Induced Recharge of an Artesian Glacial-Drift Aquifer at Kalamazoo Michigan

GEOLOGICAL SURVEY WATER-SUPPLY PAPER 1594-D

Prepared in cooperation with the city of Kalamazoo and the Michigan Geological Survey

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By J. E. REED, MORRIS DEUTSCH, *and* S. W. WIITALA

ARTIFICIAL RECHARGE OF GROUND WATER

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ARTIFICIAL RECHARGE OF GROUND WATER

INDUCED RECHARGE OF AN ARTESIAN GLACIAL-DRIFT AQUIFER AT KALAMAZOO, MICHIGAN

By J. E. Reed, Morris Deutsch, and S. W. Wiitala

ABSTRACT

As part of a program for managing its ground-water supply, the city of Kalamazoo has constructed induced-recharge facilities at the sites of several of its well fields. To determine the benefits of induced recharge in a water-management program, the U.S. Geological Survey, in cooperation with the city, conducted a series of field experiments at a city well field (Station 9). The 12 production wells at the test site penetrate about 160 feet of glacial drift, which can be separated into three general units—a lower aquifer, an intervening confining layer, and an upper aquifer. Although the upper aquifer is not tapped by any of the municipal supply wells, it serves as a storage and transmission medium for water from the West Fork Portage Creek.

The testing program consisted of four aquifer and three recharge tests. The aquifer tests show that the transmissibility of the upper and lower aquifers ranges from 50,000 to 100,000 gallons per day per foot and indicate that nearly 200 gpm (gallons per minute) leaks through the intervening aquiclude under nonpumping conditions. The object of the three recharge tests (tests 5, 6, and 7) was to observe the effects of induced recharge by varying conditions in the recharge channel. During the three recharge tests, 7 wells were pumped at a total rate averaging about 2,500 gpm. During test 5, inflow to the channel was shut off, and the water level in the channel was allowed to decline. Drawdowns measured during this test were used as a standard for comparison with drawdowns in tests 6 and 7. During test 6, the head in the recharge channel was maintained as constant as possible, and the inflow to the channel was measured. The rate of induced recharge, as indicated by the measured inflow, averaged about 300 gpm. Between tests 6 and 7, the area of the channel was increased from 27,000 to 143,000 square feet. During test 7, the head in the channel was again maintained as constant as possible, but the inflow to the larger channel increased to about 600 gpm.

The principal effect of induced recharge on the two aquifers was to reduce the amount and rate of drawdown. Therefore, where water levels and artesian pressures can be maintained at high stages, the result is lower pumping costs and increased rates of withdrawal during periods of peak demand.

INTRODUCTION

Artificial recharge by means of inducing flow of water from man-made ponds and channels to underlying aquifers has been practiced at Kalamazoo since 1946. At that time, the city constructed three recharge ponds at its well fields in the Axtell Creek area (pl. 1). Water levels in the Axtell Creek well fields have not declined significantly since 1946 although about 10 mgd (million gallons per day) is pumped from the fields. Other recharge facilities have since been added at several other well fields. Without induced recharge, water levels would have declined. Artificial recharge at these well fields seemed to be so highly effective that this activity was singled out for further study.

The city's long-range planning provides for a doubling or possibly tripling of its water-production capacity by 1975. Because artificial recharge seemed to be an important factor in expansion of its water supply, the city wanted to determine if its induced recharge facilities provided benefits sufficient to warrant the cost.

PURPOSE AND SCOPE OF INVESTIGATION

Kalamazoo's desire to conduct a study of induced recharge gave the U.S. Geological Survey an opportunity to do research on artificial recharge under actual field conditions. The facilities at Kalamazoo offered the possibility of conducting a field study with a degree of control usually found only in a laboratory model.

The study was made to help define some of the hydraulic and hydrologic principles involved in inducing recharge to complex unconsolidated aquifers and it was also designed to provide data on the advantages of inducing recharge from manmade ponds rather than from adjacent streams only. Although the exact conditions at the site probably would not be exactly duplicated elsewhere, the experiment is more representative of field conditions in many other areas than any hydraulic, electrical, or mathematical model that could be devised. Hence, data obtained could be used as a guide in determining whether artificial recharge is feasible in other areas where complex unconsolidated glacial or alluvial aquifers are the primary source of water supply..

The first step in the study was to determine the hydraulic coefficients of transmissibility, storage, and leakage. The field tests and mathematical techniques used are described in the section "Aquifer tests." Those readers who are concerned chiefly with the management aspects of the recharge operations or are not familiar with the mathematics employed in ground-water hydraulics, can refer to the results only in the section "Summary of aquifer tests."

CONCEPTS OF ARTIFICIAL RECHARGE

Artificial recharge is a means of augmenting the amount of water that enters a ground-water reservoir. The building of a lake, pond, or channel for recharging is a direct method; inducing flow of surface water into an aquifer by pumping wells is an indirect method.

When the water level in an aquifer is lowered by pumping below the level of hydraulically connected streams, lakes, or ponds, water will infiltrate into—or recharge—the aquifer. Similarly, when the piezometric surface of a leaky artesian aquifer declines to a stage lower than that of an underlying or overlying aquifer, water will leak through the confining beds into the pumped aquifer.

Recharge operations increase the yield of the well fields because surface storage capacity (available recharge) is increased, and hydraulic connection with the aquifers is created or improved. The lowering of water levels in a well field is halted when recharge is equal to pumpage. The quantity of surface water that can be recharged to the aquifer varies with the transmissibility of the aquifer, the permeability and area of the bottom of the stream or pond, the hydraulic gradient created by pumping and, of course, with the amount of surface water available.

THE TEST SITE

Kalamazoo's Station 9, the site of the recharge experiment, is located along the West Fork Portage Creek at the south edge of Kalamazoo in sec. 4, T. 3 S., R. 11 W., Kalamazoo County (fig. 1). The well-field area is just north of Interstate Highway 94 (U.S. Highway 12) and just west of Westnedge Avenue (U.S. Highway 131).

HYDROGEOLOGIC SETTING

The West Fork Portage Creek watershed includes an area of about 24 square miles in Kalamazoo County (pl. 1). More than 20 lakes and ponds, ranging in size from less an acre to about 160 acres, are in the watershed. Most of the lakes and ponds are in the headwaters, above the sources of the defined channel, and have no surface outlets. The West Fork rises near the center of Texas Township, about 8 miles upstream from the mouth. From this point, the stream flows northeastward to Limekiln Lake, then southeastward to Station 9, and then northeastward again to its junction with Portage Creek. For the most part, the stream channel is from 10 to 20 feet wide and has a shifting sand bottom with a few small cobble and gravel riffles. The total fall from source to mouth is about 70 feet, 40 feet of which occurs in the downstream 1¼ miles. The banks are low and marshy. Throughout most of its course, the stream meanders through marshes and, in the upper half, flows through Bass Lake, Atwater Pond, Limekiln Lake, and a few smaller ponds. The West Fork has only one significant tributary, Little Portage Creek, which is only about a mile

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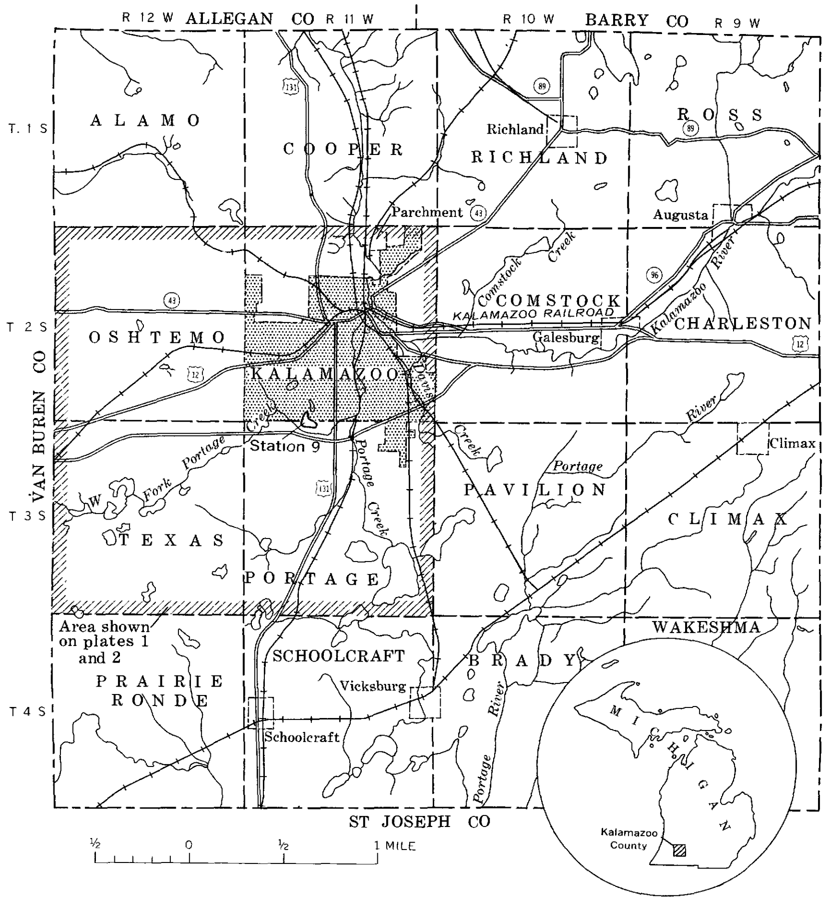


Figure 1.—Location of Station 9 and the Kalamazoo area.

long and empties into the West Fork about a mile upstream from Oakland Drive.

Station 9 lies in a low swampy area about a mile upstream from the mouth of the West Fork. A dense cover of vegetation grows on the narrow muck-covered flood plain along the creek. The creek originally meandered in a northeasterly direction through the well field. To permit farming, the creek was diverted around the south and east margins of the field. In the spring of 1960, after the testing program was completed, the channel was straightened by eliminating the bend in the creek south of the field (fig. 2).

STREAMFLOW CHARACTERISTICS

The flow of Portage Creek is remarkably steady. In the 11 years of record for the gaging station at Reed Street, the maximum monthly

FIGURE 2.—Aerial view of Station 9, June 1960. (Courtesy of of Am. Water Works Assoc.)

mean flow was somewhat less than three times the minimum monthly mean flow. This ratio indicates that much of the precipitation falling on the watershed infiltrates into the ground and appears as base flow of the stream.

The short-term streamflow records collected on the West Fork during this experiment period show a similar steadiness in streamflow (fig. 3). The two pronounced rises, in October 1959 and January 1960, are followed by a rapid return to steady base-flow conditions.

Seepage runs, wherein a number of streamflow measurements are made at selected points throughout a basin on a day when the flow is not affected by surface runoff, provide a good index of the occurrence and distribution of the dry-weather, or base, flow in the basin. Two seepage runs were made in the West Fork basin and one in the Portage Creek basin above West Fork (table 1). The run of November 10, 1959, on the West Fork was made when there was no pumping at Station 9 and when ground-water levels were recovering from the pumping for test 5 which had ended 5 days before. The run of December 1, 1959, was made during a period when pumping had been continuous since November 18, 1959. The effect of the pumping was to reduce the base flow of the West Fork below 12th Street (table 1). In the headwaters, the base flow measured was nearly the same for the two runs.

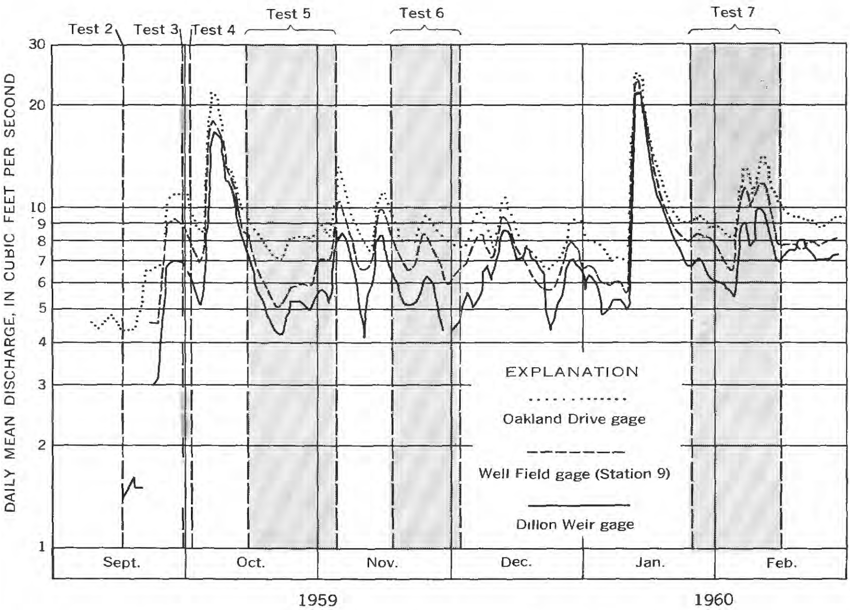


FIGURE 3.—Daily mean discharge of the West Fork near Station 9. See plate 1 for locations. (Courtesy of Am. Water Works Assoc.)

TABLE 1.—*Seepage runs in the Portage Creek basin*

[Station locations shown in pl. 1]

Station		*Drainage area (sq mi)*	*Discharge (cfs)*	
No.	*Name*		*Nov. 10, 1959*	*Dec. 1, 1959*
	West Fork Portage Creek			
1	12th Street	15.3	7.37	7.34
2	Limekiln Lake outlet	------	8.68	8.28
3	Angling Road	------	8.34	7.98
4	Oakland Drive (gaging station)	21.2	8.20	7.20
5	Morningside Drive	------	7.50	7.67
6	Station 9 (gaging station)	23.0	6.69	5.95
7	Dillon Weir (gaging station)	------	6.30	4.40
8	Mouth	24.3	6.72	4.56
	Portage Creek (main stem)			
1	12th Street	------	------	.21
2	Vanderbilt Avenue (tributary entering from south)	------	------	2.34
3	Oakland Drive	------	------	10.1
4	U.S. Highway 131	------	------	15.5
5	Milham Road	------	------	29.3
6	Lovers Lane	23.7	------	30.8
7	Reed Street (former gaging station)	[1] 50.8	([2])	-----

[1] Includes basin of West Fork. Revised from previously published area of 48 square miles (Wells, 1960, p. 117).
[2] Records available for period October 1, 1957, to September 30, 1958.

Under natural conditions, ground water discharges to the West Fork, but withdrawals of ground water since about 1949 have caused the stream to lose water in the vicinity of Station 9. The measurements in table 1 were made during periods of base flow and indicate that the West Fork picks up ground-water discharge between 12th Street and Limekiln Lake, loses some water between Limekiln Lake and Morningside Drive, and loses considerable water as it passes Station 9. Downstream from Dillon Weir, the stream again picks up a small amount of ground-water discharge.

The flow in Portage Creek above the West Fork is increased by discharge from an industrial plant between measuring points 4 and 5 (table 1). When the flows at points 5 and 6 are corrected for this inflow, which was about 14 cfs (cubic feet per second) on December 1, 1959, it becomes evident that Portage Creek picks up very little flow downstream from U.S. Highway 131. The pumping of ground water in the general area probably accounts for this condition. The base flow of the main stream is considerably greater than that of West Fork even though its drainage basin (above the West Fork) is slightly less than that of the West Fork.

GEOLOGY

The Kalamazoo area is underlain by deposits of glacial drift which overly the Coldwater Shale of Mississippian age. The Coldwater Shale and the other Paleozoic rocks that underly it are not sources of fresh water.

In its upper reaches the West Fork flows through a series of depressions in the Kalamazoo Moraine. In the lower part of its course, the creek is incised into an eastward-sloping outwash plain (pl. 2).

The drift is more than 400 feet thick at Atwater pond in the upper part of the watershed, and at Station 9 it is about 170 feet thick. The basal part of the drift is predominantly composed of dark-blue clay till, derived largely from the Coldwater Shale. The till is similar in hydrologic and lithologic characteristics to the shale, and the two are therefore considered to be a single hydrologic unit.

The glacial drift at Station 9 is composed of many lenses of sediment that vary in permeability both vertically and laterally. The drift section may, however, be separated into three general units—a lower aquifer, an intervening aquiclude or confining layer, and an upper aquifer (fig. 4).

UPPER AQUIFER

The upper 80 feet of the drift at Station 9 is designated as the upper aquifer, which consists predominantly of permeable sand (fig. 5, samples A–3, A–8, F–6). These sands will yield large supplies of water,

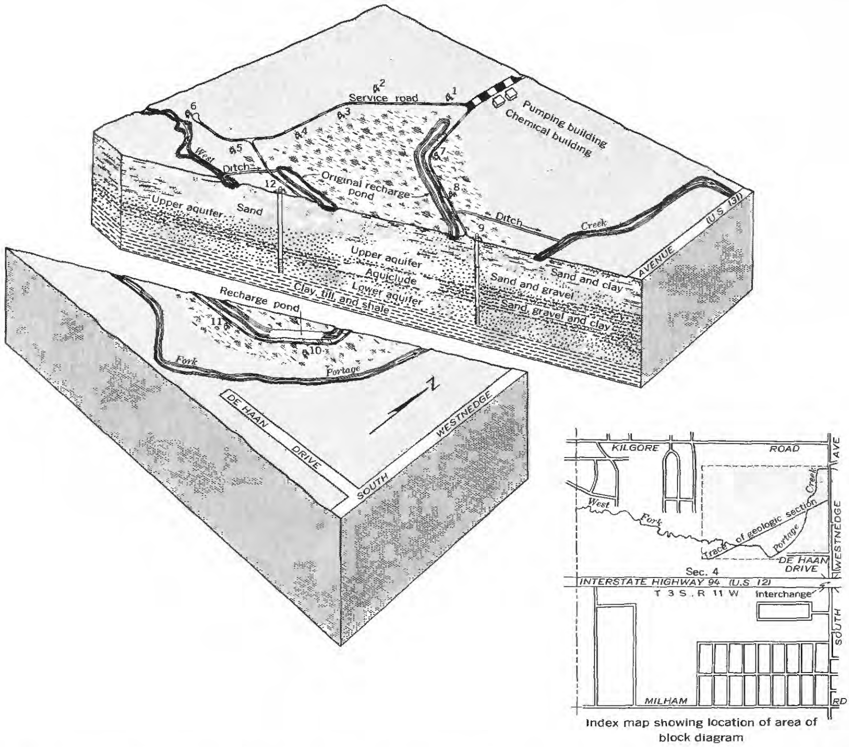


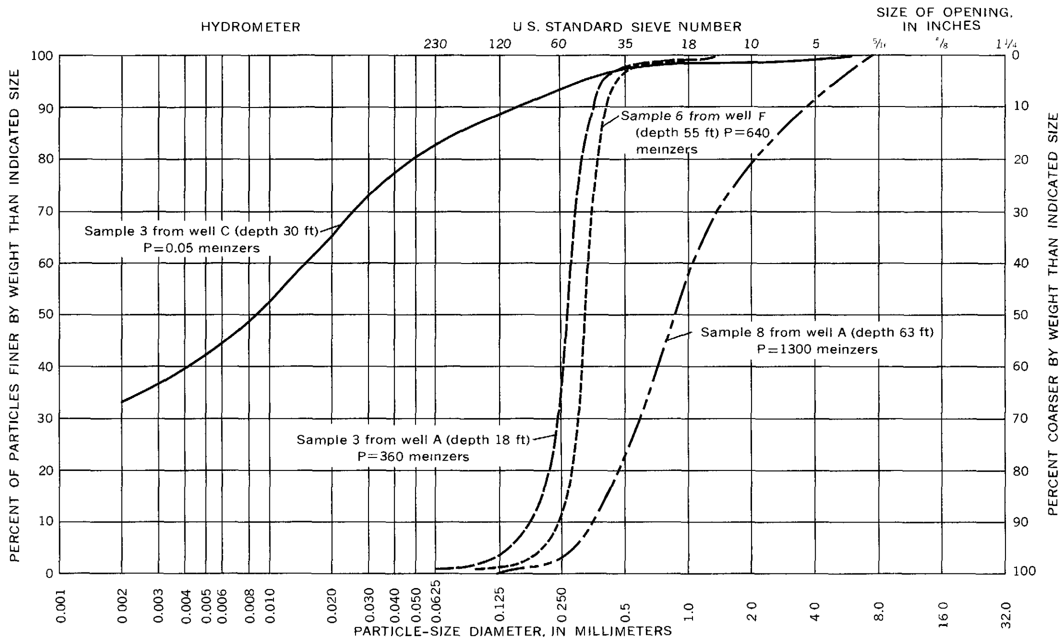
FIGURE 4.—Schematic section through Station 9. Entire area is in sec. 4, T. 3 S., R. 11 W. (Portage Township); hence, well numbers shown are 3S 11W 4–1 and so forth. See tables 3 and 4 for selected logs and records, respectively.

although they are not tapped by any of the municipal-supply wells at the station. However, lenses of poorly sorted materials of low permeability are also present (fig. 5, sample C–3) as are lenses of clay and silt. The West Fork and the recharge channel are cut into the upper aquifer.

LEAKY AQUICLUDE

The glacial drift between a depth of about 80 and 120 feet consists mainly of layers and lenses of buried outwash and till deposits. The permeability of these deposits is considerably lower than the materials in the upper and lower aquifer. This middle section functions as a leaky aquiclude or semiconfining layer which impedes the vertical movement of water from the upper aquifer to the lower.

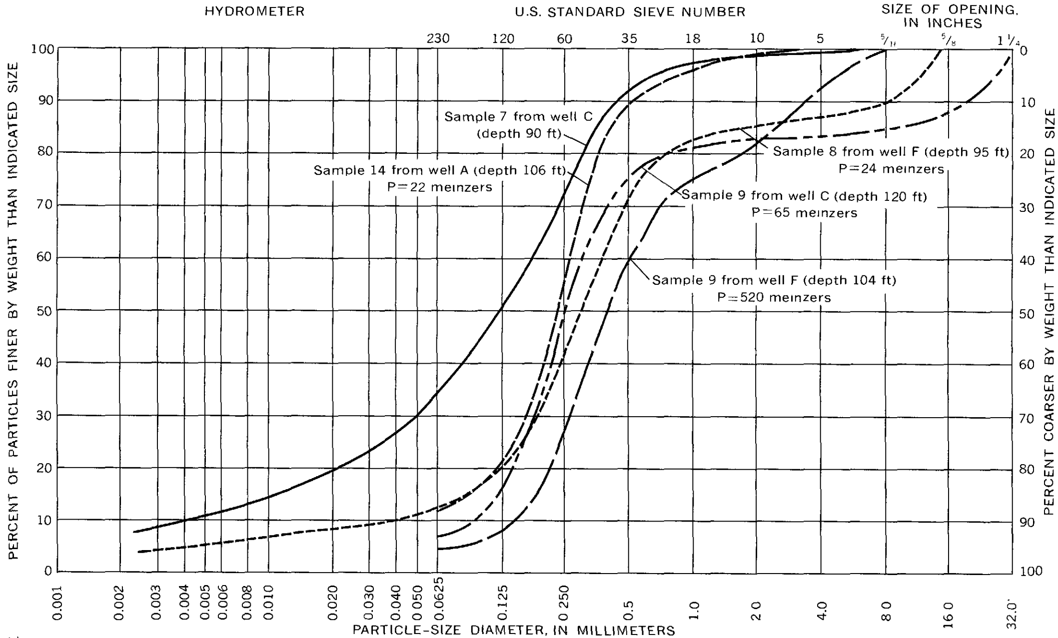
The aquiclude differs from the upper and lower aquifers in that the sediments composing many of its lenses are poorly sorted and contain more silt and clay (fig. 6, samples A–14, C–7, C–9, and F–8). Some of the sand and gravel outwash in this section is, however, highly permeable (fig. 6, sample F–9).



PERCENT OF SIZE

Sample	Clay size <0.004	Silt size 0.004-0 0625	Sand size: V fine 0.0625-0.125	Sand size: Fine 0.125-0.25	Sand size: Medium 0.25-0.5	Sand size: Coarse 0.5-1	Sand size: V coarse 1-2	Gravel size: V fine 2-4	Gravel size: Fine 4-8	Gravel size: Medium 8-16	Gravel size: Coarse 16-32
A-3	←0.6→		2.3	36.6	50.6	8.8	1.1				
A-8	←0.2→		0.0	2.8	19.9	34.1	21.3	12.9	8.8		
C-3	39.8	42.7	5.6	5.8	2.7	1.6		0.4	0.6	0.8	
F-6	←0.4→		0.6	11.5	84.3	3.0	0.2				

FIGURE 5.—Particle-size distribution curves for samples of glacial drift collected at Station 9 from the upper aquifer.



PERCENT OF SIZE

Sample	Clay size < 0.004	Silt size 0.004–0.0625	Sand size: V fine 0.0625–0.125	Sand size: Fine 0.125–0.25	Sand size: Medium 0.25–0.5	Sand size: Coarse 0.5–1	Sand size: V coarse 1–2	Gravel size: V fine 2–4	Gravel size: Fine 4–8	Gravel size: Medium 8–16	Gravel size: Coarse 16–32
A-14	←12.4→		8.4	33.5	34.8	6.3	3.4	0.9	0.3		
C-7	10.8	24.7	14.9	23.3	18.9	4.5	1.4	0.6	0.9		
C-9	←6.4→		10.5	31.2	26.7	6.1	1.7	0.7	1.3	3.2	12.2
F-9	←4.9→		3.0	19.2	32.7	15.5	5.8	11.0	7.3	0.6	
F-8	4.7	7.9	7.4	20.4	30.9	10.9	2.7	1.8	2.3	11.0	

FIGURE 6.—Particle-size distribution curves for samples of glacial drift collected at Station 9 from the aquiclude.

LOWER AQUIFER

The lower aquifer is composed principally of sand and gravel. The top of the lower aquifer is about 120 feet below land surface, and permeable glacial materials are generally not present below 160 or 165 feet. The aquifer is not a homogeneous deposit, however, as it includes some lenses of poorly sorted material of low permeability (fig. 7). The typical sand and gravel deposits are well sorted as in samples A–19, C–11, and F–14. Sample F–13 is a sample of poorly sorted material included in the aquifer. The lower aquifer is the source of water to all the municipal-supply wells at Station 9.

HYDROLOGY

Precipitation is the source of water supplies in the drainage basin of the West Fork Portage Creek. Some of this precipitation infiltrates into the ground and eventually seeps into lakes and streams, some runs off to lakes and streams or falls directly on them, but most is returned to the atmosphere by evapotranspiration. The water that is not lost by evaporation or pumped for municipal or other uses is ultimately discharged from the basin as surface runoff or as ground-water underflow.

Areas where unsaturated permeable materials are at or near the surface are favorable for infiltration of precipitation to the underlying aquifers. Precipitation on already saturated sediments or on areas underlain by materials of relatively low permeability such as clayey till or lake deposits will not result in appreciable recharge to the ground.

In the Kalamazoo area, the general direction of movement of ground water is toward the Kalamazoo River; in the basin of West Fork, the piezometric surface has a general slope to the northeast. Under normal conditions, ground water discharges to the West Fork. Where the stream is incised into permeable materials in the aquifer, water will flow from the stream into the aquifer if the water level (head) in the aquifer is lowered beneath the stream surface by pumping or any other influence. That reach of the stream then provides recharge to the aquifer potentially equal to the flow of the stream. Pumping of wells in such an area will induce migration of water from the stream toward the wells.

The percentage of precipitation that percolates to the underlying aquifers throughout the basin of the West Fork has not been determined. Water-level data from shallow wells and precipitation records collected at Station 9 indicate that a considerable part of the recharge to the upper aquifer is by direct infiltration of rainfall or melting snow. Obviously, any recharge by infiltration to buried aquifers,

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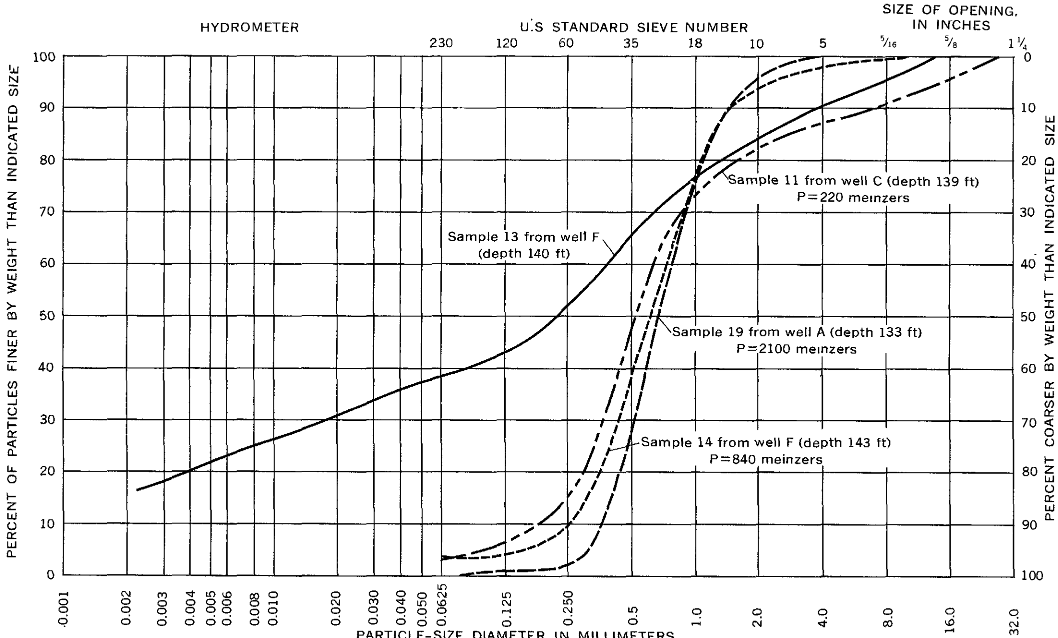

PERCENT OF SIZE

Sample	Clay size < 0 004	Silt size 0.004-0.0625	Sand size: V fine 0.0625-0.125	Sand size: Fine 0.0125-0.25	Sand size: Medium 0 25-0.5	Sand size: Coarse 0.5-1	Sand size: V coarse 1-2	Gravel size: V fine 2-4	Gravel size: Fine 4-8	Gravel size: Medium 8-16	Gravel size: Coarse 16-32
C-11	←3.1→		3.7	8.9	29.9	27.5	8.3	5.8	3.0	5.1	4.7
F-13	20.0	18.6	3.5	8.6	14.1	11.5	6.6	7.2	4.3	5.6	
F-14	←3.6→		0.7	4.7	28.9	38.0	18.1	1.0	4.5	0.5	
A-19	←0 3→		0.2	1.3	26.2	45.5	22.4	4.0	0.1		

FIGURE 7.—Particle-size distribution curves for samples of glacial drift collected at Station 9 from the lower aquifer.

water table or artesian, initially must have been to surface sediments. Table 2 shows daily precipitation and resulting rises in water levels in four shallow wells equipped with continuous recording gages. The data indicates that 1 inch of precipitation generally causes a water-level rise averaging about a quarter of a foot. The rise, however, is highly variable and depends upon the climate, the soil-moisure conditions during the period of precipitation, and the intensity of precipitation.

TABLE 2.—*Relationships between rainfall and rise in water levels in selected wells tapping the upper aquifer*

Date	Rainfall (inches)	Rise in water levels in selected wells							
		3s		9s		10s		Cs	
		Feet	Ft per unit of rainfall	Feet	Ft per unit of rainfall	Feet	Ft per unit of rainfall	Feet	Ft per unit of rainfall
1959									
Sept. 26	0.32	0.08	0.25	0.02	0.06	0.00	0.00	0.03	0.09
Sept. 27	1.00	.09	.09	.12	.12	.24	.24	.12	.12
Sept. 28	.25	.05	.20	.06	.24	.07	.28	.02	.08
Sept. 29	.23	.05	.22	.09	.39	.11	.48	.05	.22
Oct. 5	.75	.09	.12	.17	.23	.23	.31	.12	.16
Oct. 6–8	2.75	.46	.17	.62	.23	.71	.26	.45	.16
Oct. 9	.40	.05	.12	.07	.17	.08	.20	.03	.08
Oct. 11	.58	.05	.09	.11	.19	.14	.24	----------	----------
1960									
Jan. 12–13	2.75	----------	----------	1.00±	.36	.95±	.35	.95	.35

Figure 8 shows the water table in the upper aquifer on October 15, 1959, just before the beginning of the first recharge test. The water table slopes to the northeast, and the slope indicates the direction of ground-water movement. Most of this ground-water flow probably seeps directly from the creek, although part may be ground water flowing beneath the creek from the southwest. Flow of the West Fork at the inlet to Station 9 and at Dillon Weir are shown in figure 3. The stream loses water between these points, and this loss indicates recharge from the West Fork to the upper aquifer.

Under natural conditions the piezometric surface in the lower aquifer was probably higher than the water table in the upper aquifer, and water leaked from the lower aquifer, through the aquiclude, into the upper aquifer (fig. 9). Also, the water table in the upper aquifer was probably higher than the water level in West Fork, and discharge from the upper aquifer was to the creek.

Even before Station 9 was placed into operation, however, withdrawals of ground water from the lower aquifer reversed the natural conditions by causing the piezometric surface to decline to stages lower than the water table in the upper aquifer. For example, the

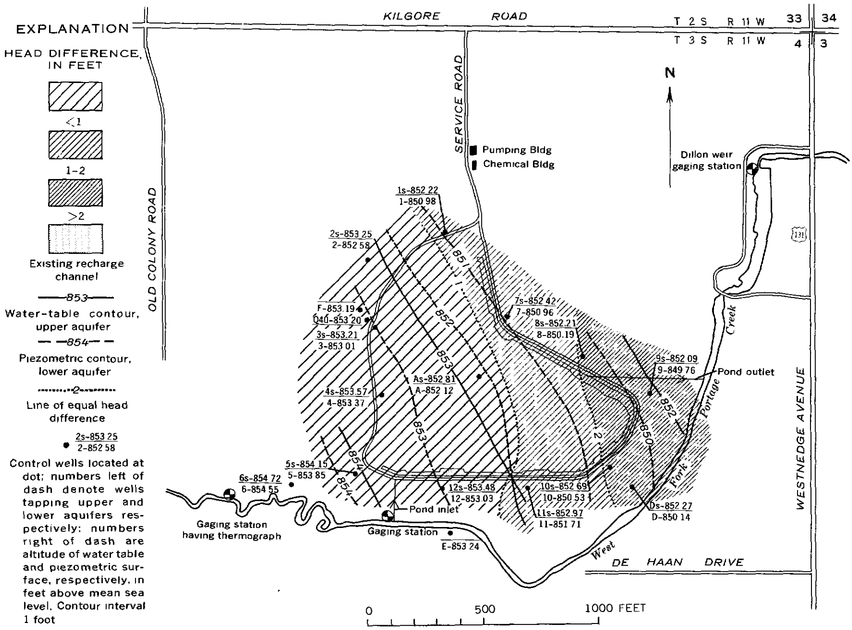

FIGURE 8.—Hydrologic map of Station 9, October 1959. (Courtesy of Am. Water Works Assoc.)

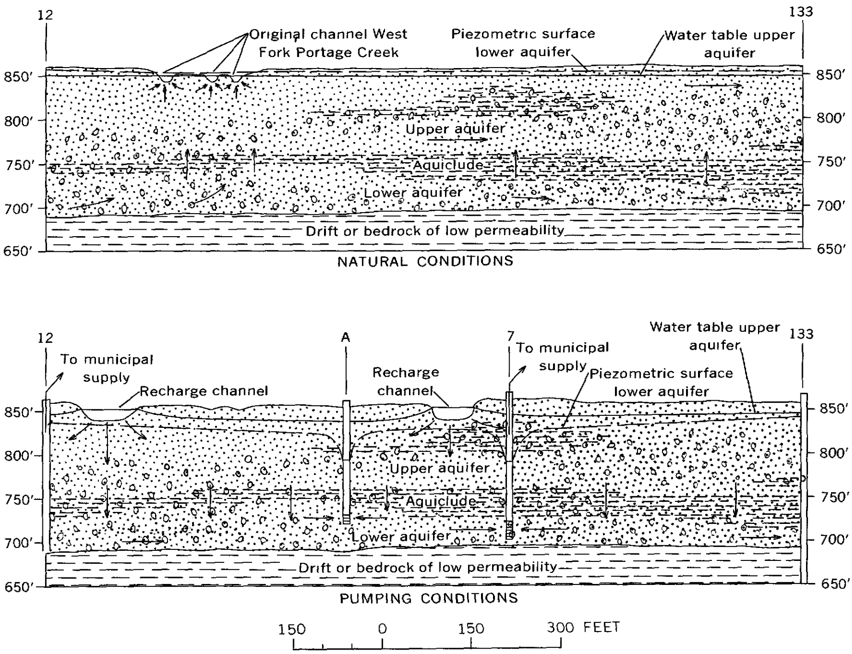

FIGURE 9.—Change in hydrologic regimen caused by ground-water development. Alinement of section is shown in figure 11. Datum is mean sea level. (Courtesy of Am. Water Works Assoc.)

piezometric surface of the lower aquifer at Station 9 is lowered below the water table in the upper aquifer when wells tapping the lower aquifer at Station 8, about 1 mile to the east, are pumped. Thus, under so-called "static" conditions at Station 9—when there is no pumping—the lower aquifer is being recharged by leakage from the upper aquifer. In addition, the lower aquifer is being recharged by underflow from the southwest. When Station 9 is pumped, these conditions become more pronounced, and recharge to the lower aquifer is increased.

Withdrawals of ground water in the vicinity of Station 9 cause the stream to lose water to the upper aquifer. The measurements shown in table 1 were made during periods of base flow and indicate that the West Fork picks up ground-water discharge between 12th Street and Limekiln Lake, loses some water to the aquifer downstream to Morningside Drive, and loses considerable water as it passes Station 9. Downstream from Dillon Weir, the stream again picks up a small amount of ground water.

The relatively flat floor of the valley of the West Fork is about 30 feet below the surface of the surrounding outwash plain. In the valley the water table is at shallow depth (there are many swampy areas), and the valley floor is covered with dense growths of lush vegetation. Evapotranspiration in these areas probably accounts for a considerable part of the ground-water discharge, especially during the summer.

Station 9 included a marsh of about 27 acres before the original recharge pond was enlarged in December 1959. Figure 10 shows potential evapotranspiration and calculated rates of water loss from

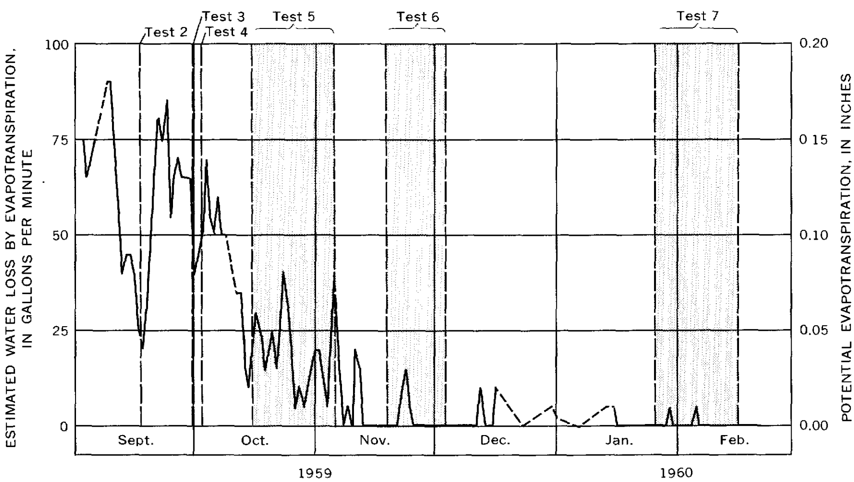

FIGURE 10.—Estimated water loss by evapotranspiration at Station 9.

September 1959 through February 1960. Potential evapotranspiration was calculated by the Thornthwaite method (Thornthwaite, 1948) from meteorological data collected at Station 9. The rate of water loss was calculated on the assumption that evapotranspiration occurred at the potential rate over the 27 acres of marsh. The Thornthwaite method generally underestimates winter potential evapotranspiration, but the error is believed to be small. During the recharge tests, evapotranspiration loss amounted to a negligible part of the total amount of surface and ground water within the area.

TEST PROGRAM

The test program consisted of four aquifer and three recharge tests. The aquifer tests, which preceded the recharge tests, were made to obtain data that could be used to calculate the hydraulic properties of aquifers and aquiclude. The recharge tests were designed to determine the increment to natural recharge that could be attributed to artificial facilities under three different sets of controlled field conditions. The tests were also made to observe the overall effects of recharge on the hydrologic regimen under actual field conditions.

FACILITIES AND INSTRUMENTATION AT STATION 9

The station consists of 12 wells each equipped with a turbine pump capable of yielding 400 gpm (gallons per minute). In figure 11 these wells are numbered 1–12. The wells are pumped under low pressure into a chemical-treatment building where the water is chlorinated, fluoridated, and treated for iron. The water is then transmitted to the pumping building where it is pumped into the city's distribution mains under high pressure.

For this experiment networks of observation wells and stream-gaging stations were installed in and near Station 9. Six deep observation wells (A–F), from 135 to 152 feet deep, were installed in the lower aquifer, and 15 shallow wells, (1s–12s, As, Cs, and Ds), from 31 to 43 feet deep, were installed in the upper aquifer (fig. 11). The shallow wells were drilled adjacent to the production wells and deep observation wells and are identified by the subscript "s." In addition, four closely clustered observation wells (Q–20, Q–40, Q–60, and Q–80) were drilled near production well F to the depths of 20, 40, 60, and 80 feet as indicated by the well number. Head loss caused by vertical percolation in the upper aquifer was measured in these wells. The pump was removed from well 9 so that a recorder could be installed. Water-level measurements were made in all other production wells by means of a steel tape inserted between the casing and pump columns. Numbers listed in the tables of well logs and

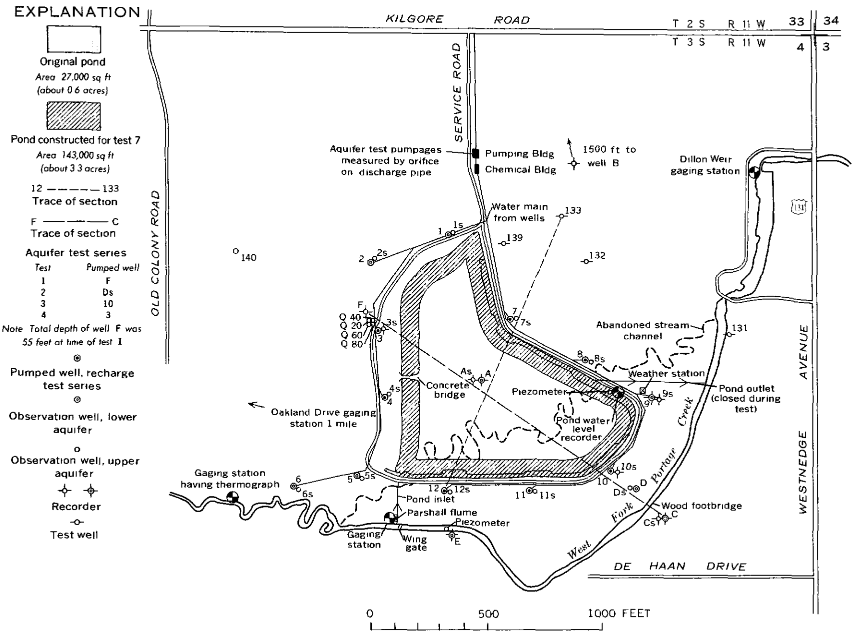

FIGURE 11.—Facilities and test instrumentation of Station 9. Sections from wells F–C and F–D are shown in figures 23 and 29, respectively. (Courtesy of Am. Water Works Assoc.)

well records (tables 3 and 4) are the standard designation for wells in Michigan, as published in areal or statewide reports.

Piezometers were driven into the bottom of the recharge pond and the bed of the creek. These shallow observation wells were used to measure head loss caused by surface-water infiltration into the ground.

Three sharp-crested weirs equipped with water-stage recorders were installed on the West Fork Portage Creek. One was at Oakland Drive (not shown in fig. 11); one was at the southwest corner of the station upstream from well 6; and one was on the Dillon property near Westnedge Avenue downstream from the pond outlet (fig. 11). A thermograph was installed at the weir in the southwest corner of the station. A parshall flume equipped with a water-stage recorder was placed at the entrance to the diversion ditch feeding the recharge pond. The pond outlet was closed during the testing period, and a recorder on the recharge pond measured changes in the water level of the pond.

In addition, a weather station consisting of a rain gage, maximum and minimum thermometer, recording microbarograph, and anemometer was installed near well 9. Wind-velocity data was obtained only for the early part of the experiment.

Measurement of ground water pumped from the field was made by continously recording meters in the pumping building. Pumping at Station 9 was regulated by city personnel throughout the period September 1959–February 1960 in exact accordance with the technical plan for the project. The area normally served by water delivered from Station 9 was supplied from other city stations during periods of recovery. During pumping periods water in excess of the needs of the area served was delivered to areas normally served from other stations.

AQUIFER TESTS

The response of an aquifer to pumping is controlled by the hydraulic coefficients of the aquifer and the location and nature of its boundaries. The purpose of an aquifer test is to determine these coefficients by relating amount and rate of drawdown to well discharge, duration of discharge, and distance from the pumping well.

The coefficients that affect the hydraulic behavior of the aquifer in the project area are the coefficient of transmissibility (T), the storage coefficient (S), and the coefficient of leakage (P'/m'). The coefficient of transmissibility is a measure of the ability of the aquifer to transmit water and is defined as the amount of water in gallons per day (gpd) that will flow through a 1-foot-wide section of the aquifer under a unit hydraulic gradient (one foot per foot) at prevailing water temperature. The storage coefficient characterizes the "reservoir" behavior of an aquifer and relates water-level changes to actual amounts of water taken from, or added to, storage. It is defined as the volume of water the aquifer releases from or takes into storage per unit surface area of the aquifer per unit change in the component of head normal to that surface. The coefficient of leakage, or leakance, characterizes the property of the confining beds to transmit water (Hantush, 1956). It is defined as the amount of water in gallons per day that will flow through an area of one square foot per foot of head difference at the prevailing water temperature.

The equation for nonsteady radial flow in an infinite leaky aquifer was derived by Hantush and Jacob (1955) and may be stated as follows:

$$s=\frac{114.6QL(u,v)}{T} \tag{1}$$

where:

s=drawdown or recovery of water level, in feet,

T=coefficient of transmissibility, in gallons per day per foot (gpd per ft),

Q=rate of pumping, in gallons per minute, and

$L(u, v)$=well function for leaky aquifers, or

$$L(u,v)=\int_u^\infty \frac{1}{u}\,e^{\left(-u-\frac{v^2}{u}\right)}\,du \tag{2}$$

and

$u=\frac{1.87r^2S}{Tt}$

r=distance from pumping well, in feet,

t=time since pumping started or stopped, in days,

S=coefficient of storage, dimensionless,

$$v=\frac{r}{2}\sqrt{\frac{P'}{m'T}}, \tag{3}$$

$\frac{P'}{m'}$=coefficient of leakage, gallons per day per square foot per foot (gpd per sq ft per ft) of head difference.

As steady-state conditions are approached, u approaches the limit of zero, $L(u, v)$ approaches the limit of $2Ko(2v)$, and the equation (Jacob, 1946) may be written

$$s=\frac{229QKo(2v)}{T} \tag{4}$$

where:

$$Ko(2v)=-(0.5772+\ln v)\left(1+\sum_{n=1}^{\infty} v^{2n}(n!)^2\right)+\sum_{n=1}^{\infty}\left(\frac{v^{2n}}{(n!)^2}\sum_{k=1}^{n}\frac{1}{k}\right).$$

As the coefficient of leakage becomes small, v approaches zero, and the leaky-well function becomes the nonequilibrium equation of Theis (1935), which may be stated as follows:

$$s=\frac{114.6Q}{T}W(u)$$

where:

$W(u)$=well function of u, or

$$W(u)=\int_u^\infty \frac{e^{-u}}{u}\,du \tag{5}$$

with symbols as defined previously.

The above equations apply rigorously only if the aquifer fits the properties of the hypothetical aquifer used in deriving the equations. Among the necessary assumptions are the following: Aquifer coefficients are constant in time and space, the aquifer is areally extensive, and flow in the aquifer is laminar and entirely radial. The Theis nonequilibrium formula also assumes that all water is derived from storage and not from recharge—either from surface sources or from interformational leakage. The leaky-aquifer equation of Hantush and Jacob assumes that the interbed leakage is proportional to drawdown, that the gradient across the confining bed is established instantaneously, and that there is sufficient contrast between the permeabilities of confining bed and aquifer so that flow lines are refracted at the interface of the two layers at an angle close to 90°, flow through the aquiclude being approximately vertical and flow in the aquifer being predominantly radial. This equation also assumes a constant head in the source bed for the leakage.

Four pumping tests were made at the project site to determine aquifer coefficients. Two tests were made on the upper aquifer (well F on the northwest side of the project area and well D on the southeast side), and two others were made on the lower aquifer (well 10 on the southeast side and well 3 on the northwest side).

TEST 1

Well F was used as the pumped well for the first test on the upper aquifer. For this test, drilling of well F, 6 inches in diameter, was interrupted when the well was 55 feet deep. The well was screened from 45 to 55 feet. Pumping was at the rate of 58 gpm. Drawdown and recovery measurements were made in wells Q–20, Q–40, Q–60, and Q–80, each 50 feet from well F; well 3s, 100 feet from well F; well 2s, 215 feet from well F; and well 4s, 300 feet from well F.

The rate of drawdown reached a maximum after a few minutes of pumping and thereafter decreased. The water levels eventually became stabilized; and the maximum drawdowns reached are shown in figure 12. In this figure, a plot of s against r, on logarithmic scales, was fitted by the curve-matching method to a plot of $Ko\ (2v)$ against v. The results of the matching are shown by the trace of $Ko\ (2v)$ in figure 12. The results obtained were:

$$T=1.0\times 10^5,$$

and

$$P'/m'=1.3 \text{ gpd per sq ft per ft.}$$

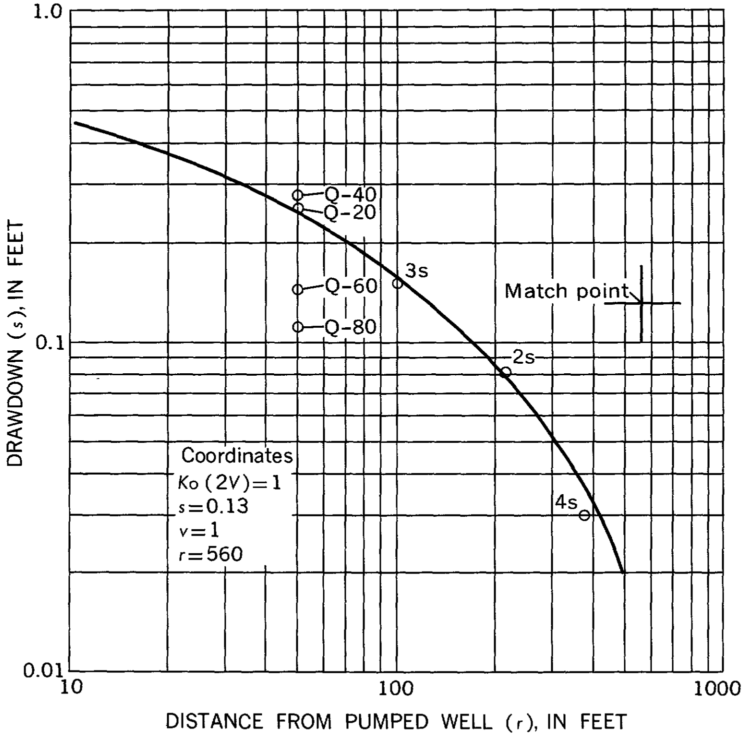

FIGURE 12.—Distance-drawdown graph for observation wells used in test 1.

Example:

$$T=\frac{229\times Q\times Ko(2v)}{S}=\frac{229\times 58\times 1}{0.13}=1.0\times 10^5 \text{ gpd/ft},$$

$$P'/m'=4T\,v^2/r^2,$$

$$=\frac{4(100{,}000)1}{(560)^2}=\frac{400{,}000}{313{,}000}=1.3 \text{ gpd/sq ft/ft.}$$

TEST 2

A second aquifer test was made to assess the hydraulic characteristics of the upper aquifer on the east side of Station 9. Well Ds, drilled to a depth of 42 feet and equipped with a 4-foot point, was used as the pumped well for the test. The well was pumped at 60 gpm. Drawdown and recovery measurements were made in well 11s, 450 feet from the pumped well, and by continuous recording gages in wells 10s and 9s, 130 feet and 410 feet, respectively, from the pumped well.

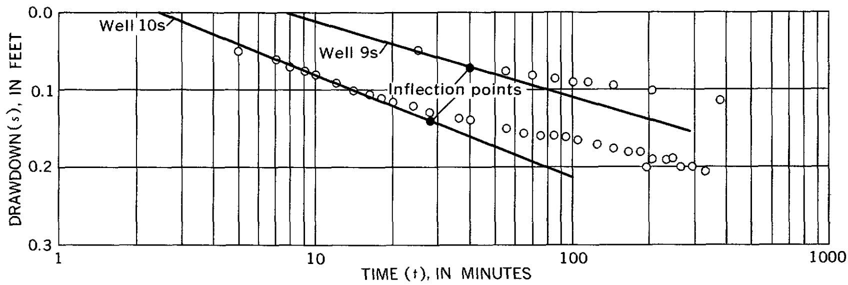

FIGURE 13.—Time-drawdown graph for wells 9s and 10s, test 2.

Figure 13 is a plot of drawdown against the logarithm of time for wells 9s and 10s. Water levels declined rapidly at first, but the logarithmic rate of decline decreased as pumping continued, a typical effect of induced recharge. The cone of depression did not stabilize during the short period of the test, and it was necessary to estimate the maximum drawdown. Test results were analyzed by a method described by Hantush (1956, p. 703): At the inflection point on the time-drawdown curve, the relation of drawdown to slope is

$$e^{2v}Ko(2v)=2.3\,\frac{s_i}{\Delta s_i} \qquad (6)$$

where

s_i=the drawdown in feet at the inflection point, and
Δs_i=the slope across one log cycle of the curve at the inflection point.

The values for $e^{2v}Ko(2v)$ have been given by Hantush (1956, table 1), and from this table, the values for v can be calculated. Since s_i is one-half of the steady-state drawdown, equation 4 becomes

$$T=\frac{114.6QKo(2v)}{s_i}. \qquad (7)$$

The latter part of the drawdown curves indicates a lesser rate of decline than would be expected from leakage alone. This lesser rate may be due to boundary effects from the West Fork Portage Creek.

Figure 14 is a plot of maximum slope (Δs_i) against distance. Using an equation modified from Hantush (1956, p. 703, eq. 7):

$$\Delta s_i=\frac{(264Q)}{T}\,e^{-2v}. \qquad (8)$$

EXAMPLE, FIGURE 13

Well 9s

$$e^{2v}Ko(2v)=\frac{2.3s_i}{\Delta s_i}$$

$$=\frac{(2.3)(0.07)}{0.10}=1.61;$$

from table 1 (in Hantush, 1956), $v=0.215$, and

$$T=\frac{114.6QKo(2v)}{s_i}$$

$$=\frac{(114.6)(60)(1.05)}{0.07}$$

$$=1.0\times10^5 \text{ gpd/ft},$$

$$P'/m'=\frac{4Tv^2}{r^2}$$

$$=\frac{(4)(1.0\times10^5)(0.215)^2}{(410)^2}$$

$$=1.4\times10^{-1},$$

$$S=\frac{\sqrt{T\frac{P'}{m'}}\,t_i}{(3.74)(1440)r}$$

$$=\frac{\sqrt{1.0\times10^5\times1.4\times10^{-1}}(40)}{(3.74)(1440)(410)}$$

$$=2.1\times10^{-3},$$

where:

$\Delta s_i=0.10$,
$s_i=0.07$,
$t_i=40$.

Well 10s

$$e^{2v}Ko(2v)=\frac{(2.3)(0.14)}{0.13}=2.48;$$

from table 1 (in Hantush, 1956), $v=0.065$, and

$$T=\frac{114.6\times60\times2.17}{0.14}=1.1\times10^5 \text{ gpd/ft},$$

$$P'/m'=\frac{4(1.1\times10^5)(0.065)^2}{(130)^2}=1.1\times10^{-1},$$

$$S=\frac{\sqrt{1.1\times10^5\times1.1\times10^{-1}}(28)}{(3.74)(1440)(130)}=4.4\times10^{-3},$$

where:

$\Delta s_i=0.13$,
$s_i=0.14$,
$t_i=28$.

As v and r approach the limit of zero, equation 8 becomes

$$[\Delta s_i]_{r=0}=\frac{264Q}{T}. \tag{9}$$

Also modified from Hantush (1956, p. 712, procedure 5), the coefficient of leakage may be derived from the following equation:

$$\frac{P'}{m'}=\frac{T}{\left[0.434\,\frac{\Delta r}{\Delta \log\,(\Delta s_i)}\right]^2}$$

where $\frac{\Delta r}{\Delta \log\,(\Delta s_i)}$ represents the slope of equation 8 plotted on semilogarithmic paper.

Hydraulic coefficients for the upper aquifer on the east side of the field calculated from the results of test 2 were as follows:

Coefficient	Time-drawdown-recovery method: Well 9s	Time-drawdown-recovery method: Well 10s	Slope-distance method
T, in gpd per ft	100, 000	110, 000	110, 000
S	2.1×10^{-3}	4.4×10^{-3}	----------
P'/m', in gpd per sq ft per ft	0. 14	0. 11	0. 11

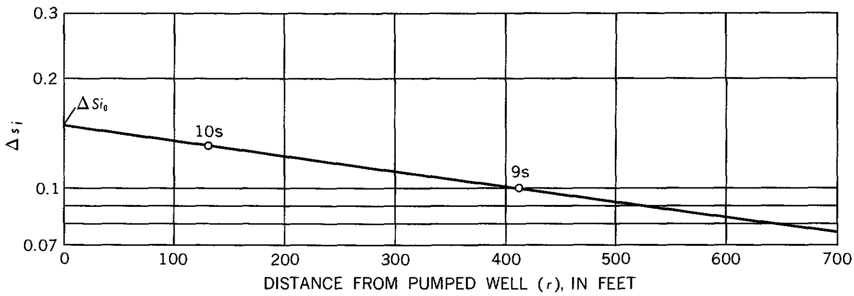

FIGURE 14.—Slope-distance graph for wells 9s and 10s, test 2.

Example:

$$T=\frac{264\,Q}{\Delta s_{i0}}=\frac{264\times60}{0.15}=1.1\times10^5 \text{ gpd/ft},$$

$$\frac{\Delta r}{\Delta \log \Delta s_i}=700/\log\frac{15}{7.6}=\frac{700}{.30}=2.3\times10^3,$$

$$P'/m'=\frac{T}{\left[0.434\,\frac{\Delta r}{\Delta \log\,(\Delta s_i)}\right]^2}=\frac{1.1\times10^5}{[(0.434)\,(2300)]^2}=1.1\times10^{-1}.$$

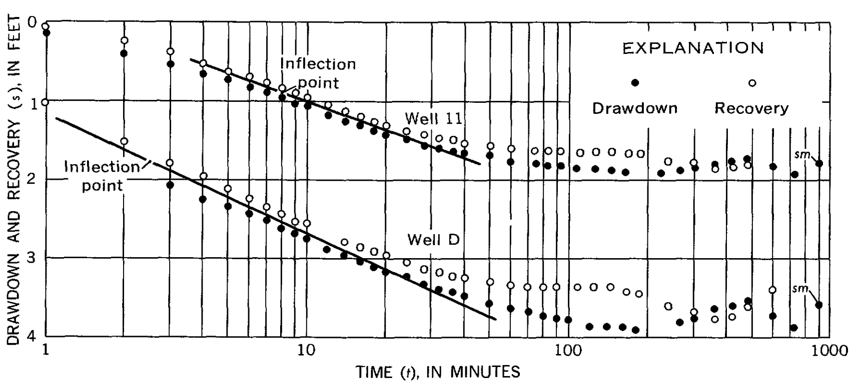

FIGURE 15.—Time-drawdown-recovery graph for wells 11 and D, test 3.

EXAMPLE, FIGURE 15

Well 11

$$e^{2v}Ko(2v)=\frac{2.3s_i}{\Delta s_i}$$

$$=\frac{(2.3)(0.9)}{1.15}$$

$$=1.80;$$

from table 1 (in Hantush, 1956), $v=0.16$, and

$$T=\frac{114.6QKo(2v)}{s_i}$$

$$=\frac{(114.6)(325)(1.31)}{0.9}$$

$$=5.4\times10^4 \text{ gpd/ft},$$

$$P'/m'=\frac{4Tv^2}{r^2}$$

$$=\frac{4(5.4\times10^4)(0.16)^2}{365^2}$$

$$=4.2\times10^{-2} \text{ gpd/sq ft/ft},$$

$$S=\frac{\sqrt{TP'/m't_i}}{(3.74)(1440)r}$$

$$=\frac{\sqrt{(5.4\times10^4)(4.2\times10^{-2})(8)}}{(3.74)(1440)(365)}$$

$$=1.9\times10^{-4},$$

where:

$t_i=8$

$s_i=0.9$

$\Delta s_i=1.15$.

Well D

$$e^{2v}Ko(2v)=\frac{(2.3)(1.8)}{1.5}$$

$$=2.76;$$

from table 1 (in Hantush, 1956), $v=0.0455$, and

$$T=\frac{(114.6)(325)(2.52)}{1.8}$$

$$=5.2\times10^4 \text{ gpd/ft},$$

$$P'/m'=\frac{4(5.2\times10^4)(0.0455)^2}{(130)^2}$$

$$=2.5\times10^{-2} \text{ gpd/sq ft/ft},$$

$$S=\frac{\sqrt{(5.2\times10^4)(2.5\times10^{-2})(2.5)}}{(3.74)(1440)(130)}$$

$$=1.3\times10^{-4},$$

where:

$t_i=2.5$

$s_i=1.8$

$\Delta s_i=1.5$.

TEST 3

Hydraulic characteristics of the lower aquifer were calculated from the two tests (tests 3 and 4) run on the east and west sides of Station 9, respectively. Well 10 on the east side of the station was pumped at 325 gpm. Drawdown and recovery measurement were made in wells D and Ds, 130 feet distant; wells 11 and 11s, 365 feet distant; wells 8 and 8s, 490 feet distant; wells 12 and 12s, 720 feet distant; and in the pumped well. Drawdown and recovery were measured in well 10s, adjacent to the pumped well, and in wells 9 and 9s, 360 feet from the pumped well. Analyses of the data were by the same methods used for test 2 (figs. 15 and 16).

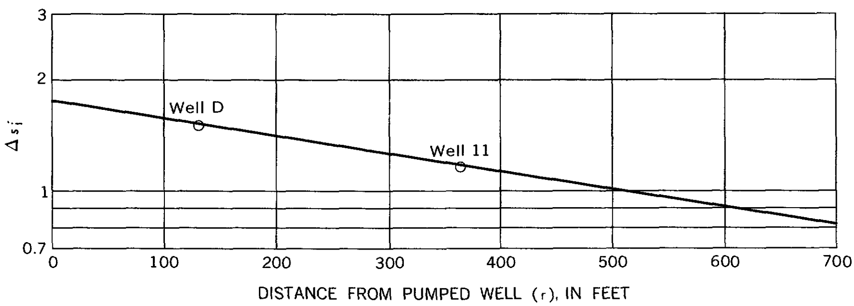

FIGURE 16.—Slope-distance graph for wells 11 and D, test 3.

Example:

$$T=\frac{264\,Q}{\Delta s_{i_0}}=\frac{264\times 325}{1.75}=4.9\times 10^4 \text{ gpd/ft},$$

$$\frac{\Delta r}{\Delta \log \Delta s_i}=\frac{700}{\log \frac{1.75}{0.82}}=\frac{700}{0.33}=2.1\times 10^3,$$

$$P'/m'=\frac{T}{\left[0.434\,\frac{\Delta r}{\Delta \log (\Delta s_i)}\right]^2}=\frac{4.9\times 10^4}{[(0.434)(2100)]^2}=5.9\times 10^{-2} \text{ gpd/sq ft/ft}.$$

Figure 16 is a plot of drawdown and recovery in relation to the logarithm of time in wells 11 and D. This plot shows the characteristic effect of leakage upon the slop of the curves. Interference from the pumping of Station 8 about 1 mile east caused the fluctuations in the latter parts of the curves; hence, it was necessary to estimate the maximum drawdowns.

Hydraulic coefficients for the lower aquifer on the east side of the field calculated from the results of test 3 were as follows:

Coefficient	*Time-drawdown-recovery method*		*Slope-distance method*
	Well 11	*Well D*	
T, in gpd per ft	54,000	52,000	49,000
S	1.8×10^{-4}	1.2×10^{-4}	---------
P'/m', in gpd per sq ft per ft	4.1×10^{-2}	2.7×10^{-2}	5.9×10^{-2}

TEST 4

To test the lower aquifer on the west side of Station 9 well 3 was pumped at 345 gpm. Water-level measurements were made in wells Q–20, Q–40, Q–60, Q–80, 50 feet distant; 2 and 2s, 295 feet distant; 4 and 4s, 290 feet distant; 1 and 1s, 515 feet distant; 5 and 5s, 635 feet distant; and in the pumped well. Drawdown and recovery were measured in well 3s, adjacent to the pumped well, and well F, 100 feet away.

The analyses for test 4 were made by a method devised by Cooper (written commun., 1959). In figure 17 the drawdown in well 4 and recovery in well 2 are plotted against time on logarithmic paper. The "early" data were matched to a type curve of $L\ (u, v)$ with the resulting match points shown. Analysis of the data indicated the following hydraulic characteristics for the lower aquifer:

Coefficient	*Well 2*	*Well 4*
T, in gpd per ft	110, 000	100, 000
S	4.7×10^{-4}	2.6×10^{-4}
P'/m', in gpd per sq ft per ft	1.3	1.2

As shown in figure 17, the "late" data fall above the trace of $L\ (u, 0.5)$. Because the water level in the upper aquifer, the source bed for the leakage, declined during the test, water levels in the lower aquifer declined also to maintain the rate of leakage. This situation prevented stabilization of the cone of depression.

SUMMARY OF AQUIFER TESTS

The aquifer tests at Station 9 show that both the upper and lower aquifers are of generally high permeability and that a considerable amount of water will leak through the intervening aquiclude under pumping conditions. Averages of the hydraulic coefficients calculated for the short-term aquifer tests were as follows:

Test	Pumped Well	Aquifer	Side of Field	T (gpd per ft)	S	P'/m' (gpd/sq ft/ft)
1	F	Upper	West	100, 000	4.2×10^{-3}	1.3
2	Ds	do	East	110, 000	3.2×10^{-3}	.12
3	10	Lower	do	52, 000	1.6×10^{-4}	.042
4	3	do	West	105, 000	3.7×10^{-4}	1.3

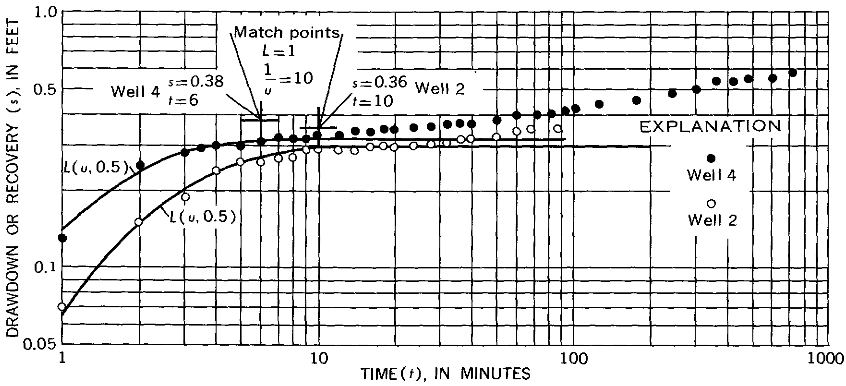

FIGURE 17.—Drawdown in well 4 and recovery in well 2, test 4.

Example:

Well 4

$$T=\frac{114.6\ Q\ L(u,v)}{s}$$

$$=\frac{(114.6)(345)(1)}{0.38}$$

$$=1.0\times10^{5}\text{ gpd/ft},$$

$$S=\frac{T\ ut}{1.87\ r^2}$$

$$=\frac{(1.0\times10^{5})(0.1)(6)}{(1.87)(290)^2(1440)}$$

$$=2.6\times10^{-4},$$

$$P'/m'=\frac{4Tv^2}{r^2}$$

$$=\frac{(4)(1.0\times10^{5})(0.5)^2}{(290)^2}$$

$$=1.2\text{ gpd/sq ft/ft}.$$

Well 2

$$T=\frac{(114.6)(345)(1)}{0.36}$$

$$=1.1\times10^{5}\text{ gpd/ft},$$

$$S=\frac{(1.1\times10^{5})(0.1)(10)}{(1.87)(295)^2(1440)}$$

$$=4.7\times10^{-4},$$

$$P'/m'=\frac{4(1.1\times10^{5})(0.5)^2}{(295)^2}$$

$$=1.3\text{ gpd/sq ft/ft}.$$

Because of the head difference between the water table in the upper aquifer and the piezometric surface of the lower aquifer (see fig. 9), a considerable amount of water was evidently leaking from the upper to the lower aquifer before the aquifer tests began. Assuming that the head difference between the upper and lower aquifers over 13.5 acres on the west half of the station averaged about 0.3 foot, the rate of leakage may be calculated using the equation

$$Q=\frac{P'/m'\Delta h A}{1440}$$

where

A=area in square feet,

P'/m'=coefficient of leakage, in gpd per sq ft per ft,

Δh=head difference, in feet, and

Q=rate of leakage, in gpm.

The rate of leakage on the west side of the field was about 160 gpm. On the east half of the station, where the head difference averaged about 2 feet but where the coefficient of leakage for the lower aquifer was only 0.042 gpd per sq ft per ft, the rate of leakage was about 30 gpm.

At Station 9 pumping from the lower aquifer averaged about 500 gpm during 1958, and there was no pumping from the upper aquifer. Under pumping conditions, therefore, a greater head difference exists inasmuch as the water level in the upper aquifer tends to be maintained by direct recharge from precipitation and inflow from the West Fork. The hydraulic characteristics of the aquifers were determined by the tests described previously. The results show that it is possible to recharge the lower aquifer by keeping the upper aquifer relatively full.

RECHARGE TESTS

Recharge experiments at Station 9 were conducted to determine the effectiveness of water-management practices in increasing the natural recharge to the well field. The tests were designed to observe effects on the total hydrologic regimen under three controlled conditions.

The objective of test 5 was to determine the hydraulic behavior of the well field under "natural" conditions. Actually, natural conditions no longer existed because a small recharge channel had already been constructed. During test 5, inflow to the channel was shut off, and the water level in the channel declined along with the water level in the upper aquifer. With inflow cut off, the channel did not act as a recharge boundary but as an area with a storage coefficient several times greater than that of the upper aquifer. Drawdowns measured during test 5 were used as a standard with which drawdowns in test 6 and 7, under conditions of induced recharge with constant head maintained in the recharge channels, were compared.

The purpose of test 6 was to determine the effectiveness of the existing small channel in recharging the two aquifers. During test 6, the recharge channel was open to the stream, and the head was maintained as constant as possible by regulating flow through the diversion ditch. Because there was no outflow, an amount almost equal to the inflow was being recharged to the upper aquifer from the channel. Evapotranspiration during the recharge tests was negligible in comparison with the amount of water pumped (see fig. 10).

The objective of test 7 was to determine the increase in recharge caused by greatly enlarging the area covered by the recharge channel and extending it along the west side of the station. Again the enlarged channel was open to the stream, the head was maintained as constant as possible, and inflow was measured. It was not possible, however, to maintain a perfectly constant head on the channel during tests 6 and 7. Water-level fluctuations in the recharge channel are shown in figure 18.

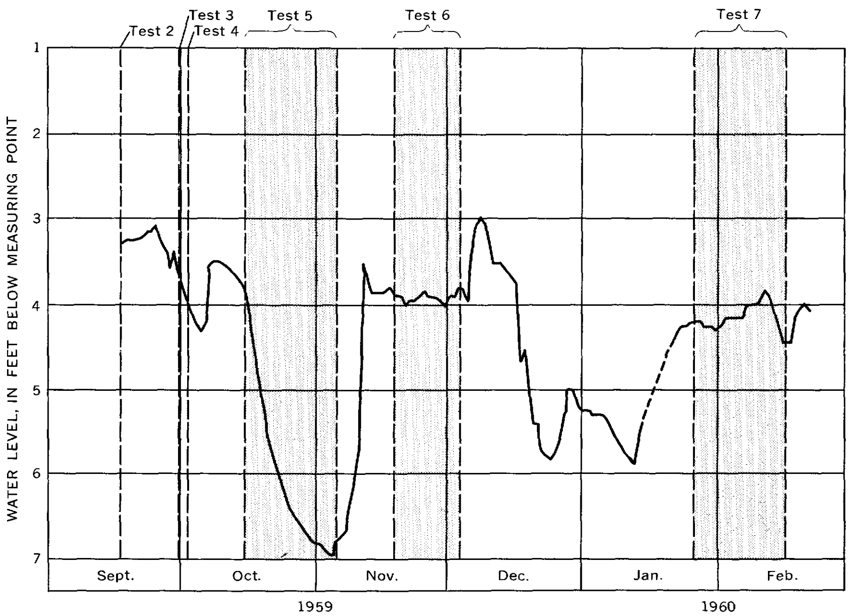

FIGURE 18.—Water level in recharge channel, September 1959–February 1960. (Courtesy of Am. Water Works Assoc.)

In each of the recharge tests seven wells were pumped (Nos. 3, 4, 7, 8, 10, 11, and 12) at a total discharge rate of about 2,475 gpm. Personnel of the city of Kalamazoo maintained the discharge rate as constant as possible by regulating the valve on the station's discharge main. The pumping rates are shown in figure 19. The duration of pumping was 14 days in test 6 and 21 days in tests 5 and 7. The pumping rate was recorded automatically and was adjusted daily for variations.

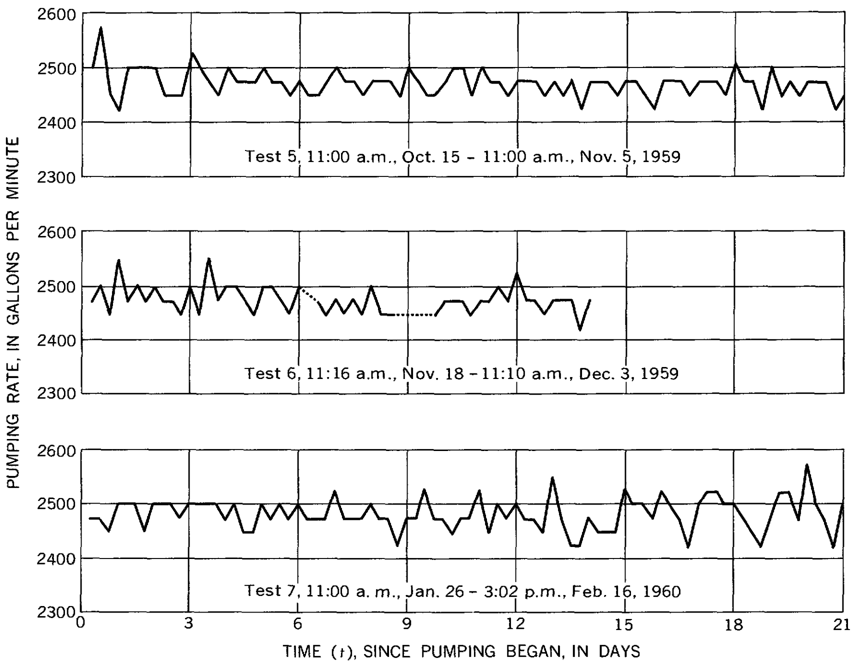

FIGURE 19.—Pumping rates at Station 9 during recharge tests.

TEST 5

The pumping phase of test 5 began at 11:00 a.m., October 15, 1959, and continued until 21 days later at 11:00 a.m., November 5. Except for the short-term aquifer tests, the field had not been pumped since September 23. Figure 20 was constructed by a method devised by Cooper and Jacob (1946). Using the equation

$$\log \bar{r} = \frac{1}{n} \sum_{k=1}^{n} \log r_k$$

where

$\bar{r}$ is the weighted logarithmic mean (and is the distance at which a single well would have the same effect as all 7 wells pumped in test 5),

n is the number of wells pumping,

r_k is the distance from any pumped well to the observation well, in feet.

Time, divided by $\bar{r}^2$, was plotted on semilogarithmic paper against the drawdown in feet. This technique permits drawdowns in various observation wells to be compared with each other and was necessary because seven wells were pumping and no single distance from an

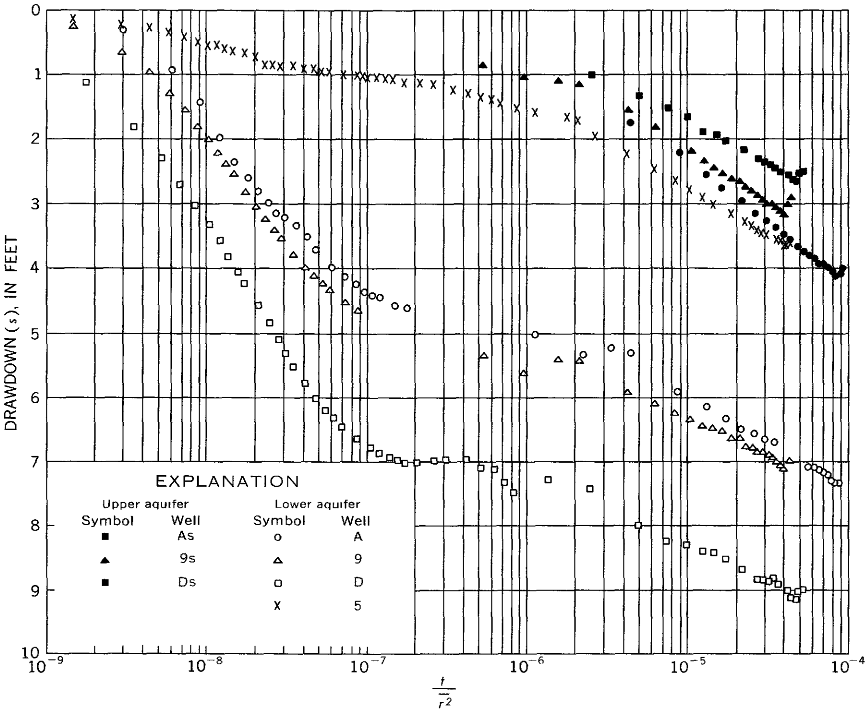

FIGURE 20.—Generalized composite drawdown graph for selected observation wells used in test 5.

observation well to a pumped well could be used as a reference. The rate of drawdown early in the test was greater on the east side of the field as shown by the steeper slopes for wells 9, A, and D as compared with well 5 in the southwestern part of the field. The logarithmic rate of drawdown in the lower aquifer increased within the first few minutes of pumping and later decreased. After about 2 hours of pumping, however, it started increasing again and reached a maximum rate after several days of pumping. The reason for the increase seems to be the decline in head in the upper aquifer. The rapid spread of the cone of depression in the lower aquifer induced a greater rate of leakage from the upper aquifer. As water was removed from storage in the upper aquifer, the water level in the upper aquifer accordingly declined. In order to maintain the rate of leakage across the confining bed, water levels in the lower aquifer declined also. The head differences across the confining beds increased during the very early part of the test and decreased slightly thereafter. This variation indicates that the rate of infiltration per square foot of recharge area was greater in the early part of the test and thereafter declined. The total amount of water being recharged probably was greater during the latter part of the test because of the increased area of the drawdown cone.

The flat-bottomed nature of the cone of depression in the upper aquifer is shown in figure 21. In the upper aquifer, radial flow becomes small in the vicinity of a well pumping from the lower aquifer, and the horizontal change in head approaches zero as most of the flow in the aquifer is vertical. The individual cones of the seven pumping wells have coalesced to form one large, slightly concave cone in the upper aquifer. Figure 21 also shows the concurrent composite cone in the lower aquifer. The individual small cones have again coalesced to form one large cone. In contrast to the cone in the upper aquifer, a small isolated cone around each well shows radial flow caused by the increase in gradient with decrease in area of flow near each well. The composite cone of depression in the lower aquifer was asymmetrical, more drawdown occurring on the east side of the field than on the west. The lower transmissibility of the aquifer is indicated by the closer spacing of the lines of equal drawdown.

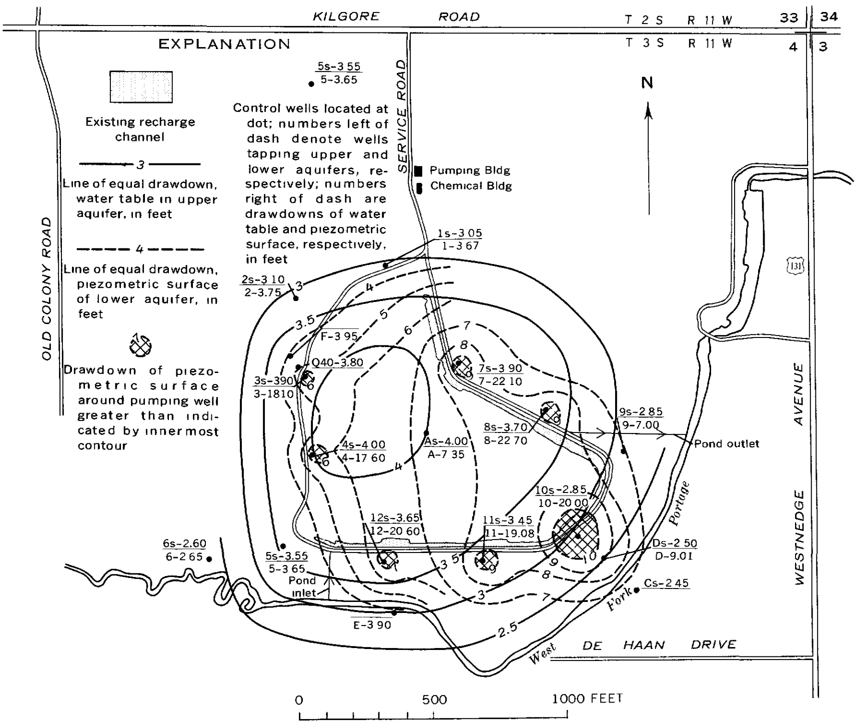

FIGURE 21.—Drawdown in upper and lower aquifers at the end of test 5.

Figure 22 shows the difference in drawdown between the upper and lower aquifers after 21 days of pumping. The greatest drawdown difference is at each pumping well because the gradient in the lower aquifer progressively increases near the pumped wells while the gradient in the upper aquifer progressively decreases. The drawdown difference is greatest on the east side of the well field. Because the rate of leakage at any point is directly proportional to the head difference and leakage coefficient, the rate of leakage per square foot through the aquiclude to the lower aquifer in the vicinity of the pumped wells was greater than elsewhere in the field. Also, the date corroborates—in a qualitative manner—the results of the aquifer tests (tests 1–4), which indicated a lower leakage coefficient on the east side of the well field.

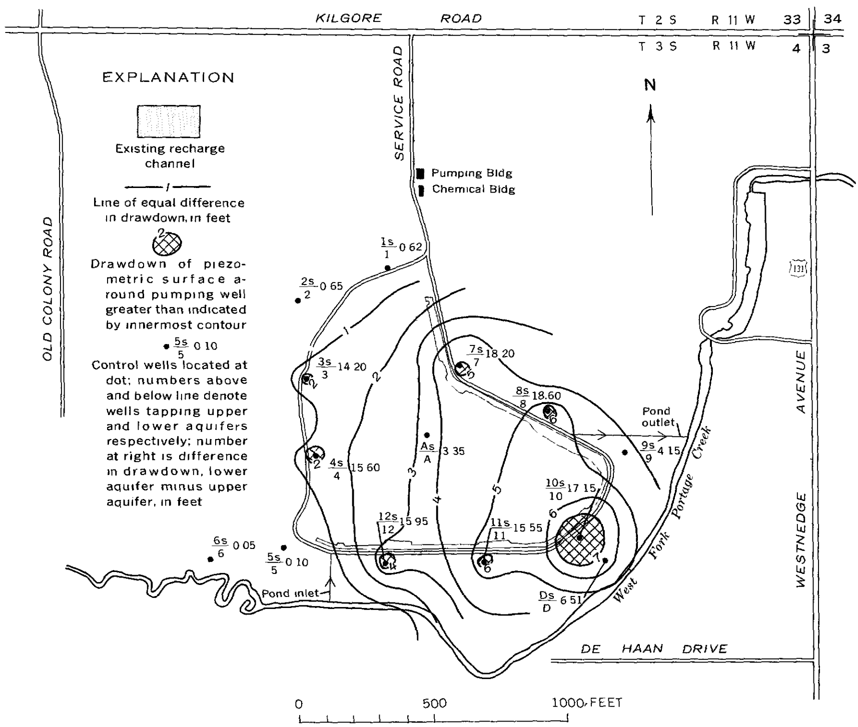

FIGURE 22.—Difference in drawdown between upper and lower aquifers at the end of test 5.

Lines of equal drawdown and directions of flow at the end of test 5 are shown in figure 23, which is a northwest-southeast section showing conditions after 21 days of pumping. The flow through the aquiclude is mainly vertical because the flow lines are refracted, as they pass through the aquiclude, owing to changes in permeability (Muskat, 1946, p. 401). The loss in head, caused by friction of the water moving through the system, is greatest in the aquiclude, especially on the east side of the field, where the coefficient of leakage is lowest.

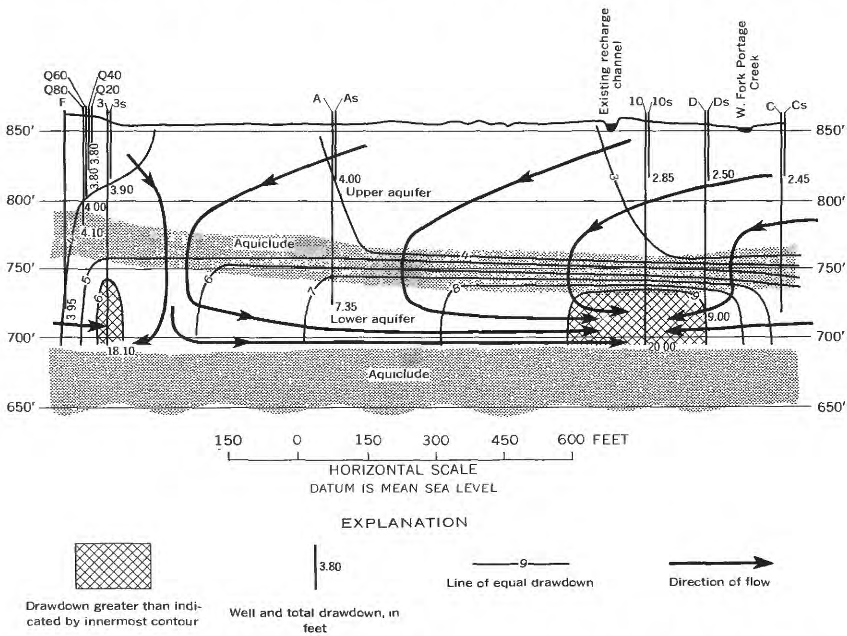

FIGURE 23.—Lines of equal drawdown and flow direction at the end of test 5.

TEST 6

Pumping for test 6 began at 11:16 a.m., November 18, 1959, and continued until 15 days later at 11:10 a.m., December 3. The discharge lines of two wells (Nos. 4 and 10) were frozen, and these wells were not pumped until about one-half hour after the others. The drawdowns during the first 2 or 3 hours were therefore different from test 5. After this time, however, water levels during the first 3–5 days compared closely with those measured during test 5 (see fig. 30). This similarity indicates that the effects of supplemental recharge to the lower aquifer were negligible during the first few days of the test.

During the last 10 or 11 days of the test, the effects of maintaining a relatively constant head in the recharge channel by diverting water from the creek were reflected by water levels in all wells at the station. Total drawdown in both aquifers throughout the field was less than at the end of 14 days of pumping the same wells at the same rate during test 5. Drawdowns in the upper and lower aquifers at the end of test 6 are shown in figure 24.

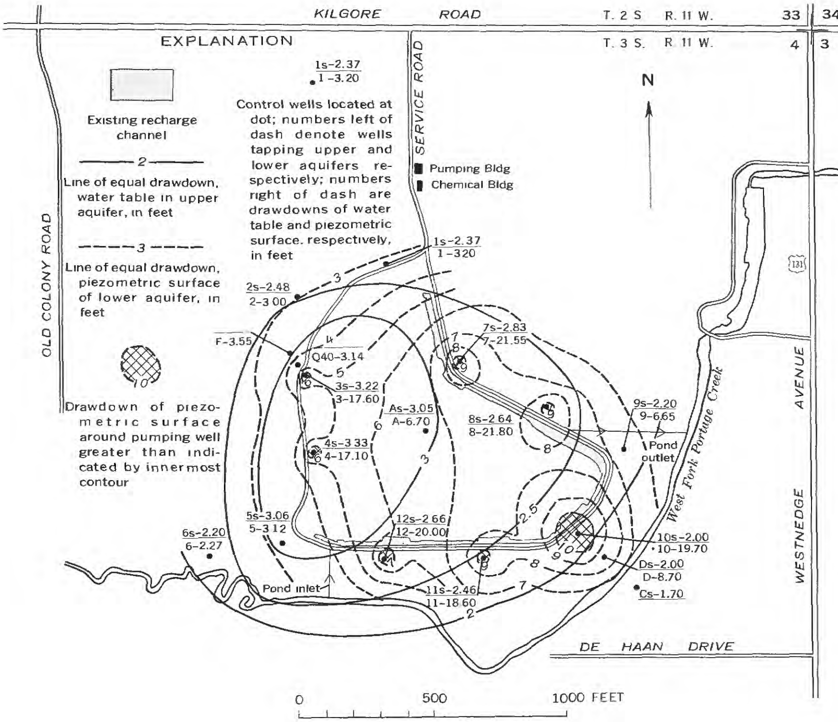

FIGURE 24.—Drawdown in the upper and lower aquifers at the end of test 6.

The cone developed in the lower aquifer is very similar in shape to that of test 5. The cone in the upper aquifer, however, was less symmetrical, and its low point shifted toward the west side of the well field, away from the recharge channel.

Rates of drawdown of water level in wells close to the recharge channel were smaller than rates in wells located at progressively greater distances from the channel. Fluctuation in the rate of drawdown in wells close to the recharge channel occurred largely as a result of fluctuations of water level in the recharge channel during the test. The total range in fluctuations in the recharge channel was limited to about 0.15 feet (see fig. 18). When the water level in the channel was rising, the drawdown curves tended to flatten out. When the channel level was declining, the drawdown curves dropped correspondingly.

The differences in drawdown between upper and lower aquifers remained relatively constant during test 6 (see fig. 32), a fact indicating that the rate of leakage remained constant instead of declining as in test 5. The drawdown differences were generally somewhat greater during test 6 than during test 5, a situation demonstrating that leakage from the upper to the lower aquifer had increased. Figure 25 shows the difference in drawdown between the two aquifers at the end of 14 days of pumping. The drawdown differences were about 0.3 foot greater in the vicinity of the pumping wells and throughout the east half of the field in test 6 than in test 5. On the west side of the field, the increase was negligible.

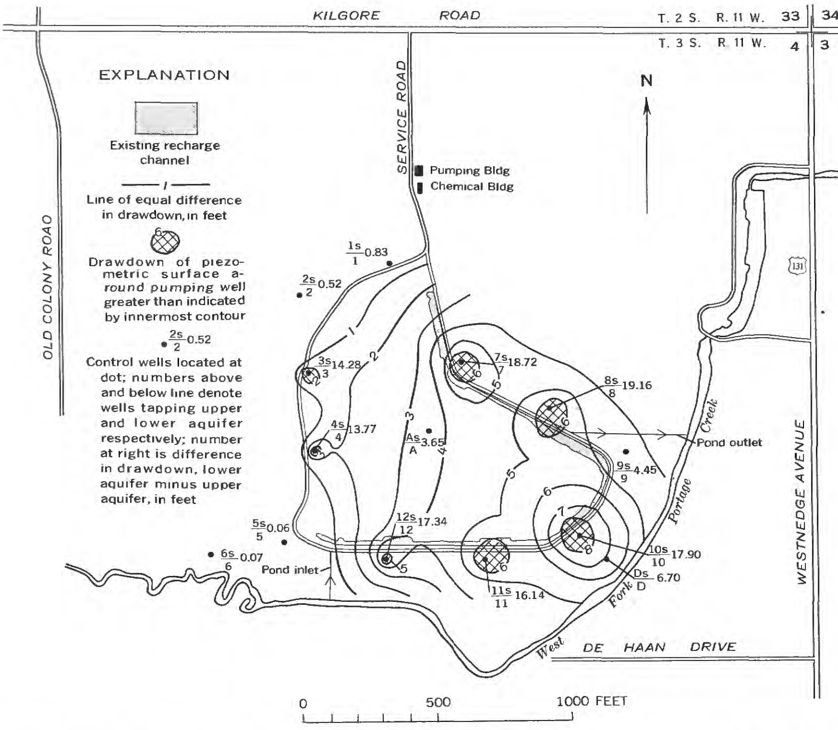

FIGURE 25.—Difference in drawdown between the upper and lower aquifers at the end of test 6.

The quantity of water recharged to the upper aquifer from the recharge pond is indicated by the amount of water diverted from the creek to the pond. During the period of the test, an average of about 0.65 cfs (290 gpm) of water was diverted through the flume to the pond in order to keep the water level in the pond as constant as possible (fig. 26). The average rate of pumping during the test was 2,475 gpm, or about 8 to 9 times the rate of diversion to the pond.

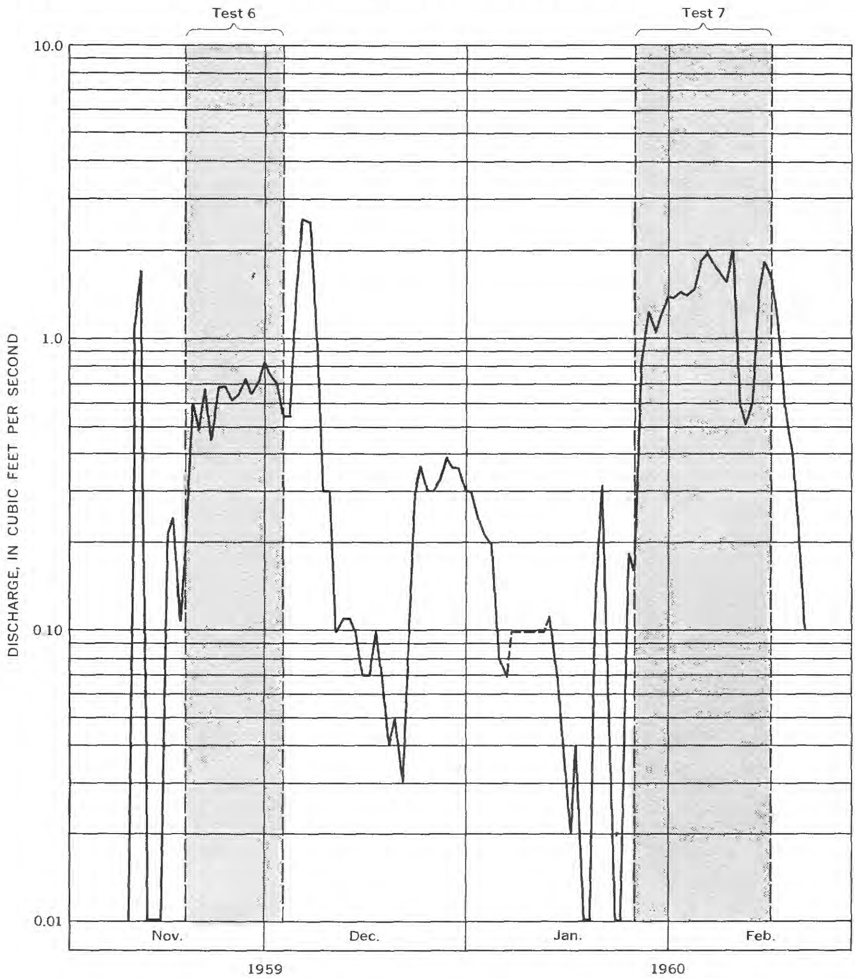

FIGURE 26.—Diversion of water from the West Fork to the recharge channel, November 1959–February 1960. (Courtesy of Am. Water Works Assoc.)

TEST 7

After completion of test 6, the recharge pond was widened and extended within the perimeter of wells to increase the rate of recharge to the wells on the west side of the field. The area of the pond was increased from 27,000 to 143,000 square feet, or from about 0.6 to 3.3 acres. The distance from the channel to the pumping wells ranged from about 50 to 100 feet.

Test pumping began at 11:00 a.m., January 26, 1960, and ended 21 days later at 3:02 p.m., February 16. As in test 6, the discharge lines of two wells (Nos. 4 and 8) were frozen, and pumping from these wells began about 45 minutes after the others. For this reason, the early drawdown measurements do not compare closely with those for test 5, but the recovery measurements do. The rate of drawdown, after several days of pumping, was less than in test 6.

Figure 27 shows drawdown in the upper and lower aquifers after 21 days of pumping. The cone of depression in the lower aquifer is similar in shape to those cones for tests 5 and 6. In most wells, there was less drawdown after 21 days of pumping in test 7 than after 14 days of pumping in test 6. The cone in the upper aquifer was radically different in shape from cones in the previous tests. The drawdowns in the upper aquifer were less in every well than they were in the shorter previous test. Rates of drawdown of water level in wells tapping both aquifers (see figs. 30 and 31) were markedly affected by fluctuations in stage of the recharge channel (fig. 18). The rate of leakage, as in test 6, remained more nearly constant than in test 5. There was an increase in drawdown difference between upper and lower aquifers. Figure 28 shows difference in drawdown between upper and lower aquifers after 21 days of pumping. The differences in drawdown on the east side of the field were about 1 foot greater than

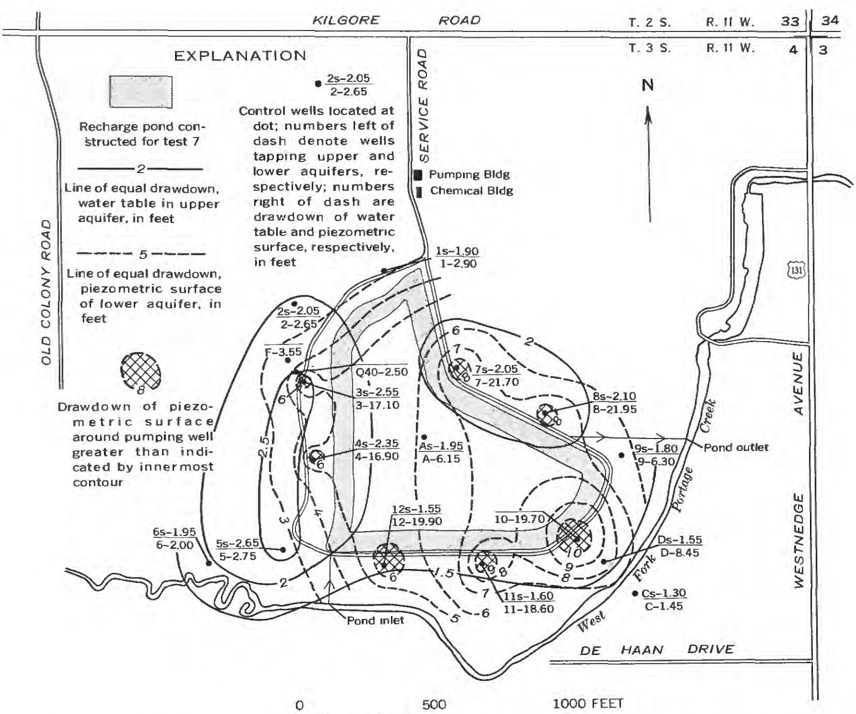

FIGURE 27.—Drawdown in the upper and lower aquifers at the end of test 7. (Courtesy of Am. Water Works Assoc.)

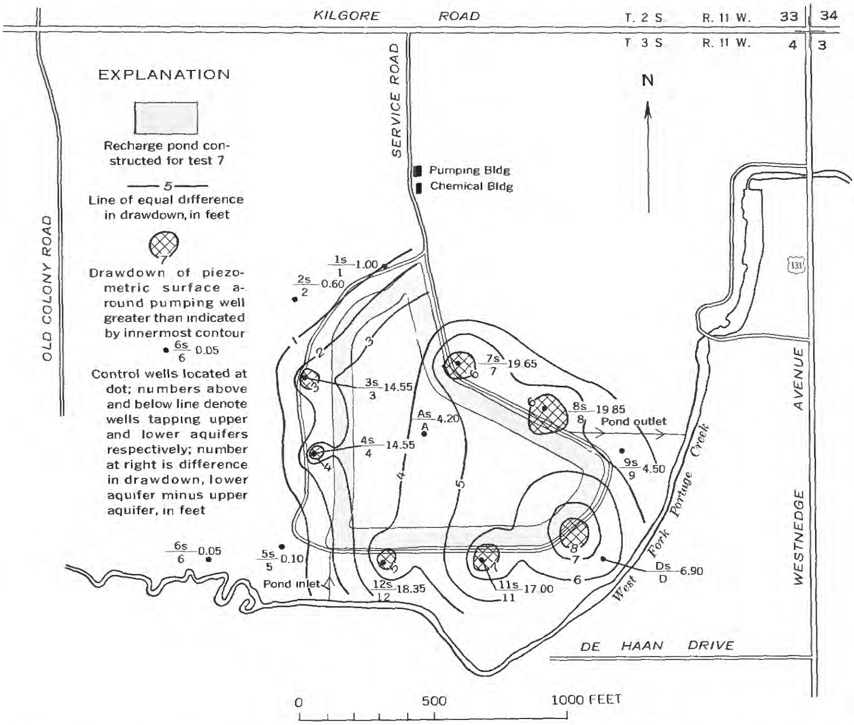

FIGURE 28.—Difference in drawdown between the upper and lower aquifers at the end of test 7.

in test 5 and about 0.5 foot greater than in test 6. On the west side, into which the recharge channel was extended for the test, the difference increased about 0.4 foot over the previous tests. The increase in drawdown differences is a measure of increased head differences and therefore resulted in greater leakage from the upper to lower aquifers.

A section through both aquifers showing lines of equal drawdown and direction of water movement is shown in figure 29. This diagram indicates the effectiveness of the enlarged recharge pond in contributing to recharge. By comparison with figure 23, which shows the same data for test 5, drawdowns are seen to be smaller because of water moving from the enlarged section of the recharge channel on the east side of the well field and because of recharge directly from the extension of the channel on the west side of the field.

During test 7, an average of about 1.35 cfs (600 gpm) was diverted from the creek to the recharge pond to maintain a relatively uniform water level in the pond (fig. 26). This rate was about double the rate of diversion—and approximate rate of recharge—recorded during test 6 and again demonstrated the effectiveness of the enlarged

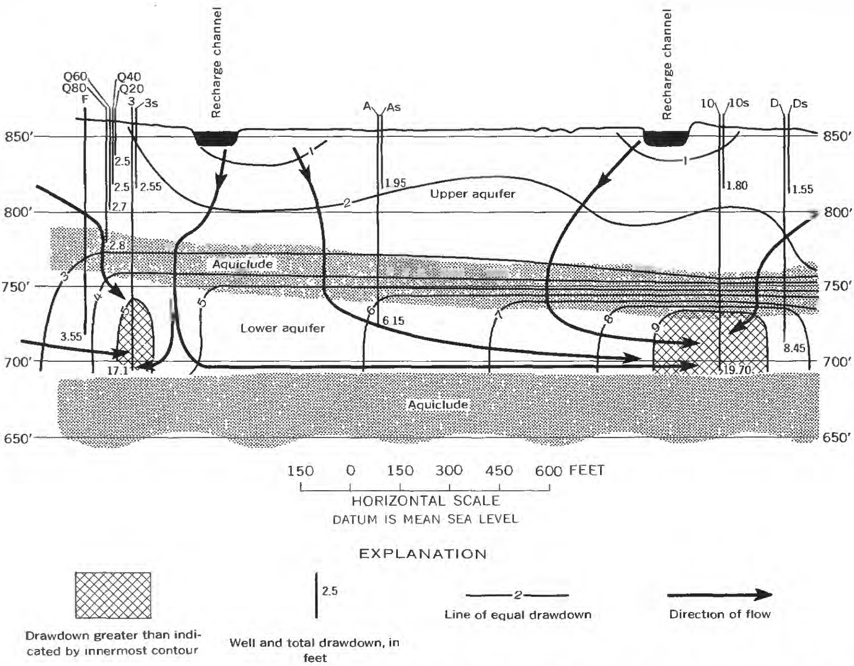

FIGURE 29.—Flow directions and drawdown at the end of test 7. (Courtesy of Am. Water Works Assoc.)

recharge channel. The average water level in the recharge channel during test 7 was about 0.4 foot lower than in test 6. Had it been possible to maintain the same head in the channel as in test 6, the recharge rate would have been greater. The range in fluctuation of the water level in the recharge channel was considerably greater than during test 6. Precipitation and subsequent runoff during the period February 6–11 caused the water level in the recharge channel to rise. Diversion from the creek was greatly reduced on February 11, 12, and 13 in an attempt to stabilize the water level in the channel.

SUMMARY OF RECHARGE TESTS

The principal effects of induced recharge upon the hydraulic behavior of two aquifers was to reduce drawdown and rate of drawdown. Figure 30 shows drawdown for tests 5, 6, and 7 as measured in various observation wells. The hydrographs demonstrate the benefit derived from maintaining the initial high head on the recharge pond in test 6 instead of permitting the decline in head as in test 5. The marked benefits in higher water levels and smaller rates of decline by greatly enlarging and extending the recharge pond for test 7 are also apparent. Water-level declines during the first few hours were about

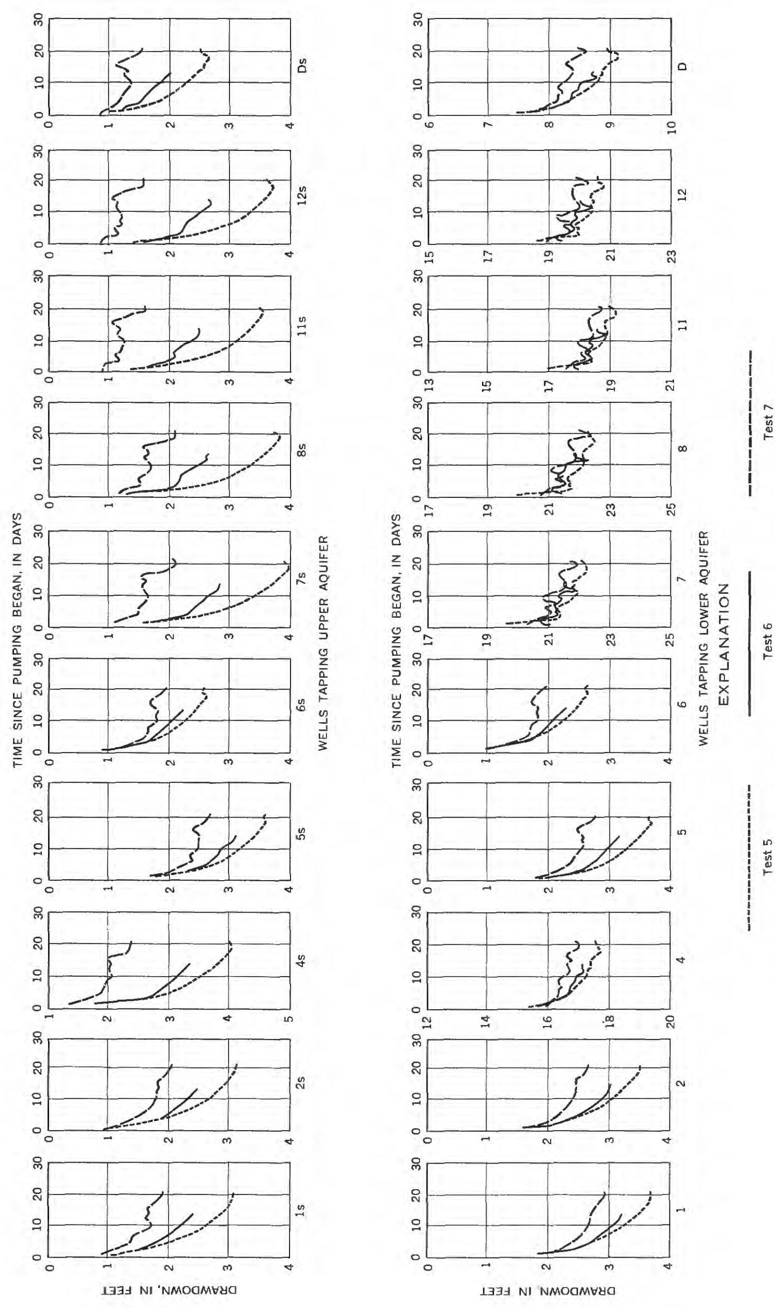

FIGURE 30.—Hydrographs of selected pairs of wells during recharge tests.

the same in all tests. After that time, the induced recharge changed the shape of the drawdown curves. The change in shape of drawdown curves appears earlier and is greater in the upper aquifer.

Rates of water-level decline were progressively reduced by induced recharge. The comparative drawdown in figure 30 is for pumping periods no longer than 21 days. These differences would tend to increase over longer periods of pumping. By further increasing the size of the recharge ponds, withdrawals can be increased without significantly greater drawdowns, although the size-discharge relationship that would result in stabilization of drawdowns cannot be calculated from the test data.

The increased infiltration rates from the surface ponds into the upper aquifer in each succeeding test is reflected in the increased amount of head difference between aquifers (fig. 31). The initial rate of leakage under "natural" conditions is relatively high soon after pumping begins; but as water is removed from storage in the upper aquifer, the head difference and, thus, the rate of leakage decline. At first the upper aquifer acts as a source for the pumped water, but later it functions more as a conduit transferring this water to the lower aquifer by leakage. The recharge ponds helped to maintain the upper aquifer as a source of water for interbed leakage by keeping head differences more nearly constant.

The differences in drawdown between upper and lower aquifers become smaller in observation wells farther from the pumped wells. This fact indicates that the major part of the interbed leakage induced by pumping occurs in and near the well field.

Another qualitative indication of the effects on the hydrologic regimen of the several recharge facilities tested at Station 9 is illustrated by figure 32. This figure shows drawdowns plotted against the logarithm of time in wells A and As, which tap the lower and upper aquifers, respectively. Adjusting for variations in slope caused by changes in stage of the recharge channel during tests 6 and 7, the time-drawdown slope is drawn as a straight line. Averaging the slopes for both wells and considering the two aquifers as a unit, an "apparent transmissibility" (T_a) for the unit was calculated. Using the equation

$$T_a = \frac{264\,Q}{\Delta s}$$

in which Δs is the change in drawdown per log cycle and Q is the rate of pumping in gallons per minute, the values for T_a were calculated to be as follows:

Test	Ta (*gpd per ft*)
5	385, 000
6	525, 000
7	750, 000

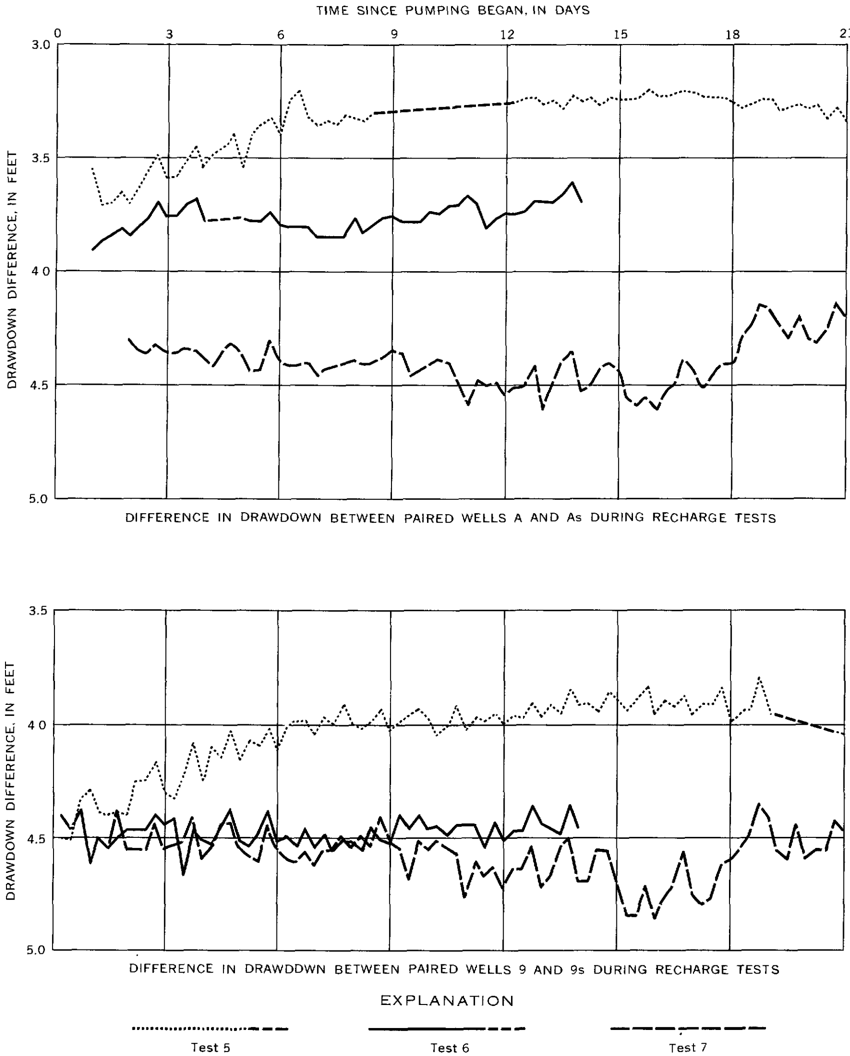

Figure 31.—Drawdown differences between wells during recharge tests. (Courtesy of Am. Water Works Assoc.)

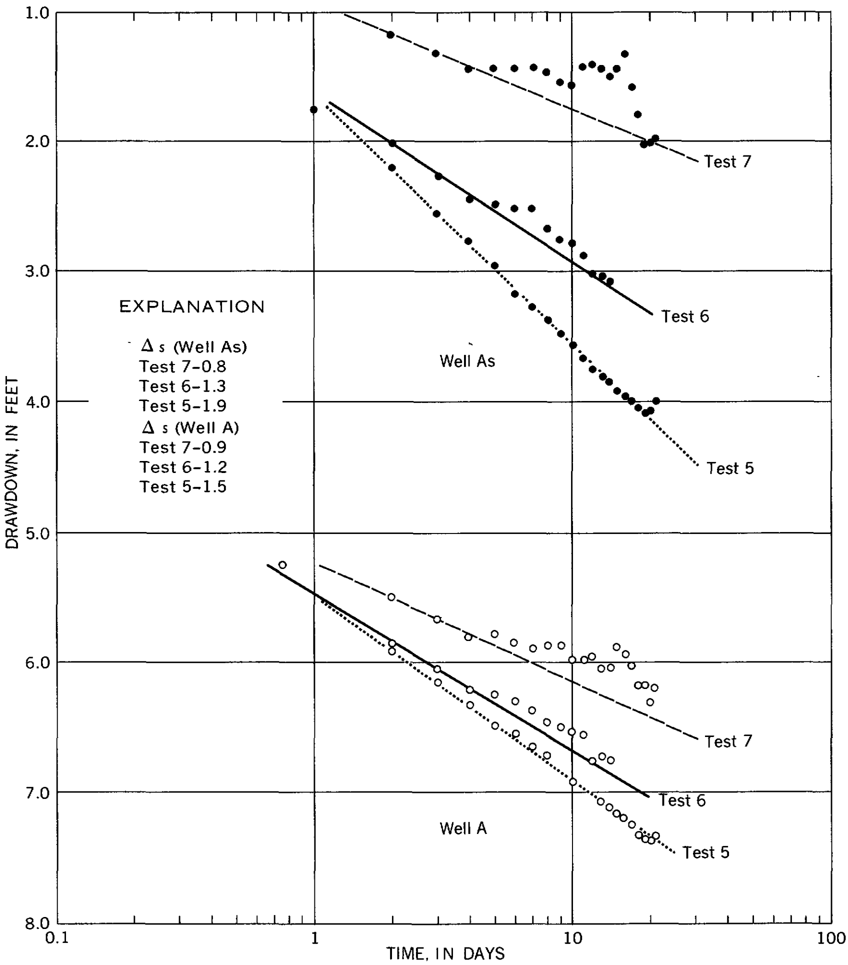

FIGURE 32.—Time-drawdown graphs for wells A and As during recharge tests. (Courtesy of Am. Water Works Assoc.)

The above values are obviously too high for true transmissibility, as the aquifer tests indicated the average transmissibility for both aquifers to be 180,000 gpd per ft. The coefficient of transmissibility for an aquifer by definition does not include the effects of recharge. The apparent transmissibilities indicate that the recharge facility resulted in significant increases over the natural water-yielding capability of the aquifers.

Another effect of the improved recharge conditions caused by enlarging the pond is a reduction in infiltration from the creek. Measurements made in the piezometer, driven into the West Fork, are compared with the stage of the creek in figure 33. The piezometer screen, which is 0.2 foot long, was set 1 foot beneath the bed of the creek. Figure 33 shows that the head difference between the stage of the West Fork and the water table in the upper aquifer beneath the creek was greater in tests 5 and 6 than in test 7. This fact shows that the amount of infiltration directly from the creek during test 7 decreased because of the enlargement of the recharge pond. The recharge channel apparently is merely substituting for the creek as a source of recharge as more extensive desaturation of the upper aquifer beneath the creekbed occurred during tests 5 and 6 than in test 7.

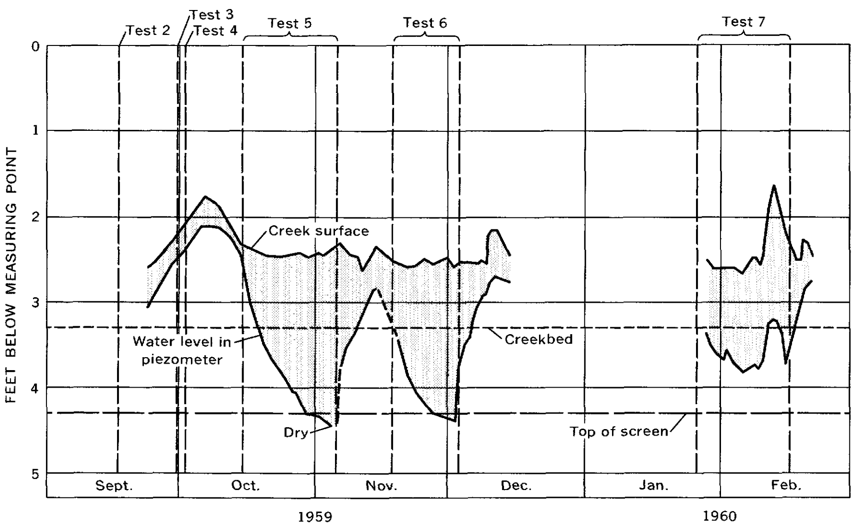

FIGURE 33.—Head difference between the West Fork and the upper aquifer 1 foot beneath the creekbed. (Courtesy of Am. Water Works Assoc.)

Figures 21, 24 and 27 show that water-level declines in the upper aquifer extended beyond the creek during tests 5 and 6 and to a lesser extent during test 7. Ground-water levels declined nearly 2 feet below the level of the stream during tests 5 and 6. The increasing head difference recorded during these tests showed that an increasing amount of water was being recharged to the upper aquifer directly from the creek. Shortly before tests 5 and 6 ended, however, the sediments below the creekbed became unsaturated, a fact demonstrating that a maximum rate of recharge from the section of the creek in the vicinity of the piezometer had been reached. Figures 21, 24, 27 and 33 show that the creek does not form a recharge boundary of zero drawdown and that water was moving northward beneath the creek in the upper aquifer.

A similar piezometer was installed in the bed of the recharge pond near the channel water-stage recorder (fig. 11), but it did not function properly as it seemed to record only the water level in the pond.

EFFECT OF RECHARGE TESTS ON STREAMFLOW

A general loss in streamflow between Oakland Drive and the Dillon Weir was recorded throughout the experiment. These losses generally increased during periods of pumping at Station 9. The differences in discharge between these gaging stations are plotted in figure 34. These graphs show that the loss of flow between the Oakland Drive gage and the well-field gage was somewhat greater than the

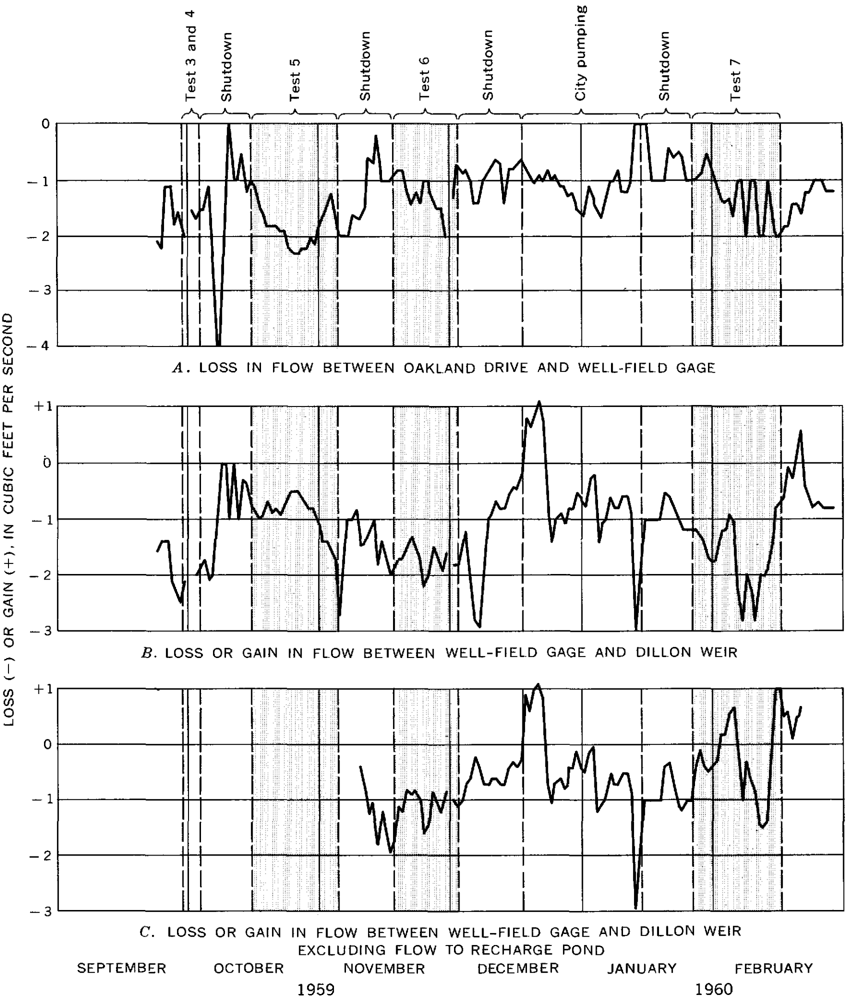

FIGURE 34.—Loss or gain in flow of West Fork near Station 9.

loss between the well-field gage and the Dillon Weir gage. The average losses between Oakland Drive gage and the well-field gage were as follows:

Test	*Average loss (cfs)*
5	1.7
6	1.3
7	1.4

Average losses between the well-field gage and the Dillon Weir gage were:

Test	*Average loss (cfs)*
5	1.0
6	1.7
7	1.7

In tests 6 and 7, however, the diversion to the recharge pond was 0.6 and 1.3 cfs, respectively. Thus, the loss directly from the creek was 1.1 cfs in test 6 and 0.4 cfs in test 7 for the reach between the well-field gage and Dillon Weir.

Probably these losses cannot be entirely attributed to pumping of ground water at Station 9. The general influent conditions created by pumping of ground water throughout the Kalamazoo area and the variations in direct surface runoff to the West Fork in some undeterminable degree affect the observed losses. Nevertheless, the pumping at Station 9 has the greatest and most direct effect upon the losses in streamflow in the immediate vicinity of the station so that the observed losses can be used to draw some general conclusions.

In the reach between the well-field and Dillon Weir gages, total losses in tests 6 and 7 were obviously greater than in test 5 because of the recharge via the diversion through the flume into the recharge pond. The total loss, or recharge, in tests 6 and 7 was about the same. In test 6, about one-third of the total was recharged from the recharge pond and the rest, directly from the creek. But in test 7, about three-fourths of the total was recharged from the recharge pond and only one-fourth, directly from the creek.

During test 5—when no water was being diverted to the recharge channel—about as much water was being recharged through the creekbed below the well-field gage as in test 6. The pumping rate at the well field was the same in both tests, but the smaller total recharge rate in test 5 resulted in greater drawdowns and an expanded cone of depression. The greater drawdowns at the well field during test 5 did not cause an increase in leakage through the creekbed in the vicinity of Station 9 because the maximum infiltration capacity of the creek bottom had already been reached (see fig. 33).

EFFECTS OF RECHARGE ON WATER QUALITY

Ground water at Station 9 and surface water from the West Fork Portage Creek are of the calcium magnesium bicarbonate type. Water from the lower aquifer is higher in mineral content than water from the upper aquifer. Surface water is generally lower in mineral content than ground water in the area. Representative chemical analyses of samples of ground and surface water are given in table 5.

Investigations in the Holland area, where the glacial-drift aquifers are also underlain by the Coldwater Shale, showed that water from municipal wells increased in sulfate content over a period of several years when water levels were lowered by pumping (Deutsch, Burt, and Vanlier, 1958). Generally, when water levels are lowered in drift aquifers overlying the Coldwater Shale, the sulfate content tends to increase because of induced migration of water of high-sulfate content from below. As a part of the present study, water samples from both aquifers, the recharge channel, and the creek in the vicinity of the station were collected periodically to determine if recharge at the station would tend to change the sulfate content of the water.

Before the recharge tests were started on October 15, 1959, the sulfate content of samples from 6 wells tapping the lower aquifer averaged 34 ppm; at the same time sulfate content of samples from 18 wells tapping the upper aquifer averaged 18 ppm. The sulfate content of two samples from the West Fork collected during August was 11 ppm. The prediction was therefore made that as recharge from the surface sources and upper aquifer to the lower aquifer was increased, the sulfate content of water taken from the lower aquifer would decrease. A slight decrease in sulfate content was noted between the beginning and end of test 7 in 3 out of 4 wells tapping the lower aquifer (fig. 35). Average sulfate content at the beginning of the test was 32 ppm compared with 27 ppm at the end of the test. During the same period, average sulfate content of water from the 4 wells tapping the upper aquifer increased slightly from about 17 to 19 ppm. The results were by no means conclusive enough to demonstrate that a declining trend in sulfate content of water from the lower aquifers had been established, but they did suggest that the sulfate content should not significantly increase as long as high water levels are maintained by recharging water of lower sulfate content.

Long-term induced recharge would also be expected to decrease the hardness of the water yielded at the station because a greater proportion of the water produced will come from surface sources and the upper aquifer in the area. Hardness of surface water sampled ranged from 168 to 198 ppm and averaged 181 ppm. Samples from the upper aquifer ranged from 165 to 331 ppm and averaged 206 ppm,

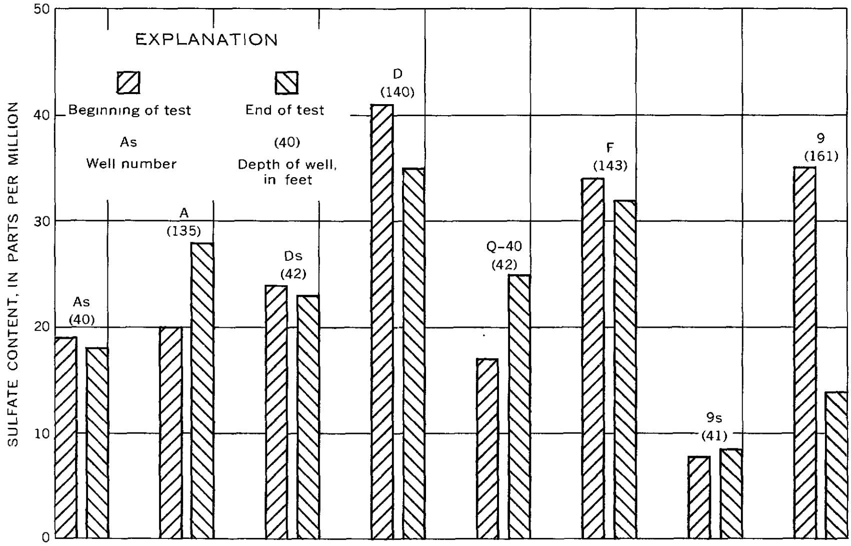

FIGURE 35.—Sulfate content of water samples from selected pairs of wells at the beginning and end of test 7.

while the average hardness of water from the lower aquifer was 251 ppm.

Plans were also made to observe changes in iron content of water from the various sources and to determine effects of recharge on iron content of the water from the lower aquifer. The data obtained, however, was inadequate for the purpose, and the results were inconclusive.

There was a considerable areal variation in temperature of ground water from the upper aquifer before the recharge tests began. The range was from 47.64° to 65.35° F, averaging 54.4° F, and reflected the effect of recharge from surface-water bodies of varying temperatures. The temperature at any point was affected by the temperature of the recharge water when the water first entered the aquifer and by the length of time that the water was in contact with the aquifer. As the distance of travel (and time of contact) increased, the temperature of recharged water approached "normal" ground-water temperature through heat exchange with the aquifer. Therefore, a water temperature higher than the average indicated initial recharge during the summer and a short period of travel from the recharge source. A temperature lower than average indicated not only a short period of travel but also that initial recharge was during the winter months. The high temperature at well 6s (65.35° F) probably reflected recharge-water temperature from the nearby West Fork Portage Creek during the summer of 1959. The low temperature at well 5s (47.64°

F) was probably due to recharge water from the previous winter (1958–59).

Efforts to observe the effects of recharge upon water temperatures from the lower aquifer by periodic sampling failed due to the fact that the sampling method allowed air temperature to greatly affect the temperature of the sample. The increased recharge would be expected to increase the ground-water temperature somewhat. This temperature increase would be caused by a greater amount of recharge during the summer months due to a greater rate of pumping and to lower water viscosities during those months.

CONCLUSIONS

If the aquifers at Station 9 were homogenous and infinite in areal extent, then by greatly expanding the cone of depression and increasing the drawdowns as much recharge could be induced from the creek alone as is presently induced from the creek and the recharge pond. The aquifers at Station 9, however, differ greatly in permeability, both laterally and vertically, and seem to be discontinuous and small in areal extent, as are most glacial-drift aquifers. In this type of aquifer, expansion of the cone of depression is limited by the presence of boundaries and by the available drawdown. Therefore, with little drawdown, more water can be induced into the aquifers from the creek and recharge pond than could be recharged from the creek alone, even with large drawdowns. Construction of the enlarged recharge pond provided greatly increased infiltration areas where optimum hydraulic gradients could be created.

Although the field has been in operation since 1957, water levels in observation wells A and As tapping the upper and lower aquifers (fig. 36) were only about 4 and 8 feet, respectively, below land surface during most of the heavy pumping season of 1962. No significant dewatering has occurred.

The field tests indicated that the following advantages may be gained from induced recharging as practiced at Kalamazoo:

1. Water levels and artesian pressures can be maintained at high stages. The results are lower pumping lifts and substantial reductions in the amount of power used for pumping. The artificial channel, therefore, provides a much more effective source of recharge than the creek.
2. Interference with nearby wells is minimized.
3. The source aquifer continues to be recharged, even when pumping is stopped.
4. The high water levels permit increased rates of withdrawal during periods of peak demand.

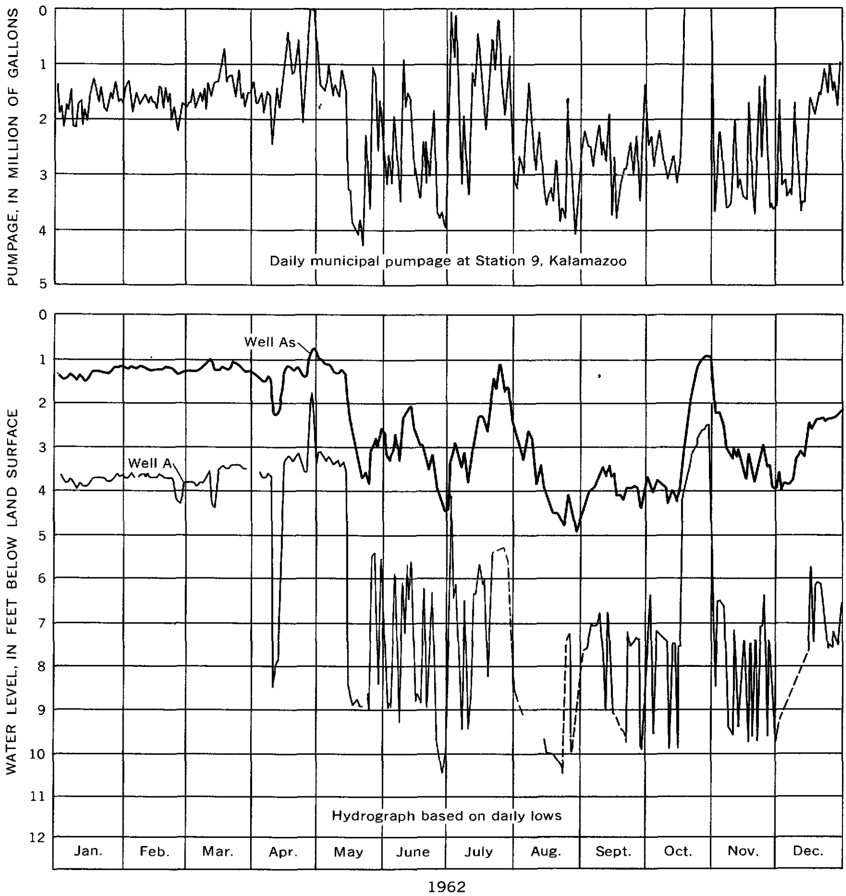

Figure 36.—Pumpage at Station 9 and hydrographs of wells A and As at Kalamazoo during 1962.

5. Encroachment of poor quality water from other aquifers is apparently prevented.
6. The surface water induced into the aquifer is filtered naturally through great thicknesses of earth materials.
7. Natural underground storage is used to conserve and protect water which otherwise would flow largely to waste.

The tests demonstrated that it is quite feasible to recharge artesian glacial-drift aquifers by induced infiltration and, further, that the hydraulics of the system can be analyzed. From the recharging facilities at Station 9, Kalamazoo has clearly gained distinct advantages which are economically significant and increase the dependability of this source of water supply. Kalamazoo's experience in induced recharging, therefore, is potentially of great value where applicable in other areas with similar hydrogeologic environments.

TABLE 3.—*Drillers' logs of wells in and near Station 9*

[Quoted verbatim. Thickness in feet. Depth in feet below land surface]

	Thickness	Depth		Thickness	Depth
1 (3S 11W 4-1)					
No record	78	78	Gravel	13	137
Sand	14	92	Sand	7	144
Gravel, brown	10	102	Gravel	13	157
Gravel, brown	14	116	Sand, fine	5	162
Gravel	8	124			
2 (3S 11W 4-2)					
No record	77	77	Clay and gravel hardpan	3	112
Sand, clay, and stones	3	80	Sand, clayey	11	123
Clay and hardpan, large boulders	2	82	Sand, coarse	7	130
Sand and gravel	10	92	Sand	5	135
Gravel and brown clay	10	102	Sand and gravel	28	163
Gravel and brown to gray clay	7	109			
3 (3S 11W 4-3)					
No record	96	96	Gravel	10	158
Sand	9	105	Sand	6	164
Sand and gravel	43	148			
4 (3S 11W 4-4)					
No record	61	61	Sand, gravel, and brown clay	12	134
Gravel, coarse, and gray clay	24	85	Gravel and sand, clayey	11	145
Sand, coarse, and boulders	11	96	Coarse gravel, fairly clean	11	156
Sand, brown, gravelly, clayey	8	104	Gravel, sandy, clayey	6	162
Clay and gravel	10	114	Hardpan	------	162
Clay and sand	8	122			
6 (3S 11W 4-6)					
Fill	3	3	Sand, fine, and brown clay	9	120
Muck	3	6	Sand	5	125
Sand	16	22	Sand, brown, coarse	8	133
Sand, clayey	16	38	Sand, coarse, and clay	7	140
Hardpan	6	44	Sand, gravelly	6	146
Gravel, clayey	26	70	Sand and gravel, clean	20	166
Sand, fine to coarse	12	100	Clay, hardpan	------	------
Gravel and brown clay	11	111			

TABLE 3.—*Drillers' logs of wells in and near Station 9*—Continued

	Thickness	Depth		Thickness	Depth
7 (3S 11W 4–7)					
Fill	3	3	Sand, gravel and large boulders	8	97
Topsoil and muck	2	5	Sand and gravel hardpan	9	106
Sand and marl	15	20	Sand and brown clay hardpan	17	123
Sand and gravel, clayey	10	30	Sand and soft clay	6	129
Sand and clay	10	40	Sand and gravel	23	152
Sand, coarse, clayey	12	52	Gravel	6	158
Sand and gravel, clayey	5	57	Hardpan	------	------
Gravel	21	78			
Sand, clean	11	89			
8 (3S 11W 4–8)					
Fill	3	3	Sand and gravel	7	136
Muck	3	6	Sand	6	142
No record	72	78	Sand and gravel	7	149
Sand	8	86	Sand, some gravel	6	155
Sand, fine	14	100	Gravel, coarse	4	159
Sand, gravel, and clay	20	120	Hardpan	------	------
Coarse sand	9	129			
9 (3S 11W 4–9)					
Fill	3	3	Clay, very hard, and boulders	8	88
Muck	3	6	Sand and gravel, clayey	16	104
Sand, marl, clayey	25	31	Sand and gravel, boulders, clayey	10	114
Clay, blue, soft	8	39	Sand and clay, gravelly	10	124
Sand, gravel with soft brown clay	6	45	Sand, gravelly	6	130
Sand and gravel	15	60	Sand and gravel	31	161
Sand and gravel, clayey	8	68	Hardpan	------	------
Sand, gravel, boulders, and clay	12	80			
10 (3S 11W 4–10)					
No record	77	77	Sand, gravelly, yellow clay	9	127
Gravel and sand	15	92	Sand with clay	10	137
Sand and gravel, boulders	8	100	Sand and gravel, clayey	17	154
Sand and gravel with clay balls	9	109	Gravel	3	157
Sand and gravel, with yellow clay	9	118	Clay and gravel	5	162
			Hardpan	------	------

TABLE 3.—*Drillers' logs of wells in and near Station 9*—Continued

11 (3S 11W 4-11)

	Thickness	Depth		Thickness	Depth
Fill	3	3	Gravel hardpan	10	93
Muck	3	6	Sand hardpan	8	101
Sand, yellow, and gravel	14	20	Gravel hardpan	8	109
Sand, brown, and gravel	26	46	Sand hardpan	6	115
Gravel	31	77	Gravel hardpan	11	126
Clay hardpan	6	83	Sand and gravel	42	168
			Hardpan	------	------

12 (3S 11W 4-12)

	Thickness	Depth		Thickness	Depth
No record	12	12	Sand, gravelly	9	143
Sand	62	74	Sand, gravel, and clay	9	152
Gravel	11	85	Sand and gravel	8	160
Sand and gravel	20	105	Sand, gray	2	162
Sand and clay	23	128	Sand, gravel, and hardpan	------	------
Gravel	6	134			

13 (3S 11W 4-13)

	Thickness	Depth		Thickness	Depth
Topsoil	2	2	Gravel and clay hardpan	34	105
Clay, blue and sand	15	17	Sand and clay hardpan	9	114
Sand, gray	18	35	Sand, fine, and clay	17	131
Sand, yellow, dirty	5	40	Sand and gravel	8	139
Sand, gray	12	52	Sand and gravel	26	165
Sand, gray, and gravel	19	71	Clay, blue and sand	5	170
			Hardpan (shale)	30	200

132 (3S 11W 4-14)

	Thickness	Depth		Thickness	Depth
Black muck	2	2	Clay balls, yellow, and sand	5	105
Sand, gray, dirty	15	17	Sand, yellow, and clay	17	122
Sand, gray, and gravel	18	35	Sand, fine and clay	8	130
Sand, black and gravel	18	53	Sand, gray, coarse and gravel	20	150
Sand, gray and gravel and boulders, dirty	25	85	Gravel, coarse	13	163
Sand, gray and gravel, clean	15	100	Hardpan (shale?)	33	196

133 (3S 11W 4-15)

	Thickness	Depth		Thickness	Depth
Muck, black	2	2	Hardpan	10	122
Sand, black, fine	8	10	Sand, gray, and clay	14	136
Sand, yellow	25	35	Sand, gray, and gravel, clayey	4	140
Sand, gray, coarse, and gravel	49	84	Gravel, gray, coarse, and sand, boulders	22	162
Sand, gray, and gravel, clayey	12	96	Hardpan	68	230
Sand, gray, fine	7	103	Hardpan, blue	10	240
Sand, yellow, and gravel, boulders	5	108	Lime and shale hardpan	16	256
Sand, yellow, and clay	4	112	Shale, black	4	260

TABLE 3.—*Drillers' logs of wells in and near Station 9*—Continued

	Thickness	Depth		Thickness	Depth
139 (3S 11W 4–16)					
Sandy topsoil	5	5	Sand and gravel	8	85
Sandy marl like mixture	10	15	Sand and gravel with clay turning to brown color	9	94
Sand—gravel with yellow clay	10	25	Sand with less gravel and a light brown color	9	103
Gravel with less clay	9	34	Coarse gravel with brown clay	6	109
Sand and gravel with yellow clay turning gray at 48 feet	14	48	Fine sand and clay	7	116
Coarse gravel and large stones in clay	10	58	Coarse sand and gravel with brown clay	9	125
Coarse gravel and stones with less clay	9	67	Sand and gravel with gray clay	5	130
Gravel—cleaner, less clay, not many large stones	10	77			
140 (3S 11W 4–17)					
Topsoil	5	5	Clay, gray, and gravel hardpan	8	111
Topsoil, sandy with brown clay	15	20	Sand and gravel, clayey	10	121
Sand, with grayish clay	10	30	Sand and gravel	9	130
Sand, clean	21	51	Sand and gravel, large boulders	10	140
Sand and fine gravel, clean	9	60	Clay, brown, and gravel hardpan	3	143
Sand and coarse gravel	10	70	Clay, brown, and sand	7	150
Sand and gravel	8	78	Sand, gray	9	159
Sand, clayey	6	84	Sand, coarse	3	162
Sand and gravel, clayey	10	94	Sand, coarse, gravelly	2	164
Sand and coarse gravel	9	103			
A (3S 11W 4–35)					
Muck	1	1	Sand, gray, fine	2	59
Clay, sandy	2	3	Sand, fine to coarse, and gravel	13	72
Gray sand, medium	1	4	Sand and clay balls	12	84
Sand, gray, coarse	7	11	Sand, coarse	8	92
Sand, brown, coarse	10	21	Sand and gravel, silty	9	101
Sand, gray, fine to coarse	6	27	Clay and gravel, silty (hard)	2	103
Sand and gravel	2	29	Silt, gravel, and clay	3	106
Sand, gray	17	46	Sand, silty, fine	8	114
Sand, clay, silt and gravel	2	48	Sand, fine to coarse	4	118
Sand, fine, and silt	1	49	Sand, coarse, clean	6	124
Sand, fine to coarse, and gravel	8	57	Sand, coarse, gravel, and silt	7	131

TABLE 3.—*Drillers' logs of wells in and near Station 9*—Continued

	Thickness	Depth		Thickness	Depth
C (3S 11W 4-37)					
Topsoil	1	1	Sand and gravel, clayey	10	70
Sand, yellow and clay	2	3	Hardpan	10	80
Sand, yellow	17	20	Gravel	10	90
Clay	6	26	Sand and clay	5	95
Sand and gravel	3	29	Sand, gravelly	15	110
Clay hardpan	1	30	Sand, gravel, and clay	10	120
Gravel	10	40	Sand and gravel	10	130
Sand and gravel, clayey	10	50	Sand and gravel with brown clay	9	139
Sand and gravel, clayey	10	60			
F (3S 11W 4-42)					
Sand, yellow	7	7	Sand and gravel	6	61
Muck	1	8	Sand	8	69
Sand, dirty	7	15	Sand	9	78
Sand, clayey	2	17	Gravel and gray clay	8	86
Sand and brown clay	9	26	Sand, gravel, and clay	9	95
Sand, gravelly, with brown clay	13	39	Sand, gravel, and clay	9	104
Sand, coarse, with brown clay	4	43	Sand and clay, gravelly	8	112
Sand, gravelly, cleaner	8	51	Gravel and silt	8	120
Sand, gravelly	4	55	Sand and gravel, clayey	10	130
			Sand and gravel, clayey	12	142
			Sand and gravel	2	144

TABLE 4.—*Record of wells at Station 9*

Well number: Most locations plotted on fig. 11.
Total depth: In feet below measuring point.
Altitude: In feet above mean sea level. MP, measuring point; LSD, land-surface datum.
Water level: In feet below measuring point.
Date: Date of measurement of water level; all measurements made in the year 1959.
Use: P, public supply; T, test well; O, observation well.
Remarks: Includes use of well during experiment; R, equipped with continuous water-level recording gage; L, indicates that log is available.

Well No.		Date drilled	Total depth (feet)	Diameter of casing (inches)	Altitude		Water level	Date	Use	Remarks
Field	Office				MP	LSD				
1	3S 11W 4–1	Mar. 1950	162	12	860.15	857.95	10.25	Sept. 23	P	Observation well.
2	4–2	Jan. 1950	163	12	860.05	857.75	8.68	do	P	L; observation well.
3	4–3	1949	164	12	860.03	858.43	8.30	do	P	L; pumped well for test 4; pumping well for recharge test series.
4	4–4	Apr. 1950	162	12	860.01	858.91	7.83	do	P	Do.
5	4–5	June 1950	167	12	863.02	861.52	10.22	do	P	Observation well.
6	4–6	Apr. 1951	166	12	863.07	861.77	9.64	do	P	L; observation well.
7	4–7		158	12	859.98	858.48	10.10	do	P	L; pumping well for recharge test series.
8	4–8		159	12	859.66	857.76	10.53	do	P	Do.
9	4–9		161	12	859.86	858.06	10.80	Sept. 29	P	L; R; observation well, pump removed.
10	4–10		162	12	859.07	857.87	9.76	Sept. 23	P	Pumped well for test 3; pumping well for recharge test series.
11	4–11		162	12	859.86	857.86	9.23	do	P	L; pumped well for test 3; pumping well for recharge test series.
12	4–12	Sept. 1951	162	12	860.01	858.31	7.98	do	P	Do.
131	4–13	1949	200						T	Abandoned.
132	4–14	Mar. 1949	196						T	Do.
133	4–15	May 1949	260						T	L; abandoned.
139	4–16	Sept. 1950	130	6					T	Do.
140	4–17	do	164	6					T	Do.
134A	4–18	Oct. 1949	133	2			6.51	Oct. 6	O	Observation well for 1949 aquifer test.
134D	4–21	do	130	2			6.04	do	O	Do. 3 ft east of well 4.

1s		4-23	Aug. 1959	31	3	859.35	858.05	8.40	Sept. 23	O	Observation well for upper aquifer.
2s		4-24	do	42	2	862.13	857.93	10.30	do	O	Do.
3s		4-25	do	42	4	859.88	858.38	7.50	Sept. 25	O	R; observation well for upper aquifer.
4s		4-26	do	42	2	863.09	859.09	10.69	Sept. 23	O	Observation well for upper aquifer.
5s		4-27	do	42	2	865.86	861.86	12.77	do	O	Do.
6s		4-28	do	42	2	865.90	861.90	12.24	do	O	Do.
7s		4-29	do	42	3	859.78	858.48	8.12	do	O	Do.
8s		4-30	do	42	3	859.46	857.86	7.89	do	O	Do.
9s		4-31	do	41	4	858.90	858.10	7.24	Sept. 25	O	R; observation well for upper aquifer.
10s		4-32	do	41	4	859.07	857.87	6.79	Sept. 3	O	Do.
11s		4-33	do	42	3	861.03	858.03	8.58	Sept. 23	O	Observation well for upper aquifer.
12s		4-34	do	42	3	860.85	858.25	7.86	do	O	Do.
A		4-35	Oct. 1959	135	3	857.63	854.03	5.46	Oct. 13	O	R; observation well, lower aquifer.
As		4-36	do	40	3	857.81	854.01	4.93	do	O	R; observation well, upper aquifer.
B	2S 11W	33-5	do	152	3			14.79	Oct. 14	O	R; distant observation well in lower aquifer.
C	3S 11W	4-37	Sept. 1959	139	4	859.24	854.84	9.29	Sept. 23	O	R; observation well, lower aquifer; screen plugged, date invalid.
Cs		4-38	do	39	3	857.51	854.71	6.46	Sept. 22	O	R; observation well, lower aquifer.
D		4-39	Aug. 1959	140	3	858.79	854.09	9.71	Sept. 23	O	Do.
Ds		4-40	do	42	3	858.20	854.20	6.76	do	O	Pumped well for test 2; observation well, upper aquifer.
E		4-41	Sept. 1959	141	3	860.92	858.92	8.96	Sept. 15	O	R; observation well, lower aquifer.
F		4-42	Aug. 1959	143	6	864.30	860.40	12.82	Oct. 1	O	R; pumped well for test 1 when total depth was 55 ft; observation well, lower aquifer.
Q-20		4-43	do	22	2	863.68	860.48	10.99	Sept. 28	O	Observation well for vertical flow and head loss analysis.
Q-40		4-44	do	42	2	864.47	860.37	11.78	do	O	Do.
Q-60		4-45	do	62	2	864.23	860.33	11.59	do	O	Do.
Q-80		4-46	do	82	2	864.26	860.66	11.69	do	O	Do.

TABLE 5.—*Selected analyses of water samples from the West Fork Portage Creek and wells at Station 9*

[Chemical constituents in parts per million. Asterisk indicates analysis by Michigan Dept. of Health]

Source	Date of collection	Temperature (° F)	Iron (Fe)	Calcium (Ca)	Magnesium (Mg)	Sodium (Na)	Potassium (K)	Bicarbonate (HCO_3)	Sulfate (SO_4)	Chloride (Cl)	Nitrate (NO_3)	Dissolved solids (total)	Hardness as Ca CO_3		Specific conductance (micromhos at 25° C)	pH
													Calcium, magnesium	Noncarbonate		
Surface water																
West Fork*	8-14-52	------	------	46	19	1.5	------	228	10	0	0	204	198	------	400	7.9
Do	8- 8-59	74.1	------	38	18	3.0	0.3	201	11	2.0	.3	179	169	4	320	7.8
Do	8-13-59	81	0.14	36	19	------	------	194	11	------	------	------	168	9	312	7.6
Recharge channel	11- 3-59	------	.02	43	19	2.3	.7	214	12	4.0	------	------	185	10	347	8.0
West Fork	4-21-61	------	.09	45	18	------	------	203	20	19	------	------	186	20	361	7.9

Upper aquifer																
Well 3s	8- 6-59	55.23	0.32	38	18	-----	-----	186	17	-----	-----	178	169	16	332	7.8
4s	8- 6-59	54.2	.19	53	19	-----	-----	239	19	-----	-----	216	210	14	396	7.8
5s	8- 8-59	47.64	.98	50	19	-----	-----	229	16	-----	-----	197	203	16	381	7.6
6s	8- 8-59	65.35	.61	39	16	-----	-----	192	12	-----	-----	172	164	6	318	7.8
7s	8-27-59	62±	4.2	41	18	-----	-----	210	7.6	-----	-----	191	176	4	337	7.5
8s	8-27-59	59.68	4.4	56	18	-----	-----	256	10	-----	-----	231	214	4	408	7.5
9s	8-27-59	55.50	2.2	54	18	-----	-----	232	22	-----	-----	226	209	18	398	7.5
10s	8-27-59	55.55	.68	52	18	-----	-----	228	18	-----	-----	214	204	16	383	7.6
11s	8-27-59	53.05	.10	53	18	-----	-----	236	18	-----	-----	209	206	12	386	7.6
12s	8-27-59	52.6	1.1	55	18	-----	-----	228	29	-----	-----	231	211	24	405	7.5
As	10-12-59	51.0	.88	83	30	-----	-----	386	10	-----	-----	314	331	14	578	6.9
Cs	9-21-59	53.9	.23	58	21	-----	-----	230	22	-----	-----	248	231	42	442	7.1
Ds	8-27-59	51.65	.09	49	18	-----	-----	218	18	-----	-----	212	196	18	377	7.6
F-55	8- 6-59	50.91	.19	56	18	2.3	0.5	240	18	3.0	3.3	221	214	17	395	7.5
Q-20	8- 6-59	50.14	.02	44	15	-----	-----	192	18	-----	-----	174	172	14	320	8.0
Q-40	8- 6-59	50.34	.04	57	18	-----	-----	228	24	-----	-----	225	216	29	403	7.8
Q-60	8- 6-59	52.25	.46	58	19	-----	-----	242	26	-----	-----	238	223	24	417	7.4
Q-80	8- 6-59	51.71	.06	36	19	-----	-----	186	18	-----	-----	183	168	16	320	7.8
Lower aquifer																
Well 2*	8-14-52	52.10	7.5	58	29	3.6	-----	293	20	4.0	0	302	265	------	500	7.6
3	5-17-61	50.5	.33	61	20	3.6	0.6	246	26	4.0	.1	247	234	32	430	7.3
Composite from 7 wells	11-27-59	51.5	.80	63	23	-----	-----	268	32	-----	-----	258	252	32	464	7.2
Do	12- 2-59	51.5	------	65	23	-----	-----	272	32	-----	-----	252	257	34	472	6.9
Do	4-21-61	-------	.24	62	22	5.1	.4	255	31	11	.1	273	245	36	449	7.9
Well A	10-12-59	51.6	14	58	25	-----	-----	240	38	-----	-----	342	248	51	442	7.0
B	10-13-59	52.15	.79	36	20	-----	-----	190	20	-----	-----	154	172	16	329	7.3
C	9-21-59	56.72	.75	52	37	-----	-----	290	43	-----	-----	290	282	44	512	7.1
D	8-27-59	55.52	.61	71	25	-----	-----	295	42	-----	-----	294	280	38	513	7.6
F-143	9-28-59	52.2	1.9	70	25	-----	-----	298	43	-----	-----	312	278	34	503	7.0

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