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Drainage by Wells—An Investigation in the Patterson Water District

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A ground-water study was undertaken to investigate the feasibility of controlling water levels and draining agricultural land in areas of the west side of the San Joaquin Valley, California. The region selected for the field study was south of the town of Patterson, in Stanislaus County.

Ground-water levels in the area had been rising due to intensive irrigation coupled with increased underground flow from new irrigated land which had been developed with water from the Delta-Mendota Canal.

A test well, 10 shallow observation wells, and 21 deep piezometer holes were drilled. Well logs were recorded and correlated with general geologic information. The water quality in the test well was determined periodically from 1959 to 1961. Shallow water-table levels and deep piezometric-table levels were recorded continuously from October 1959 to July 1961.

Several pumping and recovery tests were run and conventional as well as new methods of well hydraulic analysis were applied to the test data.

An optimum pattern and spacing were computed for the proposed drainage wells.

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INTRODUCTION

IN AN INCREASING number of situations in California, a properly designed and constructed drainage system has been found essential for maintaining satisfactory production on agricultural lands. Drainage is necessary for the growing of crops as well as for the reclamation and preservation of land. One of several possible methods of drainage is the use of pumped wells. The choice of one type of system over another depends on local conditions, such as surface and subsurface soil characteristics; irrigation management practices; source of the drainage water; water quality; reuse of drainage water for irrigation; area of influence of the system; and costs, including installation, operation and maintenance; and the preference of owners.

Drainage by a pumped well or a system of wells may be satisfactory under certain conditions, for example when ground water is moving laterally and/or vertically into an area, or when downward movement of surface water is restricted by an impervious layer.

If drainage by pumping is to be used, two major factors should be thoroughly investigated: (1) the effectiveness of wells in maintaining a reasonable water table depth during the period of usual or expected high water-table levels; and (2) the quantities and cost of water pumped and the possibility of using the pumped water for irrigation.

This study presents the results of an investigation and analysis of a drainage problem which slowly developed over a period of several years in the Patterson Irrigation District. During the past 10 to 15 years the drainage problem became more serious as a result of irrigation of recently developed lands upslope from the District, and overirrigation. In general, these lands are relatively permeable in receiving and transmitting water. It was believed that the irrigation of this area was a source of water which moved toward and into the area of low-permeability soils, resulting in an excessive buildup of the water table.

An investigation was undertaken to determine the feasibility of an interception drainage-well system which would alleviate the drainage problem. Consideration was given to the design, number, and spacing of wells which could be incorporated into a drainage system. The study was implemented by drilling a pumped well and installing a grid of piezometers around it. The data collected from the pumping tests and the piezometer readings were necessary for evaluating the characteristics of the water-bearing materials and for determining the time of the year during which pumping would be necessary.

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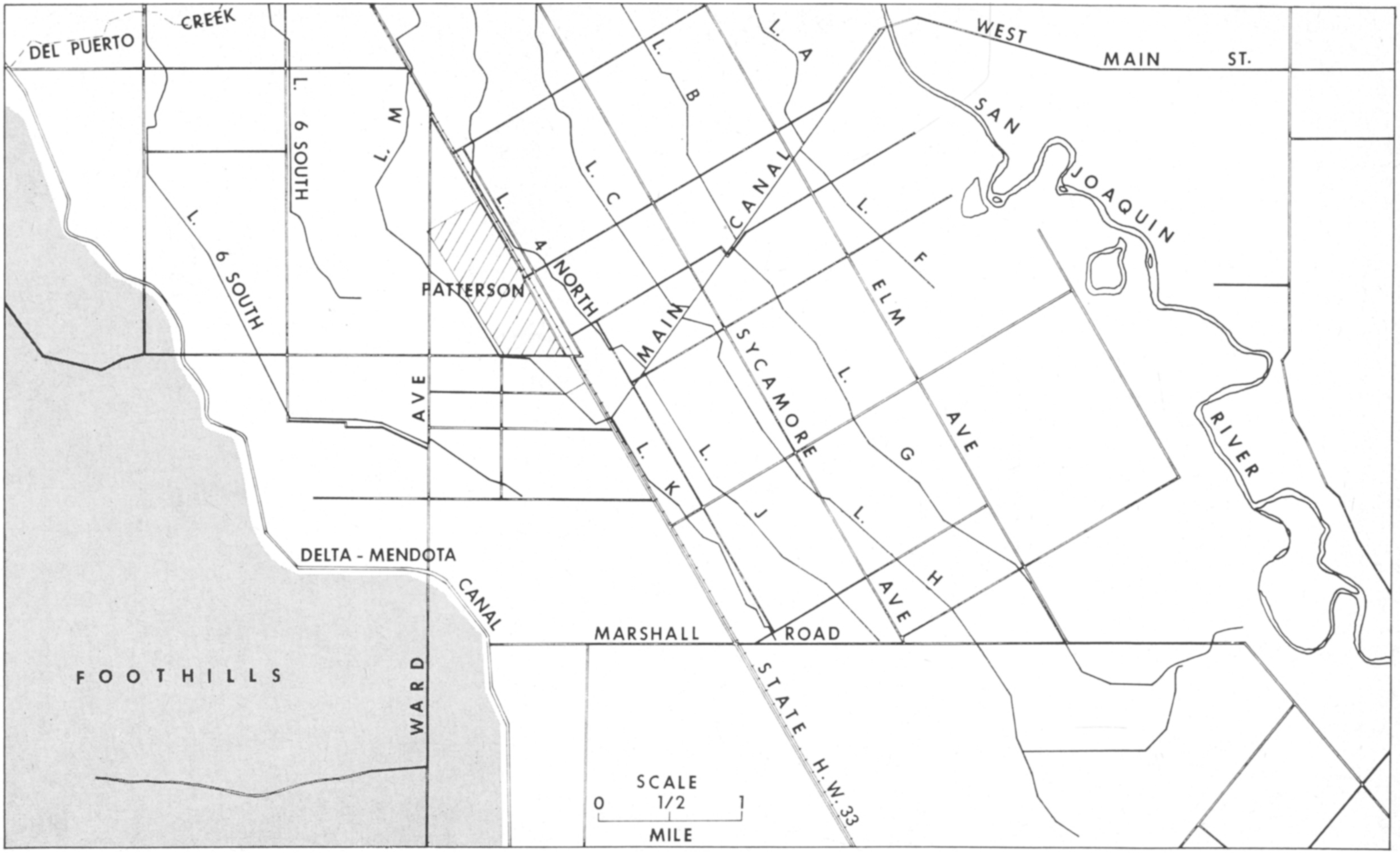


Fig. 1. General map showing Patterson area.

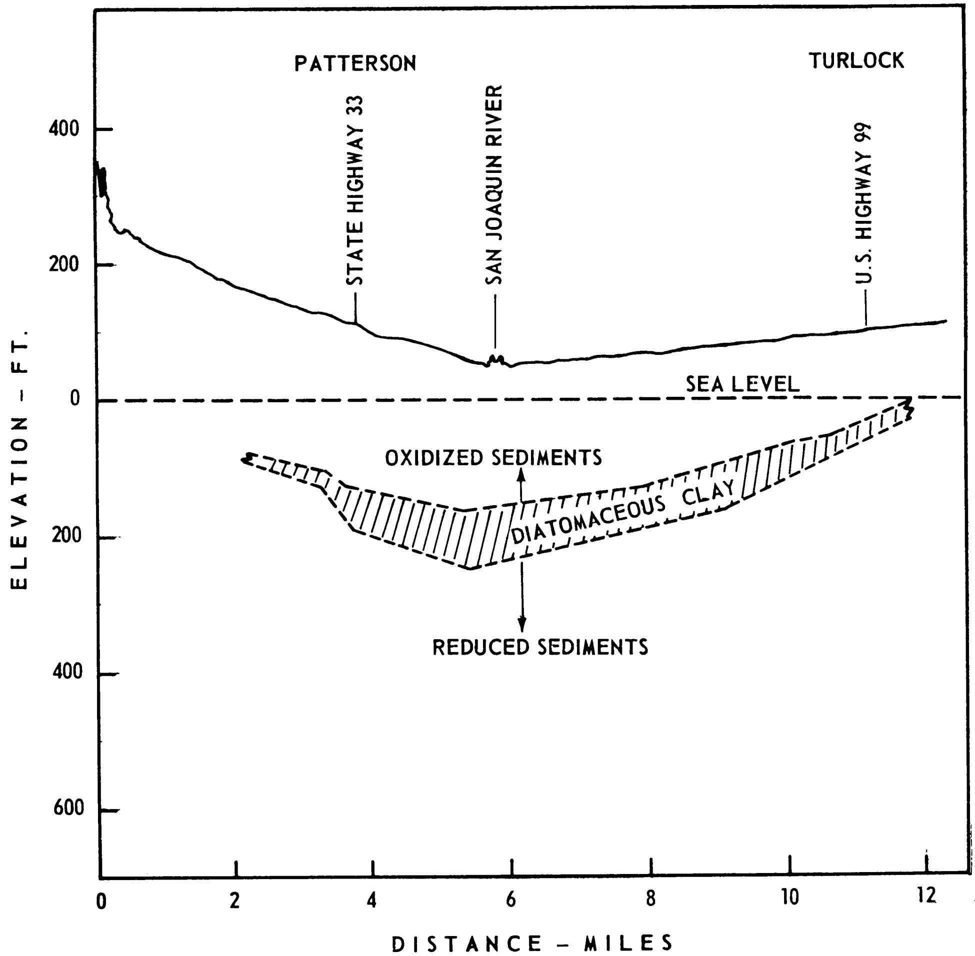


Fig. 2. Geologic section across the San Joaquin Valley.

Piezometers and shallow observation water level data were obtained for the three years, 1959, 1960, and 1961, and are included in the appendices. These data were used in plotting water level and depth contour maps for pumping and nonpumping periods. These maps provided information on ground-water movement and effectiveness of the well.

An analysis of multiple-well systems for the Patterson area was also undertaken. Both the steady- and nonsteady-state solutions were used in considering several alternative well systems. Comparisons based on economical consideration were made between these well systems, using results obtained from tests on the experimental well.

GEOLOGY AND GENERAL FEATURES OF THE AREA

The Patterson Water District is located in Stanislaus County, as shown in figure 1. The principal source of replenishment to ground water in the District is subsurface flow from the west side of the San Joaquin Valley through semiconfined aquifers overlying the diatomaceous clay. Another source is the contribution from irrigation in regions of high water table lying between State Highway 33 and the San Joaquin River. Apparently very little water is contributed to the area from the Delta-Mendota Canal because it is lined and the effect of the ground-water table in the vicinity of the canal is insignificant.

The alluvial deposits west of the San Joaquin River are predominantly ill-sorted, gravelly materials and silt. Well logs indicate that these deposits extend to depths of at least 400 feet below the ground surface. A diatomaceous clay stratum of thickness ranging from 20 to 90 feet is located at a depth of 100 feet from the ground surface near the foothills west of Highway 33 and extends down to 230 feet below the San Joaquin River. Figure 2 shows a geologic section extending southwest across the valley passing one mile south of Turlock. It presents the extent and thickness of the diatomaceous clay. The general slope of the land surface is from west to east from the foothills to the San Joaquin River.

The portion of the District between the foothills and State Highway 33 is mainly supplied by irrigation water diverted from the Delta-Mendota Canal. The area between the highway and the San Joaquin River is supplied primarily with water pumped from the San Joaquin River and distributed by gravity canals. The area north of Del Puerto Creek (figure 1) uses water diverted from the San Joaquin River at a point 3 miles northeast of Patterson.

PRELIMINARY FIELD INVESTIGATIONS

The problem area of poor drainage and a high water table extends northeast from State Highway 33 to the San Joaquin River, and includes an area of approximately 2 by 4 miles. The success of tile drainage systems in this area was doubtful because of the presence of shallow impermeable layers and lateral and vertical movement of underlying ground water which originates from overirrigation of upslope high-permeability lands and the necessity for closely spaced drainage laterals. The average specific yield for the 10- to 200-foot depth of material in the area was estimated by the U. S. Geological Survey to be 10 per cent (Anonymous, 1957).

The drainage investigation was initiated in 1959 by drilling 24 3-inch test holes to various depths up to 263 feet below the ground surface. Analyses of the hole logs were made to determine the best location of an experimental drainage well and to define the location of the aquifer. These hole logs indicated the presence of a semiconfined aquifer approximately 30 feet thick and occurring at different depths from the ground surface. The aquifer, of coarse sand and

TABLE 1

TEST HOLE NO. 3*—EXPERIMENTAL WELL LOG

Location: 5/8 32 E1, T5S, R8E, Sec. 32E MD. 1 mile S Patterson on 33, NW corner, intersection of Bartch and 33.

Depth of strata		Material
Top	Bottom	
feet		
0	4	Top soil—clay loam
4	28	Clay, short streaks of sandy silt
28	32.5	Sandy silt
32.5	40	Sandy silt, streaks ot clay
40	51	Sand, fine gravel, streaks of clay
51	63	Clay
63	71	Sand, fine gravel
71	81	Sand, coarse pea gravel
81	82	Clay
82	90	Sand, coarse gravel
90	100	Clay
100	111	Clay

* Deep piezometer at 85 feet from surface.

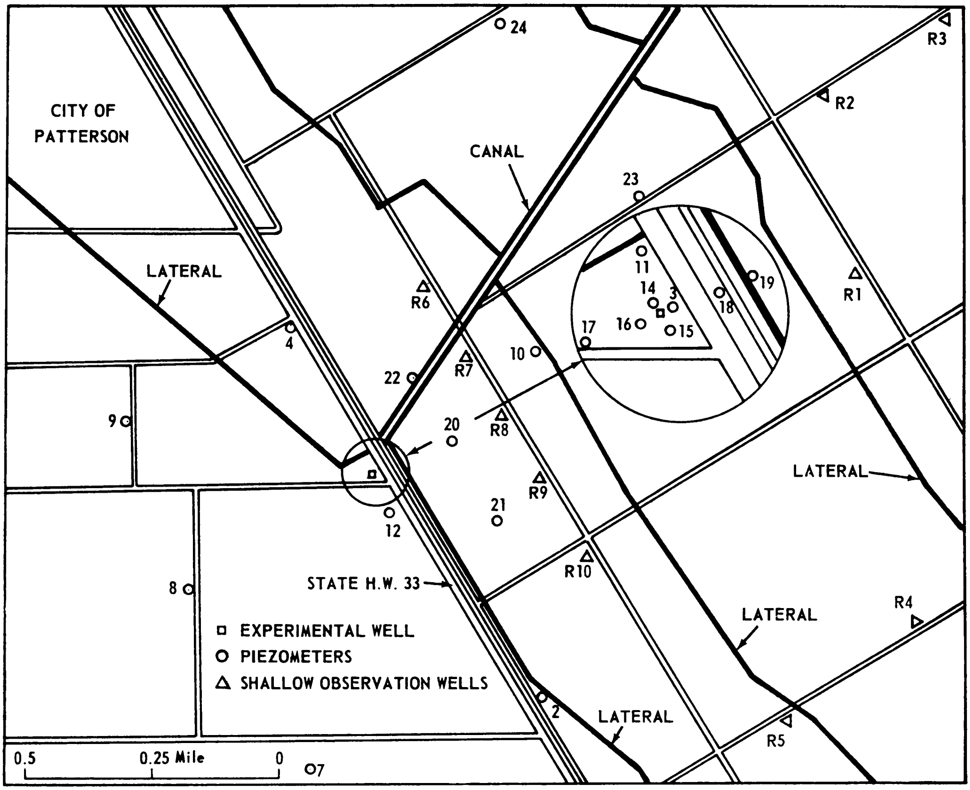


Fig. 3. Piezometers, shallow observation wells, and experimental well locations.

gravel, was semiconfined by two clay layers of variable thickness.

Table 1 gives a typical hole log for the site chosen for the experimental well. Selected other logs of test holes are given in appendix B.

At this location the test hole was enlarged to accommodate a 14-inch gravel envelope well. At 20 of the other test hole locations, piezometer pipes were installed, some of which were augered and others jetted into the soil. Figure 3 shows the piezometer and well locations.

To the northeast of Highway 33, ten other locations were chosen for large-diameter observation wells as shown in figure 3. All were located in the high water table area. Eight- to 10-inch diameter steel pipes were installed into the holes to a depth of 8 to 12 feet. A 12-inch layer of gravel was put at the bottom of each. Automatic water-stage recorders were installed to follow the shallow water table fluctuations due to irrigation and local seepage from canals.

EXPERIMENTAL WELL

A site one mile south of the city of Patterson on State Highway 33 west and upslope from the area of high water table was selected as the location for the experimental well. The selection of this site was based on the preliminary hole logs, the surface topography, and the location of the supply ditches and drains. It was intended to be upslope from the high water table area so that ground

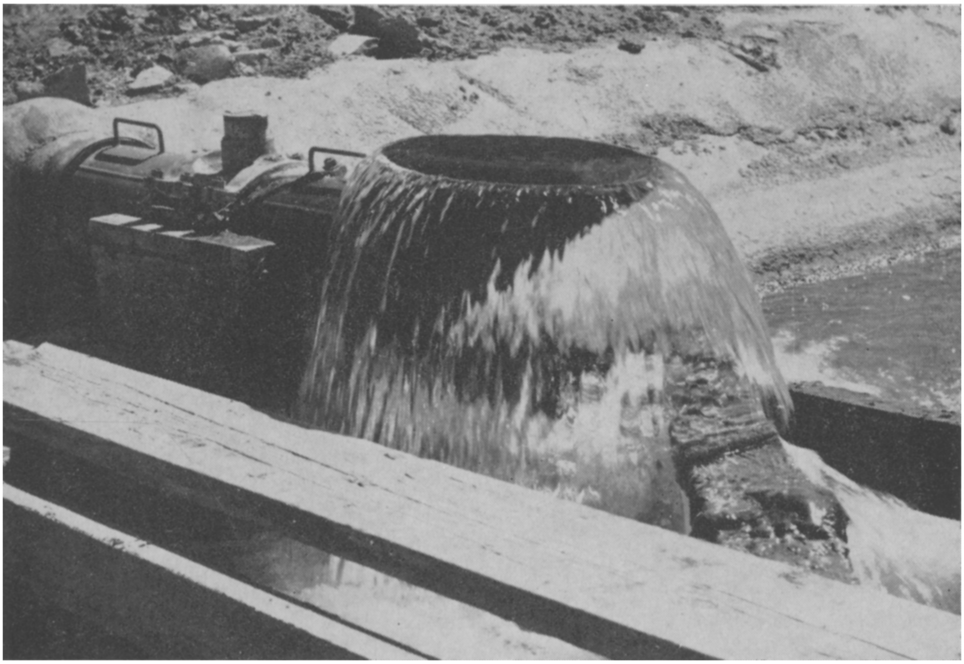


Fig. 4. Discharge pipe outlet for the experimental well.

water moving downslope toward the San Joaquin River would be intercepted.

At this location, the thickness of the principal aquifer was estimated to be 30 feet. The well hole was reamed to 36 inches, gravel-packed, and lined with 100 feet of 14-inch casing with 40 feet of perforations at the bottom. The well was equipped with a 25-horsepower turbine pump and motor. A meter was installed at the end of the discharge line to measure the flow rate during the pumping test (figure 4). A valve on the discharge line regulated the flow.

During the pumping tests, measurements of the water level in the well were made by a pressure gauge connected to the well air line. When the water level inside the well was 10 feet or less above the end of the air line, a continuous record of water depth was obtained by use of an automatic bubbling recorder installed at the well.

Normally the discharge of the well was recorded each week. During the pumping tests the flow was recorded for short intervals of time, depending on the type of test and rate of change in the flow.

PIEZOMETERS AND OBSERVATION WELLS

A network of piezometers was installed around the experimental well to provide adequate data for defining the position of the water table in the area. The network was composed of 24 piezometers installed at distances varying from 18 feet to about 1.2 miles from the well, as shown and identified with numbers 1 to 24 on figure 3. Each location usually included a deep and a shallow piezometer. An effort was made to locate these piezometers on lines at right angles to each other through the well. This would be perpendicular to and along the direction of ground-water flow. Two ¾-inch piezometer pipes were placed at each test hole location at two different depths. The deep piezometer penetrated the principal aquifer, while the shallow piezometer extended to a depth about five feet below the high water table.

These piezometers were installed to investigate the nature of the material in the water-bearing strata and to permit measurements of water-table fluctuations during different seasons of the year and in response to normal and test pumping of the experimental well.

A very deep piezometer, 250 feet, was also installed at hole location No. 11 to determine if a vertical gradient existed between the aquifer below the clay and the overlying materials.

The water level in the piezometers was measured once each week by means of an electric sounder with an accuracy of 0.1 foot. During drawdown and recovery pumping tests performed on the experimental well, the water level in the piezometers was measured at frequent time intervals.

The shallow observation wells were of value in following fluctuations produced primarily by local overirrigation and seepage from unlined supply laterals. This information provides a basis for correlating high water table conditions with crop drainage and in determining the practical limits for control of the water table.

COLLECTION OF DATA AND FIELD PROGRAM

In this study, routine measurements were taken once a week of the water level in the piezometers and the shallow observation wells, and the experimental well pumping rate. A water sample from the pumped water was taken each month for water quality determination. More frequent measurements of water levels in the piezometers and discharge rate of the pump were made during the pumping tests. These tests involved drawdown and recovery tests for both uncontrolled and constant discharge conditions.

Water level data in the piezometers were plotted on a yearly basis in days and were used to: (1) plot water depth and elevation contour maps, (2) define the cone of depression of the well, and (3) calculate the aquifer characteristics. The data for the shallow observation wells were also plotted on a yearly basis in days, and provide information on fluctuations in the shallow water table because of irrigation and seepage from adjacent irrigation canals.

During the period of study, five drawdown and three recovery tests were conducted. Four of these drawdown tests involved unthrottled discharge conditions. An additional drawdown test with a constant discharge of 300 gallons per minute was also run.

ANALYSIS OF DATA

Water-level data

Data from the shallow observation wells indicated the change in the water table as it responded to irrigation in the immediate vicinity of a particular well, as shown in figure 5 for recorder No. 8. This water depth hydrograph was typical for all the shallow observation wells. Data for all of the shallow wells are included in appendix D. The straight horizonal lines along the bottom of the graph in figure 5 indicate that the recorder float reached the bottom of the well.

No change in the water levels took place in these observation wells due to pumping of the experimental well because they all were beyond its influence. Also, for those wells which were near lined laterals, little change in water levels occurred when water was in the lateral, indicating that seepage from the concrete-lined laterals was small.

During the periods of the pumping

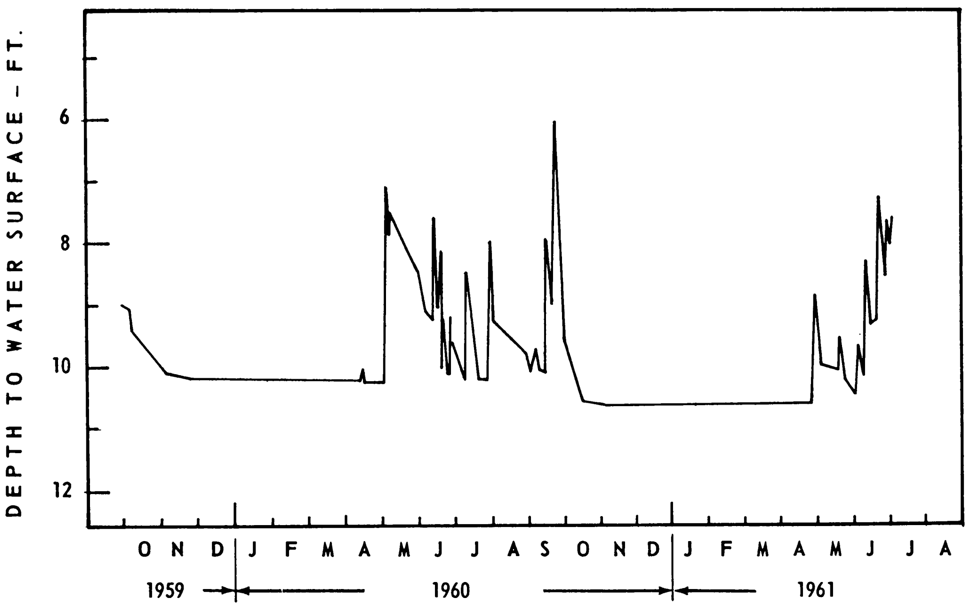


Fig. 5. Hydrograph for recorder No. 8.

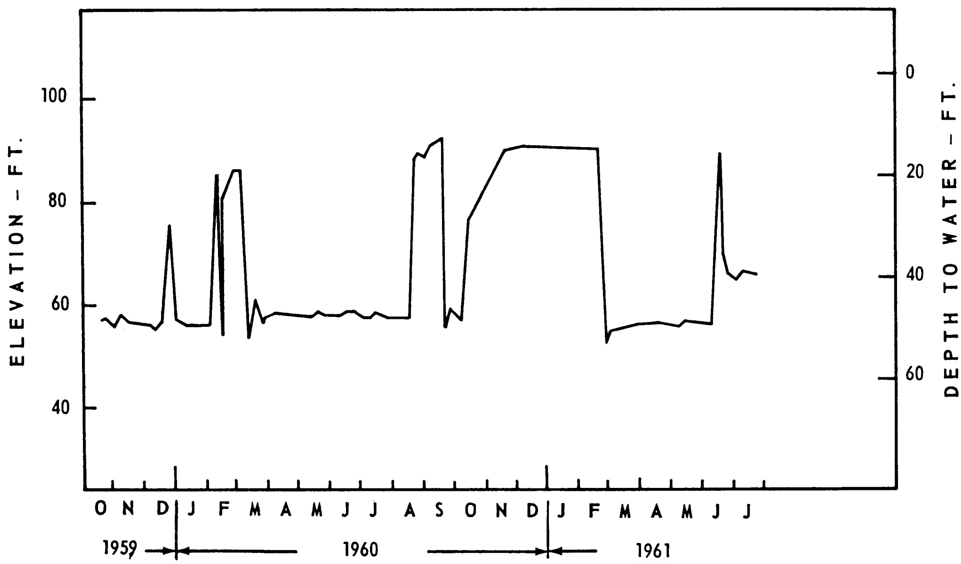


Fig. 6. Hydrograph for piezometer No. 16.

tests, the shallow and deep piezometers' water levels showed a rapid response to the influence of pumping. Shortly after the start of each drawdown test, the shallow piezometers close to the experimental well went dry. Figure 6 shows a hydrograph of piezometer No. 16, which was within the area of influence of the well—54 feet away from it. Figure 7 shows another hydrograph for piezometer No. 8 which was 2,280 feet away from the well—beyond its influence. Hydrographs for other piezometers are given in appendix C. Generally, the ground-water levels rose during the summer and declined during the

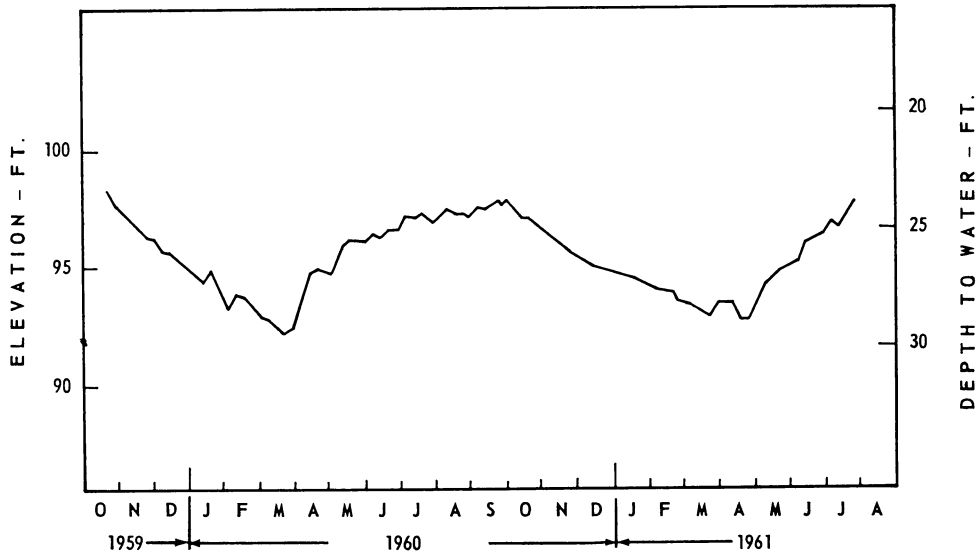


Fig. 7. Hydrograph for piezometer No. 8.

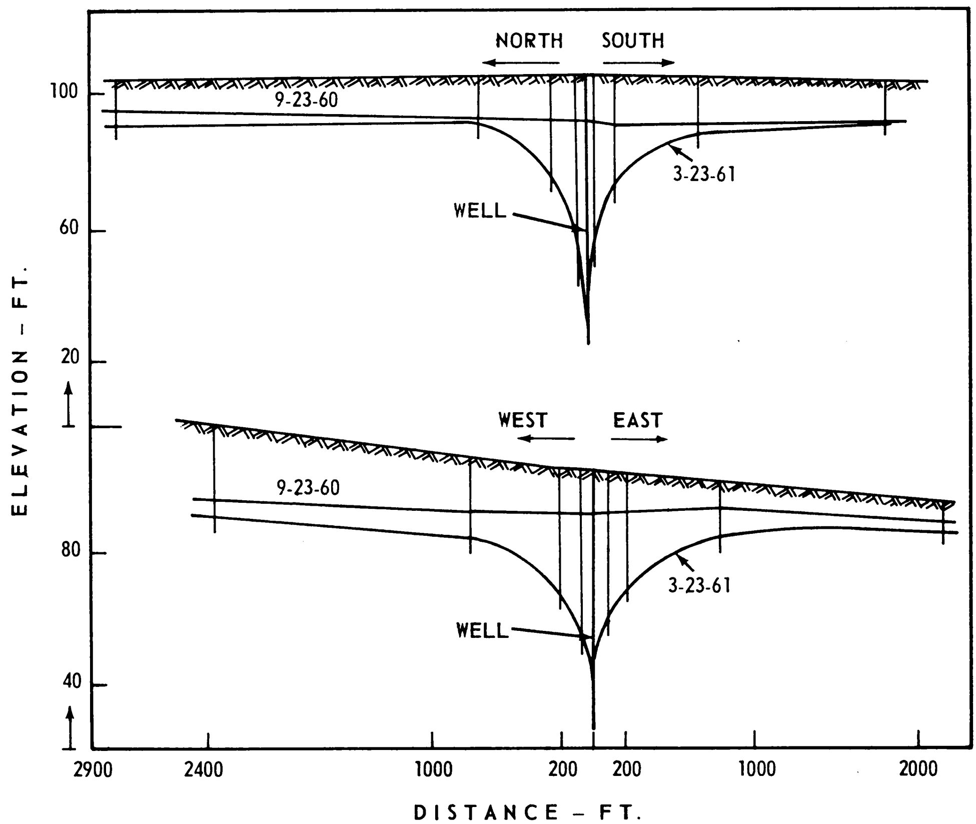


Fig. 8. Two cross-sections at the well location.

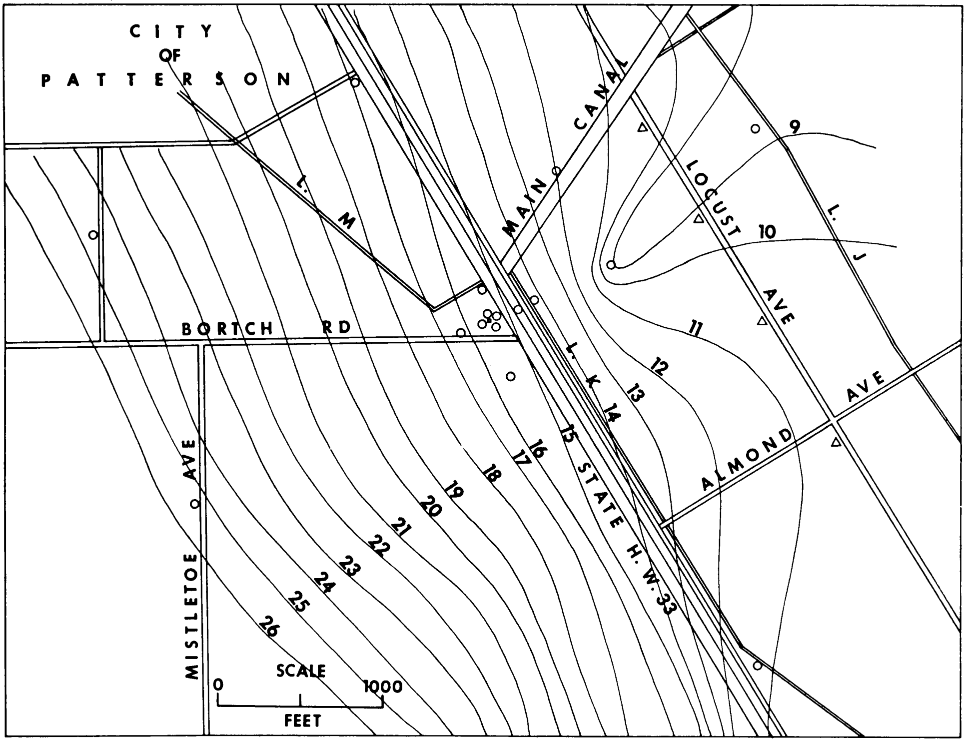


Fig. 9. Water-depth map (nonpumping, 9-23-60).

fall. The pumping influenced the water tables and piezometric pressures at distances of more than 1,000 feet west of the pumping well and about 700 feet in the east, north and south directions. Figures 10 and 12 show the extent of the pumping influence. Stratification of the materials and corresponding differences in transmissibility may account for these differences. Figure 8 shows two cross-sections at the location of the wells directed north-south and west-east. The water levels are plotted from piezometer readings for conditions in September and March. The upper level shows the nonpumping conditions for September 23, 1960. The lower water profile was obtained during a drawdown test in March, 1961.

Analysis of water levels recorded in the piezometers indicated little difference in the piezometric surface between the shallow and deep piezometers. Consequently, data from both were used in preparing the water-level contours at different times during the year. These maps established the direction of ground-water flow and were of two types—first, water-depth maps shown in figures 9 and 10, during pumping and nonpumping periods, and second, water-elevation maps (figures 11 and 12), plotted also for the same periods as the water-depth maps. In general, there was a gradient from southwest to northeast. During pumping of the experimental well, this gradient was modified substantially within the area of influence and a reversed gradient resulted from the east toward the well. The area of influence of the well was elliptical, with the major axis east-west.

Experimental-well data

A decreasing pumping rate was observed during the unthrottled pump tests. This is a common characteristic of wells and may be observed for either

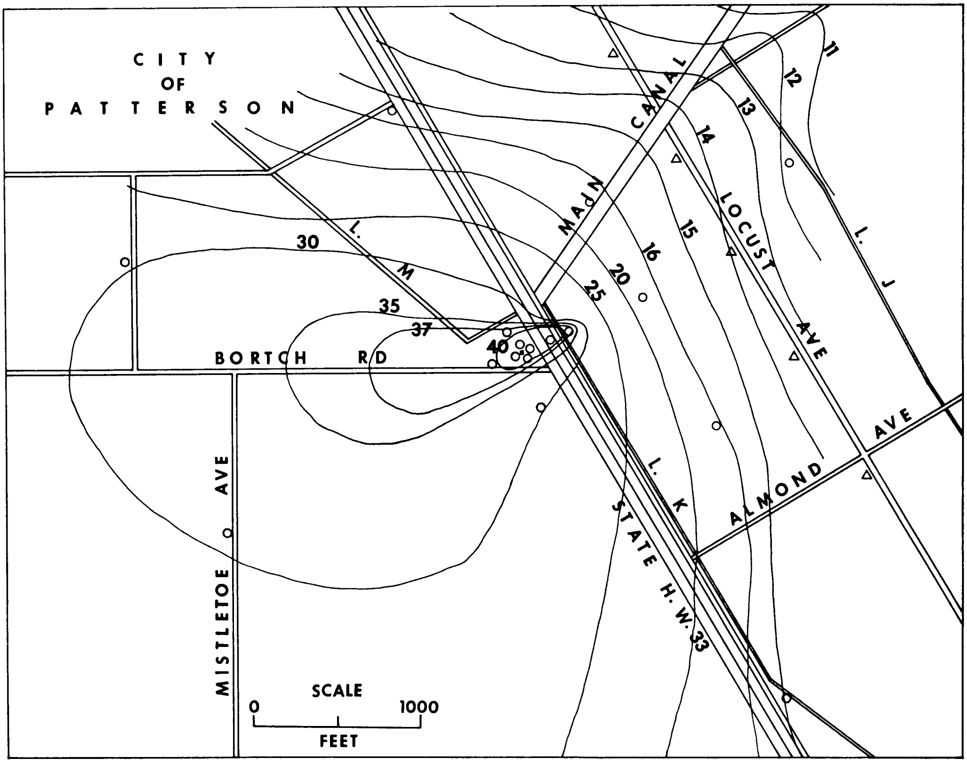


Fig. 10. Water-depth map (pumping, 3-23-60).

long- or short-term pumping periods. In both of the unthrottled pump tests the beginning rate was about 900 gpm, then decreased gradually and finally reached a steady rate of about 340 gpm, as shown in figure 13. The difference between measured and steady-state discharge was plotted in figure 14 on semilogarithmic graph paper; a best-fitting straight line was drawn through the data to obtain an exponential time-discharge relationship. With the constants obtained from this relationship, the aquifer characteristics were then computed by a method for decreasing discharge as shown in appendix E. For comparison, the characteristics were also computed by several other methods assuming an average constant discharge.

The drawdown curves under constant and decreasing rates for piezometer No. 16, which is 54 feet from the well, are shown in figure 15. These data show a significant difference in the piezometer levels under constant and decreasing discharge rates, mainly because of the differences in amounts of water pumped from storage. The required time to reach a steady state is much longer for the case of a constant-discharge throttled test than for the decreasing discharge case. Similar data were obtained at other piezometer locations within the influence of the experimental well. Figure 16 shows the recovery data for piezometer No. 16 which followed an unthrottled test.

Calculations of aquifer characteristics

For the conditions encountered in this study, it was appropriate to use an analysis based on nonsteady-state conditions, and several methods were used. All involved certain assumptions which were not met entirely by the field conditions. The methods used for calculating the aquifer characteristics were the

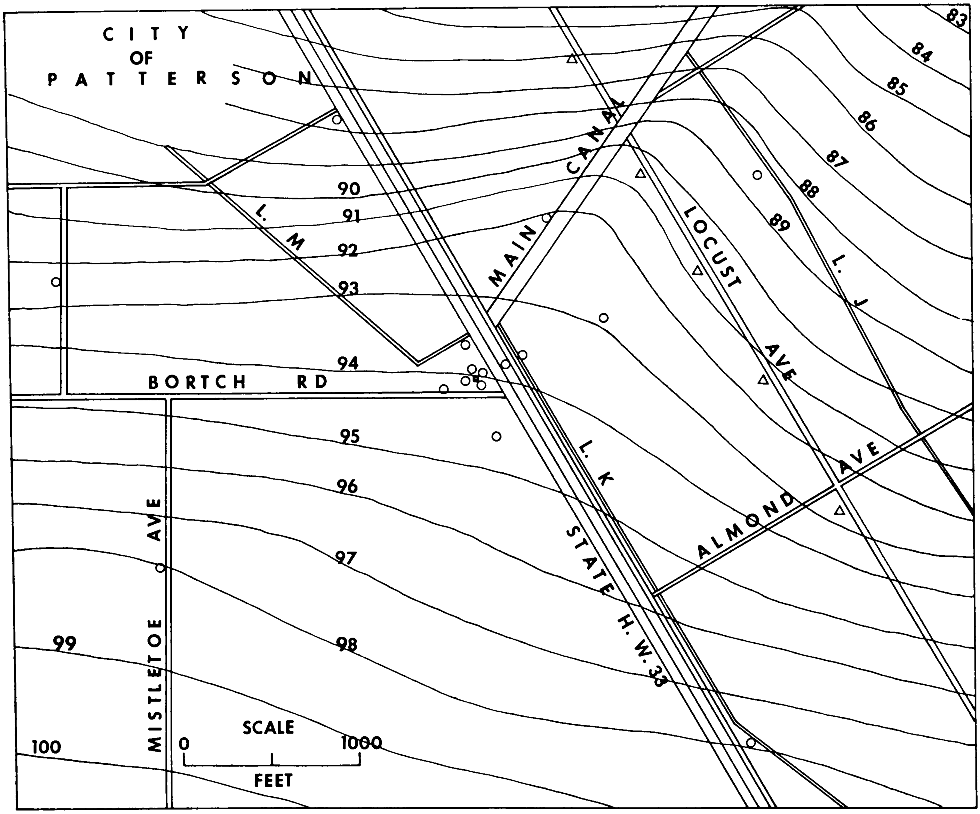


Fig. 11. Water-elevation map (nonpumping, 9-23-60).

Theis-type curve (Theis, 1935), Jacob straight-line method (Jacob, 1950), Chow modified straight-line method (Chow, 1954), and the decreasing-discharge method (Abu-Zied and Scott, 1963; Abu-Zied *et al.*, 1964).

For early pumping test data, drawdown at several piezometer locations was considered at a specific time. These drawdown values were plotted against the logarithm of the distances from the well and the transmissibility calculated from the slope of these plots.

In some cases the relationship between drawdown and time showed a slight deviation from a straight line. This is due to the fact that the aquifer from which water was being obtained was stratified —a property that contributes to slow drainage and results in a delay in the release of water from storage and piezometer response. Also, the nonequilibrium relationships do not properly define the drawdown until adjustments of the flow and pressure occur between permeable zones and within the transmitting aquifer. Consequently, there was some deviation during the early stages of pumping, but in general this could be disregarded.

Detailed computations and a summary of transmissibility values can be found in appendix E. Out of ten sets of computations, one gave considerably higher values of T than the average and was excluded. The other nine sets gave transmissibilities between 0.0124 and 0.0175 ft^2/sec, with an average of 0.0144 ft^2/sec.

Storage coefficients were also computed by each of the applied methods, but the scatter of the values obtained was so wide that no average value of S was computed.

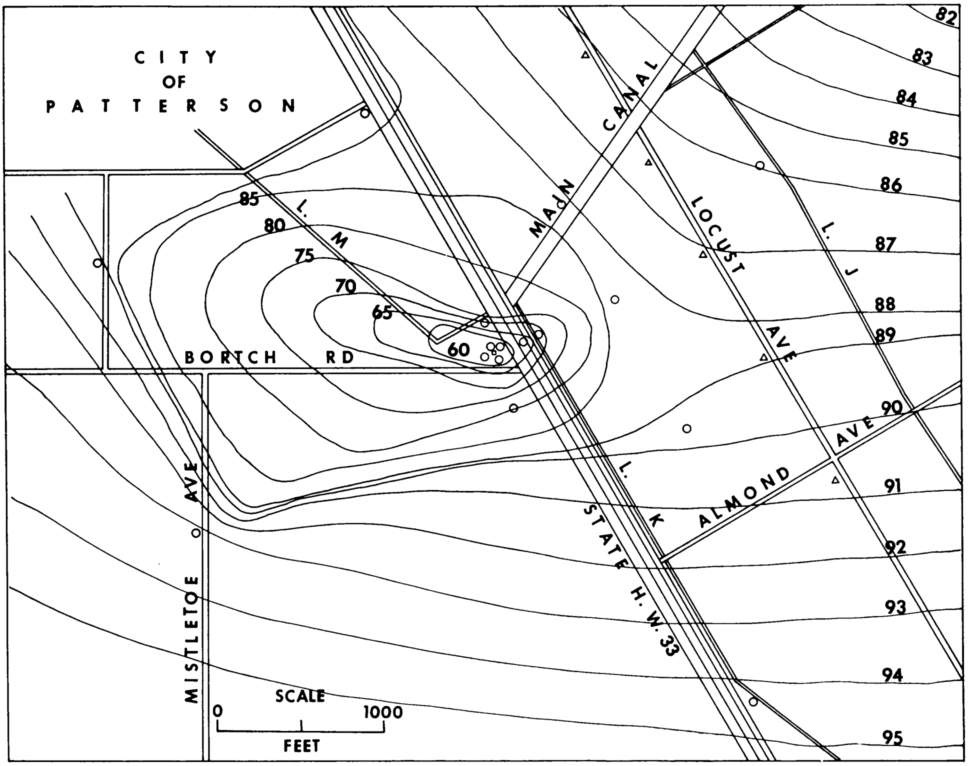


Fig. 12. Water-elevation map (pumping, 3-23-60).

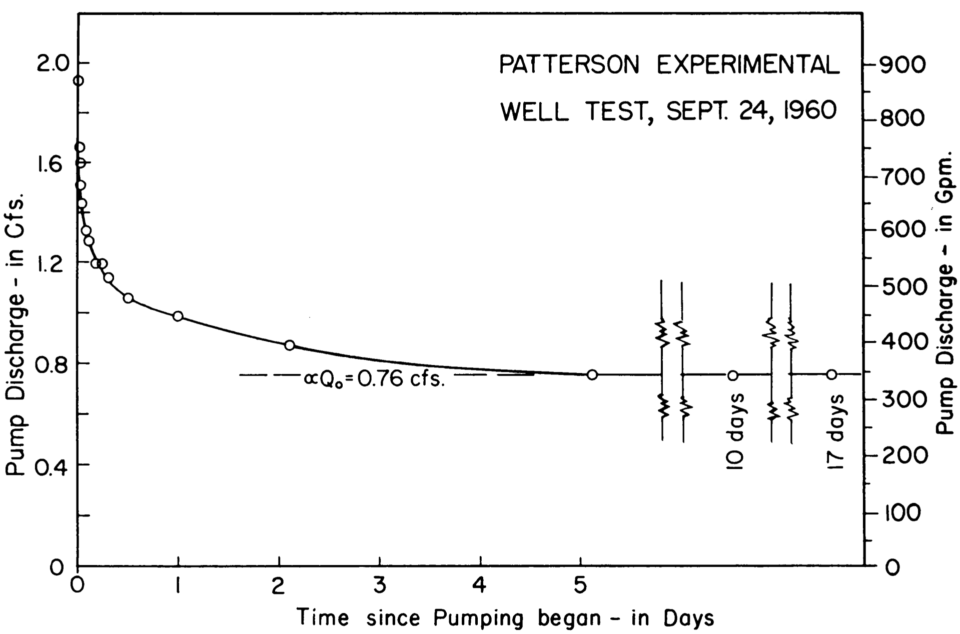


Fig. 13. Experimental-well discharge.

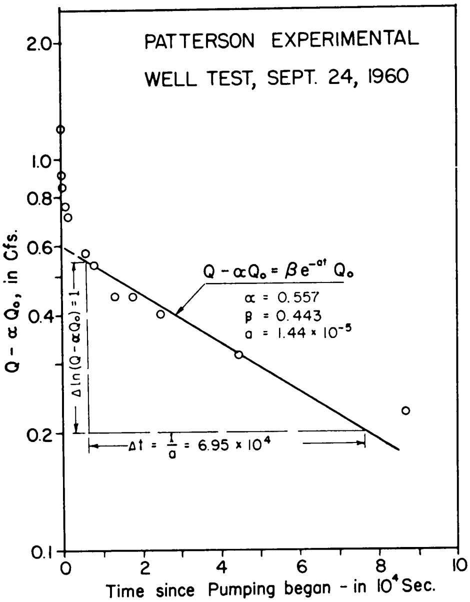


Fig. 14. Time-discharge relationship.

Water quality

Another aspect of using wells to control the water table is water quality. Samples of water were taken from the pump discharge periodically in order to determine its mineral constituents and to follow any change should it take place. Some of these analyses are presented in table 2. In general, the water quality was essentially constant during the entire pumping period. The conductivity of the water averaged 1.8 millimhos/cm. The chloride and sodium percentages averaged 45.5 and 44.6 per cent, respectively. This water would, by present standards, range between classes 2 and 3. Therefore, its suitability for irrigation of fruit trees and some sensitive vegetable crops would be questionable. For comparison, the water conductivity in the San Joaquin River was obtained near the city of Patterson during 1960 and 1961. Figure 17 shows that the conductivity generally went up from 1960 to 1961, and reached a value of 2.0 millimhos/cm during the month of March 1961.

TABLE 2

WATER QUALITY AT EXPERIMENTAL WELL

Date	Cation content, me/1				Anion content, me/1				Content in % of total		Electric conductivity
	Ca	Mg	Na	Total	$CO_3 + HCO_3$	Cl	SO_4	Total	Na	Cl	mmho/cm
10/22/59	5.0	5.52	8.37	18.89	4.91	7.60	6.38	18.89	43.1	39.4	1.89
5/13/60	5.20	3.60	8.30	17.10	4.38	7.30	5.42	17.10	48.5	42.7	1.70
7/28/60	10.00		7.20	17.20	4.62	8.00	4.58	17.20	41.9	46.5	1.72
6/6/61	10.28		8.02	18.30	5.82	8.40	4.08	18.30	43.8	45.9	1.83
7/25/61	10.20		8.60	18.80	7.80	10.00	1.00	18.80	45.7	53.2	1.88

ANALYSIS OF MULTIPLE-WELL SYSTEMS FOR THE PATTERSON AREA

In many cases, the areas to be drained are large and require more than one drainage well. Therefore, one should consider the minimum number of wells which will produce the required water depth. The spacing and well arrangement of such a system are affected by many factors. Among these are the radius of influence of each well, the minimum depth to which the water table should be drawn down, the hydraulic characteristics of the subsoil, and the sources of water moving into the area being drained. The wells may be arranged in various geometric patterns, either as isolated groups or as a continuous pattern extending over large areas. To assure adequate drawdown over the entire area, some overlapping of the wells' individual regions of influence is necessary.

Solutions for multiple-well systems may be obtained by either the steady- or nonsteady-state method. Assumptions for each are made to simplify or to make the theoretical applications possible. The steady-state solutions are based mainly on the assumption that the water table is no longer falling and an equilibrium effect of pumping is established. The Dupuit assumption of essentially horizontal flow must also be made.

Muskat (1937) derived several equations to compute the drawdown caused at certain specific points by the combined pumping action from three, four, or more wells arranged in different geometric patterns. A first approximation to the control of the water table can always be obtained by applying these steady-state solutions.

When the period required to reach an equilibrium condition is long, which is often the case, nonsteady-state analyses are necessary. Additional assumptions are needed because direct solutions for the nonsteady flow problem of multiple wells are not yet available. These assumptions are: (1) the Theis nonequilibrium solution can be applied for the existing conditions of the area under

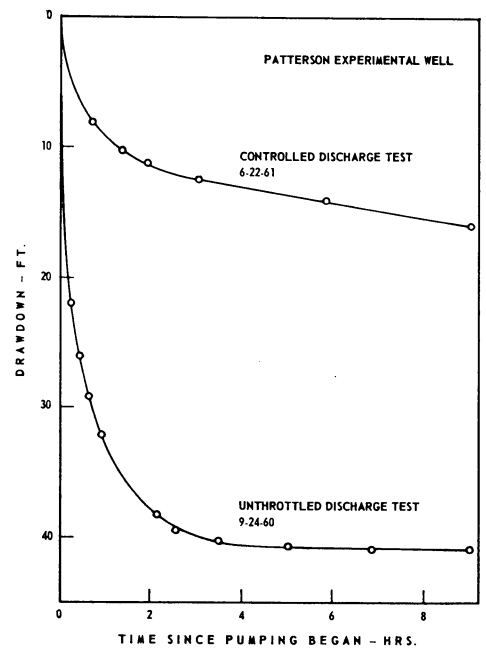


Fig. 15. Drawdown data for constant and decreasing pumping rates, piezometer No. 16.

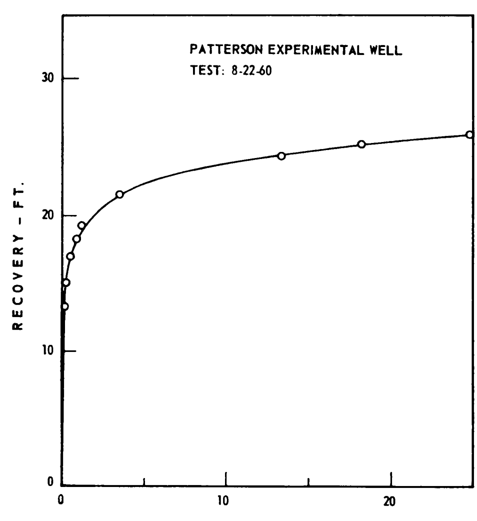


Fig. 16. Recovery data for piezometer No. 16.

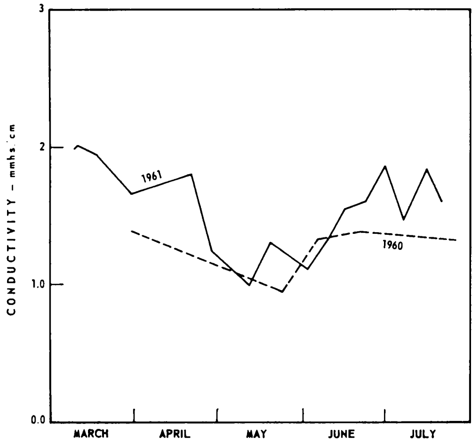


Fig. 17. Water quality of the San Joaquin River.

consideration; (2) the drawdown at any point on the cone of depression, caused by the pumping of several wells, is equal to the sum of the drawdowns caused by the discharge of each well; (3) the aquifer is homogeneous, of uniform thickness and completely penetrated by the wells. A total drawdown equation at any point in the system can be written which will involve the following:

1. The spacing of the wells and their arrangements.
2. A period of time at which the drawdown is being calculated.
3. The drawdown caused by the system at the point in question.
4. The hydraulic characteristics of the aquifer.

Therefore, one can proceed with the analysis if a minimum drawdown is specified to be reached after a certain time of pumping, and some estimate of transmissibility and storage coefficients of the aquifer is available.

Two well systems were investigated for the Patterson area to provide control of the water table. Both the steady- and nonsteady-state solutions were used in the analysis.

Steady-state flow condition

In the steady-state approach to the multiple-well flow problem the following assumptions were made:

1. The wells and pumps would be of the same diameter, depth, and capacity as the experimental Patterson well. Likewise, the soil drained by this well is representative in texture and hydraulic characteristics of the land throughout the vicinity of Patterson.
2. The minimum drawdown required for drainage at any place between the wells would be two feet.
3. The total drawdown of the water table at any point, as long as it is small compared with the aquifer thickness, is equal to the sum of the drawdowns caused individually by the neighboring wells.

From the records of the September 24, 1960, pumping test, it is apparent that the flow decreased to a fairly steady state in about 5 days after pumping started, with 70 feet of drawdown inside the well and a discharge of 340 gpm.

From piezometer data collected during the September, 1960, test, drawdown vs. log r was plotted for the fixed time $t = 5$ days, in figure 18. A straight line

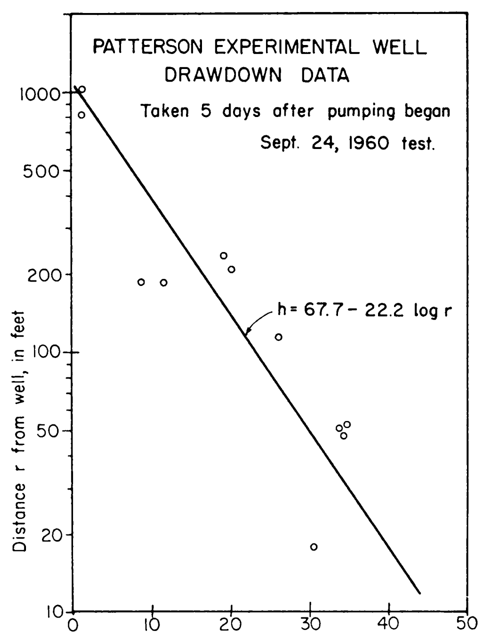


Fig. 18. Drawdown profile after 5 days.

was fitted to the points by least-square analysis and the following equation obtained:

$$h = 67.7 - 22.2 \log r$$

Replacing the single Patterson well by a network of wells arranged in a square pattern, figure 19a, or a triangular pattern, figure 19b, it is evident that the

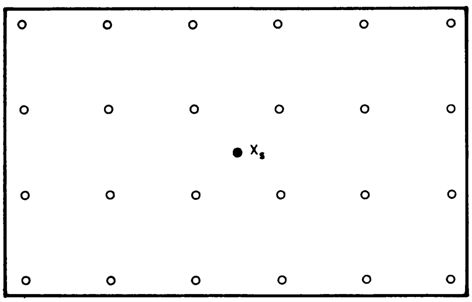


Fig. 19a. Square well pattern.

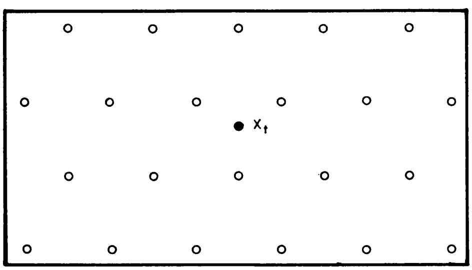


Fig. 19b. Triangular well pattern.

minimum total drawdown can be expected at the center (X_s or X_t, respectively) of the geometrical figure bounded by the wells. In the square pattern, four wells will influence the drawdown at the point X_s and the individual drawdown caused by each well would be $2.0/4 = 0.5$ ft. In the triangular pattern, three wells would contribute to the drawdown at X_t and their individual drawdown influence would be 0.67 ft.

Using the above equation, the maximum radial distances from the wells to the critically shallow drawdown point are $r_s = 1062$ ft for the square pattern and $r_t = 1043$ ft for the triangular pattern. The spacing is $L_s = 1502$ ft and $L_t = 1806$ ft, and the areas drained per well are $A_s = 2r_s^2 = 2.26 \times 10^6$ ft^2 and $A_t = 3/2\sqrt{3}r_t^2 = 2.84 \times 10^6$ ft^2, or 52 and 65 acres, respectively.

In reality, the subsoil in the Patterson region is by no means homogeneous or otherwise an ideal aquifer, and the computed spacing will most likely not produce the required minimum drawdown at all points within the well system, but the analysis nevertheless demonstrates the superior drainage efficiency of the triangular well pattern over the square pattern.

Nonsteady-state flow condition

For the nonsteady flow the average transmissibility of 0.0144 ft^2/sec, as determined from several pumping tests on the Patterson well, was used. Values of the computed storage coefficients varied so widely over the range of the test results that none of those values were used. Instead, the volume of the drawdown cone was integrated from the graph on figure 25 for time = 5 hours and divided into the total volume of water pumped during that period. The resulting value of $S = 0.0028$ was used as a reasonable approximation for the aquifer.

For the drawdown computations, Theis' nonequilibrium equation was used. Provided that the discharge in each well would be the same, the equation for total drawdown at any one point is

$$h = \frac{Q}{4\pi T} \sum W(u)$$

where $W(u)$ is Theis' well function, computed for the respective distance from the point at which the drawdown is computed to any one of the wells in the network.

For the critical center points X_s and X_t, the parameter $u = (r^2S)/(4Tt)$ was then tabulated and computed for all the neighboring wells with the distances r expressed in terms of the spacing L between the wells. Thus, for a range of

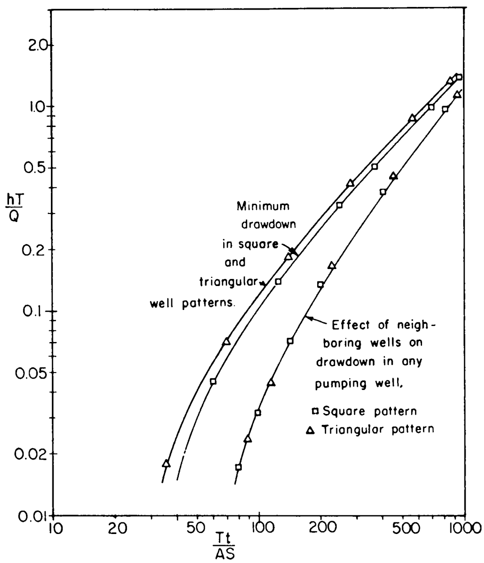


Fig. 20. Comparison of relative drawdown in square and triangular well patterns.

$u_o = (L^2S)/(4Tt)$ values the sum $\Sigma\, W(u) = (4\pi hT)/Q$ was computed for a square and a triangular well network. Finally, since the same well spacing in the two patterns leads to different extents of area drained, the parameter $u_o = (L^2S)/(4Tt)$ was converted to $(Tt)/(AS)$ where A is the area in acres drained per well, and in figure 20 $(Tt)/(AS)$ is plotted against $(hT)/(Q)$, both dimensionless.

The curves in figure 20 show that except for high values of $(Tt)/(AS)$ the minimum drawdown is somewhat larger in the triangular pattern than in the square pattern with the same number of wells per unit area, or conversely, for the same desired minimum drawdown, the spacing in the triangular pattern could be made larger. From the lower curve it seems that the effect of the neighboring wells on the drawdown in each pumping well is the same for both patterns. By choosing a triangular pattern with a slightly larger spacing, then, one should save pump installation costs and reduce the pumping head in the wells.

Using the values from the Patterson tests as representative, namely,

$$Q = 340 \text{ gpm or } 0.755 \text{ cfs}$$
$$T = 0.0144 \text{ ft}^2/\text{sec or } 1{,}240 \text{ ft}^2/\text{day}$$
$$S = 0.0028$$

and $h = 2$ ft. as required for minimum drainage, the required well spacing is shown in table 3.

Thus the nonsteady-state analysis confirms the advantage of the triangular over the square well pattern in drainage effectiveness.

The disadvantage of the nonsteady flow approach is that the effect of recharge is neglected. As mentioned before, an equilibrium between pumpage and recharge has been reached in the Patterson tests approximately 5 days after pumping started. Results from the steady-state analysis agree fairly well with those from the nonsteady-state computations for $t = 5$ days, and will therefore be used in the cost comparison.

TABLE 3

REQUIRED WELL SPACING FOR MAINTAINING A TWO-FOOT DRAWDOWN BETWEEN WELLS UNDER NONSTEADY-FLOW CONDITIONS

Parameter	Well pattern: Square		Well pattern: Triangular	
$\frac{hT}{Q}$	0.0382		0.0382	
$\frac{Tt}{AS}$	55.1		49.4	
t—days	Area drained per well	Well spacing	Area drained per well	Well spacing
	acres	*L—ft*	*acres*	*L—ft*
1	11.7	715	13.1	810
2	23.6	1,010	26.3	1,140
4	47.0	1,430	52.5	1,610
8	94.2	2,020	105.0	2,280
Steady-state analysis*	52.0	1,500	65.3	1,800

* Based on drawdown measurements taken 5 days after pumping began.

Economic considerations of multiple-well systems

In a complete economic analysis of drainage by pumping, a large number of interdependent factors would have to be considered. A few of the questions which could be the subject of an intensive economic analysis would be the following:

- What are the alternatives to well systems that would provide the necessary drainage? What are the costs and the effects of such alternatives on land use and farming operations?
- What is the basis for setting the period of time during which the drainage system would operate?
- What are the physical and economic consequences of drainage water disposal?
- Can drainage water replace imported water in the farm operations? If so, what is the value of drainage water for re-use in irrigation?
- What are the legal or institutional problems involved?

As a follow-up on the hydraulic comparison between the triangular and square well patterns, a cost comparison between the two patterns was made, taking the following five factors into consideration: (1) selection of pumps to meet head and discharge requirements; (2) cost of construction of wells including well drilling, installation of casing and screen, gravel envelope, cost of pump and installation, and well developing; (3) consideration of period of time through which the pumps will be operated; (4) power and operation costs; and (5) possibility of re-using the water for irrigation.

It was decided that all wells and pumps used in the analysis would be the same size. A well diameter of 14 inches, and 15-horsepower, deep-well turbines would meet the requirements of a total head of 70 feet and a discharge of 300 gpm. Costs of the pump and well construction were based on estimates of local drillers and pump dealers.

Physical Dimensions

Average discharge of each pump = 300 gpm
Total water pumped per well in 6 months = 238 acre-ft
Size of pumps = 15 hp
Size of well = 14-in diameter
Total annual energy for each pump = 27,000 kw-hrs

Costs

Annual service charge per pump		$ 106
Annual cost of energy..........		516
Cost of well.............	$1,300	
Cost per pump and motor	2,400	
Total cost..............	$3,700	
Amortization (20-year life, 6% interest) = 3,700 × 0.0872...		320
Maintenance (10% of pump cost)		240
Total annual pumping costs....		1,182
10% for supervision and contingencies.................		118
Total cost....................		$1,300
Value of water for re-use for irrigation at $3/acre-ft		714
Net annual cost per well.......		$ 586

Item	Square pattern	Triangular pattern
Area drained per well, acres..	52	65
Annual drainage cost per acre	$11.30	$9.00

Application of the well pattern to the Patterson drainage problem

As stated, the Patterson area requiring drainage by wells is approximately 8 square miles (figure 1). If the wells were arranged in the suggested triangular pattern, spaced 1800 ft apart, about 7 rows of 12 wells each would be needed to cover the area.

Some points along the periphery of

this well network may not experience the required 2-ft drawdown. By installing 15-hp pumps, which are calculated for only about 50 per cent efficiency, it should be possible to improve the drainage in some trouble spots by pumping the adjacent wells at a somewhat higher discharge.

SUMMARY

To determine the feasibility of pumped wells in areas along the west side of the San Joaquin Valley, which are developing serious drainage problems because of expanding irrigation on fine-textured soils, a study was initiated involving a field program of geologic and soils reconnaissance and testing, an experimental well, and an analysis of various well systems for steady- and nonsteady-flow conditions.

The field investigation, using an experimental drainage well, was made in an area near the city of Patterson where a serious high water-table condition had developed. Different pumping and recovery tests were conducted, and measurements of water levels inside a network of piezometers installed around the experimental well were used to investigate the effectiveness of the well and to calculate the aquifer characteristics using nonsteady-flow methods.

Pumping tests demonstrated the effectiveness of lowering the water in an eccentric pattern around the well. The influence of pumping extended 700 feet north and south and approximately 1,000 feet east and west from the well. Measurements of water levels in deep and shallow piezometers showed little differences in water levels.

Practically no change in water quality of the pumped water occurred during the period of investigation, and the water would be classified as satisfactory irrigation water for field crops.

Since a single well has limited influence, attention was given to an analysis of multiple-well systems for the Patterson area. Both the steady- and nonsteady-state solutions were used in considering several arrangements and spacings of wells. The use of the steady-state solution appeared to be the best approach for conditions in the area.

This analysis demonstrated that a maximum of 84 wells, 14 inches in diameter, 120 feet deep, 1,800 feet apart, and equipped with 15-hp turbine pumps would be required to drain the 2- by 4-mile, high water-table area. If all of the wells would develop the same discharge as the experimental well, the 84-well system could maintain the water table at a minimum depth of 4 feet below the ground surface after a period of 5 days of pumping. The wells could be operated for a period of approximately six months each year.

On the basis of the assumptions used, a cost analysis showed that if all the water pumped was delivered into the existing irrigation system and sold as part of the regular supply canal for irrigation purposes, about 60 per cent of the cost of pumping would be offset by the sale of water.

This study demonstrates the importance of carefully considering all factors before the feasibility of pumped wells for drainage purposes can be established. Such considerations include preliminary reconnaissance of ground-surface soil conditions, subsurface sampling by augers, cores and observation wells, detailed hydrologic analysis of ground-water movement. aquifer characteristics and responses to pumping based on various analytical methods, water quality aspects, field testing by experimental wells, and consideration of all costs and benefits of controlling the water table by a single well or a system of multiple units.

Pumped wells may offer a reasonable and economic solution to the problems of draining agricultural lands and con-

trolling a shallow water table. The field conditions for which pumped wells will be successful are, in fact, rather limited and must be examined carefully and completely in advance of a full-scale pump system program. One factor which may contribute substantially to the practicability of pumped wells is that the water may be used for irrigation if it is of a desirable quality.

APPENDIX A

GROUND-SURFACE ELEVATIONS AT PIEZOMETER LOCATIONS

Piezometer No.	Ground-surface elevation
	feet
2	107.5
3	106.2
4	103.0
7	125.6
8	121.6
9	114.6
10	95.8
11	106.1
12	106.5
14	106.0
15	103.7
16	106.3
17	107.1
18	106.3
19	105.2
20	101.9
21	101.5
22	101.8
23	88.7
24	87.3

APPENDIX B

Piezometer and Well Logs.*

Piezometer No. 2: shallow piezometer at 18-ft. depth
deep piezometer at 85-ft. depth

Location: 5/8 32P, T 5S, R 8 E, Sec. 32 P MD

Approx. 1/4 mile S 33 and Almond, in area where canal lateral bends away from railroad.

Ground elevation 107 ft. Total depth 116.5 ft.

Depth of strata (ft)	*Material*
0–5	Brownish-gray clay loam
5–28	Yellowish brown clay, plastic, some sand, impermeable
28–30	Sandy clay
30–35	Same as from 5–28, perhaps a little more sandy
35–46	Yellow-brown clay, plastic, fat
46–51	Silt and very fine-grained sand
51–82½	Clay
82½–88	Sand and some small gravel
88–97	Clay and small pebbles
97–115½	Clay
115½–116½	Sand

Piezometer No. 4: deep piezometer at 80-ft. depth

Location: 5/8 31 A1, T 5S, R 8 E, Sec. 31 A MD

North of Richfield station 33 on SW corner of Poppy Ave. & Hwy 33, 1 mile south of Patterson.

Ground elevation 103 ft. Total depth 120 ft.

Depth of strata (ft)	*Material*
0–4	Topsoil
4–20	Clay
20–22	Clay, short streaks of gravel
22–31	Clay, short streaks of silt
31–42	Silt with streaks of clay
42–47	Clay
47–48	Clay with short gravel streaks
48–56	Clay
56–62	Clay, streaks of fine silt
62–64	Clay
64–68	Clay, streaks of gravel and fine silt
68–77	Clay, short streaks of sand and gravel
77–82	Sand and gravel
82–87	Clay, streaks of gravel
87–91	Clay
91–93	Clay
93–95	Fine silt, sand
95–101	Hard clay
101–104	Hard clay
104–106	Fine sand, streaks of gravel
106–111	Coarse sand, gravel, short clay streaks
111–116	Hard clay
116–120	Coarse sand and gravel

* Logs for test hole No. 3 are found in table 1 on p. 358.

Piezometer No. 8: deep piezometer at 58-ft. depth
Location: 5/8 31 J T 5S R 8E Sec. 31J MD
S Patterson on Mistletoe between Bartch & Elfers, 100′ N of white house west of road.
Ground elevation 122 ft. Total depth 97 ft.

Depth of strata (ft)	*Material*
0–6	Topsoil
6–15	Sandy, silty clay (silt, possibly streaks of clay)
15–25	Sandy silt, silty fine-grained sand
25–28	Coarse sand and gravel
28–31	Clay, soft plastic
31–41	Silty clay (39–41 sandy silt)
41–45	Coarse sand and gravel
45–47	Clay, hard
47–51	Silty clay
51–55	Hard clay, streaks of gravel
55–60.5	Coarse sand and gravel
60.5–62.5	Clay
62.5–67	Coarse sand and gravel, short streaks of clay
67–71	Fine silty sand
71–75	Fine sand, streaks of clay
75–77	Coarse sand and gravel
77–81	Fine sandy clay, streaks of gravel
81–88	Fine sand, streaks of brown clay
88–89	Gravel
89–91	Sandy clay, traces of gravel
91–97	Fine sand, streaks of clay

Piezometer No. 10: deep piezometer at 83-ft. depth.
Location: 5/8 32 F Stanislaus Co. T 5S, R 8E, Sec. 32 F MD
SE Patterson—NE intersection Fig & Locust Ave. on Highway—SE on west bank.
First canal lateral—50′ past end of orchard, in alfalfa field.
Ground elevation 95 ft. Total depth 121 ft.

Depth of strata (ft)	*Material*
0–6	Topsoil
6–15	Slightly silty yellow-brown clay
15–21	Soft yellow-brown clay
21–25	Brown hard clay—slightly sandy
25–35	Clay, yellow-brown, hard
35–41	Brownish sandy clay, very hard
41–49	Soft brown clay, streaks of fine sand
49–58	Med. to coarse sand and gravel, small streaks clay
58–66	Hard brown clay
66–71	Clay, streaks of fine sand
71–81	Sandy clay, traces of gravel and coarse sand
81–85	Coarse sand and gravel
85–88	Silty clay
88–91	Silty clay, streaks of fine gravel, reddish-brown
91–94	Hard sand, streaks of clay
94–99	Sand, med. gravel, hard drilling
99–101	Sandy clay
101–111	Hard, fine gravel, clayey sand
111–121	Hard brown clay

Piezometer No. 11: very deep piezometer at 250-ft. depth.
Location: 5/8–32 E 2
100 ft. S on west side of Highway 33 and canal intersection
Ground elevation 107 ft. Depth of hole 263 ft.

Depth of strata (ft)	*Material*
0–5	Topsoil
5–27.5	Sandy brown clay
27.5–40	Silty fine sand—few granules, gypsum, loosely packed, brown
40–50	Very fine-grained silty sand—loosely packed, few granules, med. brown
50–70	Stiff silty clay, some sand grains, MnO_2-stained and shear planes
70–75	Hard, fine to med. sand, reddish-brown, tightly packed
75–80	Fine silty sand
80–90	Silty sand clay, traces of coarse sand and gravel
90–100	Clayey silty sand, dense reddish-brown, well graded, poor sorting to coarse sand
100–110	Fine sand, silty, clayey
110–115	Sandy clay
115–120	Sandy clay, traces of gravel
120–140	Sandy clay, streaks of sand and gravel
140–149	Silty fine sand, very fine-grained, brown, soft, moist
149–150	Hard brown clay, slightly sandy
150–156	Coarse, medium sand and gravel up to 1″
156–164.5	Sandy hard brown clay, high per cent of salts deposited in vein network
164.5–169.5	Coarse sand and gravel
169.5–171.0	Sandy gravelly clay
171–180	Stiff sandy brown clay
180–224	Silty blue clay, micaceous
224–227	Sand
227–229	Clay
229–234	Medium to fine bluish-gray sand
234–237	Very coarse sand and gravel up to 1″
237–243	Fine to medium sand
243–247	Hard coarse to very coarse sand
247–255	Hard fine to medium sand
255–260	Coarse sand and gravel
260–263	Fine to medium hard sand

APPENDIX C

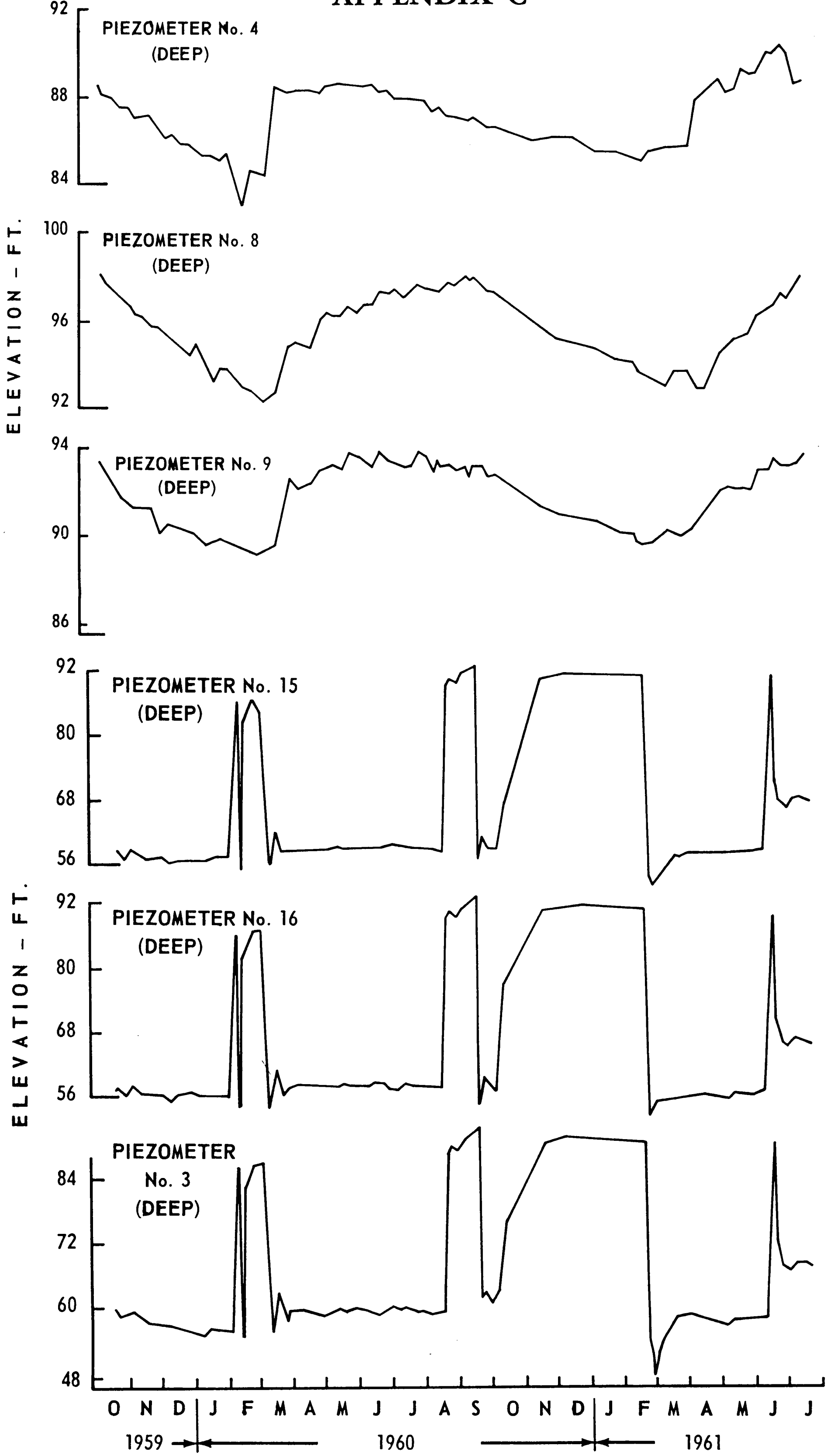


Fig. 21. Piezometer hydrographs.

APPENDIX D

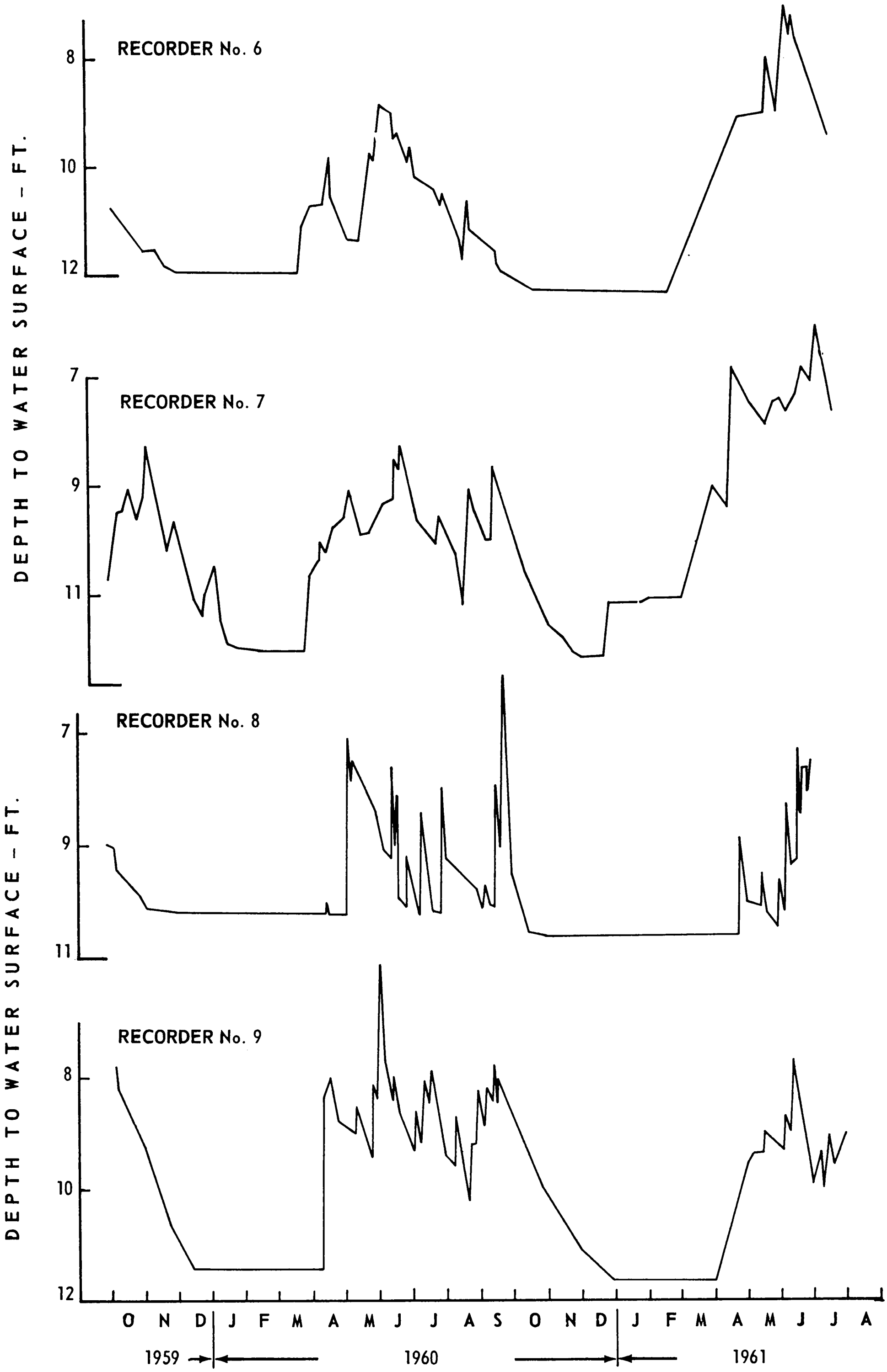


Fig. 22. Shallow observation wells hydrographs.

APPENDIX E

Details of Aquifer Characteristics Calculations

1. Methods for Constant Discharge

Although two of the four well tests performed at the Patterson well were of the unthrottled, decreasing-discharge type, the conventional solutions developed for constant-flow conditions were applied and a representative average Q chosen for the determination of the aquifer coefficients.

(a) **Theis nonequilibrium method.** This method was applied to data collected at six piezometers during a test performed on October 19, 1959. A summary of the data used is shown in table E-1.

TABLE E–1

SUMMARY OF DATA FROM OCTOBER 19, 1959, TEST

Piez. No.	Distance (r) from well	Time since pumping began (t)	Drawdown (h)	Piez. No.	Distance (r) from well	Time since pumping began (t)	Drawdown (h)
	feet	*hrs.*	*feet*		*feet*	*hrs.*	*feet*
3........	18	1.50	20.2	14.......	48	1.70	25.3
		2.50	26.6			2.50	31.1
		3.57	29.9			3.73	35.2
		4.75	32.4			4.50	37.6
		20.00	34.2			20.10	38.7
15........	51	1.62	24.8	16.......	54	1.67	27.5
		2.58	29.8			2.62	33.0
		3.60	33.8			3.62	36.0
		4.80	36.0			4.82	38.6
		20.02	36.2			20.07	38.1
18........	117	1.77	16.5	11.......	189	1.92	5.9
		2.75	21.0			2.83	9.4
		3.77	22.8			4.12	12.4
		4.86	17.3			5.17	13.7
		20.13	17.5			20.43	7.2

Data from the table were plotted in figure 23 and matched against a standard Theis well-function curve. T and S were computed by the formulas

$$T = \frac{Q}{4\pi}\frac{W(u)^*}{h^*} \qquad \text{and} \qquad S = \frac{4u^*Tt^*}{r^2}$$

in which

$W\ (u)$ is the well function of $u = (r^2S)/(4Tt)$

h is the drawdown, in ft

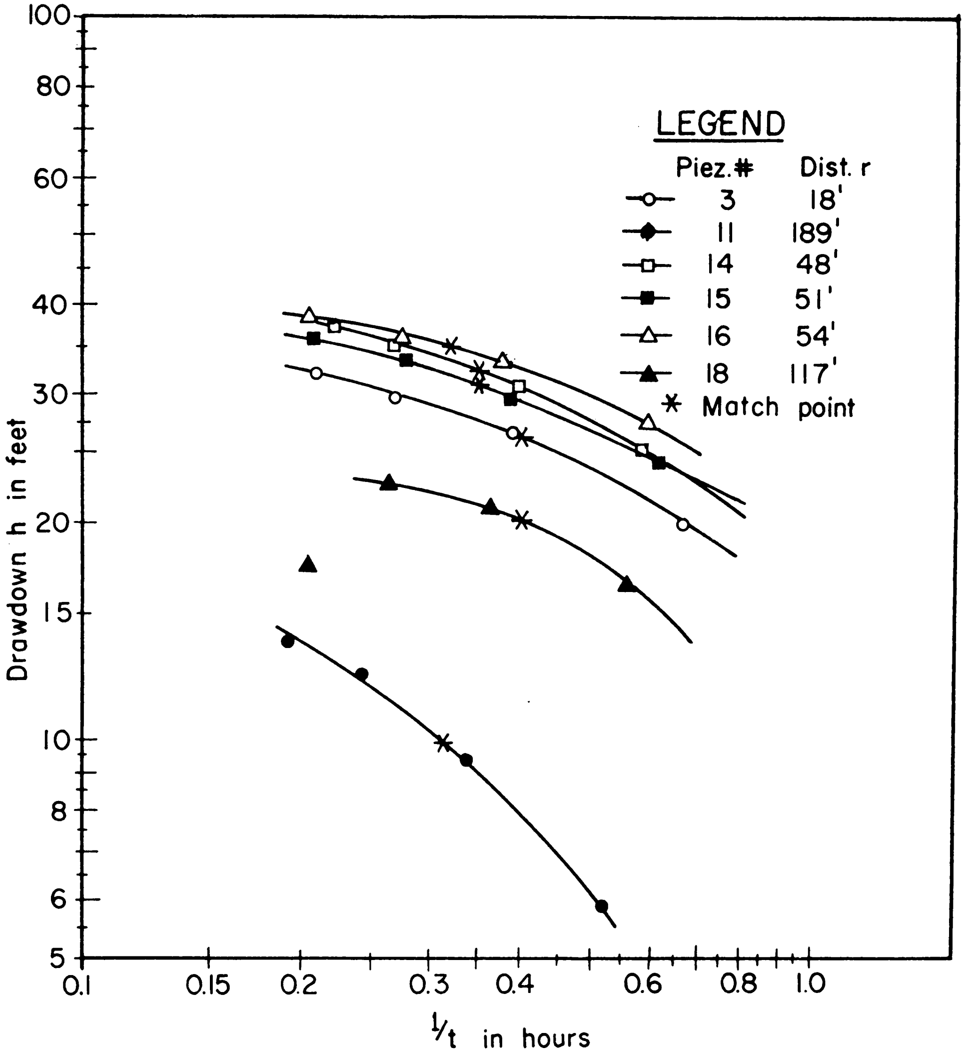
r is the distance of the piezometers from the well, in ft

t is the time since pumping began

Q is the average well discharge over the arbitrarily chosen first 5 hours

The asterisks denote matchpoints

The average transmissibility computed for the six piezometers is 0.0131 ft^2/sec. A weighted average storage coefficient was computed (because of the wide disparity between the six computed values) according to the formula $S_{ave} = (\Sigma rS)/(\Sigma r)$ and found to be 0.0076.



Piez #	r ft	$\frac{1}{t}$	Match Points h	u	W(u)	T ft^2/sec	S
3	18	0.40	26.5	0.044	2.62	0.0140	0.0685
11	189	0.31	10.0	0.275	1.00	0.0141	0.00505
14	48	0.35	32.0	0.043	2.65	0.0117	0.0090
15	51	0.35	31.0	0.038	2.75	0.0125	0.0075
16	54	0.32	35.0	0.026	3.11	0.0125	0.0050
18	117	0.40	20.2	0.088	1.95	0.0136	0.00315

Fig. 23. Theis solution for Oct. 19, 1959, test data.

(b) **Jacob straight-line methods.** This method includes the following:

(1) Drawdown at one location with varying time.

The drawdown data shown in table E-1 were plotted semilogarithmically in figure 24 versus time, as proposed by Jacob (1950), and transmissibility and storage coefficients were computed by the formulas

$$T = \frac{2.3\ Q\ \Delta\log t}{4\pi\ \Delta h} \quad \text{and} \quad S = \frac{2.25\ Tt_0}{r^2}$$

The average transmissibility and weighted average storage coefficients computed from the six best fitting straight lines were as follows:

$T_{ave} = 0.0147\ \text{ft}^2/\text{sec}$

$S_{ave} = 0.0062\ \text{ft}^2/\text{sec}$

(2) Drawdown at different piezometer locations.

Values of the drawdown 5 hours after pumping began were plotted for several piezometer locations versus the logarithm of the distances from the well as shown in figure 25. From the slope of this plot the transmissibility and storage coefficients were calculated using the following formulas:

$$T = \frac{2.3\ Q}{2\ \Delta h/(\Delta\log r)} \quad \text{and} \quad S = \frac{2.25\ Tt}{r_0^2}$$

Table E-2 shows drawdown data from a test conducted on September 24, 1960. The discharge after 5 hours of pumping was 540 gpm. Therefore,

$$T = \frac{2.3 \times 540}{2\pi \times 449 \times 25.0} = 0.0175\ \text{ft}^2/\text{sec}$$

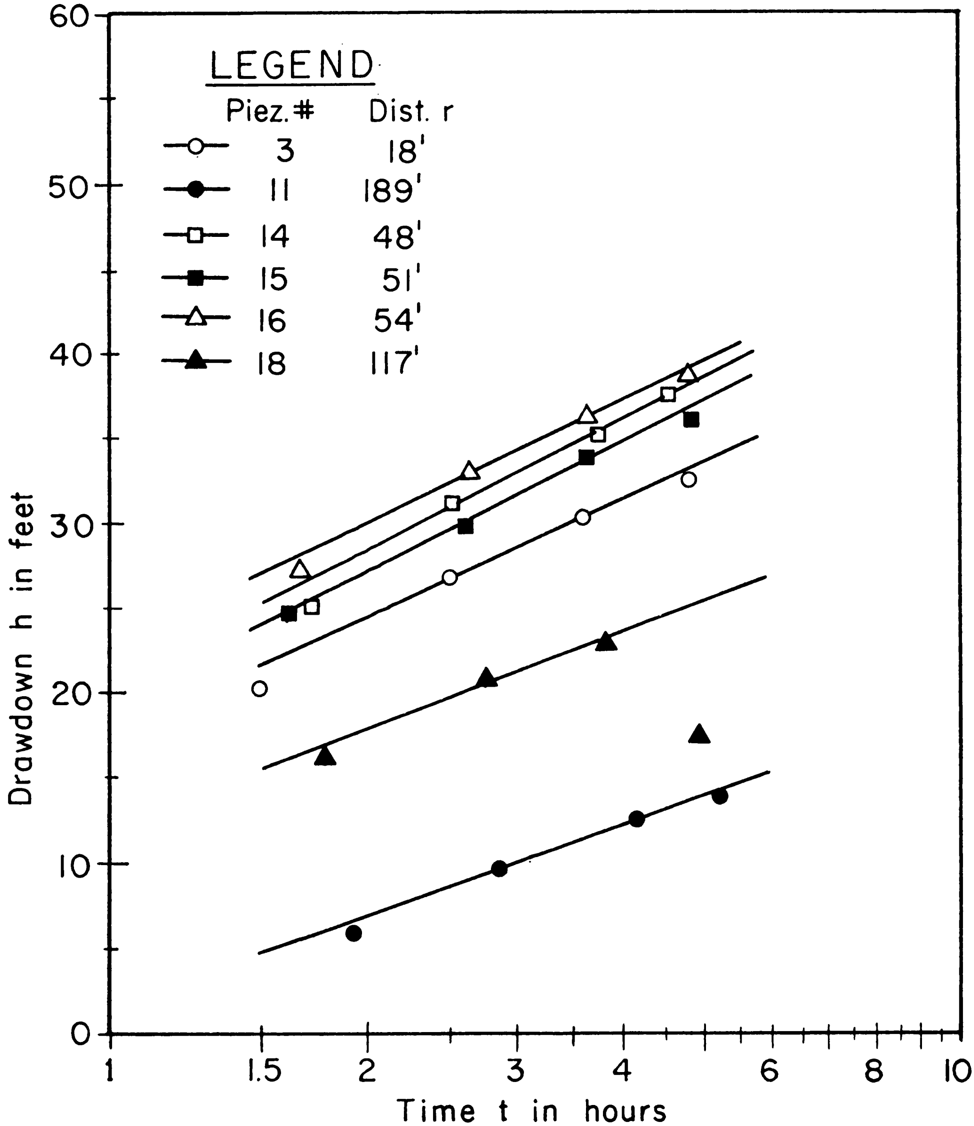
$$S = \frac{2.25 \times 0.0175 \times 18{,}000}{1120^2} = 0.00055$$

TABLE E-2

DRAWDOWN AT DIFFERENT PIEZOMETER LOCATIONS AFTER 5 HOURS OF PUMPING

Piez. No.	Distance (*r*) from well	Drawdown (*h*)
	feet	*feet*
3	18	33.3
14	48	40.4
15	51	38.4
16	54	40.6
18	117	28.6
11	189	6.8
12	189	2.4
17	207	22.8
19	237	20.6
20	802	0.1
22	1,080	0.1
21	1,375	0.1

(c) **Recovery test.** On August 22, 1960, a well recovery test was run by shutting off the pump which had been operating at a steady rate between 300 and 320 gpm for several months.



Piez #	Δh ft	t_o hrs	r ft	T ft^2/sec	S
3	23.0	0.175	18	0.0142	0.0620
11	17.8	0.80	189	0.0183	0.0033
14	25.5	0.158	48	0.0128	0.0071
15	25.3	0.172	51	0.0129	0.0069
16	24.0	0.115	54	0.0136	0.0043
18	19.2	0.230	117	0.0170	0.0023

Fig. 24. Jacob solution for Oct. 19, 1959, test data.

Recovery data of the ground-water table at three piezometers are listed in table E-3 and plotted in figure 27.

The transmissibility can be calculated from the slope of a semilogarithmic plot of $(t)/(r^2)$ versus the recovery in feet. The following formula is used.

$$T = \frac{2.3\ Q}{4\pi\ \Delta h}$$

where Δh is the change in the recovery per one logarithmic cycle of $(t)/(r^2)$.

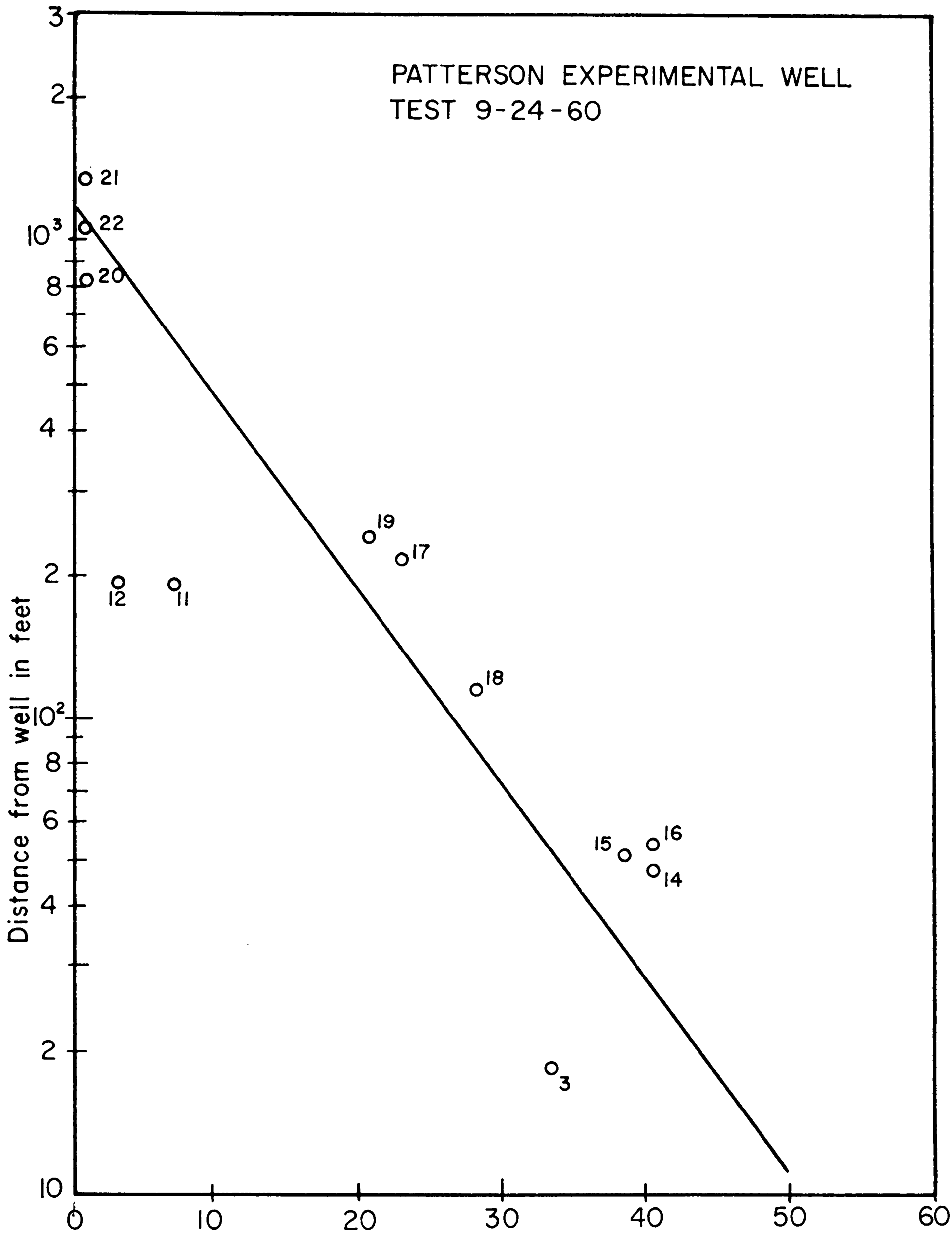


Fig. 25. Distance from well vs. drawdown after 5 hours of pumping.

TABLE E-3
DATA FOR RECOVERY TEST

Piez. No. 15			Piez. No. 16			Piez. No. 18		
Elapsed time	t/r^2	Recovery	Elapsed time	t/r^2	Recovery	Elapsed time	t/r^2	Recovery
days	*days/sq. ft.*	*feet*	*days*	*days/sq. ft.*	*feet*	*days*	*days/sq. ft.*	*feet*
0	0	0	0	0	0	0	0	0
8.35×10^{-3}	3.21×10^{-5}	8.3	9.70×10^{-3}	2.87×10^{-5}	13.1	1.18×10^{-2}	6.65×10^{-6}	6.7
1.32×10^{-2}	5.08×10^{-5}	11.7	1.46×10^{-2}	4.60×10^{-5}	14.9	1.60×10^{-2}	1.17×10^{-5}	8.0
2.56×10^{-2}	9.88×10^{-5}	16.1	2.71×10^{-2}	8.80×10^{-4}	17.0	2.85×10^{-2}	2.08×10^{-5}	10.0
3.81×10^{-2}	1.46×10^{-4}	18.5	4.03×10^{-2}	1.32×10^{-4}	18.8	4.37×10^{-2}	3.20×10^{-5}	11.4
5.05×10^{-2}	1.94×10^{-4}	18.9	5.76×10^{-2}	1.74×10^{-4}	19.2	5.98×10^{-2}	4.32×10^{-5}	12.4
1.48×10^{-1}	5.70×10^{-4}	22.0	1.55×10^{-1}	5.12×10^{-4}	21.5	1.53×10^{-1}	1.12×10^{-4}	14.8
5.60×10^{-1}	2.15×10^{-3}	24.1	5.60×10^{-1}	1.92×10^{-3}	24.4	5.60×10^{-1}	4.08×10^{-4}	17.3
$7\ 50 \times 10^{-1}$	2.88×10^{-3}	25.0	7.57×10^{-1}	2.58×10^{-3}	25.4	7.60×10^{-1}	5.53×10^{-4}	18.1
1.04	4.00×10^{-3}	25.6	1.04	3.57×10^{-3}	25.9	1.04	7.60×10^{-4}	18.7
1.56	6.00×10^{-3}	26.7	1.57	5.38×10^{-3}	27.1	1.57	1.14×10^{-3}	19.9

Using an average Q of 310 gpm or 0.69 cfs, the coefficients of transmissibility and storage calculated from the slopes of the straight lines in figure 25 are as follows:

$$T_{15} = \frac{2.3 \times 0.69}{4\pi 4.85} = 0.0260 \text{ ft}^2/\text{sec.} \qquad S = \frac{2.25 \times 0.0260}{580} = 1.0 \times 10^{-4}$$

$$T_{16} = \qquad = 0.0260 \text{ ft}^2/\text{sec.} \qquad S = \qquad = 8.1 \times 10^{-5}$$

$$T_{18} = \qquad = 0.0243 \text{ ft}^2/\text{sec.} \qquad S = \qquad = 7.9 \times 10^{-4}$$

The average transmissibility from these values is 0.0254 ft²/sec and the weighted average storage coefficient 0.00046.

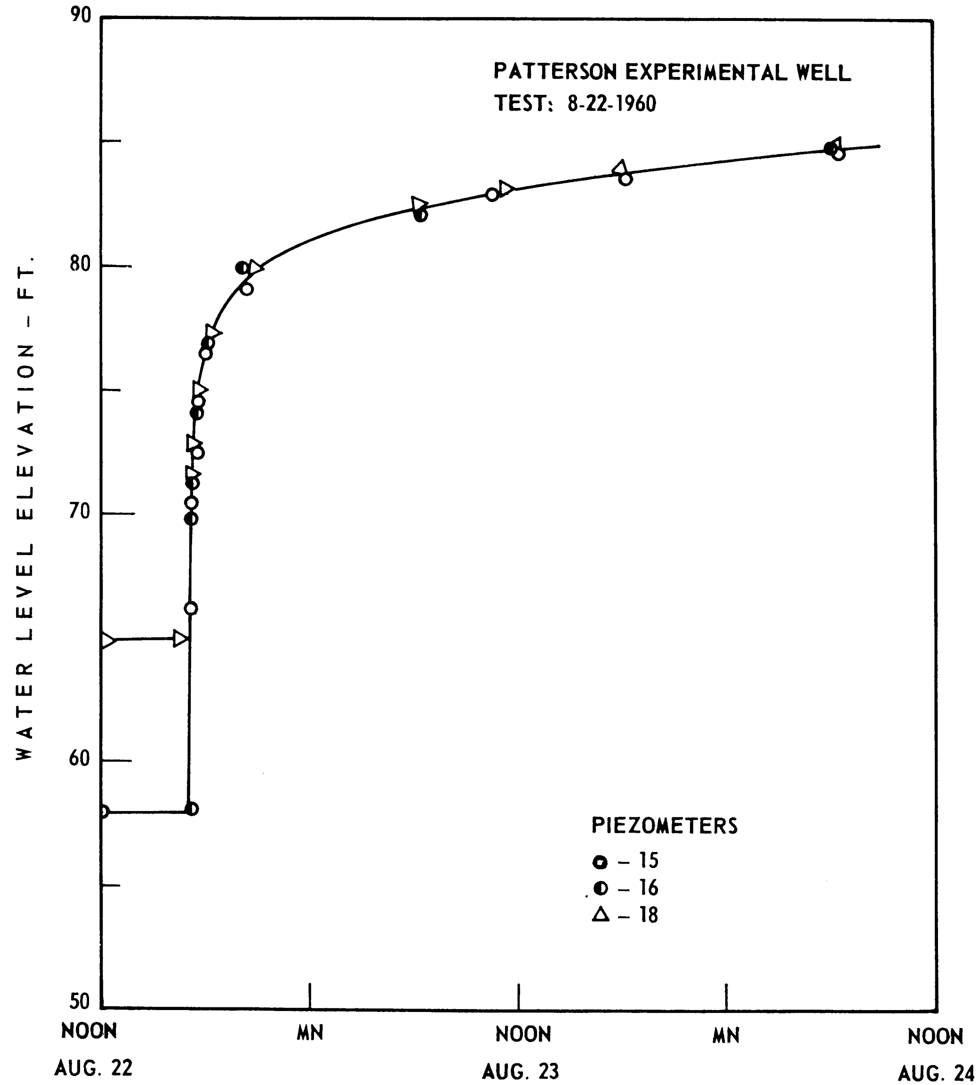


Fig. 26. Recovery-test data.

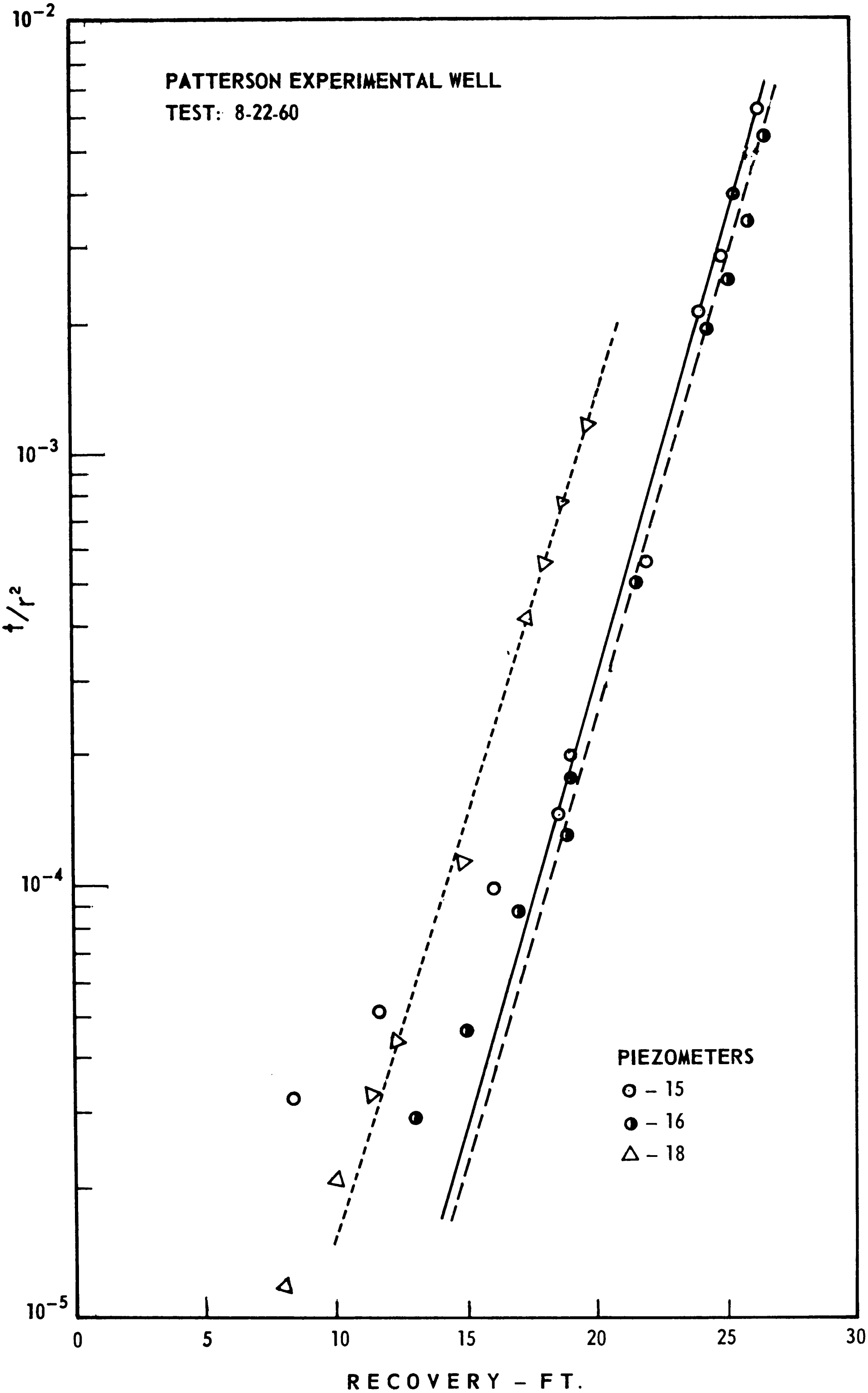


Fig. 27. Recovery of water level for three piezometers vs. t/r^2.

(d) **Chow's method.** This method is summarized in the following steps:

1. A semilogarithmic plot is prepared between the drawdown in feet and the time in minutes (figure 29).
2. Choose a point, P, on the curve, find its coordinates, and draw a tangent to the curve at this point.
3. Find the slope of the tangent.

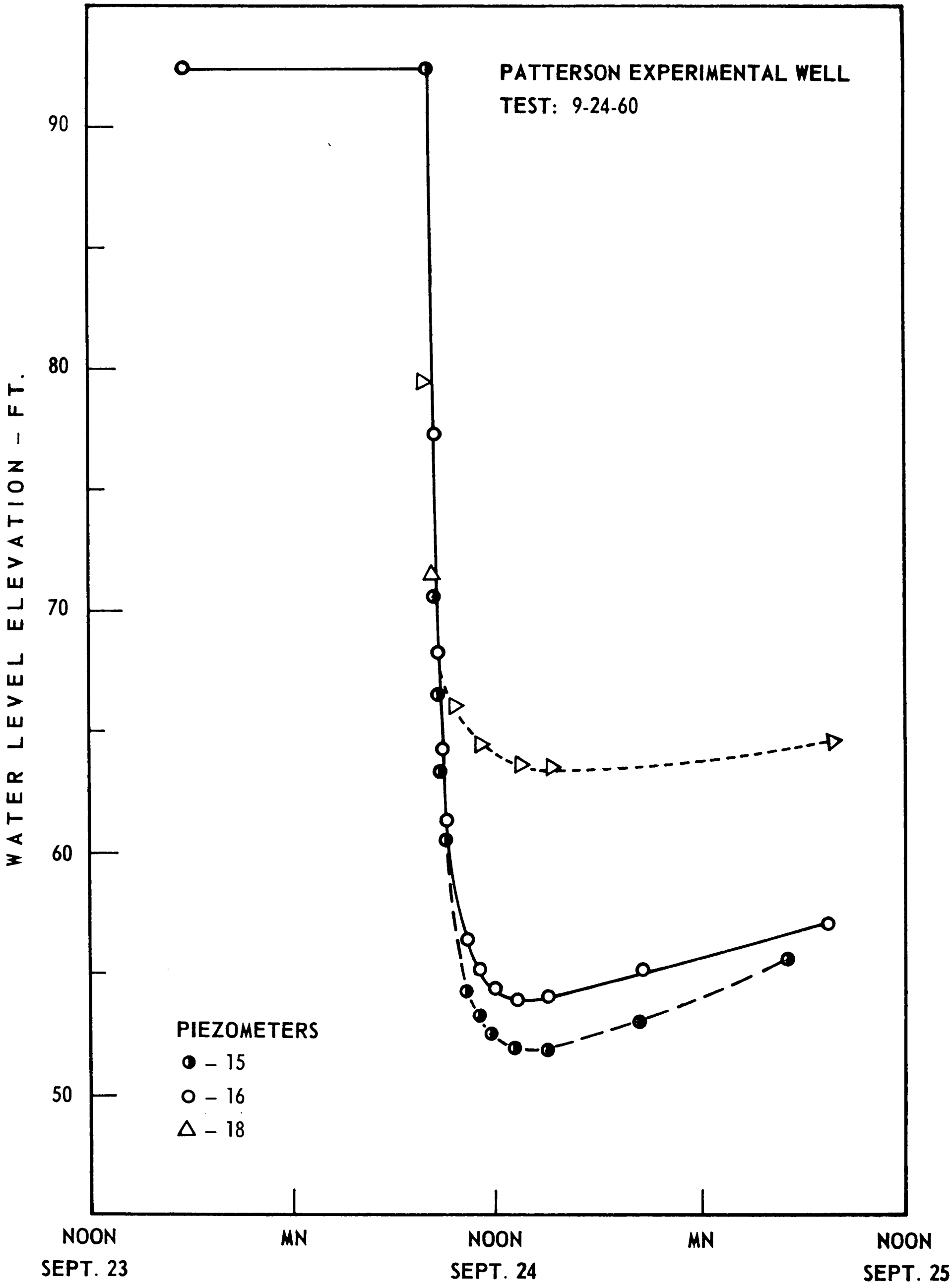


Fig. 28. Decreasing-discharge test data.

4. Calculate $f(u)$ from the following equation:

$$f(u) = \frac{h}{dh/d \log t}$$

5. Find a corresponding value for $W(u)$ from graph given in reference 4.
6. Calculate T from the formula

$$T = \frac{QW(u)}{4\pi h}$$

(1) For the unthrottled drawdown test:

Table E-4 shows the field data for piezometers Nos. 15, 16, and 18 from the drawdown test run on September 24, 1960. The drawdown data for these three piezometers are plotted on semilogarithmic graph paper for the determination of T on figure 29.

TABLE E–4
FIELD DATA FROM DRAWDOWN TEST RUN ON SEPTEMBER 24, 1960

Piez. No. 15		Piez. No. 16		Piez. No. 18	
t	*h*	*t*	*h*	*t*	*h*
min.	*feet*	*min.*	*feet*	*min.*	*feet*
11	15.1	13	22	16	13.0
25	24.0	28	26.1	42	19.0
38	28.1	40	29.2	60	21.1
56	31.1	58	32.2	142	26.4
128	36.1	130	38.3	218	28.1
167	37.1	169	39.3	308	28.8
213	37.9	214	40.1		
302	38.4	304	40.6		

The aquifer characteristics T and S are computed in table E-5. The well discharge of the time chosen for the point of tangency was 610 gpm, or 1.36 cfs.

(2) For the controlled-discharge test:

A constant-discharge test of 300 gpm was conducted on June 22, 1961. The drawdown data for piezometers Nos. 15, 16, and 18 are given in table E-6 and shown in figure 31. Table E-7 summarizes the transmissibility calculations.

TABLE E–5
COMPUTATION OF T AND S BY CHOW'S METHOD

Piez. No.	Coord. of point *P*		Δ*h*	*f*(*u*)	*u*	*W*(*u*)	*T*	*S*
	h	*t*						
	feet	*min.*	*feet*				*sq. ft./sec.*	
15	34.0	80	15.0	2.27	0.0030	5.20	0.0165	0.00055
16	34.8	80	16.6	2.10	0.0045	4.83	0.0149	0.00054
18	23.5	80	14.0	1.68	0.0125	3.84	0.0177	0.00035
	Average value of *T*						0.0164	
	Weighted average value of *S*							0.00044

2. Methods for a Decreasing Discharge.

The drawdown at Piez. No. 16 location, and discharge data of the test conducted on September 24, 1960, were used for aquifer characteristics calculations using the decreasing-discharge solution (Abu-Zied and Scott, 1963; Abu-Zied *et al.*, 1964).

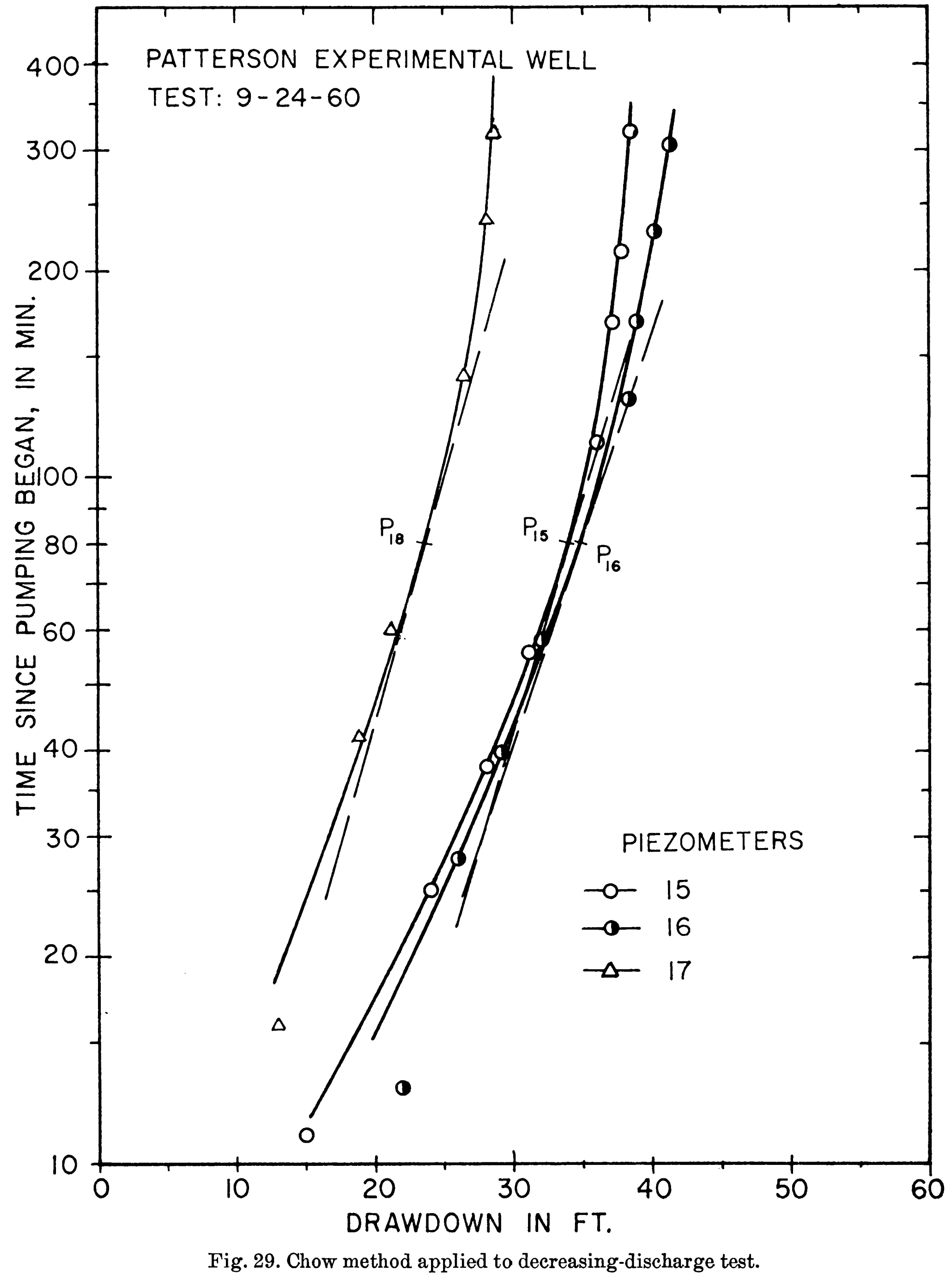


Fig. 29. Chow method applied to decreasing-discharge test.

Table E-8 shows the pumping discharge data during this test. The two methods used are: (1) the general type curve solution (Abu-Zied and Scott, 1963), and (2) modified solution (Abu-Zied *et al.*, 1964).

(a) **Type curve solution.** The drawdown at any place in the aquifer pumped at a decreasing discharge rate is expressed by the equation

$$h = \frac{Q_o}{4\pi T}\left[\alpha W(B) + \beta \bar{e}^{at} f(A,B)\right]$$

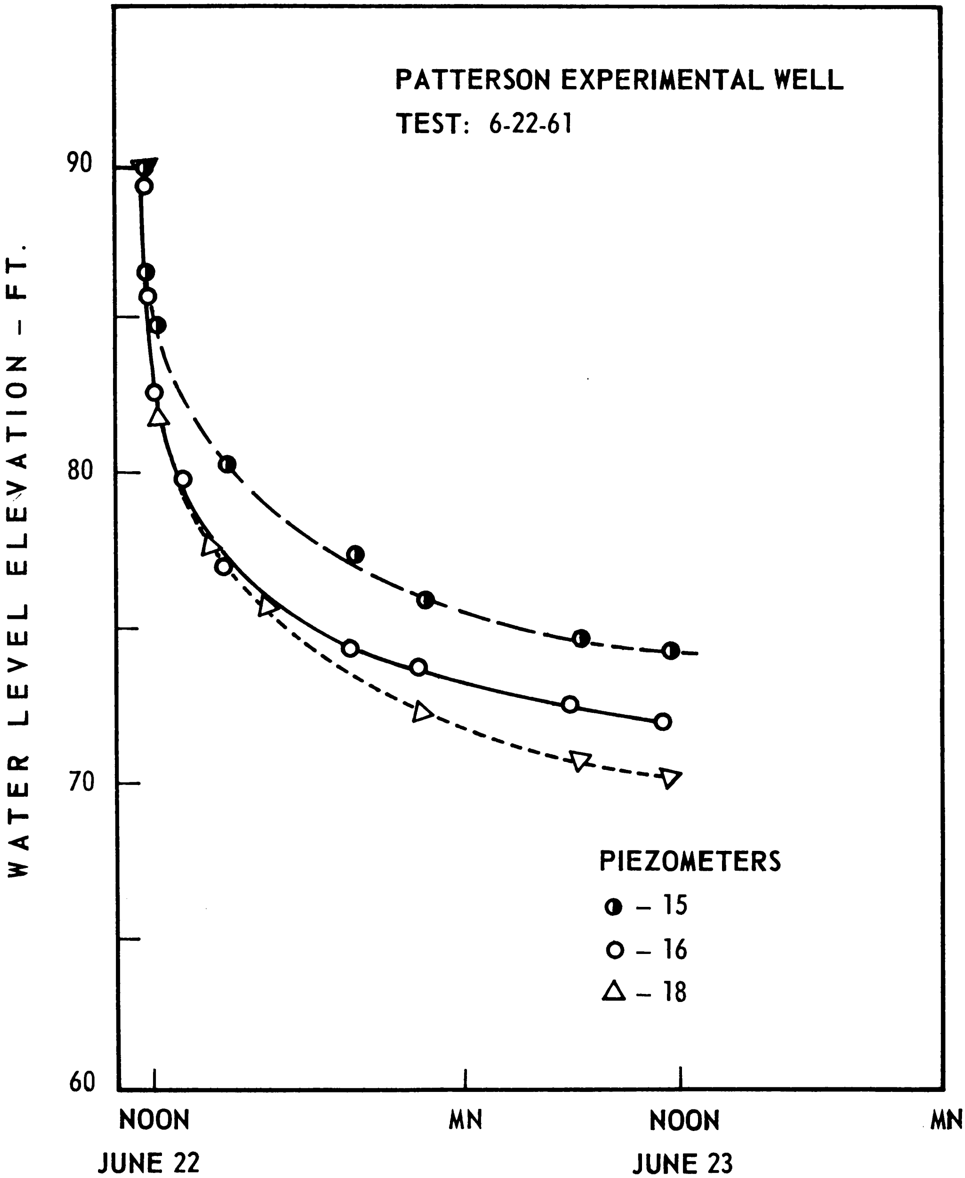


Fig. 30. Constant-discharge test data.

TABLE E-6

DRAWDOWN DATA FOR THE CONTROLLED-DISCHARGE TEST

Piez. No. 15		Piez. No. 16		Piez. No. 18	
t	*h*	*t*	*h*	*t*	*h*
min.	*feet*	*min.*	*feet*	*min.*	*feet*
3	0.75	13	5.97	18	3.15
4	1.50	34	8.02	37	5.12
5	1.75	53	8.97	48	5.67
7	2.45	80	10.17	76	6.92
9	3.15	114	11.32	107	7.99
10	3.39	141	11.50	121	8.42
21	5.43	182	12.41	151	8.72
35	6.95	213	12.47	177	9.22
110	10.00	330	14.17	220	9.97
137	10.27	574	15.40	337	11.12
176	11.05	752	17.67	587	11.98
210	11.85	1,202	19.37	770	14.22
326	12.95			1,213	15.42
570	14.23				
746	15.70				

TABLE E-7

VALUES USED IN CALCULATING TRANSMISSIBILITIES FOR THE CONTROLLED-DISCHARGE TEST BY CHOW'S METHOD

Piez. No.	Coord. of point *P*		Δh	$f(u)$	$W(u)$	u	T	S
	h	*t*						
	feet	*min.*	*feet*				*sq. ft./sec.*	
15	8	60	6.3	1.27	2.60	0.042	0.0172	0.0040
16	12.8	220	7.6	1.68	3.7	0.015	0.0154	0.0042
18	8	110	6.8	1.18	2.4	0.052	0.0160	0.0016
	Average value of *T*						0.0162	
	Weighted average value of *S*							0.0028

TABLE E-8

PUMPING DISCHARGE DATA (SEPT. 24, 1960)

Elapsed time	Pumping rate		Elapsed time	Pumping rate	
min.	*gpm*	*cfs*	*min.*	*gpm*	*cfs*
3	867	1.931	167	574	1.278
5	750	1.670	212	564	1.256
10	740	1.648	217	540	1.203
15	720	1.604	303	546	1.216
20	680	1.514	308	540	1.203
25	680	1.514	416	526	1.171
30	660	1.470	421	520	1.158
35	660	1.470	735	494	1.100
47	642	1.430	1,447	442	0.987
63	627	1.392	3,020	390	0.867
104	625	1.396	7,370	340	0.757
109	600	1.336	14,400	340	0.757
139	580	1.292	24,400	340	0.757
144	580	1.292			

In figures 13 and 14 the following values were determined:

$$Q_o = 1.360 \text{ cfs}$$
$$\alpha = 0.557$$
$$\beta = 0.443$$
$$a = 1.44 \times 10^{-5}$$

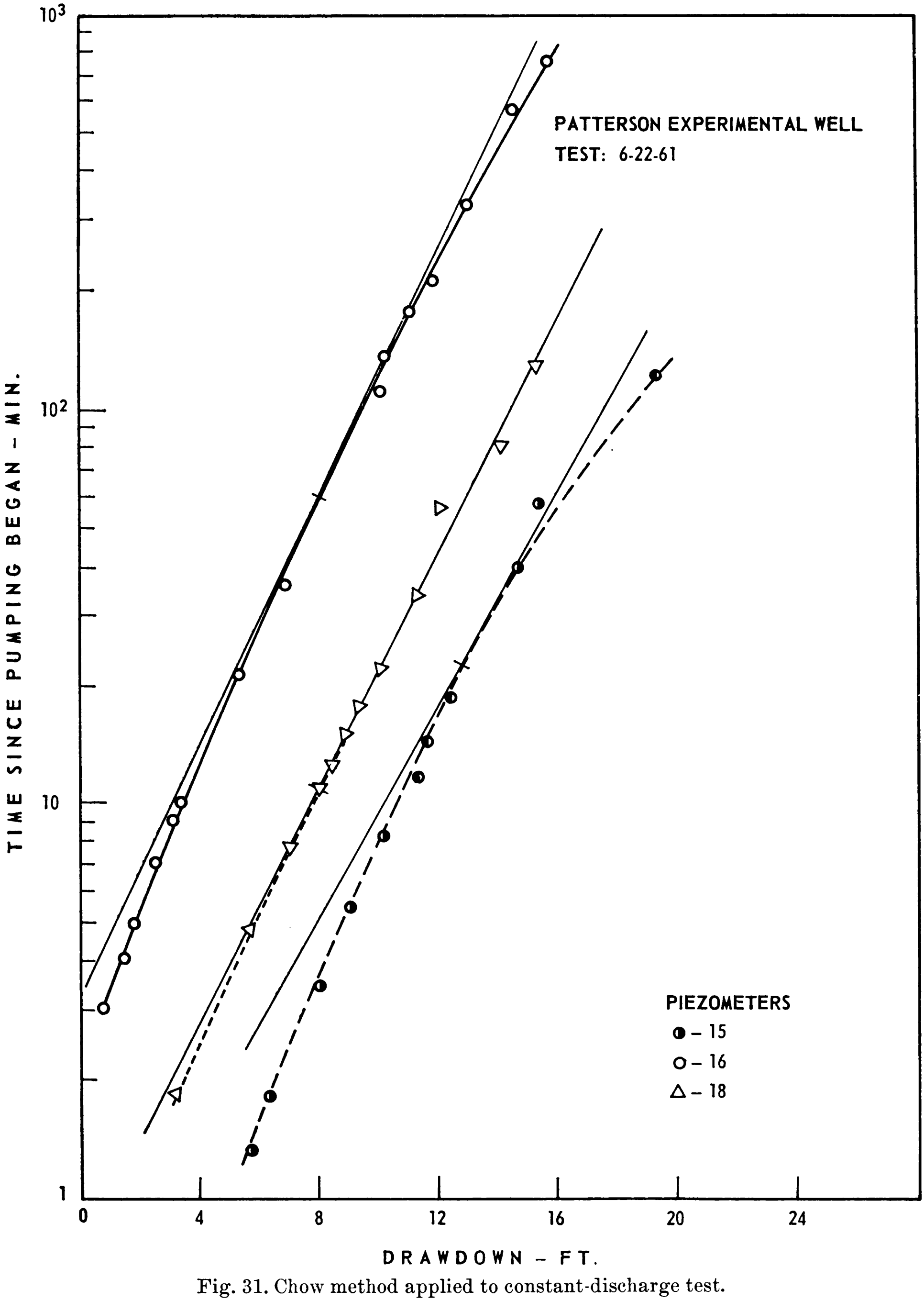


Fig. 31. Chow method applied to constant-discharge test.

Computations for the field data curve (figure 32) are given in table E-9. Values of $A = at$ in this table range between 0.01 and 0.36.

For matching with the field curve, two type curves, for $A = 0.04$ and 0.20, respectively, were constructed from published values (Aron *et al.*, 1964) of Theis' well function $W(B)$ and Abu-Zeid's function $f(A,B)$. The computations for the values of the type curves are shown in table E-10. These two type curves together with the curve of Theis' well function $W(B)$ were plotted on figure 33 vs. B. Note: Theis' well function is equal to $f(A,B)$ when $A = 0$.

TABLE E–9

DATA FOR THE FIELD CURVE, h vs. r^2/t

Elapsed time (t)	$A = at$	h	r^2/t
seconds	*seconds*	*feet*	*sq. ft./second*
780	0.0112	22.0	3.7385
1,680	0.0242	26.1	1.7357
2,400	0.0346	29.2	1.2150
3,480	0.0500	32.2	0.8376
7,800	0.112	38.3	0.3738
10,140	0.146	39.3	0.2875
12,840	0.185	40.1	0.2271
18,240	0.263	40.6	0.1599
25,200	0.363	40.7	0.1157

All three curves are so similar that there can be no major difference in match-point values. Any significant difference in computed coefficients of transmissibility will be the result mostly of the selected values of Q_{ave} (Theis method) or Q_o (Abu-Zied method).

TABLE E–10

DATA FOR THE TYPE CURVE,
B vs. $[\alpha W(B) + \beta e^{-A} f(AB)]$*

B (1)	$W(B)$ (2)	$\alpha W(B)$ (3)	$f(AB)$ (4)	$\beta e^{-A} f(AB)$ (5)	Col. (3) + Col. (5) (6)
a Curve for $A = 0.20$					
0.001	6.33	3.52	6.45	2.38	5.90
0.002	5.64	3.14	5.85	2.12	5.26
0.004	4.95	2.76	5.15	1.87	4.63
0.01	4.04	2.25	4.24	1.54	3.79
0.02	3.35	1.87	3.55	1.29	3.16
0.04	2.68	1.49	2.86	1.04	2.53
0.10	1.82	1.01	2.19	0.80	1.81
b Curve for $A = 0.04$					
0.001	6.33	3.52	6.37	2.71	6.23
0.002	5.64	3.14	5.68	2.41	5.55
0.004	4.95	2.76	4.99	2.12	4.88
0.01	4.04	2.25	4.06	1.72	3.97
0.02	3.35	1.87	3.37	1.43	3.30
0.04	2.68	1.49	2.70	1.15	2.64
0.10	1.82	1.01	1.85	0.79	1.80

* Constants: $\alpha = 0.557, \beta = 0.443$

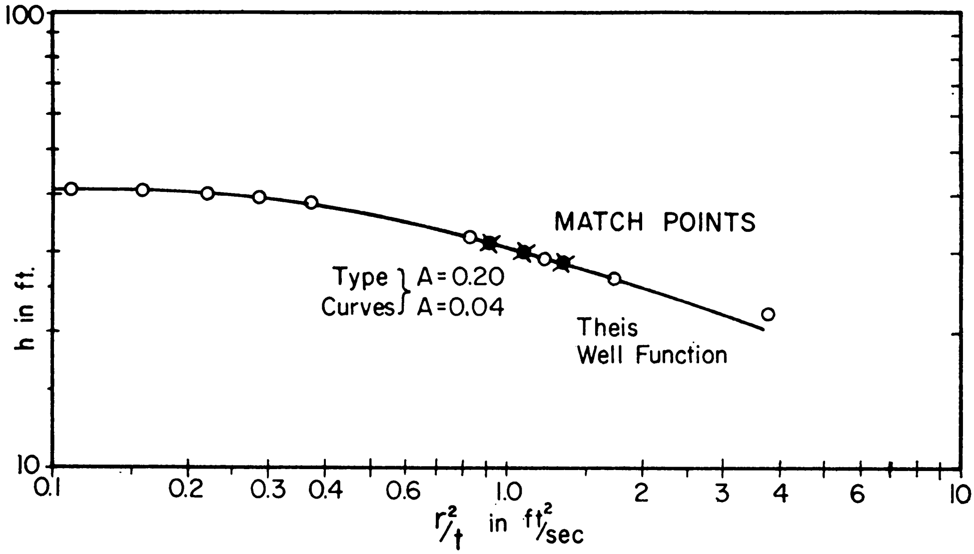


Fig. 32. Field data curve of drawdown vs. $\frac{r^2}{t}$ at a piezometer 54 feet from pumped well.

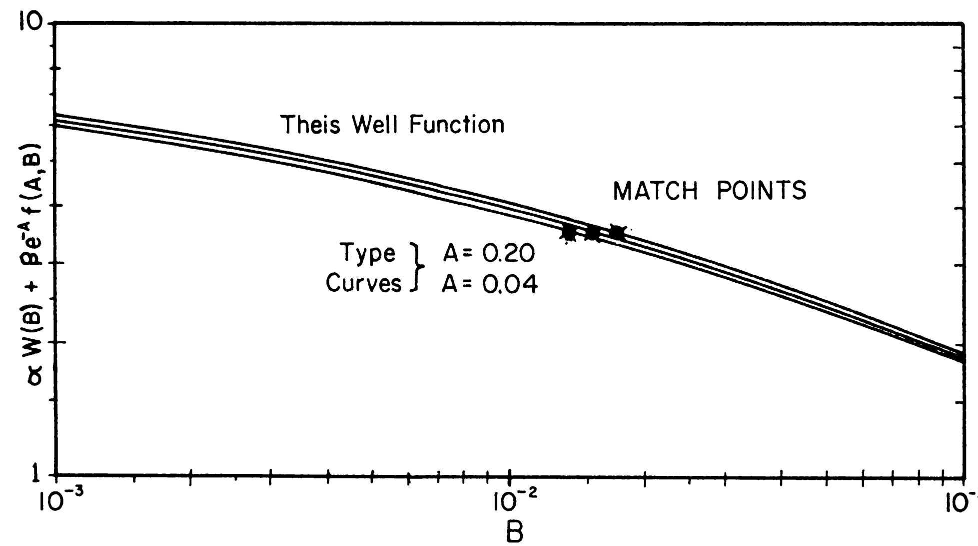


Fig. 33. Theis and type curves.

The aquifer coefficients of transmissibility and storage were determined by the following equations: in Theis' solution

$$T = \frac{Q_{ave}}{4\pi}\,\frac{W(B)^*}{h^*} \text{ and } S = 4T\,\frac{B^*}{(r^2/t)^*}$$

in Abu-Zied's general solution

$$T = \frac{Q_o}{4\pi}\,\Phi\,\frac{(AB)^*}{h^*} \text{ and } S = 4T\,\frac{B^*}{(r^2/t)^*},$$

in which the asterisks denote matchpoint values of the respective parameters.

TABLE E-11

COMPUTATION OF TRANSMISSIBILITY AND STORAGE COEFFICIENTS

Method	Q_o	$\Phi(AB)^*$	h^*	B^*	r^2/t^*	T	S
	cu. ft./sec.		*feet*		*sq. ft./sec.*	*sq. ft./sec.*	
Abu-Zied							
A = 0.04	1.36	3.5	29.9	.0155	1 12	0.0127	0.00071
A = 0.20	1.36	3.5	31.5	.0137	0.92	0.0121	0.00072
Theis							
	Q_{ave}	$W(B)^*$					
	1.31	3.5	28.1	.0173	1.34	0.0131	0.00068

TABLE E-12

DATA FOR MODIFIED-METHOD CURVE*

t	h	A	e^{-A}	$\alpha + \beta e^{-A}$	$\frac{h}{\alpha + \beta e^{-A}}$
seconds	*feet*				
780	22.0	0.0112	0.989	0.995	22.1
1,680	26.1	0.0242	0.976	0.989	26.4
2,400	29.2	0.0346	0.966	0.985	29.6
3,480	32.2	0.0500	0.951	0.978	32.9
7,800	38.3	0.1115	0.894	0.953	40.2
10,140	39.3	0.146	0.864	0.940	41.8
12,840	40.1	0.185	0.831	0.925	43.4
18,240	40.6	0.263	0.769	0.898	45.2
25,200	40.7	0.363	0.696	0.865	47.0

* Constants: α = 0.557 β = 0.443

The value of Q_o as found in figures 13 and 14 was 1.36 cfs. To obtain a representative Q_{ave} for the Theis solution, the first 5 hours of pumping were considered, because the main drawdown trend in the observation well was established during this period. From water-meter readings, the average pump discharge over these 5 hours was 591 gpm or 1.32 cfs.

The coefficients of transmissibility and storage were computed in table E-11 for the Abu-Zied general solution and the Theis methods.

(b) **Modified solution.** In the modified solution (Abu-Zied *et al.*, 1964) for decreasing well discharge, the approximation

$$h \approx \frac{2.30}{4\pi T} Q_o(\alpha + \beta e^{-A})\left(\log \frac{2.25\,Tt}{r^2 S}\right)$$

is used to solve for the aquifer characteristics, T and S.

In figure 34, values of $h^* = h/(\alpha + \beta e^{-A})$, computed in table E-12, are plotted vs. log t. As in the Jacob method, T is then computed from the slope of the best-fitting straight line, and S from the intercept with the h^* axis, yielding the following results.

$$T = \frac{2.3\,Q_o}{4\pi\,\Delta h^*}\,\Delta \log t = \frac{2.3 \times 1.36 \times 1}{4\pi\ 19.3} = 0.0128\ \text{ft}^2/\text{sec}$$

$$S = \frac{2.25\,Tt_o}{r^2} = \frac{2.25 \times 0.0128 \times 68}{54^2} = 0.00067$$

For comparison, the h vs. log t data from the September 24, 1960, test were also plotted in figure 34 according to the Jacob method. One can clearly see the noticeable curvature of the plot due to the discharge decrease with advancing time. Applying this method to a decreasing discharge test it becomes not only difficult to find a best-fitting straight line for the plot, but almost impossible to choose a representative average value of Q.

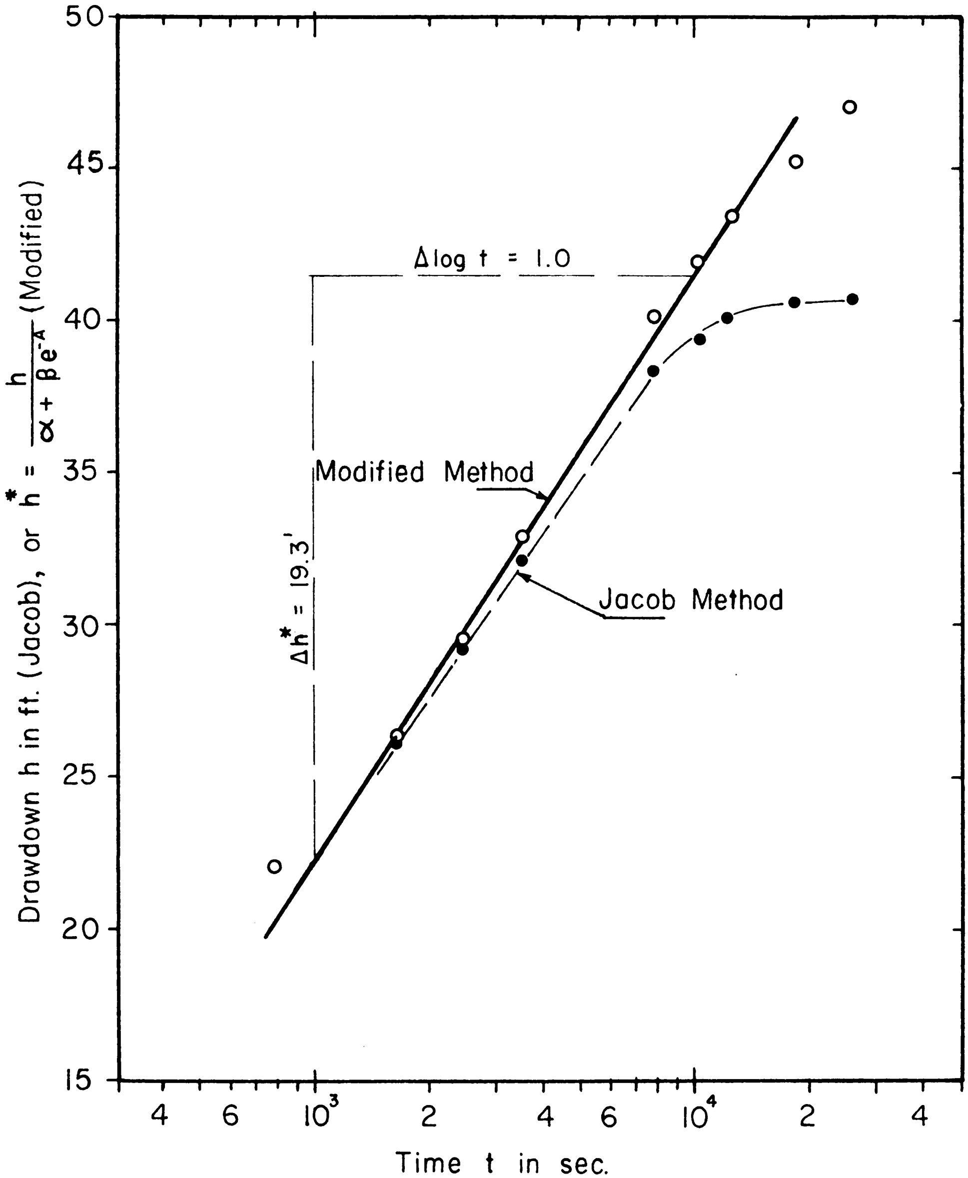


Fig. 34. Modified and Jacob methods applied for comparison.

TABLE E-13

SUMMARY OF COEFFICIENTS OF TRANSMISSIBILITY AND STORAGE

Flow condition	Test date	Method of solution	T	S
			sq. ft./sec.	
Discharge decreasing with time	10-19-59	Theis	0.0131	0.0076
" " " "	"	Jacob*	0.0147	0.0062
" " " "	9-24-60	Theis	0.0131	0.00068
" " " "	"	Jacob*	0.0136	0.00061
" " " "	"	Jacob†	0.0175	0.00055
" " " "	"	Chow	0.0164	0.00044
" " " "	"	Type curve	0.0124	0.00072
" " " "	"	Modified	0.0128	0.00067
Constant discharge	6-22-61	Chow	0.0162	0.0028
Recovery test	8-22-61	Jacob*	0.0254	0.00046

* Computations for individual observation wells, time varying.
† Computations drawdown profile, several observation wells at a chosen instant.

Using the rather arbitrary value of 1.32 cfs (average Q over the first 5 hours of the test), the coefficient of transmissibility and storage would be

$$T = \frac{2.3 \times 1.32 \times 1}{4\pi\ 17.9} = 0.0136 \text{ ft}^2/\text{sec}$$

$$S = \frac{2.25 \times 0.0136 \times 58}{54^2} = 0.00061$$

Comparison of results

In table E-13 a comparison of transmissibility coefficients obtained by various methods and for four well tests is given.

Table E-13 shows generally good agreement among values of T, except for the solution of the Jacob method applied to the recovery test. In this test, the discharge prior to shutdown of the pumps may not have been very closely controlled, which could lead to errors. Disregarding this one high value, the average coefficient of transmissibility of the aquifer would be 0.0144 ft²/sec. Since the coefficient of storage in the values of S was scattered over a wide range, no averaging was attempted.

APPENDIX F

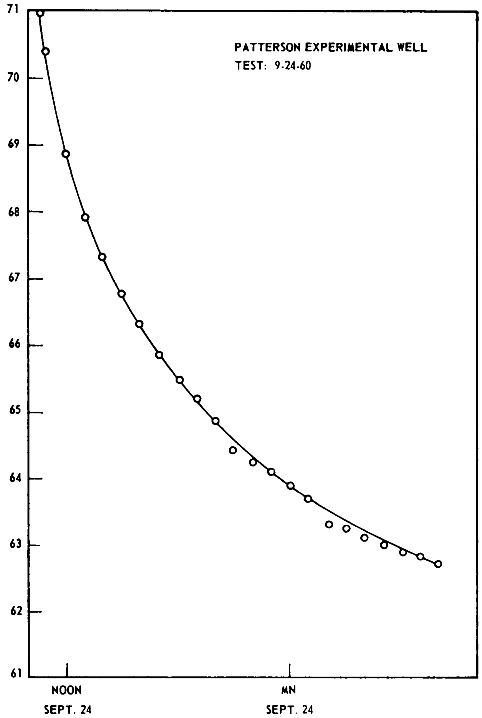


Fig. 35. Water level in pumped well during decreasing-discharge test.

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