquifer per unit change in the component of head normal to that surface. In artesian aquifers, where water is confined under pressure and the saturated thickness remains constant, very little change in storage takes place when the head changes in the aquifer. Consequently, the storage coefficient computed for artesian aquifers is low, a typical range of values being about 0.0001 to 0.0003. In water-table or unconfined aquifers, however, a change in water level results in either a dewatering or a refilling of that part of the aquifer through which the water table moves. The volume of water involved is significant and the coefficient of storage, which is virtually identical to the specific yield, is relatively large. A typical range in values for unconfined aquifers is about 0.10–0.25.

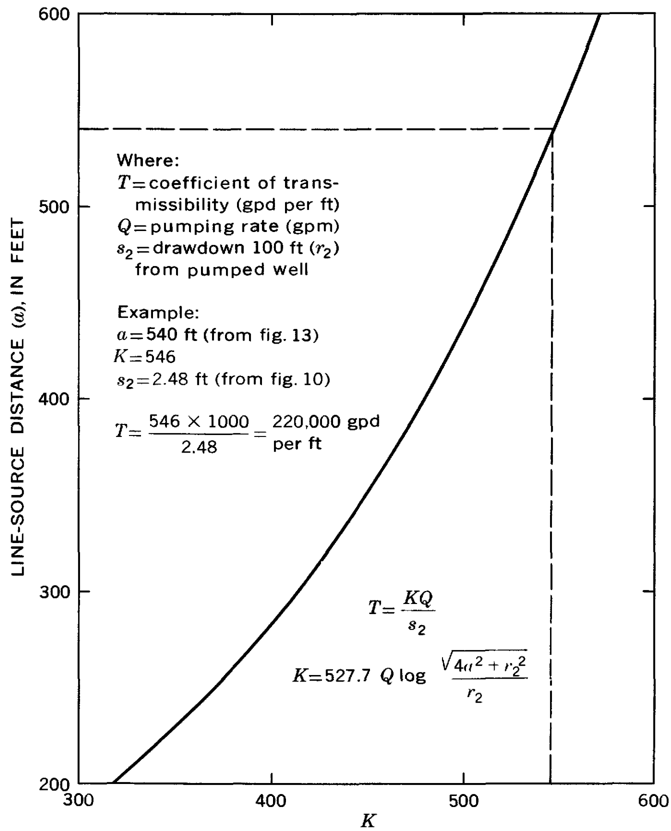


FIGURE 14.—Determination of coefficient of transmissibility from K for parallel line of wells (after Schaefer and Kaser, 1965).

The apparent coefficient of storage was calculated from distance-drawdown data by a straight-line graphical method developed by Cooper and Jacob (1946). Drawdown or recovery values are plotted on semi-logarithmic paper against distance from the pumped well, as is done, for example, in figures 9 and 10. A line through the plotted points is extended to the intercept of distance with zero drawdown, and this value is substituted into the following nondimensional equation:

$$S=\frac{2.25Tt}{r_0^2}$$

or, as expressed in common units,

$$S=\frac{Tt}{4{,}790r_0^2},$$

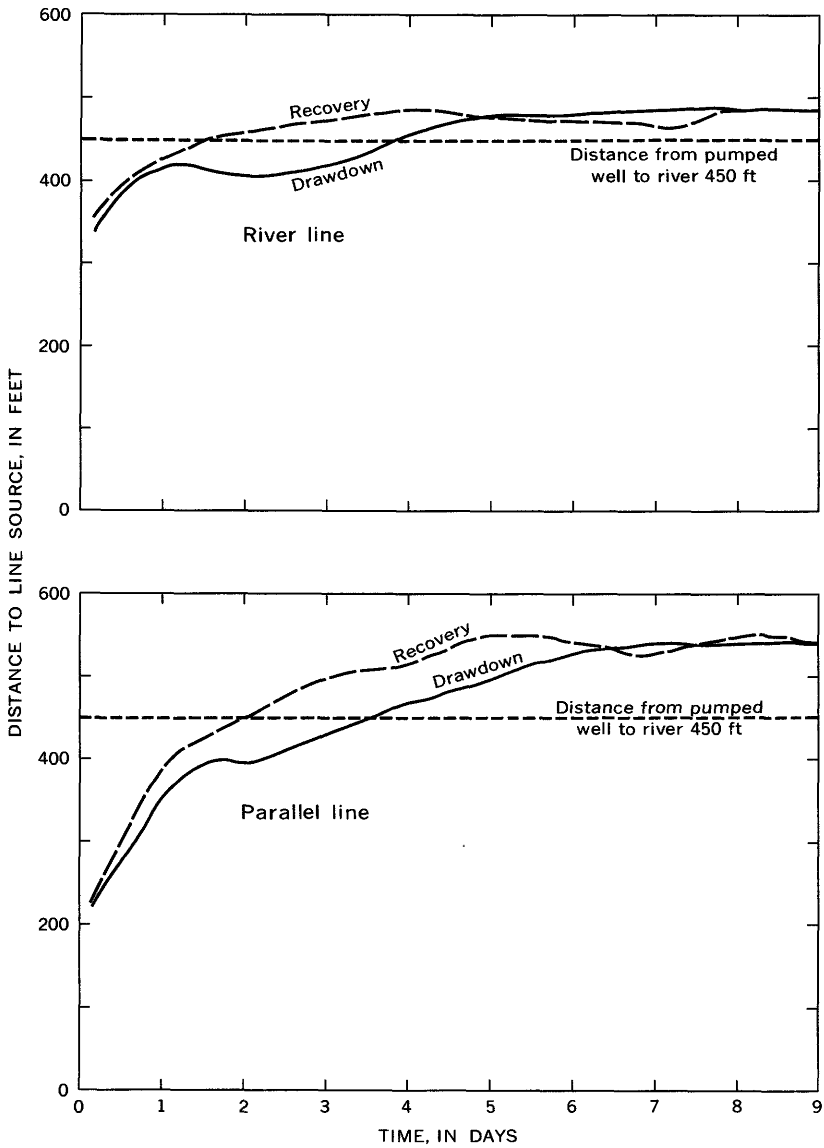


FIGURE 15.—Line-source distance at selected times, computed from river line and parallel line of observation wells.

where

S= coefficient of storage, in percent;

T= coefficient of transmissibility, in gallons per day per foot;

t= time in minutes since beginning of drawdown or recovery;

r_0^2= distance, in feet squared, at which the drawdown on the projected line is zero.

The Cooper and Jacob method, based on the Theis nonequilibrium formula (Theis, 1935), strictly applies to artesian aquifers and is not applicable to data collected at large distances from the pumped well or in the very early part of the drawdown or recovery periods. In unconfined aquifers, pumping or recovery must go on for a considerable length of time before essentially radial flow conditions are established; only then will the method yield a true value for the coefficient of storage.

Figure 16 is a graph showing variation with time of the apparent coefficient of storage, as determined by the Cooper and Jacob straight-line method. Recharge became effective several hours before the calculated storage coefficient had reached a value considered representative of water-table conditions. Then, with the onset of recharge, the calculated value increased rapidly and soon became unrealistically high, exceeding the porosity of the aquifer. For the river line the apparent value of the storage coefficient approached unity. This large value and the increase of the apparent value throughout the test are clear evidence of river infiltration.

DETERMINATION OF THE COEFFICIENT OF STORAGE

The true value of the coefficient of storage was determined from time-drawdown data by a method suggested by R. W. Stallman, of the U.S. Geological Survey. This method makes use of special type curves, which are constructed for a two-well image system.

The ratio $r_i/r_r=K$, where r_i is the distance from the observation well to the image well and r_r is the distance from the observation well to the real well, can be computed for each of the observation well locations. Response curves for the image system, as functions of K, have been given by Stallman (Bentall, 1963, p. 45–47). The shape of each curve, which is drawn as an appendage to the Theis nonequilibrium-type curve (see Ferris and others, 1962, p. 92–98), depends only on the relative position of the observation well with respect to the pumping and image wells.

For each of the 6-inch diameter observation wells, the time-drawdown plot was matched to the appendage curve having the same K value. The K value was computed for each observation well by using its known distance from the pumped well and then its hypothetical distance from the image well, the location of which had been determined by distance-drawdown methods. The position of each of the image well curve limbs used in the analysis was adjusted slightly by Week's (1964) method to compensate for partial penetration of the pumped well. In most plots, a good fit between observed drawdowns and the two-well curves was obtained for the later test data. After matching the field data plot to the two-well curves, match-point values were selected and the coefficients of storage and transmissibility were computed. The coefficient of storage computed from the two-well curves for the 10 observation wells ranged from 0.18 to 0.22 and averaged 0.20. Computed transmissibility values for all wells were close to that obtained by distance-drawdown methods.

COEFFICIENT OF VERTICAL PERMEABILITY

Most outwash aquifers are anisotropic, the permeability measured vertically, or transverse to the bedding, typically being much lower

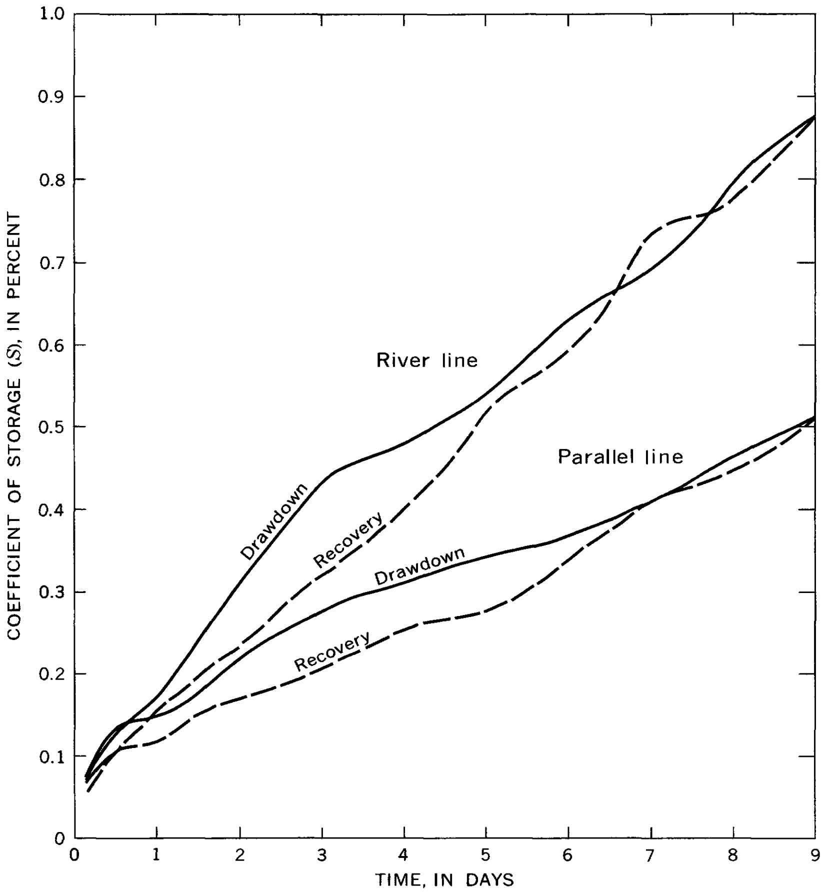


FIGURE 16.—Variation with time of calculated value of coefficient of storage.

than the permeability measured parallel to the bedding. Relatively low vertical permeability of a sand and gravel aquifer commonly results from the presence of interbedded layers of fine-grained sediment such as silt or clay, which retard downward percolation. Another factor contributing to low vertical permeability is the asymmetric shape of many of the constituent particles and the preferred orientation of their flat surfaces parallel to the bedding plane (Weeks, 1964, p. 193). The vertical permeability of an aquifer can be a principal factor in controlling the rate of induced stream infiltration, owing to the predominately vertical flow components which exist beneath an infiltrating stream. Partial penetration losses in pumping wells are increased by aquifer anisotropy in which the vertical permeability is relatively low.

The authors studied the anisotropy of the aquifer at Piketon, using several analytical methods in an effort to determine or make an intelligent estimate of P_z, the coefficient of vertical permeability (Norris and Fidler, 1966b). The principal method, developed by Stallman (1965), was believed to have yielded an accurate P_z value; it was a type curve method which utilized time-drawdown data. The result was essentially confirmed by a simple, graphical approximation method, using drawdown data at the top and bottom of the aquifer. Also tried, but considered highly inaccurate, was a finite-difference method, using distance-drawdown data from a special array of wells (Stallman, 1963). Techniques associated with the use of Stallman's type curve method, of interest to the hydrologist, are discussed briefly here. The significance of the coefficient of vertical permeability relative to utilization of ground water at Piketon is evaluated. For a more complete discussion of the analytical methods, the reader is referred to the original paper and to the work of Stallman.

Using electric analog simulation, Stallman (1965) studied the effects of several factors on the specific yield of unconfined aquifers; these factors principally involved a difference between the horizontal and vertical permeability, the effect of vertical flow components, and the partial penetration of the pumped well. Stallman constructed families of type curves showing analog model response as a function of the dimensionless quantities, Tt/r^2s versus sT/Q, for fully and for partially penetrating wells,

where

T=coefficient of transmissibility, cubic feet per day per foot;
t=time, in days, since beginning of pumping;
r=distance, feet, from pumping well to point of observation;
S=coefficient of storage, in percent;
s=drawdown, feet, at distance r from pumped well;
Q=pumping rate, cubic feet per day.

Stallman's response curves (see fig. 17) are drawn as limbs to the nonequilibrium well function curve of Theis (see Ferris and others, 1962, p. 92–98) and represent selected values of the group $(r/m)(P_z/P_r)^{0.5}$,

where

r=distance, in feet, from the pumping well to the point of observation;

m=saturated aquifer thickness, in feet;

P_z, P_r=coefficients of vertical and horizontal permeability, respectively, gallons per day per square foot.

Figure 17 is a logarithmic plot of time versus drawdown from well E–4; superposed on it are two appropriate response curves of Stallman which bracket the field data. The shape of the time-drawdown curve for this well is fairly typical for the 6-inch-diameter observation wells. After the time-drawdown data were matched to Stallman's water-table type curves as shown, match point values were read out and the coefficient of vertical permeability was computed. The procedure was somewhat more complicated than this implies, however, as the field data plots could not be matched directly to the type curves. Radial flow had not become established by the time the control of the river became dominant, and at no time during the aquifer test did the time-drawdown plots sensibly follow the Theis curve. The position of the Theis curve on the field plot is necessary for identification of the type-curve limb corresponding to the time-drawdown data (fig. 17). The position of the Theis curve was determined indirectly and traced onto the field data plot by matching the latter part of the test data to the appropriate limb of the two-well type curves for an image well system, as described in the preceding section. The superimposing of the field data sheet on Stallman's water-table type curves by matching the location of the Theis curve on the data plot with the parent Theis curve on the water-table response curves permitted the selection of match point values and computation of P_z. The average coefficient of vertical permeability determined by this method was 365 gpd per sq ft, about one-ninth the value of the horizontal permeability.

The P_z value determined by Stallman's method, used to compute the theoretical drawdown in the well pumped during the 9-day aquifer test, yielded a figure of 19.5 feet, or about 16 percent greater than the observed drawdown (subtracting dewatering and well loss effects) of 16.8 feet. The computation was based on a table of values presented by Butler (1957, p. 159–164) from which the drawdown in a partially penetrating well can be computed if the ratio of vertical

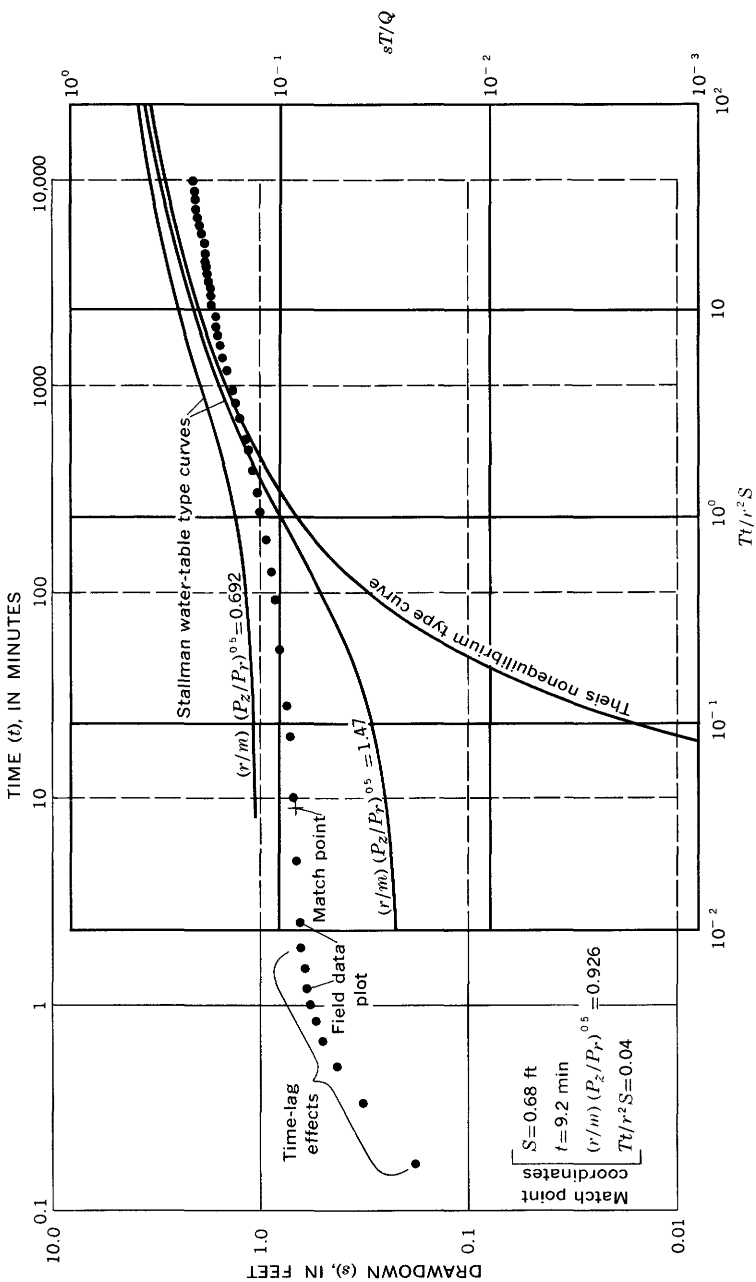


FIGURE 17.—Time versus drawdown in well E–4, compared with Theis and Stallman type curves.

to horizontal permeability is known. Although the table is based on flow in confined aquifers, it is believed that only small error was introduced in applying the values to the Piketon data. Anisotropy at the pumped well was somewhat reduced by the method of well construction, and this partly accounts for the difference between the observed and computed drawdowns.

The table Butler presents indicates that although small changes in well penetration can have significant effect on drawdown, the drawdown in a partially penetrating well is relatively insensitive to the degree of aquifer anisotropy. For example, if the vertical permeability of the aquifer at Piketon were equal to the horizontal permeability, the theoretical drawdown in the well pumped during the aquifer test—which was screened in the lower one-third of the aquifer—would be 16.0 feet, or about 3.5 feet less than the value computed for a vertical to horizontal permeability ratio of 1:9. If, however, the pumped well were fully penetrating, the ratio of vertical to horizontal permeability would not enter the calculations and the theoretical drawdown would be only 8.25 feet, or 11.2 feet less than that for a well open to one-third the aquifer thickness. Thus, although aquifer anisotropy produced a theoretical increase of about 22 percent in the drawdown in the pumped well, the fact that the well was open to only one-third the aquifer thickness, rather than to the full thickness, theoretically increased the drawdown by 136 percent. Although somewhat inexact, because the calculations do not strictly apply to unconfined aquifers, the comparison is nonetheless meaningful and indicates that aquifer anisotropy at Piketon is of little importance in the design of vertical wells. Similarly, the vertical permeability had little effect on infiltration conditions. Though relatively low compared with horizontal permeability, the vertical permeability was considerably greater than the permeability of the streambed, which chiefly controlled the infiltration rate.

RECHARGE CONDITIONS

STREAMFLOW DEPLETION DUE TO PUMPING

The quantity of water diverted or induced to reenter the aquifer from the Scioto River during the 9-day test was determined from flow-net analysis and by a mathematical method developed by Theis (1941). The flow-net analysis served also as a check on the computed value of the coefficient of transmissibility. (For an explanation of flow-net methods, see Bennett in Ferris and others, 1962, p. 139–144.)

A flow net representing a plan view of the flow field in an aquifer consists of two families of curves, orthogonal to each other, forming roughly a system of rectangles. One family of curves, termed equipotential lines, represents contours of equal head. These contours are intersected orthogonally by flow lines, or stream lines, each of which represents the mean or average path of motion of a representative particle of water as it progresses through the aquifer. The flow net (pl. 4) was constructed from drawdown data collected after 9 days of pumping. The altitude of the water table was determined by averaging the drawdown in the deep and shallow observation wells. The position of each equipotential line was determined approximately from straight-line, semilogarithmic plots of average head versus distance from the pumped well. The position of each equipotential line, after its altitude was transferred to the flow net, was adjusted to obtain the best average fit for all wells.

In a homogeneous infinite aquifer receiving no recharge, the equipotential lines would form a series of concentric circles around a discharging well. Equipotential lines tangent to a recharge boundary such as an infiltrating stream are asymmetric with respect to a discharging well and reflect the steepening of the cone of depression on the side toward the stream. Each equipotential line on plate 4 is drawn as a circle whose center is on a line which is perpendicular to the hypothetical line source and which passes through the pumped well. The center of each successive circle of decreasing head is progressively closer to the pumped well on the land side of the perpendicular extending from the stream through the well. The flow lines, drawn as arcs of circles whose centers are on the hypothetical line source, were adjusted to the equipotential lines to produce, as nearly as possible, a regular system of rectangles.

The coefficient of transmissibility was calculated from the flow net by the formula

$$T=\frac{q}{\frac{nf}{nd}\times h},$$

where

T=coefficient of transmissibility, gallons per day per foot;
q=quantity of water being pumped, gallons per day;
nf=total number of flow paths;
nd=number of equipotential drops;
h=total decrease in head, feet.

Substituting values:

$$T=\frac{1{,}440{,}000 \text{ gpd}}{\frac{14}{4}\times 2 \text{ ft}}=206{,}000 \text{ gpd per ft.}$$

The coefficient of transmissibility determined from the flow net is practically the same as that determined by Rorabaugh's method; however, as the graphical technique is subject to variation in interpretation, such close agreement may be due in part to coincidence. From the same data an equally plausible appearing flow net could have been constructed to yield a slightly higher or lower value of the coefficient of transmissibility.

The chief purpose of the flow-net analysis was to determine the diversion or infiltration of water from the Scioto River during the 9-day test. The method is based on the fact that the quantity of water moving towards the pumped well between any two points on the flow net is proportional to the area of the flow field between these points. In the reach opposite the pumped well equal in length to twice the line-source distance, or 1,020 feet, approximately 581,000 gpd was being induced or diverted from the Scioto River after 9 days of pumping. This figure is the quantity of water moving through 5.65 of a total of 14 flow paths, as determined graphically from the flow net.

The percentage of water derived directly or indirectly from the Scioto River after 9 days of pumping was also determined mathematically from the following equation of Theis (1941, p. 735):

$$P=\frac{2}{\pi}\int_0^{\pi/2} e^{-k \sec^2 u}\, du$$

where

P=percentage of pumped water being derived from streamflow;
k=1.87 a^2s/Tt;
a=distance from well to line source of recharge, feet;
S=coefficient of storage, in percent;
T=coefficient of transmissibility, gallons per day per foot;
t=time since pumping began, in days;
u=arc tan x/a;
x=distance along line source measured from perpendicular intersecting well, in feet.

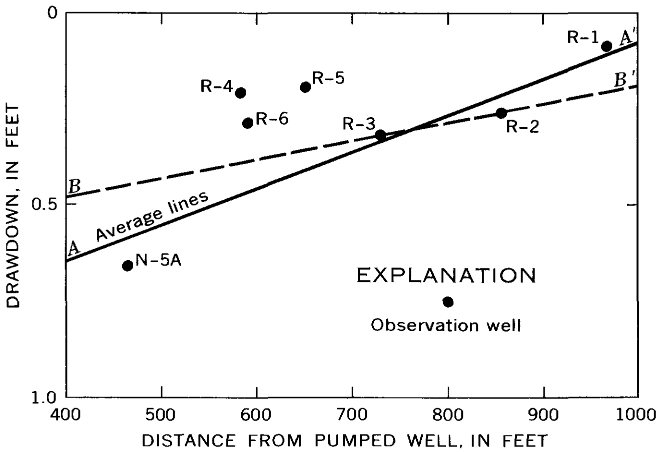
Theis presents a solution to this equation as a graph of values of P for various values of k. As determined from the graph, about 78 percent, or approximately 1.12 mgd, was from reduction of groundwater flow to the stream in combination with direct recharge through the streambed after 9 days of pumping. It can be shown mathematically from Theis' equation that an amount equal to a little more than half the quantity derived from the stream, or approximately 562,000 gpd, originated opposite the pumped well between points on the recharge boundary whose distance apart is equal to twice the line-source distance. Analysis by Theis' method thus agrees fairly closely with the value (581,000 gpd) determined from the flow net. The average value obtained by the two methods is 572,000 gpd, which represents the approximate quantity of water induced or diverted from the reach of the river opposite the pumped well equal in length to twice the line-source distance, or 1,020 feet. The measured area of the streambed in this reach is approximately 6 acres; thus, the unit infiltration rate at the end of the 9-day pumping period was about 95,000 gpd per acre.

PERMEABILITY OF THE STREAMBED

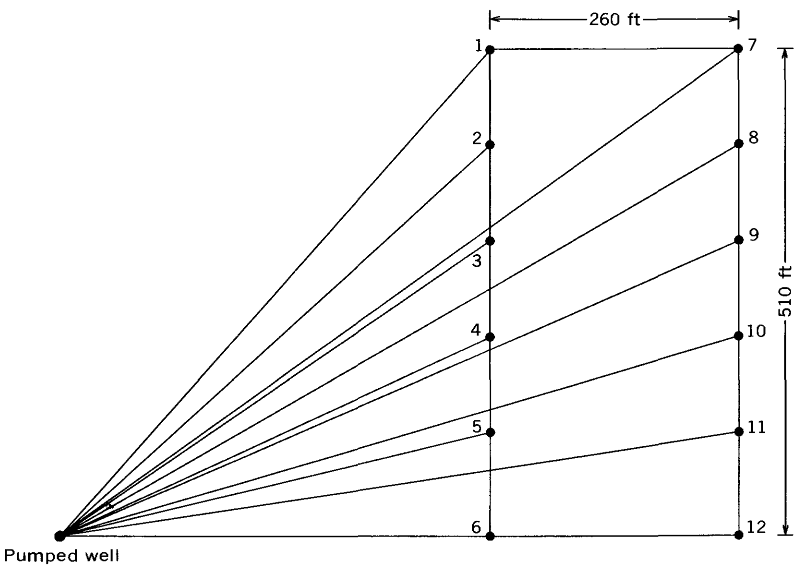
The unit infiltration rate was related to head loss under the river by (1) plotting values of drawdown in the riverbed drive-point wells against distance from the pumped well, (2) drawing a straight line through selected points—generally those representing the greatest drawdown, (3) plotting a regular grid of reference points on a map of the riverbed and finding the drawdown at each point by scaling its distance from the pumped well and referring to the drawdown-distance graph, and (4) averaging the drawdown at the reference points (fig. 18). Dividing the unit infiltration rate by the average drawdown under the streambed yielded a value expressing the relationship between the unit infiltration rate and the head difference between the stream level and the underlying water table.

The drawdowns at points beneath the river do not necessarily fit a distance-drawdown curve based on radial flow in the aquifer. In determining the position of average lines *A–A′* and *B–B′* (fig. 18*A*), it was assumed that most of the riverbed wells were relatively inefficient and did not respond fully to head changes in the streambed sediments near the top of the aquifer. For example, values of drawdown in drive-point wells R–4, R–5, and R–6 fall significantly above the position of lines *A–A′* and *B–B′*. Had these wells been ideally efficient, that is to say, had the wells been so effectively sealed where they penetrated the river bottom sediments that they prevented all flow between the river and the aquifer along the outside of the well pipe, it is assumed that the drawdown values would have been greater and would have more nearly fit on line *A–A′* or line *B–B′*. Two of the riverbed wells, R–7 and R–8, showed little or no drawdown during the test, and their records were not used in the analysis.

Of the two lines constructed in figure 18*A*, line *A–A′* gives special weight to the drawdown in well N–5A; line *B–B′* is based only upon the drawdown observed in wells R–2 and R–3, which were considered to have been the most efficient of the riverbed wells. Well N–5A, at the edge of the river and surrounded on three sides by water, responded essentially as expected of a well on the river line of wells, and the observed drawdown at the end of the 9-day test falls on the distance-drawdown curve for all other wells on this line. (See fig. 9.) Well N–5A, however, was within the region where vertical flow largely prevailed, and the drawdown is thought more nearly to reflect effects of the permeability of the streambed than of the aquifer. Observation



A. Drawdown in riverbed wells versus distance from pumped well



B. Idealized plan of part of streambed, showing selected reference points

FIGURE 18.—Drawdown in riverbed observation wells after 9 days of pumping and at selected reference points on an idealized plan of a part of the streambed.

wells R–2 and R–3 are believed to show the drawdown accurately for wells close to the center of the river.

The drawdowns for the selected reference points shown in figure 18*B* are tabulated below for lines *A–A′* and *B–B′*:

Reference point	*Distance* (r) *from pumped well, in feet*	*Drawdown from indicated lines in feet*	
		A–A′	B–B′
1	680	0.38	0.35
2	610	.45	.38
3	550	.51	.37
4	500	.55	.43
5	465	.58	.46
6	450	.61	.47
7	875	.20	.25
8	820	.25	.28
9	775	.29	.30
10	740	.32	.32
11	715	.35	.33
12	710	.36	.33
Average		0.40	0.35

The average drawdown determined from line *A–A′* is 0.40 feet; that determined for the same reference points from line *B–B′* is 0.35 feet. Relating the unit infiltration rate to the average drawdown beneath the stream yields values, respectively, of

$$I=\frac{95{,}000 \text{ gpd per acre}}{0.40 \text{ ft}}=0.235 \text{ mgd per acre per ft}$$

and

$$I=\frac{95{,}000 \text{ gpd per acre}}{0.35 \text{ ft}}=0.270 \text{ mgd per acre per ft,}$$

where I expresses the unit infiltration rate for each foot of head difference between the stream level and the underlying water table. The more conservative of these values, 0.235 mgd per acre per ft, was used as a minimum infiltration rate for design purposes.

The vertical permeability of the sediments immediately underlying the river, in the 5-foot-thick interval between the streambed and the top of the screens in the riverbed observation wells, was determined by the equation:

$$P_{vs}=\frac{Im}{A\Delta h}$$

in which

P_{vs}=coefficient of vertical permeability of the streambed sediments, gallons per day per square foot;

I=infiltration rate, gallons per day per acre;

m=thickness of sediments, feet;

A=area of infiltration, acres;

Δh=average difference between the stream level and the piezometric head in the riverbed wells, feet.

Substituting values:

$$P_{vs}=\frac{95{,}000 \text{ gpd x } 5 \text{ ft}}{43{,}560 \text{ sq ft x } 0.4 \text{ ft}}=27 \text{ gpd per sq ft.}$$

The vertical permeability of the streambed sediments is thus only about one-thirteenth as high as the vertical permeability (365 gpd per sq ft) measured across the full thickness of the aquifer. A layer of silt, mud, and organic debris, possibly having penetrated no more than a few inches into the underlying sediments, is thought to be chiefly responsible for the relatively low permeability of the streambed.

QUALITY OF WATER

CONSTITUENTS IN GROUND WATER AND SURFACE WATER

Chemical constituents in samples of ground water from the 12-inch-diameter well pumped during the aquifer test, from observation well E–1B, from each of the four 24-inch diameter production wells, and from one of the three original supply wells at the Portsmouth facility are given in table 1, and the chemical constituents of two water samples from the Scioto River, one of which was collected at a time of low flow and the other at a time of relatively high flow, are given for comparison. Analysis of a water sample from Lake White also is included in table 1. The water is of a calcium magnesium bicarbonate type and is considered to be moderately hard to very hard. Total dissolved solids in the ground-water samples range from 348 to 463 ppm (parts per million); total dissolved solids in the samples from the Scioto River were 337 ppm for the sample collected at relatively high flow and 468 ppm for the sample collected at low flow.

The chemical quality of the ground water can be expected to approach the average quality of the river over a period of time, as a progressively higher percentage of streamflow reaches the wells by induced infiltration. Chemical-quality data collected elsewhere in Ohio, under hydrologic conditions similar to those anticipated at Piketon, indicate that changes in chemical quality of the ground

TABLE 1.—*Chemical constituents in ground water and surface water in the Piketon area, Ohio*

[Results in parts per million except as indicated]

Source of water	Date of collection	Temperature (degrees F)	Silica (SiO_2)	Iron (Fe)	Manganese (Mn)	Calcium (Ca)	Magnesium (Mg)	Sodium (Na)	Potassium (K)	Bicarbonate (HCO_3)	Carbonate (CO_3)	Sulfate (SO_4)	Chloride (Cl)	Fluoride (F)	Nitrate (NO_3)	Dissolved solids (residue at 180°C)	Hardness as $CaCO_3$: Calcium, magnesium	Hardness as $CaCO_3$: Noncarbonate	Specific conductance (micromhos at 25°C)	pH	Color
Test well 12 in. diameter	10- 9-63 [1]	53	11	4.3	0.12	105	31	3.3	1.2	420	0	50	4.0	0.1	0.2	427	390	46	703	7.4	5
Do	10-23-63 [2]	53	11	4.1	.12	111	31	2.7	1.1	440	0	52	5.0	.2	.3	438	405	44	734	7.2	3
Observation well E-1B	9-26-63	53	11	4.6	52.	128	30	2.6	1.0	480	0	52	2.0	.0	.1	463	443	50	772	7.1	2
Production well 1	6-11-65 [3]	[4] 58	17	-----	-----	113	32	3.2	1.0	414	0	60	7.0	.1	2.3	452	414	74	724	7.4	10
Production well 2	6-18-65 [3]	[4] 54	16	-----	-----	114	36	3.6	1.1	449	0	52	5.0	.2	.4	440	433	64	736	7.5	10
Production well 3	6-25-65 [3]	[4] 53	15	-----	-----	103	30	3.4	1.0	400	0	46	6.0	.2	.2	418	381	52	663	7.5	2
Production well 4	7- 2-65 [3]	[4] 54	16	-----	-----	106	30	3.5	1.3	414	0	44	6.0	.2	.1	436	388	49	678	7.4	2
One of three original supply wells	9- 7-53	54	9.0	1.9	.26	76	34	4.4	1.6	348	0	48	3.8	.0	.3	348	328	----	597	7.2	0
Scioto River	5-15-63 [5]	------	4.6	2.5	.37	61	24	16	2.9	210	0	75	20	.3	8.4	337	251	78	540	7.1	20
Do	9-27-63 [6]	------	7.6	.87	.63	77	27	44	4.2	294	0	96	42	.4	.8	468	303	62	748	7.4	70
Lake White	5-15-63	68.5	5.5	.25	.06	10	6.6	3.7	2.3	18	0	40	4.0	.2	2.9	93	52	37	143	6.6	25

[1] After pumping about 5 hrs, at 300–1,000 gpm.
[2] After pumping 9 days, at 1,000 gpm.
[3] After pumping approximately 18 hrs, at 800 gpm.
[4] Temperature determined from flow through long discharge line.
[5] Discharge 3,400 cfs.
[6] Discharge 450 cfs.

water will be comparatively slow and probably predictable. Some idea of the magnitude, though not of the rate of change to be expected in the quality of the ground water, can be inferred from results of analyses made of water pumped under conditions analogous to those at Piketon, from a sand and gravel aquifer in the Great Miami River valley near Hamilton, Ohio. Here, since 1952, the Southwestern Ohio Water Co. has pumped 6–11 mgd from a ground-water collector located 750 feet from the Great Miami River. Although the quality of the ground water has become increasingly similar in certain of its properties to that of the Great Miami River, the changes have proceeded slowly at a steady, generally predictable rate; the ground water has not been subject to the wide and rapid changes in quality that characterize the river water. Given below are selected constituents, based on analyses by the U.S. Geological Survey, of water samples collected at various times since the Southwestern Ohio Water Co. began pumping. Also shown, for comparative purposes, are analyses of water from the Great Miami River at different stages. Chemical constituents are given in parts per million:

Date of analysis	*Sulfate* (SO_4)	*Chloride* (Cl)	*Hardness as* $CaCO_3$	*Dissolved solids*	*Temperature* (*°F.*)	*Discharge* (*cfs*)
			Ground-water collector 1			
7–11–52	38	5.5	288	335	54	----------
1–29–54	64	12	340	383	56	----------
11– 7–56	72	16	340	401	56	----------
3–27–57	75	21	360	420	56	----------
6– 4–58	79	16	339	410	56	----------
6– 4–63	82	24	354	423	59	----------
10–14–64	103	35	356	474	62	----------
2–16–65	121	38	380	486	63	----------
			Great Miami River at Hamilton			
[Mean annual discharge, 3,323 cfs, which is equaled or exceeded 25 percent of the time; flow equaled or exceeded 90 percent of the time, 450 cfs. (Cross and Hedges, 1959, p. 62, 147)]						
5–13–46	78	11	331	390	----------	3, 460
9–19–46	141	31	378	517	----------	496
10–11–49	102	18	360	438	----------	1, 060
10– 2–63	144	60	356	570	----------	280
3–18–64	83	23	264	341	----------	6, 800
4–15–64	99	26	316	423	----------	3, 140

The above table shows that, over the years, the sulfate content of the ground water has increased to a concentration approaching that of the Great Miami River at low discharges. Although part of the increase in sulfate may be due to movement of water into the aquifer from the underlying shale bedrock, most of the change represents the progressive increase in the quantity of water being induced from the river.

Changes in ground-water quality that are likely to occur at Piketon probably can be anticipated fairly soon after pumping becomes established, and after time is allowed for orderly and economic modifications of treatment practices as may be required. Probably the most significant of the differences between the chemical character of the surface and ground waters at Piketon, from the standpoint of treatment and well maintenance costs, are the higher sulfate content of the surface water and the higher iron content of the ground water. If the sulfate in the ground water should increase to such an extent that more chemicals would be required to treat the water, eventually the present large cost advantage in the use of ground water over surface water might be reduced somewhat. Iron in solution in the ground water that becomes partially oxidized as it enters the well may result in the precipitation of iron compounds on the well screens and in the interstices of the sand and gravel particles surrounding the well screens. This precipitation will contribute to well maintenance problems. To minimize effects of chemical-quality changes in the ground water, recommended distances between the production wells and the river are as large as possible, consistent with allowable drawdown. The larger the distance between the wells and the river, the greater the percentage of ground water derived from the landward side.

RESULTS OF CHEMICAL QUALITY AND TEMPERATURE MONITORING

Pumping during the 9-day aquifer test did not induce an appreciable quantity of river water into the aquifer, nor did river water advance far into the aquifer towards the pumped well. Prior to the test the piezometric surface in the riverbed wells was about 0.2 foot above river level. Only in the comparatively small area of the streambed where the drawdown exceeded this amount did actual infiltration of river water occur. As determined from line *A–A′*, figure 18*A*, the distance from the pumped well to the point beneath the streambed representing a drawdown of 0.2 foot is approximately 875 feet. The area of the streambed opposite the pumped well bounded by a circle whose radius is 875 feet and whose center is at the pumped well is approximately 7.8 acres. Within the 7.8-acre area the piezometric surface was generally below river level, and infiltration of river water theoretically could have occurred. The average difference in head between the river and the piezometric surface in this area, determined by averaging the head difference at selected reference points, was 0.16 foot. At the end of 9 days, the quantity of river water entering the aquifer in the 7.8-acre area was approximately 292,000 gpd; based on an infiltration rate of 0.235 mgd per acre per ft, this quantity, amounts to about 20 percent of the water being pumped. Thus, most recharge to the pumped well represented reduction of ground-water flow to the river rather than direct recharge through the streambed. Although this analysis is highly generalized, the result, though inexact, is probably of the correct order of magnitude.

The total quantity of water that entered the aquifer during the 9-day test is small by any estimate compared with the volume of the sediments in the cone of depression. To illustrate, if 300,000 gallons of river water had entered the aquifer during each of the 9 days of the test, the total quantity, 2,700,000 gallons, could be stored in a strip of the aquifer 1,000 feet long—which is approximately twice the line-source distance—and only 28 feet wide.

The average velocity of ground-water flow is calculated from the formula,

$$V=\frac{PI}{7.48p},$$

where

V=velocity of flow, feet per day;
P=coefficient of permeability, gallons per day per square foot;
I=hydraulic gradient, foot per foot;
p=porosity of aquifer; assumed to be 30 percent.

The hydraulic gradient in the vicinity of the river, along the perpendicular between the river and the pumped well, was approximately 1 foot per 315 feet after 9 days of pumping, as determined from the flow net, plate 4.

Substituting values:

$$V=\frac{3{,}300\times\frac{1}{315}}{7.48\times0.30}=4.67 \text{ ft per day.}$$

Thus, in 9 days and under the hydraulic gradient prevailing at the end of the test, river water would have moved shoreward only about 40 feet along the most direct route to the pumped well. As the gradient during most of the aquifer test was less than that indicated by the flow net, it is evident that river water did not move far into the aquifer unless, by chance, it followed highly preferential flow paths.

As might be inferred from the foregoing discussion, chemical-quality changes in the aquifer were slight during the 9-day test. Changes in selected chemical constituents during the test are shown in table 2, which is based on analyses of samples collected at various times from the pumped well and the 6-inch-diameter observation wells, screened at the bottom of the aquifer. For comparison, samples were collected simultaneously from the Scioto River.

The ground-water samples show a progressive increase in alkalinity and hardness; changes in chloride and sulfate are less marked. Exceptions to the general pattern of water-quality changes due to pumping are indicated by samples from wells N–1, N–2, and N–4, which are located on the river line. The samples showed a slight decrease in alkalinity and hardness, the trend suggesting the possibility of mixing of river water with the well water. However, only by relatively rapid movement along local and highly preferential flow paths could river water have reached these wells during the 9-day test. This possibility is deemed slight, and it is not suggested by results of temperature monitoring.

Chiefly, the changes that did take place represented the downward movement of relatively hard water which occurred near the top of the aquifer. This slight difference in chemical quality of water near the top of the aquifer is shown by a comparison of analyses of water from observation well E–1B, which is relatively shallow, with analyses from the 12-inch-diameter pumped well (table 2). Water from well E–1B was about 8 percent higher in dissolved solids and about 14 percent higher in total hardness than water from the pumped well prior to the start of the aquifer test.

During the aquifer test, temperature measurements were made nearly every day in the river and in the observation wells at 5-foot intervals of depth, using a thermistor-type electric thermometer accurate to approximately 0.2° F. Temperature changes during the 9-day test were insignificant. Prior to and during the test the water temperature in the upper few feet of the aquifer was 1° or 2° colder than water in the lower part of the aquifer. Most temperature changes observed during the test reflected the downward movement of the colder water. The largest observed temperature change occurred in well N–1, 10 feet from the pumped well, between depths of 50 and 65 feet. The average temperature in this zone declined from 51.6° F at the start of the test to 50.4° F at the end of the test.

Table 2.—*Chemical constituents in ground*

[Analyses by Goodyear Atomic

Sampling point	Alkalinity as $CaCO_3$								
	October 1963								Percent change 10/13–22/63
	3	13	14	15	17	19	21	22	
Well N-1	295	328			320	316		300	−8.5
2	306	328	310	308	230	310	316	306	−6.7
3	260	248	240	244		238	260	256	+3.2
4	200	202	202	204	200	204	200	198	−2.0
Well W-1	308	290	310	328	320	326	342	328	+13.0
2	304	320	314	322	332	332	334	330	+3.1
3	274	314	306	278	310	330	344	336	+7.0
Well E-1B	390	424							
2	286	324	312	324	328	332	346	340	+4.9
4	306	292	280	286	300	308	318	310	+6.2
Well S-1	288	286	308	310	322	340	360	352	+22.0
Pumped well			346	354	350	356	362	356	[1] +2.9
Scioto River	248	264		270	216	262	268	266	+0.8

[1] Oct. 14-22; 9-day test started Oct. 14, 1963.

The temperature of the river water ranged between 59° F and 61° F, or 6° to 8° above the average ground-water temperature. Despite the higher river temperature the ground-water temperature in well N–5C, a drive-point well about 70 feet deep at the edge of the river, was virtually unaffected. Only at two depth zones in this well did the temperature of the ground water rise. At the depth of 20 feet the temperature rise was 0.7° F, and at 30 feet the rise was 0.3° F between October 13 and 23. At other depth zones the ground-water temperature in well N–5C declined slightly during the test. The temperature data provide further evidence that relatively little river water entered the aquifer during the 9-day test.

RECOMMENDATIONS FOR DRILLING PRODUCTION WELLS

Recommendations based on the results of the test drilling and aquifer test were presented to the Atomic Energy Commission for the location and construction of wells. Although criteria pertaining to vertical wells were submitted, it was stated that either vertical wells or horizontal collector wells would be feasible as long as (1) the wells were placed sufficiently close to the river and far enough apart to avoid excessive drawdown, (2) the rate of pumping was not consistently to exceed the infiltration capacity of the streambed, and (3) well loss was kept to a minimum by proper construction, development, and maintenance.

Three alternative plans were presented. Plans *A* and *B*, each requiring 10 wells, were designed to yield 20 mgd. Plan *C*, developed in anticipation of a reduction in water use at the Portsmouth facility, was for an array of six wells designed to yield a total of 14 mgd. Plan *A*, calling for 1,400 gpm wells, and Plan *C*, calling for 1,620 gpm wells,

water and surface water in aquifer test area

Corp., in parts per million]

Hardness as $CaCO_3$									Chloride (Cl)								Sulfate (SO_4)			
October 1963								Percent change	October 1963											
3	13	14	15	17	19	21	22	10/13-22/63	3	13	14	15	17	19	21	22	3	13	14	15
342	359	----	----	356	354	----	339	−5.5	3	3	----	----	6	4	----	4	61	47	----	----
351	326	342	349	268	351	349	340	+4.3	3	5	3	4	6	3	3	3	45	45	47	45
292	278	274	280	----	275	280	286	+2.9	5	6	5	6	----	6	6	5	37	43	43	44
233	235	231	231	229	224	224	221	−6.0	8	6	8	7	11	8	9	8	31	36	34	31
363	348	352	363	364	364	351	364	+4.6	5	3	4	5	5	3	3	4	50	56	55	68
347	364	332	356	378	374	371	376	+3.3	3	3	4	3	4	4	3	3	52	49	53	52
320	324	323	316	352	373	378	378	+17.0	4	5	5	4	5	5	4	2	51	46	48	44
439	468	----	----	----	----	----	----	----	8	4	----	----	----	----	----	----	57	54	----	----
333	353	361	364	373	378	385	392	+11.0	5	3	3	2	5	3	4	3	51	48	50	51
349	331	327	325	345	344	361	349	+5.4	3	3	4	4	6	3	3	3	40	48	50	48
337	318	351	354	368	386	388	402	+26.0	4	3	4	4	4	4	3	5	50	53	51	72
----	----	393	398	402	398	405	402	[1] +2.3	----	----	4	4	6	3	4	4	----	----	52	50
316	338	----	337	280	327	333	332	−1.8	47	54	----	54	41	54	56	52	160	140	138	129

specified that the wells be drilled in the incised bedrock channel in the general area of the aquifer-test site. Plan *B*, designed to reduce pipeline costs, called for five wells, yielding 1,800 gpm each, to be drilled in the incised channel and for five shallower wells, yielding 1,000 gpm each, to be drilled along the river between the incised channel and the X–608 pumphouse.

Calculation of yield and drawdown was based on a formula by Rorabaugh (1956, p. 156, eq. 30), as follows:

$$Q=\frac{\frac{m_1+m_2}{m_1}\pi Ts}{2.30\log\left\{\left[\frac{2x}{r_w}\right]\left[1+\left(\frac{2x}{d}\right)^2\right]\left[1+\left(\frac{2x}{2d}\right)^2\right]\cdots\left[1+\left(\frac{2x}{nd}\right)^2\right]\right\}}$$

where

Q=gallons per day per well.

T=coefficient of transmissibility; assumed, conservatively, to be 200,000 gallons per day per foot in the incised channel and 128,000 gallons per day per foot between the incised channel and the X–608 pumphouse.

m_1=saturated thickness of aquifer prior to pumping, in feet; assumed to be 60 feet in the incised channel and 40 feet between the incised channel and the X–608 pumphouse.

m_2=saturated thickness of aquifer at pumped well during pumping, in feet.

s=drawdown in aquifer outside well, in feet.

x=distance from the center of the well screen to the line source, in feet.

d=well spacing, in feet.

r_w=radius of well, in feet; taken as 24 inches for wells in the incised channel and 18 inches for wells in the shallower part part of the aquifer.

n=number of intervals between wells.

In the aquifer-test the average width of the Scioto River was 260 feet, and the average depth was 3.6 feet. Assuming all water to be pumped will be derived from filtration through the streambed, the minimum well spacing for a line of wells parallel to the river is

$$L=\frac{Q}{WDI},$$

where

L=distance between wells, in feet;

Q=pumping rate, million gallons per day;

W=width of river in feet;

D=average depth of river in feet:

I=infiltration rate, million gallons per day per acre per foot.

For wells yielding 2 mgd (1,400 gpm), the equation is

$$L=\frac{2 \text{ mgd} \times 43{,}650 \text{ ft}^2 \text{ per acre}}{260 \text{ ft} \times 3.6 \text{ ft} \times 0.235 \text{ mgd per acre per ft}}=400 \text{ ft.}$$

For wells yielding 1.44 mgd (1,000 gpm) and wells yielding 2.6 mgd (1,800 gpm), the minimum spacing is 285 and 500 feet, respectively.

Because the width and depth of the Scioto River in the test area seem to be representative of the entire reach below the X–608 pumphouse at least as far as the first major bend in the river, a constant relationship between stream length and infiltration capacity was assumed in determining minimum well spacing.

In calculating the yield of the wells, the position of the line source was taken as the far bank of the river for wells in the incised channel and the middle of the river for wells in the shallower part of the aquifer. These hypothetical positions were assumed to represent conditions in periods of minimum recharge coincident with maximum pumping. Recommendations called for wells in the incised channel to be located 200–250 feet from the river bank, and wells in the shallower part of the aquifer to be located 100 feet from the bank. Values of drawdowns substituted in the calculations were 22 and 24 feet for the wells in the incised channel and 15 feet for the shallower wells. To minimize partial penetration losses, recommended screen lengths were 30 feet for the deeper wells and 20 feet for the shallower wells; the lengths were chosen to be as long as practicable and to be consistent with expected drawdown.

When constructing wells in a typical sand and gravel aquifer, selection of the screen slot size is commonly based on the sieve size that will retain 40 percent of the material. Sieve analyses of samples taken from wells drilled for the 1963 aquifer test were made by the Goodyear Atomic Corp.; they found that for most wells below 50 feet the sieve size was 0.156 inch or larger. A No. 150 slot size (0.150 inch), therefore, could have been used to develop most of the wells.

The slot size, combined with the length and diameter of various commercially available screens, must be such that the velocity of water entering the well is low enough to prevent entrance of an excessive amount of sand and to keep friction losses and rates of incrustation and corrosion to a minimum. Recommended maximum entrance velocity is 6 feet per minute (The Johnson Drillers Journal, 1963, p. 2). Hypothetical screen velocities calculated for slot size openings of 0.080–0.150 inch fell below this limit for all wells in the three recommended plans.

DRILLING AND TESTING PRODUCTION WELLS

WELL LOCATION AND CONSTRUCTION

On the basis of the recommendations of the Geological Survey, but in line with a substantial reduction in the demand for water, the Goodyear Atomic Corp., in 1965, contracted for the drilling of four wells designed to yield 1,650 gpm each, or a total of 9.5 mgd. The U.S. Atomic Energy Commission requested the Geological Survey to monitor the drilling and test pumping of the wells and to report on their performance and adequacy.

Drilling of the production wells started about April 30 and was completed about June 30, 1965. Because of local irregularities in the depth to bedrock, two of the wells were a few feet shallower than anticipated. Also, the need to avoid drilling wells on flood-protection levees caused the intervals between wells and their distances from the river to deviate somewhat from the recommended plan. Depth and spacing of the wells are tabulated below:

Well	*Elevation of top well casing, approx 7½ ft below land surface*	*Depth from land surface (ft)*	*Distance from river (ft)*	*Distance between wells (ft) (1)*	*(2)*	*(3)*	*(4)*
1	542.7	72	205	----			
2	541.2	74	338	380	----		
3	544.1	81	255	880	510	----	
4	543.0	80	252	1,300	940	450	----

Well 1 was located as close as possible to the east side of the incised channel to minimize the length of pipeline between the wells and the X–608 pumphouse (fig. 19). Well 4, the well farthest from the X–608 pumphouse, is approximately in the middle of the incised channel and almost exactly at the former site of test well N–3, which was drilled for the 1963 aquifer test.

Each well is cased with 24-inch-diameter steel pipe and is finished with 30 feet of brass well screen. A submersible pump is installed in each well; a 10-inch discharge line leads from the well through the pit wall to the pipeline.

CHARACTER OF THE AQUIFER

A 6-inch-diameter test hole was drilled in advance at each of the four sites initially selected for a production well. Sand and gravel samples representing the materials in depth intervals of about 5 feet were collected by the driller for particle-size analysis. Although the full thickness of the aquifer was sampled in each of the four 6-inch test holes, the portion to be screened was sampled again when the 24-inch production wells were drilled. As in the 1963 investigation, drilling was by the cable-tool method and the procedure was to alternately drive the casing a few feet into the material and bail out the sand and

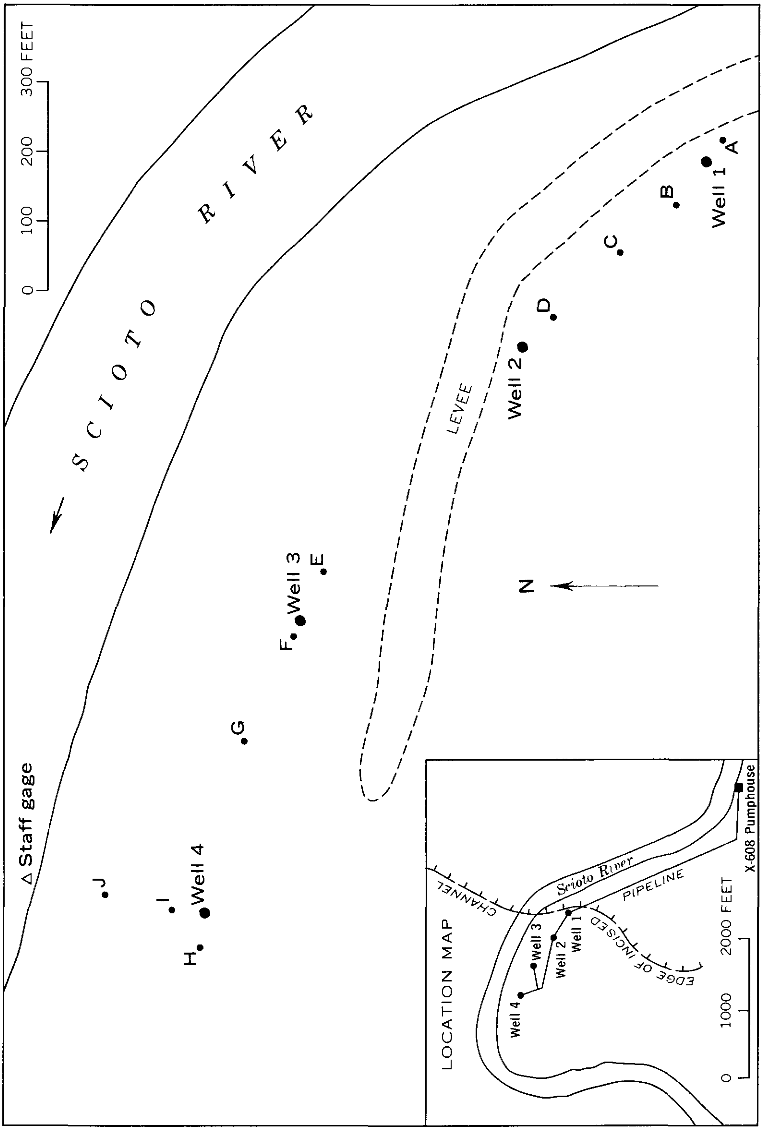


FIGURE 19.—Location of production wells (numbers) and drive-point observation wells (letters).

gravel from within it. Water was poured into the casing from time to time, as the drilling progressed, to prevent the heaving of the material into the casing. This procedure was adopted following the drilling of test well 1, when it became evident that heaving of the material was occurring.

The aquifer materials are relatively fine grained at the site of well 1, ranging from medium to coarse sand, with a minor quantity of fine gravel. The aquifer materials at the other three well sites are relatively coarse, mostly ranging from coarse sand to medium gravel.

Samples from the four sites confirm the 1963 findings relative to the character of the aquifer. Interbedded with the coarser grained sand and gravel, constituting the bulk of the aquifer, are layers a few feet to several feet thick of uniform, relatively fine-grained material. Most of the fine-grained material occurs near the middle of the aquifer at, or a few feet above the tops of the screens in the production wells. A relatively thin zone of the finer grained material also occurs a few feet above the bedrock.

During the 1963 investigation, efforts were made to relate the particle-size distribution of the samples to the permeability of the aquifer. Several parameters of the particle-size distribution curves were studied, including the effective size (identified as the particle diameter such that 10 percent of the particles by weight are of smaller diameter), the 60 percent finer size, and the uniformity coefficient. The uniformity coefficient indicates the degree of sorting and is defined as the ratio of the 60 percent finer size to the effective size.

A low value of the uniformity coefficient, denoted by a steeply sloping particle-size curve, indicates a comparatively well-sorted mixture of uniform grain size. Ordinarily, a well-sorted deposit is more permeable than a mixture having less uniform grain size; however, it was found that the higher values of the uniformity coefficient—averaging 13.9—were associated with the more permeable, coarser grained material, and the lower values—averaging about 6—with the less permeable, finer grainer material. Using the trial and error method, it was found that the uniformity coefficient of the samples multiplied by a factor ranging between 250 and 275 yielded reasonable values for the coefficient of permeability.

From the values determined by this method, the average coefficient of permeability of the less permeable material was estimated to range from 1,500 to 1,650 gpd per sq ft and that of the more permeable material from 3,480 to about 3,820 gpd per sq ft. When these values were weighted by multiplying them by their respective depth intervals, a value was obtained for the average coefficient of transmissibility which closely approximated the value, about 215,000 gpd per ft, determined from the 9-day aquifer test. The method used in this analysis and the results obtained are described by the authors in a paper published by the U.S. Geological Survey (Norris and Fidler, 1965).

The results obtained by applying the empirical method to sample data from the four production well sites show little consistency with results obtained from the 1963 data. Using 275 for the value of the arbitrary multiple, computed values for the coefficient of transmissibility for the 6-inch-diameter test wells are 102,000 gpd per ft for test well 1; 180,000 gpd per ft for test well 2; 140,000 gpd per ft for test well 3; and 136,000 gpd per ft for test well 4. These values are low compared with results obtained from the 24-hour discharge tests of the production wells. The inconsistency between the respective sets of values, compared with the good agreement obtained in the 1963 investigation between sample analysis and pumping test results, means that the samples are not strictly comparable; the samples probably reflect differences in the sampling techniques of the respective drillers.

SELECTION OF SCREEN SLOT SIZES

The screen slot sizes for three of the production wells were based on the 30 percent retained size; the slot sizes for well 1 were based on retained sizes ranging from 30 to 40 percent. Except for well 3, the samples from the 24-inch production wells were used by the project engineer in determining the screen slot sizes. The materials at the site of well 3 were relatively clean and well sorted and considered so favorable for well development that samples from the 6-inch test hole were used in selecting the slot sizes for the production well.

The screens were installed by the pull-back method, in which the well casing is sunk to the full depth of the well, the screen is lowered to the bottom and the casing is then pulled back, exposing the screen to the formation. The wells were developed by surging with a surge block attached to the drilling tool. The specifications required that development be continued until "30 minutes of surging brings in less than 6 inches of material to be bailed out." Tabulated below are the screen characteristics of the production wells:

Well	*Depth interval (ft)*	*Slot No.*[1]	*Open area per foot of 24 in OD screen (sq in)*	*Incremental open area (sq in)*
1	42–50	80	266	2, 128
	50–58	60	217	1, 736
	58–64	80	266	1, 596
	64–69	20	87	435
	69–70	40	158	158
	70–72	60	217	434
	72–74	Blank		
Total				6, 487
2	44–52	40	158	1, 264
	52–54	80	266	532
	54–74	150	389	7, 780
Total				9, 576
3	51–60	150	389	3, 501
	60–65	60	217	1, 085
	65–67	100	307	614
	67–81	150	389	5, 446
Total				10, 646
4	50–70	150	389	7, 780
	70–75	100	307	1, 535
	75–80	80	266	1, 330
Total				10, 645

[1] Designates widths of openings in thousandths of an inch.

Standard methods were used in constructing production wells 1 and 3; modifications were ordered by the project engineer in constructing wells 2 and 4. In sinking the casing for well 2, a large hole was bailed between depths of 37 and 54 feet, bringing down relatively coarse material from the depth zone 35–40 feet to fill the annular space around the well screen. This method, similar to that which the driller used in constructing the well pumped in the 1963 aquifer test, permitted use of a larger screen slot size in this depth interval than otherwise would have been warranted. The land surface immediately surrounding the well casing subsided somewhat during the bailing of the well, and the resulting depression was filled with the bailed-out material. Long clamps were affixed to the well casing to prevent it from sinking too fast during the bailing process. A similar procedure was used in constructing well 4, with relatively coarse material being brought down by bailing in the depth interval from 55 to 72 feet.

If the pumping rate is the criterion that maximum entrance velocity should not exceed 6 feet per minute, production well 1 could be pumped as high as 2,000 gpm; well 2—2,970 gpm; wells 3 and 4—3,300 gpm. These hypothetical pumping rates are well above those for which the production wells are designed; actual entrance velocities, therefore, should be very much on the "safe" side, with respect to keeping friction losses and rates of incrustation to a minimum.

TWENTY-FOUR HOUR DISCHARGE TESTS

Preparatory to testing the production wells, drive-point observation wells, designated by letters A through J, were installed at selected locations, all but two being on a line parallel with the river (fig. 19). The distances, in feet, from the production wells to the drive-point wells and the river are given in the following table:

Well	Drive-point wells										River
	A	B	C	D	E	F	G	H	I	J	
1	40	75	183	317							205
2	420	306	200	65	435						338
3					75	25	198	500	460	490	255
4					525	425	258	49	50	150	252

Open approximately in the middle of the saturated portion of the aquifer, the drive-point wells were developed with a pitcher pump to insure their quick response to changes in ground-water levels.

The specifications provided for a discharge test of 24 hours duration at a constant rate of 800 gpm for each well. The procedure was changed somewhat in the field; the 800 gpm rate was held constant for about 22 hours and then was raised to 1,000 and 1,200 gpm, respectively, for each of the final 2 hours to provide data for computation of well loss.

During the tests, periodic depth to water measurements were made in the well being pumped and in the drive-point wells. A staff gage, installed in the Scioto River, was also read periodically. The river stage remained essentially constant during all the tests, and ground-water levels were relatively stable, contributing to the excellent environmental conditions under which the tests were made. Drawdowns, in feet, in the drive-point observation wells at the end of the 800 gpm discharge tests, unadjusted for dewatering, are tabulated below:

Well pumped	Drive-point wells										Change in river stage [1] (ft)
	A	B	C	D	E	F	G	H	I	J	
1	3.26	2.26	1.31	0.79							−0.14
2	.57	.82	1.23	2.29							−.15
3					2.13	2.81	1.03	0.34			−.01
4					.32	.39	.78	2.33	2.20	1.01	−.01

[1] Effect of change in river stage assumed negligible in all tests.

From each of the tests, measurements of drawdown in the drive-point wells closest to the pumped well, after being adjusted to compensate for the slight dewatering of the aquifer caused by pumping, were plotted on logarithmic coordinate paper versus the square of the distance from the pumped well. By matching these data to the Theis nonequilibrium type curve (see Ferris and others, 1962, p. 92–98), an approximate value for the coefficient of transmissibility was obtained at each of the production well sites. Also, by standard methods of analysis, the distance to the line source of recharge and the well-loss coefficient were determined (Rorabaugh, M. I., 1956, p. 149–152). Values of these coefficients and additional data relative to the aquifer tests are listed in table 3.

Although the discharge tests of the production wells were of short duration, the aquifer coefficients which were determined from the data are believed to be close to their true value. As indicated in table 3, the coefficient of transmissibility is lowest at well 1 and highest at well 4, which is in the deepest part of the channel.

Well loss was low in all wells, but highest in well 2. Well loss may become important at high pumping rates as the drawdown due to well loss is proportional approximately to the square of the discharge. For example, the part of the drawdown due to well loss in well 1 is 0.50 foot at a pumping rate of 800 gpm ($7.84\times10^{-7}\times800^2$) and 2.1 feet at a pumping rate of 1,650 gpm ($7.84\times10^{-7}\times1{,}650^2$).

EXPECTED YIELD OF WELLS

Before the four production wells were drilled, tentative plans called for a 1,650-gpm capacity pump in each well and a total yield of 9.5 mgd. Because of differences in well performance during the 24-hour discharge tests, these plans were modified. Presently, pumps of 1,650 gpm capacity are installed in wells 1 and 2 and pumps of 2,400 gpm capacity are installed in wells 3 and 4; all pumps are rated against a 300-foot head. Wells 1 and 2 will be pumped at the rate of 1,300 gpm each and wells 3 and 4 at rates up to 2,400 gpm each, for a total yield of 10.4 mgd.

The transmissibility values determined from the 24-hour discharge tests were used in computing the drawdowns to be expected at these pumping rates under near minimal conditions of infiltration. The estimated components of drawdown due to partial-penetration effects, well loss, and dewatering were considered. In this analysis, it was assumed that partial-penetration losses would be directly proportional to values determined for each well during the 24-hour tests at the 800 gpm pumping rate. These values were arrived at by (1) computing the theoretical drawdown in each well, using the coefficient of transmissibility and the line-source distance that were determined from drawdowns in the drive-point observation wells, (2) adding to the theoretical drawdown the components of drawdown due to well loss and de-

TABLE 3.—*Characteristics of wells and aquifer properties determined from the 24-hour discharge tests*

Well	Date	Depth to water below land surface[1] (ft)	Pumping rate (gpm)	Duration of pumping (hr)	Duration of pumping (min)	Observed drawdown (ft)	Specific capacity (gpm per ft)	Well loss (ft)	Well-loss coefficient (ft per gpm²)	Distance to line source (ft)	Coefficient of transmissibility (gpd per ft)	Coefficient of permeability (gpd per ft²)
1	6-10-65	16.4	800	21	36	11.07	72.3	0.50	7.84×10^{-7}	280	160.000	2,860
1	6-10-65		1,015	1	1	14.05						
1	6-10-65		1,200	1	8	16.58						
2[2]	6-17-65	14.6	801	22	0	7.60	105.2	.90	1.41×10^{-6}	340	191,000	3,240
2	6-17-65		1,003	1	0	9.57						
2	6-17-65		1,200	1	0	11.61						
3[3]	6-24-65	19.6	800	23	6	7.09	113.0	.50	7.82×10^{-7}	280	190,000	3,130
3	6-24-65		1,000	1	0	8.13						
3	6-24-65		1,200	1	3	9.94						
4[4]	7- 1-65	17.8	800	22	0	7.95	100.5	.52	8.09×10^{-7}	310	209,000	3,370
4	7- 1-65		1,006	1	0	9.27						
4	7- 1-65		1,200	1	5	11.24						

[1] Under nonpumping conditions.
[2] Well on for 13 min approximately 1 hr prior to test.
[3] Well on for 35 min approximately 2 hr prior to test.
[4] Well on for 15 min approximately 1 hr prior to test.

watering, and (3) deducting the sum of these values from the observed drawdown in each well.

The following table shows the estimated drawdowns under the present pumping plan. The calculations are based on a line-source distance of 500 feet, a position approximately at the far bank of the river:

Well	*Estimated allowable drawdown* [1] *(ft)*	*Pumping rate (gpm)*	*Estimated drawdown* [2] *(ft)*
1	26	1, 300	23
2	29	1, 300	20
3	31	2, 400	33
4	32	2, 400	33
Total		7, 400 (10.4 mgd).	

[1] To top of screen, based on measurements made in June 1965.
[2] With all wells pumping.

Although the table shows that under assumed minimal conditions of recharge the estimated drawdown in wells 3 and 4 will be such that slight unwatering of the screens might occur, drawdowns will be less than those indicated during most of each year when ground-water levels are above their seasonal lows and infiltration conditions are relatively good.

As the operating characteristics and performance of the four production wells can be determined precisely only from field data collected under actual operating conditions, a comprehensive monitoring program was recommended. The advantage of maintaining systematic records of pumpage, ground-water levels, chemical quality, and temperature changes are obvious. Such records will give ample warning of overpumping or of possible deterioration of the wells caused by incrustation or sand pumping, and thus contribute to an orderly and effective maintenance program.

In the Piketon investigation a unique opportunity has been afforded to compare well performance and aquifer response to predictions based on a controlled discharge test. The results, together with other hydrologic data that will be collected as pumping continues, will be increasingly useful as a case history record of the well field.

FUTURE GROUND-WATER DEVELOPMENT IN THE LOWER SCIOTO RIVER VALLEY

Extending from Columbus south to the Ohio River valley at Portsmouth and constituting an essentially similar hydrogeologic environment over most of its approximately 100-mile length, the Scioto River valley watercourse aquifer system is virtually the only source of large industrial and municipal ground-water supply in the basin. Compared with other watercourse aquifers in the State, the Scioto River valley aquifer system has been only moderately developed. The Mead Paper Co. at Chillicothe, the largest industrial water user, pumps approximately 40 mgd from large-diameter collector wells in the outwash deposits. Chillicothe, a city of about 25,000, gets its water supply from wells tapping the same source. Smaller users include nearly all the municipalities and many industrial plants in the lower Scioto River valley.

Walker, Schmidt, Stein, Prée, and Bailey (1965) discuss the present meager development of the Scioto River valley aquifer and state that of the State's four principal watercourse aquifer systems, namely those of the Great Miami, Scioto, Tuscarawas, and Ohio River valleys, only the outwash aquifer system in the Great Miami River valley has been extensively developed; it accounts for almost half the total ground-water use in Ohio. The authors call attention to promising undeveloped areas in the Scioto River valley where conditions for development of ground-water supplies are as favorable as in much of the Great Miami River valley. They state that "the large ground-water potential of the Scioto River valley, combined with other favorable factors, should ultimately result in important industrial development."

Considerable economic growth will occur in the Scioto basin in the years immediately ahead, according to forecasts made by the Division of Economic Research of the Ohio Department of Development. Citing the wide diversification of manufacturing and the growth of industry in and around Columbus, Division researchers (Bryant and Buttress, 1964) estimate that the population in the "Scioto Valley Region," one of eight economic regions into which the State has been divided, will increase more than 45 percent between 1960 and 1980—the greatest rate of growth projected for any region in the State. Although Pike County (which includes Piketon) and Scioto County to the south, lie just outside the Division of Economic Research's 10-county Scioto Valley Region, there is little doubt that accelerated economic development and population growth will inevitably involve the entire watercourse aquifer system below Columbus.

According to a report published by the U.S. Army Corps of Engineers, Huntington, W. Va. (1962), over the past 60 years the population of the Scioto basin has increased at an annual rate of 2 percent, as compared with 1.6 percent for the State and 1.5 percent for the United States. The population in the basin is expected to reach 1,920,000 by 1985.

The growth of population and industry will add a new dimension to the problem which caused the Atomic Energy Commission officials to change from a surface-water to a ground-water supply at the Piketon site—the deterioration in quality of the Scioto River water. Although much progress has been made in the adequate treatment of municipal and industrial wastes in the Scioto basin, many such improvements have barely kept pace or have lagged behind the rate of economic and population growth.

A report by the Ohio Division of Water (1963, p. 24–25) discusses the general problem of pollution abatement in the basin and states that of 43 municipalities, 29 villages with a combined population of 22,115 are without sewage treatment facilities. Municipal wastes, however, are of minor importance compared with industrial wastes discharged directly to the streams by 30 plants. The report states: "Far more significant are the problems created by wastes from the other 30 establishments: 15 organic and 15 inorganic. Seven of the 15 organic wastes create an aggregate load of 330,000 population equivalent, calculated on a biochemical oxygen demand basis. Major portion of this load is attributed to three paper mills."

Although the report states that significant load reductions have been achieved and additional corrective measures are being planned, it is evident that serious problems remain in the continuing battle for pollution abatement.

CONCLUSIONS

This investigation has shown that one of Ohio's most important watercourse aquifers, sand and gravel outwash in the Scioto River valley, will yield large quantities of ground water at a typical site, the quantity being sustained by the induced infiltration of streamflow. Values of the coefficient of transmissibility, 215,000 gallons per day per foot, and the rate of streambed infiltration, 0.235 million gallons per day per acre per foot, were determined from a 9-day aquifer test made at a time of near-minimum flow. These figures proved to be a valid basis for design criteria for the location, spacing, and construction of four supply wells.

Because the four supply wells performed about as predicted, the hydraulic properties of the aquifer, as determined by standard methods, can be taken as fairly representative. Aquifer properties determined at Piketon, therefore, can be extrapolated with some confidence to make quantitative estimates of ground water availability at other locations in the same hydrogeologic environment.

Thus, the Piketon investigation, an example of a systematic approach to problems associated with development of a large ground-water supply, should add significantly to knowledge of the watercourse aquifer system in the lower Scioto River valley. Moreover, a description of the analytical techniques used to evaluate the aquifer-test data and to determine the hydraulic properties of the sand and gravel aquifer should be a useful guide to hydrologists making investigations of watercourse aquifer systems in other areas.

In all liklihood the development of ground-water supplies in the lower Scioto River valley will increase significantly in the years ahead. Not only will the growth of industry and population demand more water, but it seems highly probable that for many years to come the superior quality of the ground water over that of surface water in the Scioto River valley will commend the ground-water resources to the attention of industry and municipal officials in their future plans for development.

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