Research Report 120

Ground-Water Investigation at Jake Wolf Fish Hatchery, Mason County, Illinois

by
Adrian P. Visocky and Mark E. Sievers

ILLINOIS STATE WATER SURVEY
DEPARTMENT OF ENERGY AND NATURAL RESOURCES

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RESEARCH REPORT 120

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Abstract: As part of the expansion plans for Jake Wolf Fish Hatchery in Mason County, the well field that supplies its process and drinking water was tested and evaluated. Step tests were conducted on each of the eight production wells in order to assess their hydraulic condition and determine whether or not deterioration had occurred in the years since they were constructed. Weekly water-level measurements were also made in newly constructed observation wells in order to estimate natural recharge to the aquifer and to evaluate the impact of the well field on the regional ground-water resource. The wells were found to be pumping near capacity, and the current well field is not considered capable of producing significantly greater quantities of water. Recommendations include the addition of two and possibly three new wells located approximately two miles south of the existing well field.

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Indexing Terms: Ground water, aquifer, Mason County, production wells, step-drawdown tests, natural recharge, water levels, flow-net analysis, potentiometric surface, water supply, well field, pumpage, water table.

STATE WATER SURVEY DIVISION
MARK E. PEDEN, Acting Chief

2204 GRIFFITH DRIVE
CHAMPAIGN, ILLINOIS 61820-7495

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by Adrian P. Visocky and Mark E. Sievers

ABSTRACT

The Jake Wolf Memorial Fish Hatchery, located west of Manito in Mason County, Illinois, pumps water for its operational needs from eight wells located one mile southeast of the hatchery. The wells are finished in thick Pleistocene sand-and-gravel deposits.

Step-drawdown tests were conducted on each of these production wells in order to assess the hydraulic condition of their screens. Analysis of the step-test data indicated that with the exception of well 3, their hydraulic condition is good. Since they were constructed, specific capacities have increased at all wells but 3 and 4. Well losses at well 3 are significant

Since the wells are not equipped with discharge meters, pumpage is estimated by calculating total dynamic head and applying this number to pump performance curves supplied by the pump manufacturers. Several methods were used to estimate natural recharge:

1. **Hydrographs from wells remote from the well field suggest that water levels were approaching an apparent equilibrium (recharge was balancing pumpage) prior to the drought of 1988-1989. A modest correlation between water levels and pumpage also suggests equilibrium conditions.**
2. **A method suggested by Stallman (19S6) was used to estimate recharge: weekly water-level data from wells in three "five-spot" arrays were substituted into a finite-difference equation of ground-water flow. Calculated recharge rates at each five-spot were adjusted to an "effective" rate by subtracting a visually interpreted base level to account for the sloping water table. Effective recharge rates from the three arrays varied between 292,000 and 507,000 gallons per day per square mile (gpd/sq mi) or 6.1 and 10.6 inches per year.**
3. **Recharge rates calculated from flow-net analysis of monthly potentiometric surface maps averaged 855,000 gpd/sq mi (18.0 inches).**
4. **A simple water-balance analysis was conducted using measured precipitation, estimated potential evapotranspiration, and changes in soil moisture. The analysis indicated 19.76 inches of surplus soil moisture for 1990, which is equivalent to 941,000 gpd/sq mi of available recharge.**

Supplemental water supplies of 2,500 gallons per minute are required for operations at the fish hatchery. In all probability two additional wells will be needed and are recommended for construction about two miles south of the existing well field.

INTRODUCTION

The Jake Wolf Memorial Fish Hatchery near Manito, Illinois, has been in operation since 1982. To supply its water needs, it has relied on a well field consisting of eight production wells. Estimated daily water use at the hatchery ranges from about 5 to 10 million gallons a day (mgd) and averages about 8.5 mgd.

The Illinois Department of Conservation (IDOQ) contracted with Cochran & Wilken Inc., of Springfield to examine hatchery operations in a major ongoing study. The Illinois State Water Survey was contracted in fall 1988 to conduct the ground-water portion of the research. IDOC had three major concerns regarding the operation of the well field at the hatchery:

1. The condition of the eight production wells at the well field, and how their present hydraulic performance compares with that at the time of their completion.
2. The effect of ground-water withdrawals from this well field on the regional aquifer, and whether or not withdrawals are being balanced by natural recharge to the aquifer.
3. The feasibility of additional pumping from the present well field to serve additional water needs if, as anticipated, the fish hatchery expands its facilities. If a new well field were necessary, where could it be sited to minimize interference with the present well field?

Scope of the Study

The initial contract for this study was received in November 1988, and work was scheduled over a period of 14 months. Unanticipated conditions in the well field and in the data results required additional supplemental contracts and time, extending the contract through June 1991. The study was divided into three parts or phases in order to address the concerns outlined above: 1) step testing each of the eight production wells, 2) determining the recharge rate to the regional aquifer tapped by the production wells, and 3) locating suitable sites for additional production wells.

In the first phase of the study, selected hydraulic parameters associated with the production wells were evaluated in order to assess the hydraulic condition of the well screens. These parameters were then compared to those existing at the time each well was completed. This work necessitated the installation of air lines to the wells in winter 1989-1990 so that readings could then be taken.

Phase two involved the installation of 13 observation wells and the routine monitoring of water levels in these wells and in five other observation wells already in the study area. Installation and monitoring began in July 1989, and monitoring concluded in March 1991. Water-level data collected during this period were analyzed by several methods in order to estimate rates of natural replenishment (recharge) to the aquifer. To assess the impact of pumpage, recharge was then compared to average ground-water withdrawals from the well field.

Phase three involved an assessment of the feasibility of additional ground-water withdrawals. This phase was not conducted as a separate, distinct study; instead, it evolved into an ongoing concern throughout much of the first year or so of the study. During this period, the prime contractor for the hatchery study, Cochran & Wilken, frequently sought the comments and recommendations of Water Survey staff as to the potential location of additional production wells. Water Survey staff responded by letter and maintained ongoing communication with the contractor and with hatchery engineering staff regarding potential sites. In addition, the Water Survey drilled a test boring at one of the potential sites in order to assess its feasibility.

Physical Setting of the Study Area

The Jake Wolf Fish Hatchery is located in Sand Ridge State Forest, about four miles west of Manito in' Mason County (figure 1). According to the brochure from the Illinois Department of Conservation, which operates the hatchery, the complex includes a 36,000-square foot hatchery building, a 22-acre solar pond to heat water for fish production, numerous raceways and rearing ponds, and a wastewater treatment system. Additional fish ponds are currently under construction as part of an expansion of operations. Eight production wells, located about a mile southeast of the hatchery building, provide all the water for operations at Jake Wolf.

Sand Ridge State Forest, the largest of Illinois' state forests, covers 7,500 acres. It consists of 3,916 acres of native oak-hickory forest and 2,494 acres of pine plantations. The remainder is open fields and sand prairies. The forest's extremely sandy soil supports a variety of desert flora and fauna, which are the remnants of a dry period following the last glaciation. Physiographically, the forest lies just east of the Illinois River valley in an area of Wisconsinan terraces formed during glacial stages when the river's flow was greater than it is today (Walker et al., 1965).

The area around the forest is heavily agricultural. Annual precipitation at Havana, about ten miles southwest, is 35.5 inches. Center-pivot irrigation systems are used to supplement precipitation for growing melons, seed com, green beans, and other truck crops.

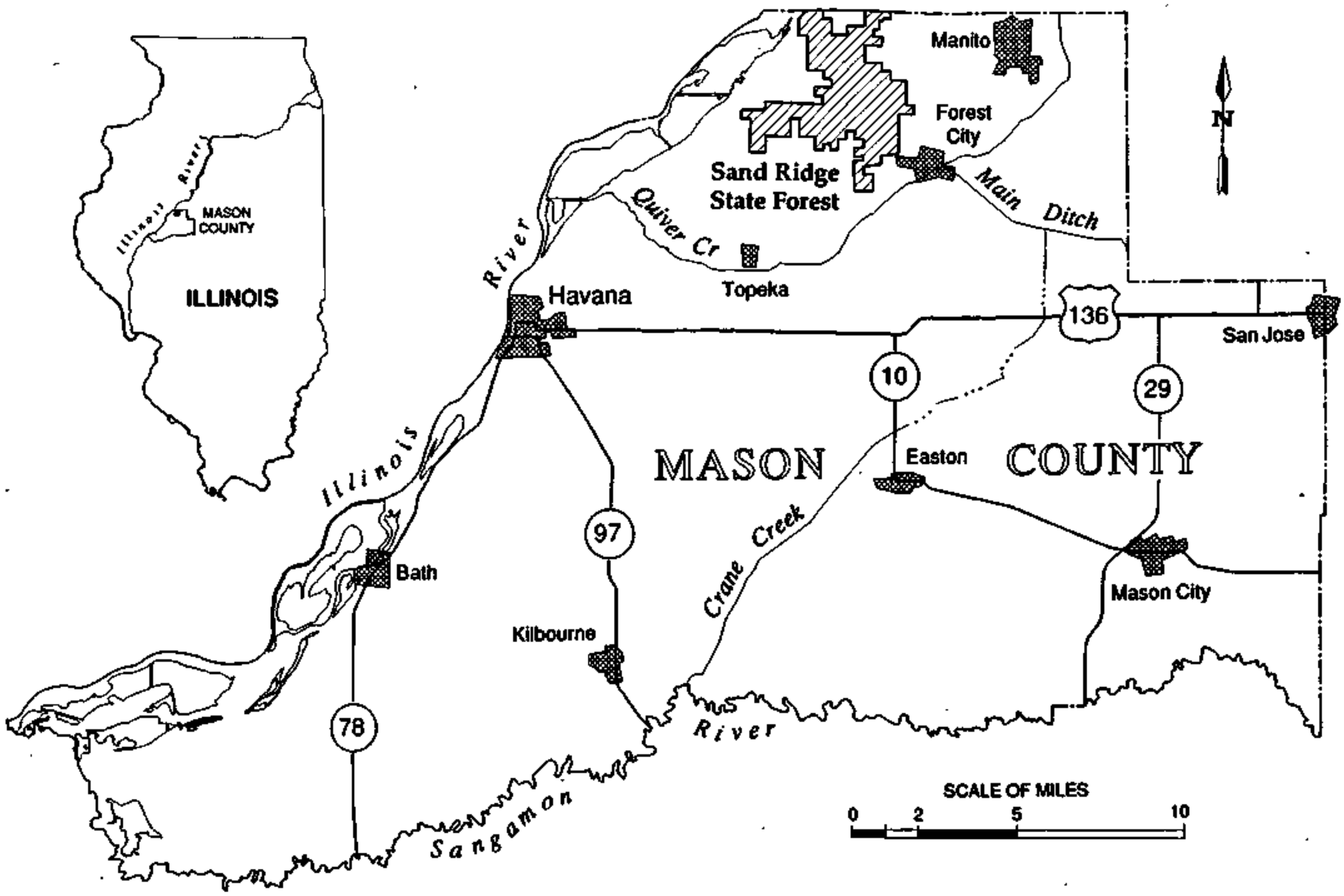

Figure 1. General location of Sand Ridge State Forest

Well Field

The well field that supplies water for operations at the Jake Wolf Fish Hatchery is located in Section 27, T.23N., R.7E., about one mile southeast of the hatchery building (figure 2). The wells are located along forest roadways at approximate spacings of 500 feet from one another. Each well is equipped with an Allis-Chalmers vertical turbine.pump with two to four stages (a Goulds is in well 1). Table 1 summarizes the construction and pump features of the wells. Their depths range from 98 to 110 feet, the screen diameters are either 12 or 16 inches, and the screens are all 40 feet in length. The only exception is well 4, whose screen is 30 feet. Rated pump capacities range from 400 to 1,200 gallons per minute (gpm).

Figure 3 shows a generalized construction schematic for the production wells. The annulus between the casing and the well bore is filled with gravel pack in order to improve the efficiency of water flow to the screen, which is located below the pump. Inside the well, the pump is connected to the motor through a column pipe with flanged couplings. As originally constructed, the only way to measure water levels in the well was through a piezometer suspended in the annulus just before the gravel pack was installed. Annular piezometers were installed in all wells but well 4. Modifications during the current study included adding a stainless steel air line inside each well to the depth of the pump. The air line was connected to a pressure gage used to determine its depth of submergence at any given time. (The contractor noted

Table 1. Construction and Pump Features of Production Wells at Jake Wolf Fish Hatchery

Well	*Depth (feet)*	*Screen length (feet)*	*Screen diameter (inches)*	*Number of pump stages*	*Pump capacityl dynamic head (gpmlft)*
1	98	40	16	3	1,200/180
2	107	40	16	2	1,200/160
3	104	40	16	2	1,200/160
4	110	30	12	3	400/160
5	108	40	16	2	1,200/160
6	106	40	12	4	1,000/170
7	104	40	12	4	1,000/170
8	105	40	12	4	1,000/170

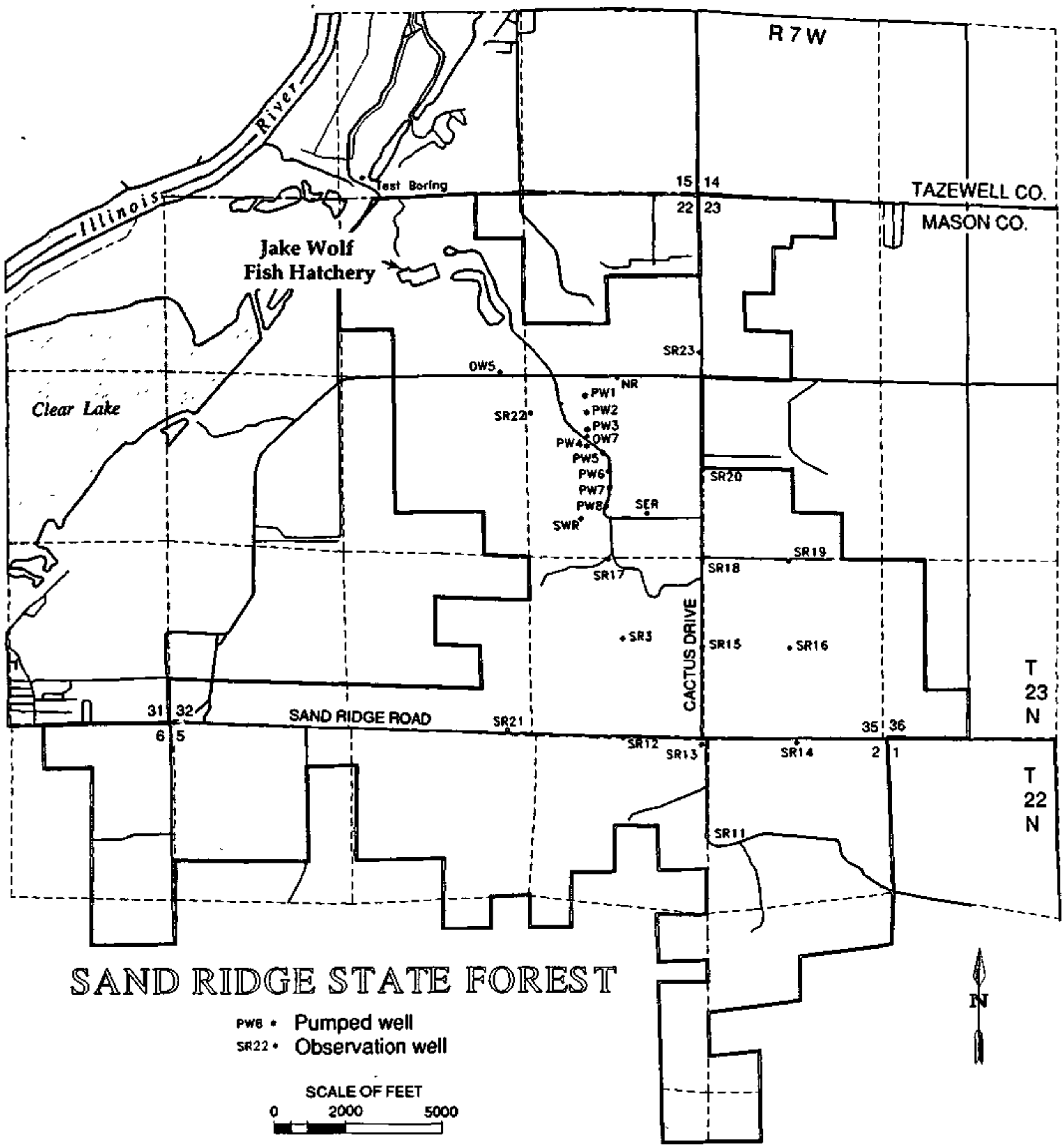

Figure 2. Site of Jake Wolf Fish Hatchery

during the modification of well 3 that he could not raise the pump bowls above a certain point. He speculated that the casing might have been egged-shaped when welded together, thus causing a constriction. This situation has definite implications on the feasibility of performing future service work on this well.)

Acknowledgments

The authors wish to express their thanks to Paul D. Frey, hatchery system engineer at Jake Wolf, for his advice during the study, for his cooperation in facilitating the contractual arrangements, and for supplying well operations information. Thanks are also offered to Dan Riggs, site superintendent at Sand Ridge State Forest, for his valuable help in locating drilling sites. We wish to acknowledge the contributions of Ron Clark of R & R Supply, Manito, for installing the first six observation wells; John Raynolds, Jr., for his surveying work in the monitoring network; and Wehling Well Works, Inc., for their well modification and step-test work. Several Water Survey staff members contributed much-appreciated effort in installing the remaining seven observation wells, including Andy Buck, Steve Burch, and Thad Wilson.

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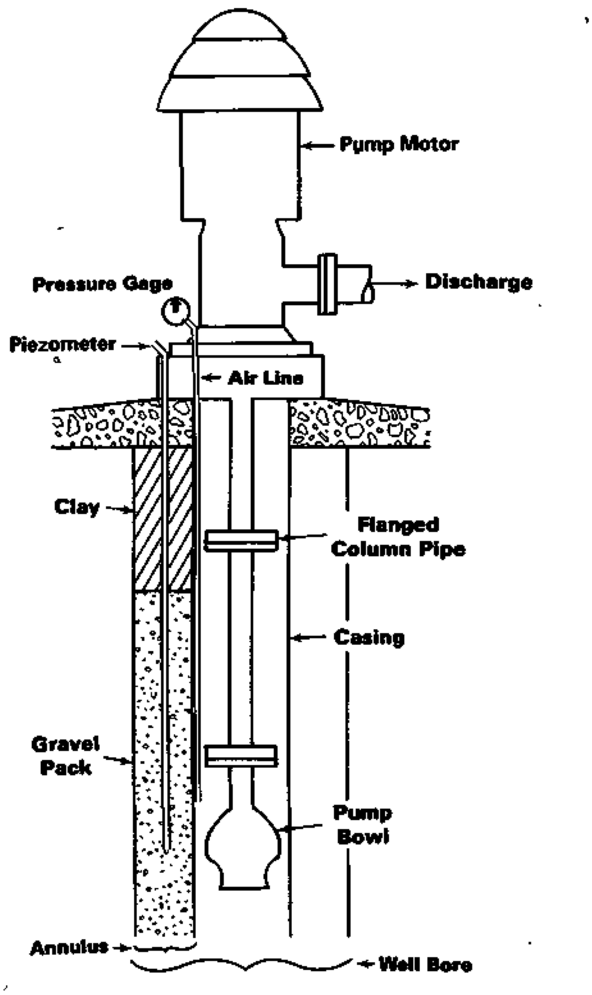

Figure 3. Construction schematic for production wells

ASSESSMENT OF INDIVIDUAL WELL CONDITIONS

Step-Drawdown Tests

Two parameters are useful in assessing the hydraulic condition of a production well: specific capacity and well loss. Specific capacity (yield per foot of drawdown) is a measure of the productivity of a well. It is a function of time, pumping rate, and the hydraulic properties of both the aquifer and the well itself. Well loss, on the other hand, is a function of pumping rate and well hydraulics only; for a given set of conditions it remains constant with time.

The observed drawdown (s_0 in a pumping well consists of the formation loss (s), which results from laminar-flow (and sometimes turbulent-flow) head loss within the aquifer; and well loss (s_w), which results from the turbulent flow of water into and inside the well, as shown in figure 4 and equation 1:

$$s_o = s + s_w \qquad (1)$$

Jacob (1947) expressed these components as being proportional to pumpage, Q, as:

$$s_o = BQ + CQ^2 \qquad (2)$$

where B is the head loss in the formation per unit discharge, and C is referred to as the "well-loss constant" In the Theis equation for drawdown (1935), B is equal to $W(u)/4\pi T$, where W(u), the "well function," is an exponential integral, and T is the aquifer transmissivity, a measure of the ease with which water can move through the aquifer. Rorabaugh (1953) suggested that the well-loss portion should be expressed as "CQV where n is a constant greater than 1.

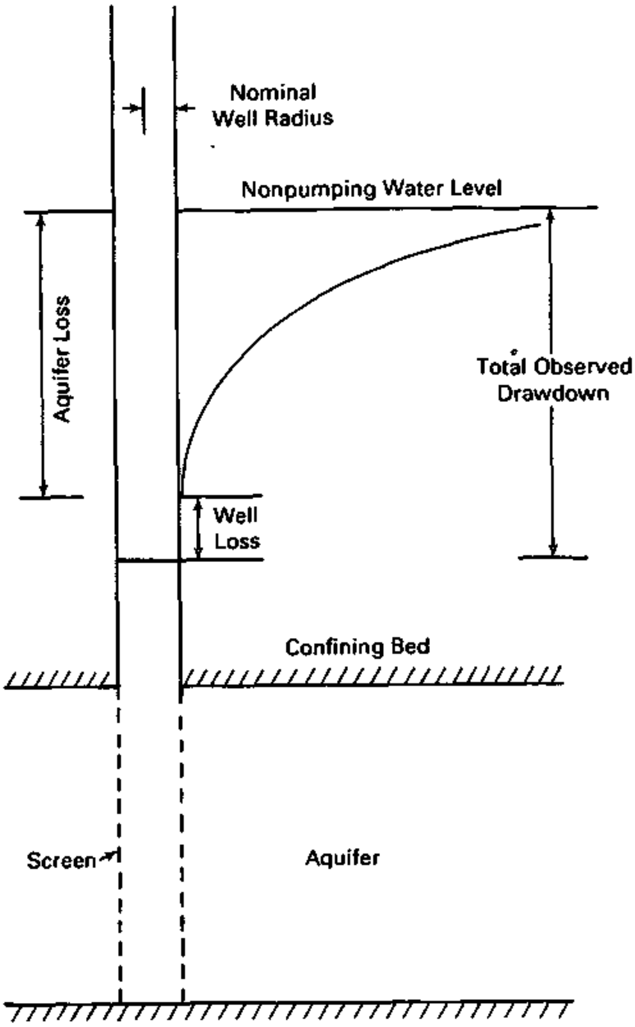

Figure 4. Components of observed drawdown

If Jacob's equation is used to express drawdown, then the coefficients B and C must be determined. Jacob proposed an algebraic method to solve for these coefficients, involving incremental values of drawdown and pumping rate. The authors, however, prefer a graphic method based on a rearrangement of equation 2:

$$s_o/Q = B + CQ \qquad (3)$$

In this method, values of sQ are plotted against values of Q on arithmetic graph paper. The slope of a line fitted to this plot is equal to C, and the y-intercept is equal to B. For consistent units in the application of this method, discharge Q is given in cubic feet per second (cfs). The well-loss coefficient is given in seconds^2/feet^5 (sec^2/ft^5). According to Walton (1962), the value of C for a properly developed and designed well is generally less than 5 secVft^5. Values of C between 5 and 10 secVft^5 indicate mild deterioration, and clogging is severe when C is greater than 10 secVft^5.

Procedure

Field data to be analyzed by the Jacob method were collected by means of a step-drawdown test, in which water is pumped during successive periods at different withdrawal rates, while water levels are monitored in the pumped well. In an ideal step test, discharge rates are selected at more or less constant fractions of the final rate. Three or more pumping periods (steps) are necessary for graphic analysis.

For the hatchery's production wells, discharge rates were monitored by a propeller-driven flow-measuring device that sends a current signal to a laptop computer. Water levels in the wells were monitored by connecting a pressure transmitter through a tee to the air line installed in each well. Like the flow meter, the pressure transmitter also sends a current that is converted to a digital signal for interpretation by the computer.

Water-level data also were collected manually with a steel tape inserted through the piezometers in the gravel packs. These data were collected as a means to correlate water levels in the pumped wells with those in the gravel-pack piezometers. Hatchery staff have historically estimated water levels in the wells by observing levels in the piezometers, assuming that they would correspond.

Pump discharge pressure was monitored by observing a pressure gage located on the discharge pipe near the pump. These data were collected as a means to calculate the total dynamic head (pumping lift plus discharge pressure) for a given pumping rate. A plot of total dynamic head versus pumping rate was made from this information and compared to the pump rating curve supplied by the pump manufacturers. (See the discussion of pumping rates under Estimates of Pumpage in the next chapter.)

Step Tests

Upon installation of the air lines in each of the production wells, step-drawdown tests were conducted in January 1990. Step tests were performed on January 11 (well 6), January 12 (well 5), January 22 (wells 7 and 8), January 23 (wells 3 and 4), and January 24 (wells 1 and 2). A summary of data from the tests is presented in table 2. Four steps were conducted in each test, and pumping rates varied from as little as 48 gpm in well 5 to 1,343 gpm in well 1. Each step lasted 30 minutes.

Results

Data from the step-drawdown tests were analyzed by the Jacob graphic method only. This analysis indicated that further evaluation by the Rorabaugh method was not warranted. The results are presented in table 3, along with those of the original tests, which were run at the time the wells were constructed.

Table 2. Summary of Step-Drawdown Test Data at Jake Wolf Fish Hatchery

Well	*Date of test*	*Pumping rates (gpm)*	*Drawdowns per step (feet)*	*Depth to water in well (feet)*	*Depth to water in piezometer (feet)*	*Discharge pressure (feet of water)*
1	1/24/90	262	2.17	50.36	50.51	161.49
		537	436	52.61	51.08	138.42
		1,055	6.73	55.03	51.79	119.96
		1343	8.98	5734	52.74	99.20
2	1/24/90	207	1.71	58.51	59.15	184.56
		354	2.41	5938	59.92	17533
		571	334	6037	60.86	170.71
		701	4.25	6135	61.85	161.49
3	1/23/90	301	4.53	52.38	53.76	17533
		496	634	54.18	55.48	168.41
		618	8.53	5636	5737	161.49
		675	11.17	59.03	60.21	149.96
4	1/23/90	152	238	54.14		138.42
		291	4.01	55.78		126.88
		410	5.92	57.69		108.43
		605	6.69	61.00		66.90
5	1/12/90	48	1.44	51.28	52.63	184.56
		486	4.11	53.96	5532	173.02
		801	6.55	56.42	58.00	161.49
		1.159	8.61	58.52	60.10	149.96
6	1/11/90	84	1.24	53.81	55.03	196.10
		795	4.63	57.18	58.64	161.49
		937	5.63	58.24	59.64	149.96
		1,301	7.34	60.02	61.42	126.88
7	1/22/90	326	1.98	52.53	53.07	198.40
		416	3.42	53.95	54.46	186.87
		650	4.92	55.47	55.92	170.72
		897	6.65	57.19	57.60	154.57
8	1/22/90	379	3.43	52.32	50.16	196.10
		554	4.58	53.43	50.38	186.87
		818	7.01	55.80	50.71	170.72
		1,075	9.74	58.55	51.17	149.96

Table 3. Results of Step Tests

Well	*Orig. 30-min. specific cap. (gpm/ft)*	*Retest 30-min. specific cap. (gpm/ft)*	*Orig. well-loss coeff. (sec^2/ft^3)*	*Retest well-loss coeff. (sec^2/ft^5)*
1	87.2	151.5	0.10	0.22
2	96.9	147.4	0.27-038	0.31
3	79.7	32.7	negl.	4.20
4	92.4	693	0.2-0.4	negl.
5	70.7	137.9	0.11	negl.
6	133.0	173.9	negl.	negl.
7	93.7	123.6	negl.	0.48
8	73.2	109.4	0.03	0.29

As seen in table 3, well-loss coefficients in wells 1-3 and 7-8 ranged from 0.22 to 4.2 secVft^5. Well losses in wells 4-6 were too small to assess the well-loss coefficients. These values represent small to moderate changes in the turbulent conditions in the wells, since they were originally tested at the time of construction. For example, well 3 originally had negligible well losses, but the recent tests suggested a substantial increase in turbulent conditions, with a well-loss coefficient of 4.2 secVft^5. Well losses during the fourth step of the well 3 test amounted to 85 percent of the observed drawdown. In wells 1, 7, and 8 the well-loss coefficients increased at least 100 percent but were still less than 0.S secVft^5. While the values of C for the most part remained well within the low well-loss criteria suggested by Walton (1962), some of the changes suggest modest to significant deterioration in well-screen flow conditions since the wells were originally tested.

On the other hand, with the exception of wells 3 and 4 (with declines of 59 and 25 percent, respectively), specific capacities actually increased from their original values by 30.7 to 95 percent One possible explanation for the apparent inconsistency between well-loss coefficients and specific capacities could be that, over the operating lives of the wells, simultaneous processes were working at odds with one another. At the well screen, conditions deteriorated somewhat (from some type of chemical or bacteriological clogging), causing small increases in well losses. Simultaneously, significant well development occurred outside the screen in the gravel pack. The overall effect was to reduce observed drawdowns. Thus, with the exception of well 3, the production wells were in generally good condition as of January 1990.

Of more concern are the water-level declines in the well field since the original tests. These declines have impacted the available yields of some wells, and a few pumps have been lowered to recover some of this yield. As will be discussed later in this report, however, much of this decline can be attributed to the drought conditions that prevailed through 1988 and 1989.

Piezometer Water Levels Versus Water Levels in Wells

Water levels observed in the gravel-pack piezometers were correlated with corresponding water levels within the wells during the step tests using LOTUS® data regression software. The regression equations generated during the correlations are presented in table 4, where DW = depth to water (feet) in the well, and DG = depth to water (feet) according to the gravel-pack piezometer. Well 4, of course, is not represented since no piezometer could be installed. These equations can be used to estimate water levels in the wells by applying them to the data routinely collected from the piezometers by the hatchery staff. If one applies these equations to the observed range of DG, a close agreement appears to exist between DG and DW for all but wells 1 and 8. This agreement suggests that overall, the methodology used to estimate DW is reasonably close to the correct value.

Table 4. Relationship between Water Levels in the Gravel Packs and in the Wells

Well	*Regression equation*	*R^2*
1	**DW = 3.48801DG - 125.236**	**0.941115**
2	**DW = 1.15799DG - 10.1795**	**0.992496**
3	**DW = 1.03672DG - 3.34377**	**0.999081**
5	**DW = 0.96733DG + 035584**	**0.999525**
6	**DW = 0.96424DG + 0.71190**	**0.998517**
7	**DW=1.04391DG-2.93471**	**0.997904**
8	**DW = 7.15846DG - 306.712**	**0.868686**

Care must be used with the equations for wells 1 and 8 because the effective range of application appears to be confined near the midrange of observed DG values, while significant errors occur at the extreme ends of the DG range.

Pump Curves

As described in the section on step-test procedures, discharge pressures were recorded during each test, along with the depth to water in the well and the discharge rate, thus establishing the relationship between the discharge rate and the total dynamic head (TDH) for each pump stage. The result is similar to the pump curves supplied by the pump manufacturers and provides a means of comparison between the two. Because there are no flow meters on the discharge lines at each well, hatchery staff use such curves, along with total dynamic head information, to estimate pump discharge. Because well 4 has no gravel-pack piezometer, pump discharge has historically been assumed to be approximately equal to the rated capacity of the pump (400 gpm).

The relationship between discharge Q and total dynamic head per pump stage for each well was derived by regression analysis, and the regression coefficients were used to construct graphs of Q versus TDH. The regression equations used to calculate Q for various values of TDH are presented in table 5.

Table 5. Observed Relationships between Discharge and Total Dynamic Head per Fwmp Stage

Well	*Regression equation*	*R^2*
1	**Q= 6,219.45-116.15TDH+0.4448TDH^2**	**0.973475**
2	**Q= 12,078-148381TDH+b.4162TDH^2**	**0.968263**
3	**Q= -57,882+1111392TDH-5.2729TDH^2**	**0.999793**
5	**Q= 5.368.52+188.224TDH-1.206TDH^2**	**0.997911**
6	**Q= 88835+72.428TDH-13644TDH^2**	**0.998338**
7	**Q= 11.216-309357TDH+2.162TDH^2**	**0.997916**
8	**Q= 2.577.4+187.4TDH-2.251TDH^2**	**0.999970**

A significant uncertainty surrounds the temporal stability of the regression equations in table 5. Inasmuch as it is not possible to determine the applicability of these pump discharge curves to historic TDH data, they should be used with caution. They do, however, suggest that discharge estimates made in the past might have been underestimated.

The regression coefficients derived from the regression analysis are shown in figures 5-11, in which the pump curves predicted by the pump manufacturers are compared with the observed relationships. The figures indicate that for all but well 1, the manufacturers' pump curves tend to predict lower discharges than those observed over most of the range of discharges used in the step tests. In well 1, the pump curve predicts higher discharges than those observed during the step tests, and the disparity increases at lower pumping rates. Since the step tests for wells 2 and 3 did not include final pumping rates as high as the rated pump capacities of those wells, the validity of the graphic relationships determined in these tests becomes uncertain over a wider range of discharges. At well 5, for instance, the two curves diverge at higher rates, while at well 7 the curves appear to have opposite shapes (convex versus concave), diverging at both ends of the range of discharges.

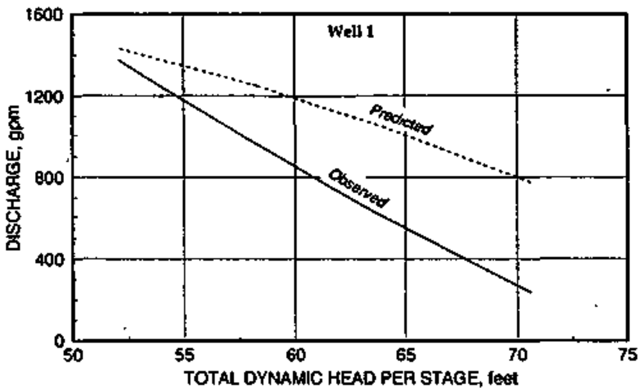

Figure 5. Discharge versus dynamic head at well 1

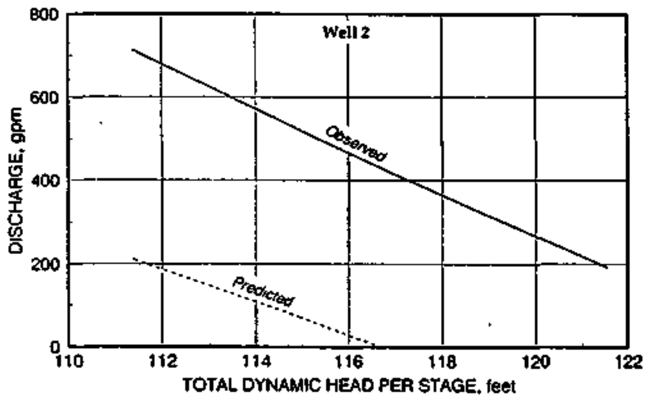

Figure 6. Discharge versus dynamic head at well 2

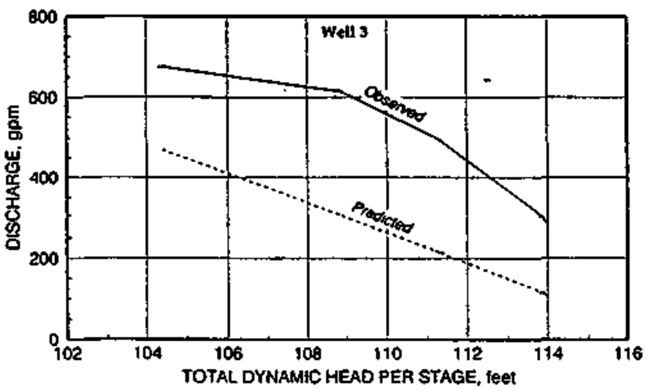

Figure 7. Discharge versus dynamic head at well 3

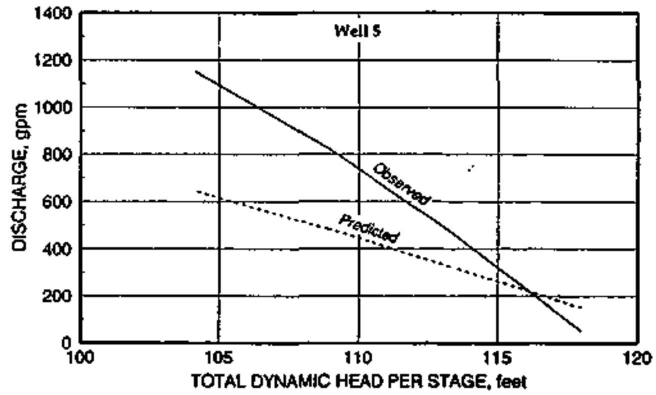

Figure 8. Discharge versus dynamic head at well 5

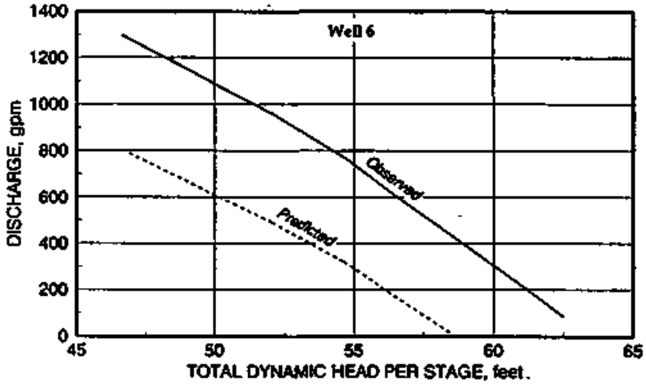

Figure 9. Discharge versus dynamic head at well 6

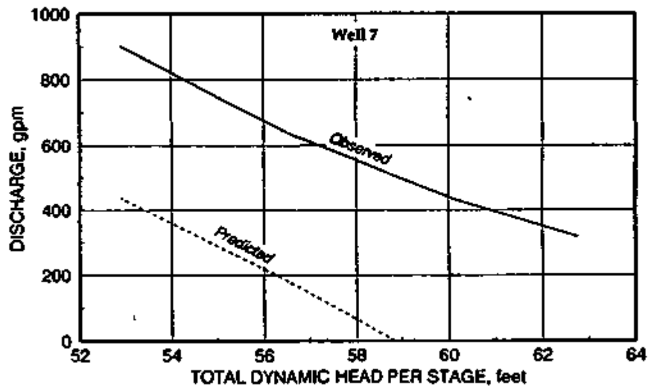

Figure 10. Discharge versus dynamic head at well 7

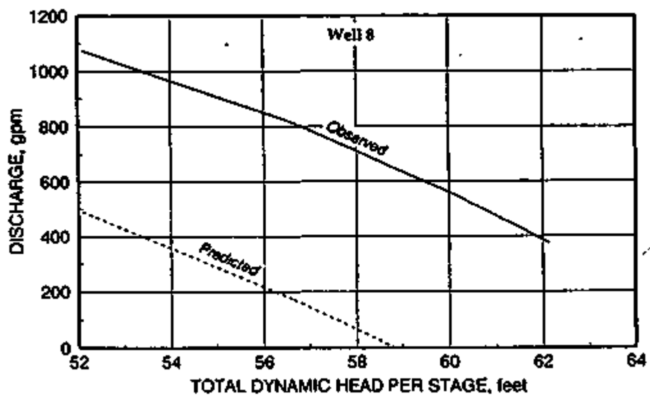

Figure 11. Discharge versus dynamic head at well 8

Conclusions

1. While the turbulent well losses within the production wells have increased by varying degrees in wells 1, 3, 7, and 8, the overall performance of the wells suggests that they are generally in good hydraulic condition. With the exception of wells 3 and 4, they have actually improved in specific capacity, suggesting that development has taken place through the gravel pack since the wells were constructed. At well 3 the specific capacity suffered a 59 percent decline, and the turbulent head losses at the well screen have increased. Well 3 therefore merits close attention in order to monitor any further deterioration. Short-term, specific-capacity tests are suggested on a monthly basis. Such monitoring would signal the possible need for rehabilitation work before serious deterioration could occur.

   Although turbulence at well 4 appears to be negligible, its specific capacity has declined by about 25 percent since construction. The hydraulic performance of this well, therefore, should also be monitored periodically as a precaution against any deterioration.

   In general, it is always prudent to monitor production wells periodically, regardless of their hydraulic condition. This is especially true of wells in which pumps have recently been lowered, since turbulent conditions may have been created by placing pump intakes opposite the screens in these wells. Monitoring of water levels inside the well screens was, of course, greatly facilitated by the installation of air lines. However, pumping levels in some wells are below the lowest reading on the air pressure gages, so water levels must be measured in the gravel-pack piezometers.

2. Since none of the wells has meters, the only manner in which discharge can be estimated is by the methodology historically employed by the hatchery staff (the use of pump-rating curves). The most critical improvement in this methodology was the installation of air lines, which make it possible to measure the depth to water in the well by subtracting the column of water indicated on the pressure gage from the length of the air line. As indicated in conclusion 1, however, pumping levels in some cases may be below the lowest readable level on the air gage.

3. The most obvious improvement that could be made in the wells is the addition of individual discharge meters. It is good management practice to record discharges from individual wells as well as from the total well field; and pumping rates are an essential part of the record keeping of a good well performance monitoring program. Finally, pumping records are necessary in any ongoing assessment of the ability of natural recharge to balance pumpage.

ASSESSMENT OF THE WELL FIELD

Introduction

One of the concerns that prompted this study was whether or not pumpage from the fish hatchery's well field was being balanced by recharge. Normally the cone of depression created by pumpage from a well or well field deepens and grows until it intersects a recharge boundary, such as a large body of water. Or it could deepen and grow until it intercepts enough natural recharge to balance the discharge. The following methods appeared to be viable options for assessing recharge to the well field of the Jake Wolf Fish Hatchery:

1. One means of assessing whether or not recharge is balancing ground-water withdrawals is by examining the historical records of pumpage and water levels. In a scenario with a continuous ground-water balance, each increase in pumpage would cause a proportionate decline in water levels, i.e., the relationship between the two variables is linear. If, however, pumpage exceeds recharge, then the relationship will deviate from linearity, and water-level declines will accelerate.
2. A second approach is to construct a potentiometric surface map from water-table measurements and solve for recharge by using a flow-net analysis. To use this procedure, information on the hydraulic properties of the aquifer is necessary.
3. Lastly, recharge can be examined by measuring it directly or estimating it in the field. Typically, recharge is derived indirectly through ground-water balance equations that incorporate the various elements of a ground-water budget This study, however, used a direct procedure for measuring recharge, which was described by Stallman (1956). The measurements were verified through the use of flow-net analyses and solving for water-balance equations.

Estimates of Pumpage

The production wells at the Jake Wolf Fish Hatchery are not equipped with discharge meters, so pumping rates cannot be measured directly at the individual wells. Since there is no master meter on the water line from the wells to the hatchery, there is also no way to measure the total pumpage from the well field. Historically, individual pumping rates at the wells have been estimated indirectly by comparing pump performance curves (graphs of pumping rates) with total dynamic head per pump stage. Pump performance curves are supplied by the pump manufacturer. Total dynamic head is the sum of the pumping lift and the discharge pressure (converted to feet of water). Discharge pressure is measured directly with a pressure gage on the discharge line at the well.

Access to the insides of the wells had not originally been possible because of construction features such as the small annular spaces between the pump columns and the casing. Until the wells were reconstructed during the study and air lines were installed, no direct means of measuring water levels was possible. Instead, water levels were estimated in all but well 4 by measuring the depth to water in a piezometer installed in the gravel pack. Well 4 had neither a piezometer in the gravel pack nor direct access to the inside of the well; thus, total dynamic head (and therefore, pumping rate) could not even be estimated. Instead, pumpage in well 4 has been assumed to be approximately equal to the rated capacity of the pump, about 400 gpm.

With the air lines installed in the wells during the study, the pumping lift portion of the total dynamic head can now be measured directly. However, pumping rate estimates may continue to rely on the pump performance curves. A trade-off is involved, however, because of the difference in precision between measurements in the well and those in the piezometer. Measurements in the piezometer can be made with steel tapes (which are precise to 0.01 foot) or with electric droplines (which are precise to 0.02 to 0.0S foot), whereas measurements in the well are routinely made with a pressure gage attached to the air line (precise to approximately one foot).

To improve the accuracy of the air line measurements, a pressure transmitter and a small computer would have to be used, as was done during the step tests. As discussed previously, the historic pumping lift estimates from piezometer measurements probably are fairly close to those obtained from a direct measurement of pumping lift in the well. As shown in the step testing, well losses at the Jake Wolf well field were relatively small, and water levels in the piezometers generally corresponded closely to water levels in the production wells.

Pumpage history at the well field from 1984 to 1990 was estimated by examining operational records furnished by the hatchery to determine pumping lifts, discharge pressure, and the number of hours of cumulative operation for each well in a given period. Pumping rates for given periods were estimated by three methods: pump performance curves, pump curves derived from the step tests, and rated pump capacities. Total pumpage for the period was the product of the pumping rate (from each of the three data sets) and the hours of operation.

Average pumping rates for a given period were determined by dividing the total pumpage by the number of days of operation for the period. The results are shown in figure 12. The pump curve in the graph was derived from the pump performance curves supplied by the pump manufacturers. The step-test curve utilized the pump curve obtained from

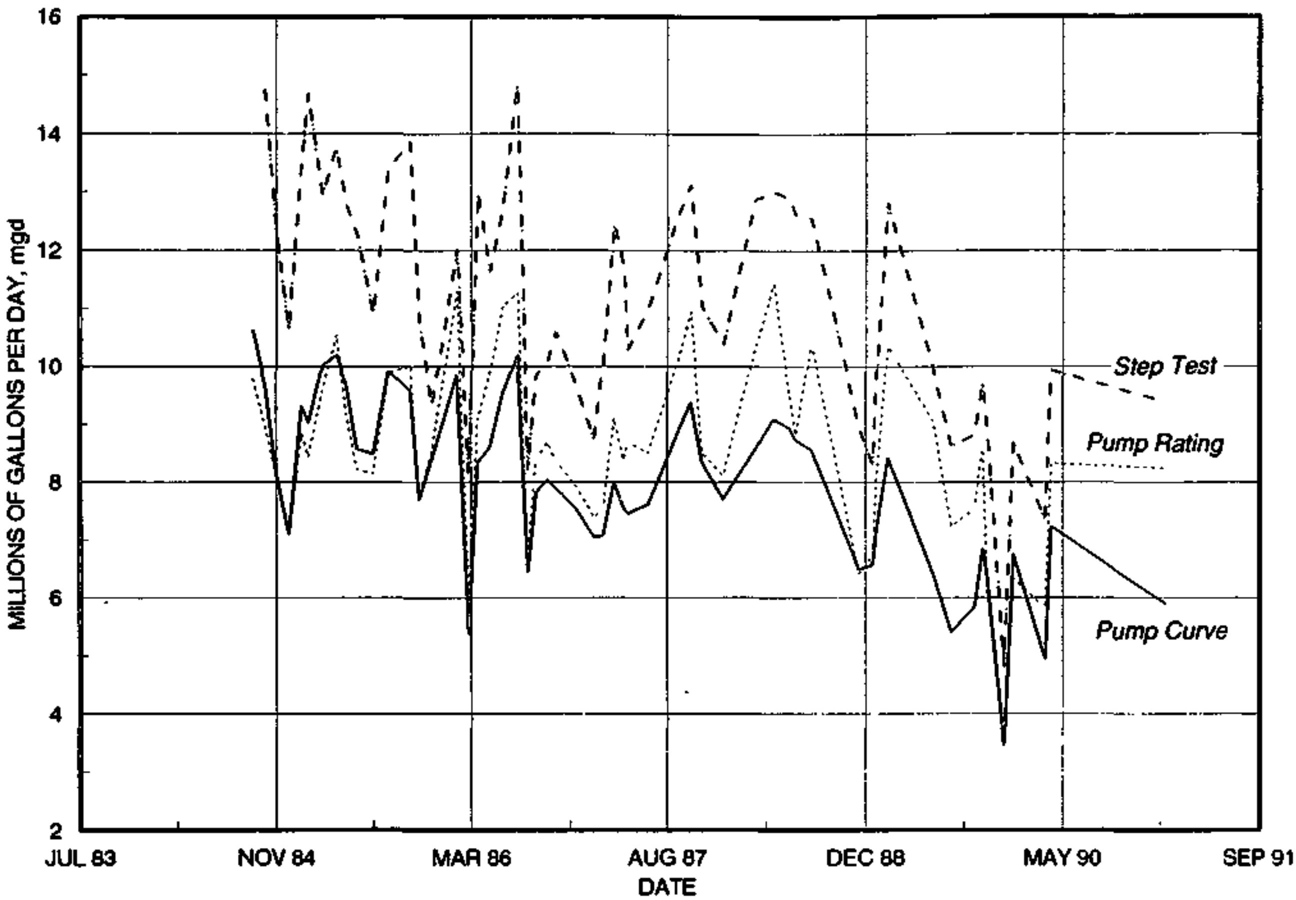

Figure 12. Monthly average pumpage according to three methods

the step-test data. The pump rating curve was produced using the rated pump capacities.

Total pumpages from the well field between 1984 and 1990 varied from 3.5 to 10.6 mgd, with an average of 7.9 mgd, as computed by the pump curve method. When calculated by use of the pump curves derived from the step tests, the pumpages ranged from 4.8 to 14.8 mgd, with an average of 11.1 mgd. If calculated by the rated pump capacities, the pumpages ranged from 3.6 to 11.4 mgd, with an average of 8.6 mgd. The pumpage curve derived from the rated pump capacities generally falls between the curves derived from the two pump curve methods and appears to lie closer to the pump performance curve than to the step-test curve.

Table 6. Observation Wells in the Vicinity of the Well Field at Jake Wolf Fish Hatchery

Well location	*Name*	*Depth (feet)*	*Measuring point elevation (ft above msl)*	*Measurement frequency*
MSN23N7W-21.1a	OW5	130	518.17	Monthly
MSN23N7W-27.3b	SER	80	513.31	Continuous
MSN23N7W-27.3h	NR	?	501.13	Continuous
MSN23N7W-27.6b	SWR	52		Disc.1988
MSN23N7W-27.6f	OW7	109	497.39	Monthly
MSN23N7W-34.4e	SR3	40	501.51	Monthly

All of the curves suggest that pumpages declined by about 2 mgd in 1989 and 1990. This could be a misleading relationship, however, since hatchery operational data indicate that the hatchery staff monitored pumpage less frequently during this period. It is possible, therefore, that average pumpages over the relatively longer periods between monitoring in 1989 and 1990 are not representative of actual conditions.

Monthly Water-Level Measurements

Monthly water-level measurements in the vicinity of the well field were begun in 1981, prior to the beginning of pumping, at four observation wells; a fifth observation well was added in 1985 (figure 2 and table 6). Three of the wells were equipped with continuous water-level recorders, and two were measured manually on a monthly basis. Measurements at the southwest observation well MSN23N7W-27.6b (SWR) were discontinued in 1988 after water levels dropped below the bottom of the well. In addition, water levels have been monitored monthly in well SR3, which was constructed about a mile south of the well field for an experimental study in 1982.

Water-level hydrographs for the periods of record are shown for two of the four close-in observation wells, OW5 and OW7, that are still in service (figure 13) and for observation well SR3 (figure 14). The hydrographs contrast water-level responses close to the production wells and at a greater distance. Water levels rose in 1982 and 1983 above their initial levels due to the above-normal precipitation in those years. The subsequent steep declines in wells OW5 and OW7 were caused by the initiation of ground-water withdrawals in 1984. Seasonal cycles are superposed on the gradual decline of water levels, especially in SR3. The hydrograph for SR3 is much smoother than those for the close-in observation wells, because it is far enough away to avoid some of the cyclic effects of well field withdrawals.

Three features are noteworthy in the hydrograph for well SR3 (figure 14). The first important feature is the apparent trend toward a more or less stable water level (approximate elevation: 472 feet) just prior to August 1987. Pumpage records illustrated in figure 12 indicate that annual withdrawals were fairly consistent then (between 8 and 10 mgd), while water levels in the vicinity of the well field also appear to have been close to an equilibrium with pumpage. This suggests that recharge to the aquifer was sufficient to balance withdrawals on an annual basis.

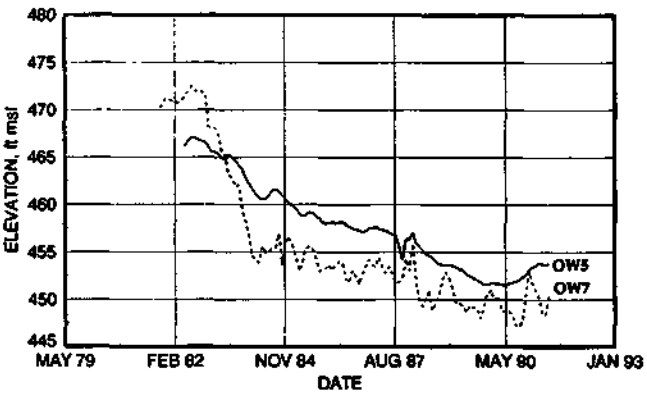

Figure 13. Historic water levels in observation wells OW5 and OW7 near the well field

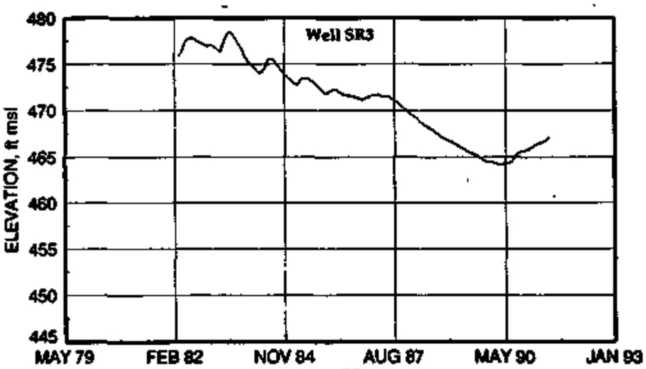

Figure 14. Historic water levels in observation well SR3 at the field site

The second feature of the hydrograph for well SR3 is the uninterrupted decline in levels beginning in late summci 1987 and continuing until just prior to spring 1990. This decline, of course, was the result of the severe drought that began in 1987 and lasted through 1989. Water levels at SR3 declined approximately 8 feet during this drought period.

The third feature is the water-level recovery at the end of the hydrograph. With the end of the drought in 1990, water levels began to recover in early spring; the recovery continued through the rest of 1990 and was still evident in spring 1991. The remarkable characteristic of this recovery is its continuity through the summer and fall months of 1990, when water levels would normally exhibit seasonal declines. This could be due to the fact that annual precipitation at Havana was nearly 16 inches above normal and was fairly well distributed through the year. If precipitation remains at least near normal for the next few years and if pumpage remains at present rates, water levels may return to the near-equilibrium levels that trends indicated prior to the drought. As of March 1991, water levels at SR3 had recovered about 30 percent of the nearly 8 feet of decline that occurred as a result of the drought.

Water Levels Versus Pumpage

An attempt was made to identify any significant correlation between average well field pumpage and water levels. Pumpage was determined as described above and correlated with monthly water-level elevations recorded in the five observation wells. A correlation was attempted for the north observation well, but little correlation is apparent in the plot of monthly water levels and monthly average total pumpage (figure 15). Similar results were observed in the other four observation wells close to the well field (those in Sections 21 and 27). Correlation coefficients were poor, ranging from 0.0002 to 0.0259.

The most probable explanations for the lack of correlation among the close-in observation wells are the uncer-

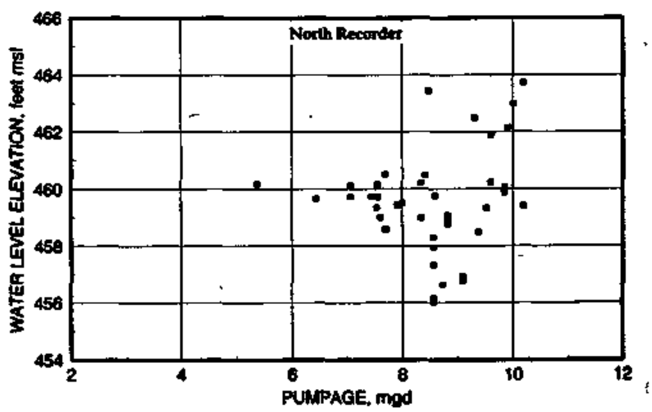

Figure 15. Monthly water levels and average total pumpage for the north observation well

tainties associated with estimating monthly pumpages and the hatchery's policy of rotating pumpage among the eight production wells. For a given total well field pumpage, the effects on water levels in the close-in observation wells depend on which of the eight production wells were being pumped at the time of the water-level measurement Apparently the observation wells were located too close to the well field, so the results of the plots were affected by the particular distribution of pumping wells at any given moment.

Better results were obtained when pumpages were correlated with water levels in observation well SR3, located one mile south of the well field (figure 16). A correlation coefficient of 0.5764 was obtained from a regression of these variables. Thus, when taken as a whole, about 58 percent of the variation in water levels can be attributed to pumpage. Since figure 16 is made up of drought-period and normal-period data, the data points appear to separate into two groups, each with its own slope. Individual correlations of water-level data from drought and normal periods with pumpage, however, do not improve the correlation coefficient Thus, the relationship between pumpage and the entire monthly water-level record at SR3 suggests that a modest linear correlation exists and that recharge has kept up with pumpage over a significant portion of the period of record.

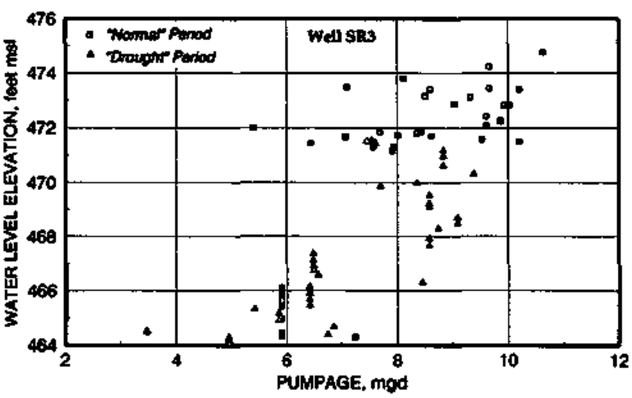

Figure 16. Monthly water levels and average total pumpage tor observation well SR3

Stallman Method of Estimating Recharge

Stallman (1956) introduced a method for estimating the natural recharge rate for aquifers that uses numerical techniques to analyze water-level information gathered in a detailed data-collection program. The method combines the features of a simple data-collection program over a significant study area with the details of a field-test approach. The Stallman method examines ground-water flow in a two-dimensional field under nonsteady conditions, as shown in figure 17. The basic components of the flow field are:

1. Accretion (recharge) to the water table, W, is the net rate at which water is gained or lost through the aquifer surface in response to external forces.
2. Changes in the volume of water stored in the aquifer augment or diminish the rate of flow at each point by an amount equal to S(əh/ət), where S is the storage coefficient The direction of this flow component is parallel to the direction of accretion, and the sum of the two components equals the net rate at which water is added to or lost from the zone of saturation.
3. The net amount of water movement $F_x + F_y$, through the zone of saturation is related to aquifer transmissivity, T (and its variation from place to place within the aquifer), and to the configuration of the potenuometric surface.

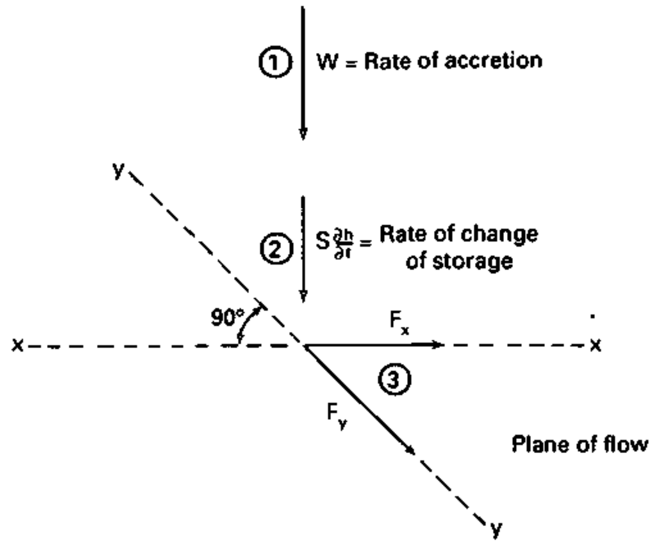

Figure 17. Components ot flow in a two-dimensional flow field

For a homogeneous aquifer, the relationship among the three flow components for any point in the flow field is:

$$T\left(\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}\right)=S\frac{\partial h}{\partial t}-W \qquad (4)$$

or

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}=\left(S\frac{\partial h}{\partial t}-W\right)/T \qquad (5)$$

Using equation 5, the transmissivity and storage coefficient of the aquifer can be evaluated at any point in space and time by measuring the partial derivatives. This is not possible in a practical sense, however. In this case, estimates of head differentials can be obtained by applying the methods of Southwell (1948). By this technique, head differentials that apply to finite areas can be estimated from water levels measured at widely spaced points. Assume, as shown in figure 18, that the aquifer is subdivided into squares of equal area, a^2. This subdivision can be thought of as an approximation of the area of a corresponding mathematical grid, dxdy.

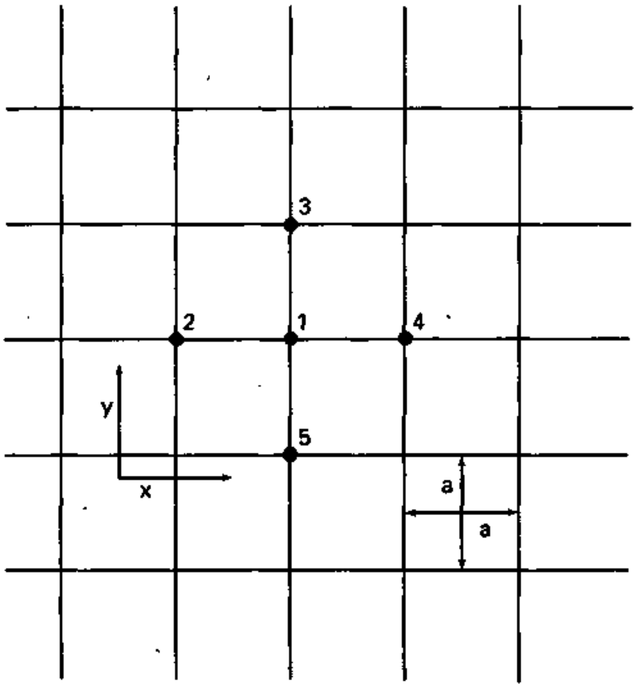

Figure 18. Finite-difference grid on a flow field

The nodes formed by the intersection of the grid lines in figure 18 are presumably points at which the head, h, is known. At node 1, the second derivatives of h can be approximated by:

$$\frac{\partial^2 h}{\partial x^2} = \frac{h_2 + h_4 - 2h_1}{a^2} \tag{6}$$

and

$$\frac{\partial^2 h}{\partial y^2} = \frac{h_3 + h_5 - 2h_1}{a^2} \tag{7}$$

Also,

$$\frac{\partial h}{\partial t} = \frac{h_{1,t} - h_{1,t+\Delta t}}{\Delta t} \tag{8}$$

or

$$\frac{\Delta h}{\Delta t} = \frac{\Delta h_1}{\Delta t} \tag{9}$$

By substituting the approximations in equation S, the finite-difference form of the equation is produced, which can be used for numerical analysis of water-level data:

$$h_2 + h_3 + h_4 + h_5 - 4h_1 = \left(S\frac{\Delta h_1}{\Delta t} - W\right)\frac{a^2}{T} \tag{10}$$

If the values of T and S are known, equation 10 affords a means of computing the rate of natural recharge, W, using water-level observations at five wells located in a five-spot pattern, as shown in figure 18. From aquifer tests conducted at the time the production wells were constructed, the average values of T and S were determined to be 356,800 gallons per day per foot (gpd/ft) or 47,700 feet per day (ft/day) and 0.13, respectively. Each of the flow components may be expressed as a volume flux per surface area of the aquifer in feet per day, gallons per day per square mile (gpd/sq mi), or as an equivalent depth of water per year (inches).

If the nodal spacing is not uniform, that is, the distances from the center node to the surrounding nodes are not equal, equation 10 can be modified to a more general case in a manner suggested by Karplus (19S8). Then equation 10 becomes:

$$\frac{\frac{h_4 - h_1}{d_4} + \frac{h_2 - h_1}{d_2}}{0.5(d_2 + d_4)} + \frac{\frac{h_3 - h_1}{d_3} + \frac{h_5 - h_1}{d_5}}{0.5(d_3 + d_5)} = \frac{1}{T}\left(S\frac{\Delta h_1}{\Delta t} - W\right) \tag{11}$$

where d_2 d_3, d_4, and d_5 are the distances from the center well to wells 2, 3, 4, and 5, respectively.

Observation Well Network

In order to implement the Stallman method for estimating natural recharge rates in the study area, a network of eleven observation wells was constructed during the summer of 1989. The wells were constructed in a grid network spaced approximately one-half mile apart along the main north-south road, Cactus Drive, in Sand Ridge State Forest (see figure 2). The well array was designed so that it consisted of three overlapping five-spot patterns, which will be referred to as the "south," "middle," and "north" five-spot patterns. The use of three arrays made it possible to identify the areal variability of recharge from south to north, which is of particular interest because of the proximity of the hatchery well field at the north end of the study area.

The wells were constructed of 2-inch-diameter casing and were finished at depths approximately 10 to 15 feet below the point at which the water table was encountered. Wells SR11, 12, 13, 14, 15, and 16 were driven with a cable-tool rig outfitted with a falling weight. They consisted of a 4-foot drive point and galvanized drive casing. Wells SR17, 18, 19, and 20 were constructed with the Water Survey auger rig and consisted of PVC plastic screens (5-foot lengths) and casing to a point approximately 2 feet below grade. A 5-foot length of threaded galvanized pipe was attached to the top of the PVC pipe. Well SR3 was constructed for use in an earlier study and was made of galvanized pipe and screen.

Table 7 shows the relationship of the wells in the three five-spot patterns and the distances (nodal spacings) between the center well and its four neighbors in each five-spot.

Measuring-point elevations of the wells in the five-spot patterns and in all of the other observation wells in the study (along with hatchery production wells) were surveyed by a registered land surveyor. Weekly measurements of water levels in the south five-spot commenced on July 25, 1989, and in the middle and north arrays on September 22, 1989. Weekly measurements continued until November 2, 1990, and were resumed briefly in March 1991.

Table 7. Wells Used in Five-spot Patterns
to Estimate Recharge

Well	*Spot number*	*Distance to center well (feet)*
	South five-spot	
SR13	**1**	**Center well**
SR12	**2**	**2,372**
SR15	**3**	**2,916**
SR14	**4**	**2,817**
SR11	**5**	**2,726**
	Middle five-spot	
SR15	**1**	**Center well**
SR3	**2**	**2,400**
SR18	**3**	**2,595**
SR16	**4**	**2,582**
SR13	**5**	**2,916**
	North five-spot	
SR18	**1**	**Center well**
SR17	**2**	**2,788**
SR20	**3**	**2,685**
SR19	**4**	**2,564**
SR15	**5**	**2,595**

Results

Values of recharge were calculated for each of the three five-spot patterns using equation 11 and the water-level data. As an example of one such calculation, the water-level elevations in the south five-spot will be examined for June 8, 1990. In order to approximate the rate of change of water levels, the elevations of the previous week (June 1) are subtracted from the levels of the following week (June 15). The selected data for these weeks are shown in table 8.

Table 8. Selected 1990 Water Elevations
for South Five-Spot Array

Well	*Spot number*	*June 1*	*June 8*	*June 15*
SR13	**1**	**469.39**	**469.43**	**469.49**
SR12	**2**	**467.35**	**467.38**	**467.43**
SR15	**3**	**467.42**	**467.45**	**467.51**
SR14	**4**	**471.58**	**471.63**	**471.71**
SR11	**5**	**471.00**	**471.04**	**471.11**

By applying the water-level elevations for June 8 to equation 11 and the distances between wells as given in table 7, the terms on the left-hand side of the equation are calculated to be -6.34 x 10^{-8} ft^{-1}. Estimates of the rate of change of water levels at the center well (SR13) would approximately equal the differences in elevations at that well observed on June 1 and June 15, divided by the number of days between these dates, or 469.49 minus 469.39 divided by 14. However, the rate of water-level changes in the individual wells of the south five-spot was not uniform, and the rate of change calculated at the center well alone might not be as representative as desirable. Stallman (1956) suggested that a weighted average could be taken among the five wells as follows:

$$\left(\frac{\Delta h_1}{\Delta t}\right)_{wt\ av} = \frac{\Delta h_2+\Delta h_3+\Delta h_4+\Delta h_5+4\Delta h_1}{8\Delta t} \qquad (12)$$

Using the weighted average method, the rate of change of water levels was computed to be 0.00723 ft/day. By substituting into equation 11 the calculated value of the rate of water-level change, along with the average values of T and S (47,700 ft/day and 0.13, respectively), the rate of recharge for the period around June 8, 1990, was computed to be 0.00396 fl/day or 826,000 gpd/sq mi, which is equivalent to 17.3 inches per year.

Figures 19-21 show calculated recharge values for the south, middle, and north five-spot arrays, respectively. The figures show the total recharge, along with the two components of the calculation for the 12-month period November 1989 to October 1990. The portion of recharge attributed to the storage coefficient multiplied by the rate of change of water levels is labeled "storage." The portion calculated from the finite-difference form of the partial derivatives of head with respect to x and y is labeled "gradient." Negative values of the storage portion indicate periods of water-level decline. Calculated recharge at the south five-spot ranged from 471,000 to 1,358,000 gpd/sq mi (9.9 to 28.5 inches) and averaged 761,000 gpd/sq mi (16.0 inches). At the middle five-spot, recharge ranged from 2,852,000 to 4,096,000 gpd/sq mi (59.9 and 86 inches) and averaged 3,808,000 gpd/sq mi (80 inches). Recharge at the north five-spot ranged from 6,972,00 to 8,057,000 gpd/sq mi (146 to 169 inches) and averaged 7,375,000 gpd/sq mi (155 inches).

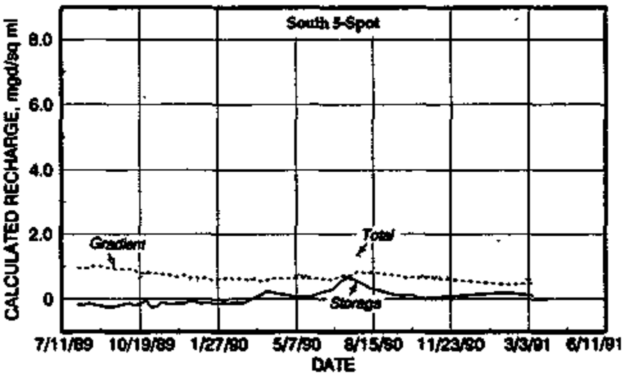

Figure 19. Recharge at the south five-spot

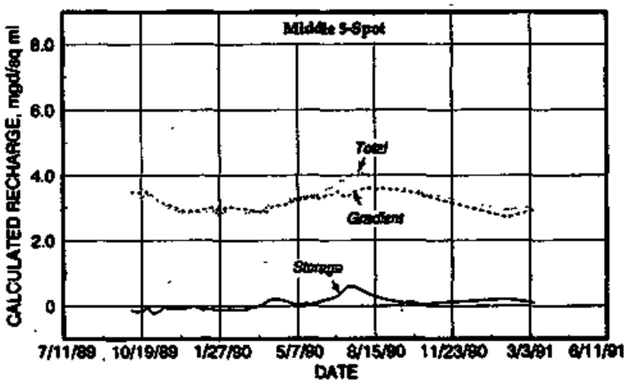

Figure 20. Recharge at the mddle five-spot

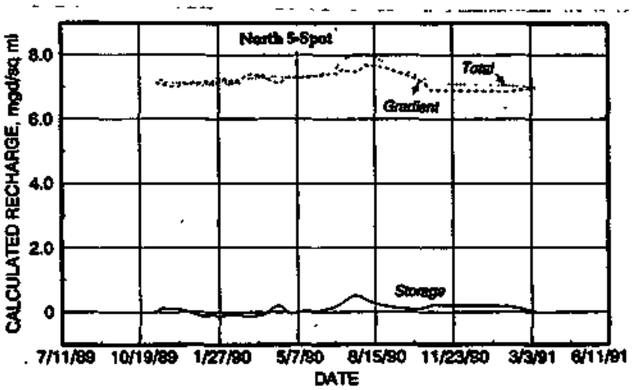

Figure 21. Recharge at the north five-spot

Several characteristics of the Stallman calculations were noteworthy. One obvious feature was the relative importance of the storage and gradient portions of the recharge total. It is clear that the gradient calculations produced numbers considerably larger than did the storage calculations; this relationship became more pronounced from south to north.

A second notable feature of the recharge calculations was the large disparity from south to north between total recharge amounts: values were about ten times as high at the north five-spot as at the south five-spot Since the flow gradient was found to be the dominant factor in the recharge calculation, the disparity in total recharge values suggests that the large gradients in the vicinity of the well field, which is north of the five-spot array, were overwhelming the calculation itself.

A third characteristic of the results was the unexpectedly high values that the recharge calculation produced. As shown above, the average recharge values at the south, middle, and north well arrays were 761,000, 3,808,000, and 7,375,000 gpd/sq mi (16, 80, and 155 inches), respectively. Obviously, recharge rates cannot exceed precipitation. Walker et al. (1965) did flow-net analyses of the potentiometric surface in the Mason-Tazewell Counties area and calculated recharge rates at 258,000 to 500,000 gpd/sq mi (5.4 to 10.5 inches). These values agree with results in similar environments elsewhere in Illinois. The significant difference between the Walker et al. results and those found in this study also suggests that the Stallman calculations, under the prevailing high-gradient field conditions, were artificially inflated.

Figures 19-21 show the most compelling evidence of the need to adjust the raw numbers produced by the Stallman calculations to account for local conditions. The graphs show that total recharge was a positive number throughout the entire period August 1989 through March 1991. Yet during the first eight months of this period, water levels were *declining* because of the effects of the drought As a result, the storage portion of the Stallman calculation was negative; on the other hand, because the gradient portion was much larger, the net result was positive. Since it is unlikely that recharge was actually occurring during the drought period, the results suggest the need for a base value for the gradient portion (and therefore for recharge) at each of the three five-spots. This base level for recharge could be an artifact of its position in the regional flow field, and it could have existed even during periods when no recharge was occurring.

Based on the above evidence, adjustments were made to the recharge calculations for each of the three five-spots. First it was assumed that recharge would be virtually zero during periods when water-level changes were negative (falling water levels). This assumption is compatible with the conventional definition of recharge, i.e., that recharge is the addition of water to the saturated zone, and that such recharge should cause water levels to rise. Next figures 19-21 were examined, along with the numerical data, to determine whether or not recharge values appeared to more or less stabilize around "base" values at each five-spot during non-recharge periods. Finally, these base values were subtracted from the totals, and the adjusted values of recharge were identified as values of "effective" recharge. The base values determined for the south, middle, and north five-spot recharge data were 480,000, 2,900,000, and 6,900,000 gpd/sq mi (10.1, 60.9, and 145 inches), respectively. The resultant values of effective recharge are plotted in figure 22.

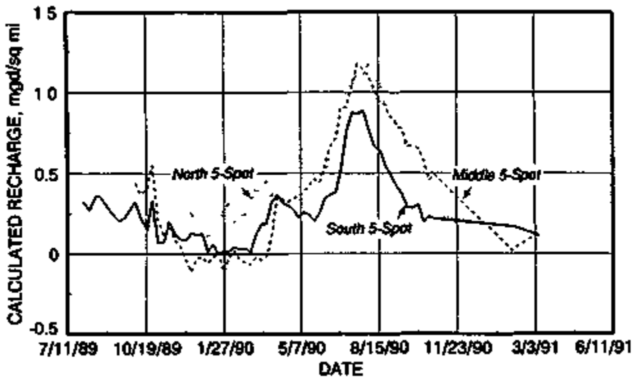

Figure 22. Effective recharge according to the Stallman method

The effective recharge rates at the three five-spots are thus much closer in value to one another than they were prior to the adjustments. Average effective recharge rates for the south, middle, and north five-spots for the period November 1989 to October 1990 were 292,000, 400,000, and 507,000 gpd/sq mi (6.1, 8.4, and 10.6 inches), respectively. These rates are in good agreement with the values obtained by Walker et al. (1965).

Flow-Net Analysis Method to Estimate Recharge

In addition to the data collected weekly at the network of 11 wells used in the three five-spot patterns, water levels were also monitored monthly at the five observation wells described earlier, as well as in three new wells constructed during the study (SR21, 22, and 23). At the same time, water levels were also monitored at each of the production wells. The resultant data set was sufficient to construct monthly potentiometric surface maps, using LOTUS® files and SURFER® contouring software (Lotus Development Corp. and Golden Software, Inc., respectively). To represent the year 1990, maps were produced for the period February 2, 1990, to January 4,1991. An example of one such map, for December 7, 1990, is shown in figure 23.

Conventional flow-net analysis (Cedergren, 1977) was employed to determine recharge rates for each map. Three flow channels were drawn, roughly corresponding to the

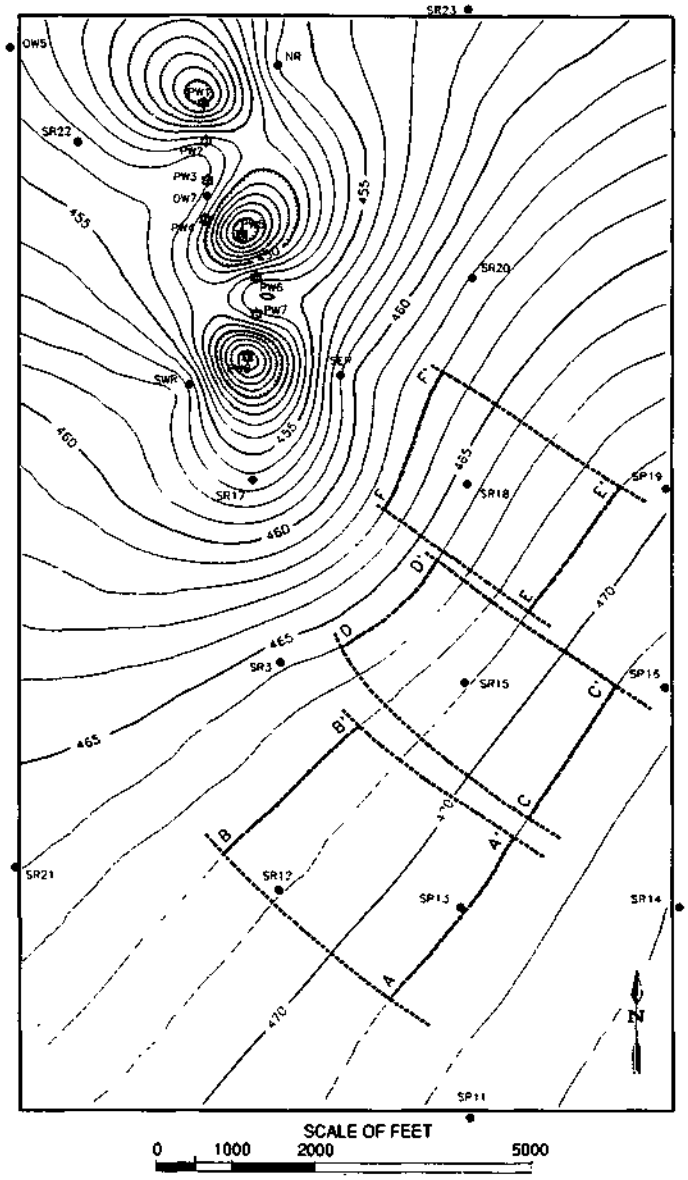

Figure 23. Flow-net analysis of the potentiometric surface, December 7,1990

areas covered by the three five-spot patterns. Ground-water flow gradients were measured on the maps at the upstream and downstream flow cross sections, and Darcy's law (see Todd, 1980) was solved at both cross sections. The rate of change of storage over the area of the flow channel was determined by multiplying the rate of change of water levels over time (taken from the Stallman method data) by the average water-table storage coefficient for the well field, which was determined to be 0.13. The difference between the downstream and upstream cross-sectional flows, plus the change in storage, was assumed to be the recharge rate for the area of the flow channel. The recharge rate per square mile was calculated by dividing the recharge over the flow channel by the area of the flow channel.

To test the sensitivity of the flow-net analysis to the aquifer transmissivity, Darcy's law was calculated for the upstream and downstream cross sections, using both uniform and variable values of transmissivity. The uniform value used was the average value employed in the Stallman method, 356,800 gpd/ft. For the variable-T approach, an average value of hydraulic conductivity derived from the well field aquifer tests was multiplied by the estimated saturated thickness of the aquifer at each cross section. The average recharge values for the variable-T calculations were found to be lower than those for the uniform-T approach at the south, middle, and north flow channels by 17, 4.S, and 6.8 percent, respectively.

The results of the flow-net analyses were similar to those of the Stallman method: 1) the calculated recharge increased from south to north; 2) the portion of the total due to the difference in flow between cross sections was generally an order of magnitude larger than that due to storage changes; and 3) the results were positive for each of the 12 months of data. The overall average recharge rates (uniform-T and variable-T) calculated at the south, middle, and north flow channels were determined to be 1,748,000, 4,298,000, and 5,804,000 gpd/sq mi (36.7, 90.3, and 122 inches), respectively.

Since the inordinately large values of recharge were obviously an artifact of gradients, the sensitivity of the flow-net analysis method was tested against the contouring methodology. Potentiometric surface maps were constructed with the SURFER® contouring software and again manually. The manual method involved contouring the data by assuming curvilinear (logarithmic) interpolations, rather than linear. Interpolations were of necessity approximate, since the manual countour drawing technique is subjective. For the sake of brevity, only one flow channel was constructed, centered on the middle five-spot array. The estimated recharge rates based on the flow-net analyses are shown below.

Date	*Recharge (gpd/sq mi)*
2/2/90	0
3/2/90	0
4/6/90	626,000
5/4/90	1,000,000
6/1/90	1,250,000
7/6/90	1,130,000
8/3/90	1,675,000
9/7/90	615,000
10/5/90	1,100,000
11/2/90	1,100,000
12/7/90	848,000
1/4/91	920,000
Average =	855,000

The flow-net analysis based on manually constructed potentiometric surface maps produced results closer to those obtained with the modified Stallman method: recharge rates were established as zero for the early months of 1990, when water levels were still in recession. Moreover, the average value for the year, 855,000 gpd/sq mi (18.0 inches), is closer to the values derived by Walker et al. (1965).

Water-Balance Method to Estimate Recharge

For further insight into natural recharge in the study area and its measurement, a simple water balance was calculated for 1990, using precipitation data from Havana and reasonable assumptions about field conditions in the area. The water balance described in this section was performed by Jean Bowman of the Illinois State Water Survey.

A simple water balance for the study area shows that maximum recharge for 1990 would have been approximately 20 inches. The water-balance computations are shown in table 9. Potential evapotranspiration was estimated using the Blaney-Criddle equation (Blaney and Criddle, 1950), along with crop coefficients for a moderately deep-rooted plant system such as corn (Dunne and Leopold, 1978). Potential evapotranspiration is defined as the amount of water lost naturally from a system through evaporation and transpiration when soil moisture is not limiting. Field capacity for soil moisture was assumed to be 6 inches in the upper 60 inches. The change in soil moisture was estimated using the accumulated potential water-loss values derived from a Thornthwaite and Mather linear equation for soil moisture (Thornthwaite and Mather, 1957). The actual evapotranspiration numbers represent the amount of moisture actually available to the system from precipitation plus contributions from the moisture in the root zone. Since the two combined are often less than potential evapotranspiration, the difference between potential and actual evapotranspiration represents a moisture deficit in the upper 60 inches. This condition most commonly occurs in Illinois during the summer months.

Precipitation in 1990 totaled approximately 52 inches, about 16 inches above normal, exceeding potential evapotranspiration by approximately 22 inches. Because of the high rainfall amounts, potential evapotranspiration exceeded precipitation by only very small amounts in the months of July, August, and September. The small discrepancies resulted in lower than normal accumulated potential water losses, which in turn led to very small soil-moisture deficits, totaling only 1.58 inches for the entire year. The difference between precipitation and potential evapotranspiration, minus the seasonal soil-moisture deficit, amounted to 19.76 inches (equivalent to 941,000 gpd/sq mi). That relatively large annual volume of surplus moisture is consistent with the high recharge numbers estimated for the study area for 1990.

For purposes of comparison, similar water-balance computations are shown for the study area for 1989 and 1988 (tables 10 and 11). The record drought year of 1988

Table 9. Water-Balance Computations for 1990
(inches)

Month	*Precipitation*	*Potential evapotran-spiration (PET)*	*Precipitation minus PET*	*Accumulated potential water loss*	*Soil moisture*	*Change in soil moisture*	*Actual evapotran-spiration*	*Soil moisture deficit*
Jan.	2.06	0.74	132		6.0	0	0.74	
Feb.	4.77	0.67	4.10		6.0	0	0.67	
Mar.	337	0.73	2.64		6.0	0	0.73	
Apr.	1.84	1.14	0.70		6.0	0	1.14	
May	5.98	2.48	3.50		6.0	0	2.48	
Jun.	9.41	6.68	2.73		6.0	0	6.68	
Jul.	6.40	7.93	-1.53	1.53	5.6	-0.4	6.90	1.03
Aug.	3.68	6.44	-2.76	4.29	3.0	-2.6	634	0.10
Sep.	1.15	2.22	-1.07	536	2.4	-0.6	1.77	0.45
Oct	3.29	1.08	2.21		4.7	2.3	1.08	
Nov.	5.00	0.60	4.48		6.0	4.5	0.60	
Dec.	5.10	0	5.10		6.0	5.2	0	
Totals	52.05	30.71						1.58

Note: Total precipitation (52.05) minus total PET (30.71) minus seasonal soil-moisture deficit (1.58) equals 19.76 inches of maximum recharge.

Table 10. Water-Balance Computations for 1989
(inches)

Month	*Precipitation*	*Potential evapotran-spiration (PET)*	*Precipitation minus PET*	*Accumulated potential water loss*	*Soil moisture*	*Change in soil moisture*	*Actual evapotran-spiration*	*Soil-moisture deficit*
Jan.	1.10	0.45	0.65		6.0	0	0.45	
Feb.	1.40	0	1.40		6.0	0	0	
Mar.	1.03	0.55	0.48		6.0	0	0.55	
Apr.	4.57	1.14	3.43		6.0	0	1.14	
May	2.95	2.51	0.44		6.0	0	2.51	
Jun.	0.99	6.42	-5.43	5.43	2.4	-3.6	4.59	1.83
Jul.	1.75	8.50	-6.75	12.18	0.8	-1.6	3.35	5.15
Aug.	3.79	6.52	-2.73	14.91	0.6	-0.2	3.99	2.53
Sep.	2.39	1.84	0.55		1.2	0.6	1.84	
Oct	1.73	1.29	0.44		1.6	0.4	1.29	
Nov.	0.80	0.45	0.35		2.0	0.4	0.45	
Dec.	0.70	0	0,70		2.7	0.7	0	
Totals	23.20	29.67						9.51

Note: Total precipitation (23.20) minus total PET (29.67) minus seasonal soil-moisture deficit (9.51) equals a water deficit of 15.98.

Table 11. Water-Balance Computations for 1988
(inches)

Month	*Precipitation*	*Potential evapotran- spiral ion (PET)*	*Precipitation minus PET*	*Accumulated potential water loss*	*Soil moisture*	*Change in soil moisture*	*Actual evapotran- spiration*	*Soil- moisture deficit*
Jan.	2.27	0	2.27		6.0	0	0	
Feb.	1.36	0	1.36		6.0	0	0	
Mar.	2.47	0.57	1.90		6.0	0	0.57	
Apr.	1.17	1.20	-0.03	0.03	6.0	0	1.20	
May	1.28	3.22	-1.94	1.97	5.6	-0.4	1.68	1.54
Jun.	0.72	7.09	-6.37	8.34	1.4	-4.2	4.92	2.17
Jul.	1.05	8.84	-7.79	16.13	0.4	-1.0	2.05	6.79
Aug.	2.70	7.70	-5.00	21.13	0.2	-0.2	2.90	4.8
Sep.	1.15	2.18	-1.03	22.16	0.1	-0.1	1.25	0.93
Oct	1.64	0.95	0.69		0.8	0.7	0.95	
Nov.	4.89	0.50	4.39		5.2	4.4	0.50	
Dec.	2.81	0	2.81	.	8.0	2.8	0	
Totals	23.51	32.26						16.23

Note: Total precipitation (23.51) minus total PET (32.26) minus seasonal soil-moisture deficit (16.23) equals a water deficit of 24.98.

had less than 24 inches of precipitation, with more than 32 inches of potential evapotranspiration. Soil-moisture deficits in 1988 were estimated at more than 16 inches. Observations of irrigation operations revealed that many irrigation farmers in the area applied more than 20 inches of irrigation water in 1988. The difference between precipitation and potential evapotranspiration, minus the soil-moisture deficit, indicates that the potential for recharge from surplus soil moisture was nil. Steadily falling water levels observed in the study area in 1988 are consistent with these water-balance conclusions.

Although 1989 was also a dry year, it was not as hot as the previous year. Cooler temperatures resulted in lower rates of potential evapotranspiration, which in turn resulted in lower seasonal soil-moisture deficits. The 1989 soil-moisture deficit totaled 9.51 inches, which is within a normal range for the study area. Lower than average precipitation in 1989, combined with the continued effects of the 1988 drought, are consistent with the continued falling water levels observed throughout 1989.

Area of Diversion

The area of diversion is the area of the water table or potentiometric surface from which ground water moves toward a point, a line, or an area of ground-water discharge. It is also known as a "flow field" or "capture zone." In the study area, ground water moves from the area of diversion toward the well field. The area of diversion developed after the well field was put into operation, when the drawdown caused by pumping lowered the water table. Areas of diversion stabilize when recharge balances discharge. Since recharge is distributed unevenly during the year and from year to year, while pumpage may be constant, areas of diversion expand or contract with these changes.

In a flat water-table aquifer with uniform hydraulic properties, the shape of the area of diversion would be approximately circular, corresponding to the layout of wells in the well field. But in an aquifer with a sloping water table, which is indicative of a regional flow system, the shape and boundary of the area of diversion may change from a circle to a curve whose open end points upslope from the well field. Moving upslope from the center of pumpage, the flow boundary asymptotically approaches a straight line on both sides of the flow field.

Downslope from the well field, the boundary extends to a stagnation point, beyond which flow moves away from the area of diversion. Todd (1980) described the shape of the area of diversion for a confined aquifer with a sloping potentiometric surface and presented formulas to estimate the distance to the stagnation point and the limiting size of the flow cross section. Figure 24 shows the approximate shape of the flow field around a pumping well in a confined aquifer. The same general shape applies to a water-table aquifer, and the formulas for the confined case are good approximations of the water-table situation in the study area. Todd pointed out that for a well field pumping for an infinite time, the boundary would extend upslope to the limit of the aquifer.

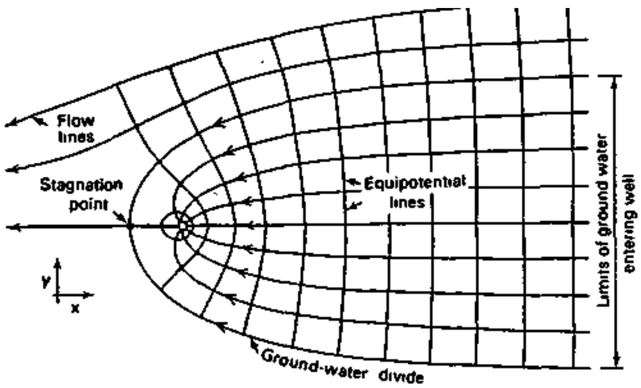

Figure 24.. Flow field around a pumping well in a confined aquifer with a sloping potentiometric surface (after Todd, 1980)

The formula for the distance downslope to the stagnation point is given by Todd (1980) as:

$$x = -\frac{Q}{2\pi Ti} \qquad (13)$$

where x is the distance downslope to the stagnation point, Q is the pumping rate, T is the aquifer transmissivity, and i is the slope of the water table.

Ultimately, the width of the flow cross section can be solved by applying Darcy's law, as done in flow-net analysis:

$$y = \frac{Q}{Ti} \qquad (14)$$

Walker et al. (1965) presented a potentiometric surface map for 1960 of the aquifer underlying the Havana Lowlands in Mason and Tazewell Counties (figure 25). The map shows a regional water-table slope of 0.00059 foot per foot (ft/ft) in the study area in 1960. This compares with typical gradients of about 0.001 ft/ft observed in the southern portion of the area during this study. If one substitutes into equation 13 the average pumping rates of 8 to 10 mgd, the 1960 gradient of 0.00059 ft/ft, and an average regional transmissivity value of about 370,000 gpd/ft (based on an average value of hydraulic conductivity from the well field and an average saturated thickness of the regional aquifer), the distance to the stagnation point downslope from the well field is between 5,800 and 7,200 feet (or about 1 to 1.4 miles). By substituting the above elements into equation 14, the upstream width of the flow cross section is found to range from 36,600 to 45,800 feet (6.9 to 8.7 miles).

Because the fish hatchery's well field is in an aquifer with a sloping water table, the capture zone or area of diversion will extend upslope virtually to the end of the aquifer or until it reaches a drainage divide created by flow towards another well field or to a stream. For a year of normal recharge (292,000 to 500,000 gpd/sq mi) and with well field pumpage of 9 mgd, the area of diversion necessary to balance pumpage is approximately 18 to 31 square miles. If the width of the flow cross section is 7.8 miles (the average of 6.9 and 8.7 miles determined for pumpages of 8 and 10 mgd), the length of the flow field would be 2.5 to 4 miles. It is not unreasonable to assume, therefore, that the majority of the recharge to the hatchery's well field comes from local rainfall within Sand Ridge State Forest, while the remaining recharge comes from areas upslope from the forest

Summary and Conclusions

Based on the results of an examination of field conditions at the Jake Wolf well field, a number of conclusions and observations can be made.

1. The historical practice of estimating pumpage from manufacturers' pump-discharge curves and hours of operation yields an approximation. But that practice is only one of three methods that could be employed to estimate pumpage. A second set of curves, derived from data collected during step testing, gave pumpage estimates that averaged 40 percent higher than those calculated from the manufacturers' curves. On the other hand, pumpages estimated by simply multiplying nominal rated pump capacities by hours of operation averaged only 9 percent higher than the method used, that is, estimating from the pump manufacturers' curves. As observed earlier, the obvious means to improve accuracy would be to install in-line flow meters or at least a master meter in the main line to the hatchery.

2. Water-level data from observation wells distant from the well field suggest that water stages at each well were approaching an equilibrium level in 1987. If so, this would indicate that recharge was balancing discharge. Correlations of water levels with pumpage, however, show only weak to moderate support for such a conclusion. Subsequent declines in water levels in these wells indicate that the effects of the 1988-1989

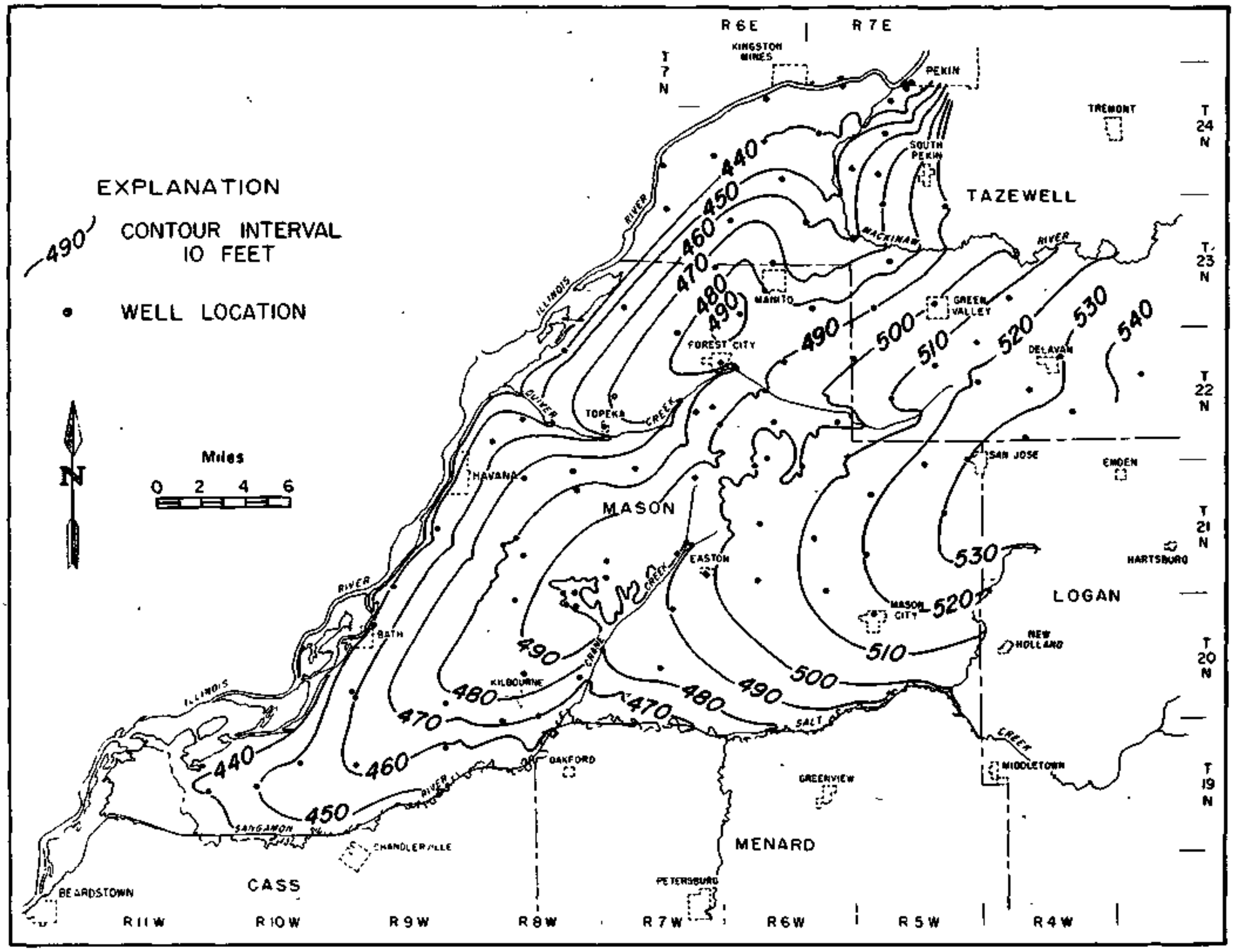

Figure 25. Approximate elevation of the water table in Mason and Tazewell Counties, 1960 (from Walker et al., 1965)

drought lasted well into 1990, and that recoveries did not begin until early May 1990. Nevertheless, the recovery of water levels that began at that time continued without interruption through spring 1991. The remarkable feature of this recovery is that it continued through summer and fall 1990, when water levels would normally exhibit seasonal declines. It is tempting to conclude that the recoveries might represent a natural tendency for levels to return to the apparent near-equilibrium stages that existed prior to the drought. Since the Water Survey will continue monthly water-level monitoring in its network of long-term wells at Sand Ridge for the foreseeable future, it will be possible to confirm or deny the accuracy of this conclusion.

3. Analysis of weekly water-level data from wells in five-spot arrays (using the Stallman finite-difference method) indicated that the average effective recharge rate in the study area between November 1989 and October 1990 ranged from 292,000 gpd/sq mi (6.1 inches) at the south five-spot to 507,000 gpd/sq mi (10.6 inches) at the north five-spot. The unadjusted recharge rates that were calculated with this method were highly sensitive to flow gradients and had to be adjusted to account for the natural water-table slope and the gradient created by well field pumpage. The results were in close agreement with those determined by earlier workers.

4. Flow-net analyses of potentiometric surface maps were constructed from monthly water-level data from the well network. These were also highly sensitive to flow gradients and produced results that were inordinately high. A subjective approach for constructing the potentiometric surface maps manually (using a curvilinear interpolation) produced results more in agreement with the Stallman method. Using this manual approach, the

average recharge rate for the study area for 1990 was 855,000 gpd/sq mi (18.0 inches).

5. A simple water-balance method for approximating available moisture for recharge indicated that soil-moisture deficits were quite small in 1990 since precipitation was 16 inches above normal, and rainfall was plentiful during the summer months. As a result, the excess soil moisture available for recharge was 19.76 inches, which is equivalent to a recharge rate of 941,000 gpd/sq mi. Again, these results were consistent with the results from the Stallman method and with apparent recoveries in observation wells.

6. By comparing the results of various methods of estimating recharge with the average range of pumpage at the hatchery well field, the approximate area of diversion can be determined. Since the aquifer has a sloping water table, the area of diversion extends upslope as far as necessary to capture sufficient water to balance pumpage. For years of normal recharge and pumpage, the area of diversion may vary from approximately 18 to 31 square miles. Given the shape of the area of diversion, most of the recharge to the aquifer at Sand Ridge apparently comes from precipitation that falls within the forest, while the remainder comes from areas upslope of the forest.

ASSESSMENT OF THE POTENTIAL FOR ADDITIONAL WATER

As expected, the Jake Wolf Fish Hatchery has been expanding its facilities, including the construction of new rearing ponds. Therefore, a supplemental water supply of 2,500 gpm will be needed. The source for this additional water must be addressed in a timely manner to facilitate the hatchery expansion.

Three potential water sources are being considered: 1) pumping more water from existing wells in the current well field, 2) constructing additional wells in the well field, and 3) constructing additional wells in areas remote from the well field.

Existing Well Field

The feasibility of pumping at higher rates from existing wells in the present well field depends on such factors as the amount of available drawdown, mutual interference, well design, and well losses. Examination of monthly air line measurements during the study indicated that water levels under pumping conditions are close to the pumps in all but wells 4 and 5. Generally less than S feet and often less than 3 feet of water is left above the pump intakes during operation of these wells. At wells 4 and 5, however, records indicate that 5 to 10 feet of water is above the pump intakes during operation.

The potential for well operation at higher pumping rates is very slim, not only because of the marginal available drawdown in most cases, but also because of the possible hydraulic impairment that such operation might cause in the wells. Higher pumping rates also mean higher entrance velocities at the well screens, and the long-term effects of higher velocities could include deterioration of the screens by either mechanical or chemical incrustation. If this occurred, well yields would decrease because of the reduction in specific capacity.

The second option for obtaining supplemental water supplies from the well field is to construct additional wells. But available drawdown—the same limitation that makes higher pumping rates in the existing wells infeasible—would also preclude additional wells in the immediate vicinity of the well field. Mutual interference effects among wells would reduce pumping levels even further, so that the total yield from the well field would not increase to any significant degree.

In all probability, therefore, the current well field is not capable of producing significant quantities of water beyond what it has been able to produce until now. Additional water supplies clearly must come from elsewhere.

Additional Wells

Based on the typical pumping rates from the current production wells, a supplemental water supply of 2,500 gpm would probably require two additional wells, and possily a third as a standby. In order to minimize the impact of withdrawals from these wells on the existing well field, the new wells should be constructed at sites remote from the current well field.

The relationship between pumpage and drawdown can be examined by constructing a theoretical distance-drawdown curve. (figure 26). This curve was made by interpolating average aquifer properties (T = 356,800 gpd/ft, and S - 0.13); a pumping rate of 2,500 gpm; and a continuous pumping period of six months (180 days) into the Theis equation (Theis, 1935). The distance-drawdown curve thus produced shows that drawdowns of about 4.0 feet would be observed at a distance of 1,000 feet from the pumping well; 2.5 feet would be observed at a distance of one-half mile; 1.45 feet would be observed at one mile; and 0.55 foot would be observed at two miles. It is obvious, therefore, that the limited amount of drawdown still available at the existing wells makes it necessary for the new wells to be constructed as far away from the current well field as practicable.

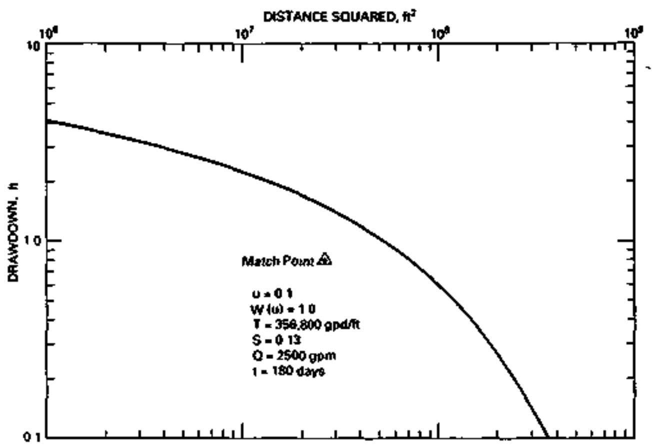

Figure 26. Theoretical distance-drawdown curve for a pumping rate of 2,500 gpm at the study site.

From the standpoint of distance alone, the most logical location for the new wells would be south on Cactus Drive or Forest City Road or east on Sand Ridge Road (i.e., sites in Sections 2 or 3, T.22N., R.7W.) as shown in figure 2.

This would also agree with Walker et al. (1965), who indicated in their report that the saturated thickness of the aquifer increases in a southeasterly direction from the well field. Thus, aquifer conditions at those sites could be expected to be at least as good or better than at the well field. At any site selected for the new wells, long-term aquifer tests (three to seven days) should be conducted to verify the well yields and determine aquifer hydraulic properties.

Results of Test Drilling

The saturated thickness of the aquifer reportedly decreases toward the Illinois River (Walker et al., 1965). But if sufficient sand deposits could be found in close hydraulic connection with the river or the lakes in the Illinois River valley, significant quantities of ground water might be made available by inducing infiltration from these surface waters.

This concept was investigated by the Water Survey in one location close to the hatchery. The Water Survey auger rig was employed to drill a test well at the base of the bluff adjacent to Spring Lake in the southwest quarter of Section 16, T.23N., R.7W. (Figure 2). On July 17, 1990, a test boring was made to a depth of 35 feet at the site. The driller reported that at that depth the auger bit could not penetrate the material. He concluded that he had encountered either bedrock or extremely dense clay. Following is the log of materials reported by the driller:

Depth (feet)	*Material*
0-9	Brown, fine to medium sand
9-10	Dark brown to grey, fine to medium sand with silt or clay and pebbles
10-14	Grey clay to silty clay (wet)
14-35	Saturated grey clay - no cuttings
35+	Drilling difficult

Based on the results of the test boring, the likelihood of locating sufficient aquifer material in the bottoms area is not good, especially in view of the limited amount of state-owned property in that area. The bedrock surface is apparendy close to the land surface in the bottoms, causing the alluvial materials to be fairly thin. However, if land were made available, a limited exploratory drilling program in the bottoms would be warranted.

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