FEASIBILITY OF ARTIFICIAL RECHARGE TO THE OAKES AQUIFER, SOUTHEASTERN NORTH DAKOTA: PRELIMINARY COST ANALYSIS OF A PROJECT-SCALE AND PILOT-SCALE WELL FIELD AND ARTIFICIAL RECHARGE FACILITIES

By Robert B. Shaver

Water Resources Investigation No. 8
North Dakota State Water Commission

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INTRODUCTION

In 1957, the U. S. Bureau of Reclamation redesigned the Pick–Sloan Missouri River Basin Plan enacted by Congress in the Flood Control Act of 1944. Under the redesigned plan, 1,007,120 acres of land were to be irrigated in central and eastern North Dakota using Missouri River water diverted eastward from the Garrison Reservoir. The plan designated 108,000 acres of land to be irrigated in the Oakes area, southeastern North Dakota.

In 1965, Congress enacted legislation to authorize construction of the 250,000–acre Garrison Diversion Unit as the initial stage of the ultimate 1,007,120–acre project. The 1965 authorization designated 45,980 acres to be irrigated in the East and West Oakes irrigation development tracts of the Garrison Diversion Unit. Missouri River water would be diverted eastward to the James River via the McClusky Canal, Lonetree Reservoir, New Rockford Canal, and the James River Feeder Canal. Because channel capacity of the James River was insufficient to meet peak irrigation demands for the East and West Oakes irrigation development tracts, the U.S. Bureau of Reclamation proposed construction of Lake Taayer Reservoir.

The Garrison Diversion Unit, as authorized in 1965, raised significant issues of environmental, economic, and international concern. As a result, in accordance with Public Law 98–360, Sec. 207, enacted by Congress July 16, 1984, a 12–member commission was appointed by the Secretary of the Interior to "examine, review, evaluate, and make recommendations with regard to the contemporary water development needs of the State of

North Dakota." Concerning irrigation in the Oakes area, the Garrison Diversion Unit Commission recommended the following in December 1984:

1) Reduce the 45,980 acres to be irrigated under the 1965 authorization to 23,660 acres (West Oakes = 19,660 acres; West Oakes extension = 4,000 acres).
2) Deauthorize construction of Lake Taayer Reservoir.
3) Initiate a feasibility study to assess artificial recharge to the Oakes aquifer as an alternative to a surface reservoir (Garrison Diversion Unit Commission, 1984).

Based on recommendations of the Garrison Diversion Unit Commission, the Congress of the United States passed the Garrison Diversion Unit Reformulation Act of 1986. The act directed the Secretary of the Interior to submit a comprehensive report to Congress no later than the end of fiscal year 1988. The report would include the results of an artificial–recharge feasibility study for the Oakes aquifer. Under the proposed artificial–recharge plan, the Oakes aquifer would function as a storage reservoir. Water would be diverted from the Missouri River to the James River and then into recharge facilities at selected sites in the aquifer. Withdrawals for irrigation would be from wells completed in the Oakes aquifer.

In July 1985, the North Dakota State Water Commission and the U.S. Geological Survey entered into a cooperative agreement with the U.S. Bureau of Reclamation to investigate the feasibility of artificial recharge to the Oakes aquifer, southeastern North Dakota (fig. 1). The feasibility study was divided into three phases. Phase I defines the geometric, hydraulic, and hydrochemical properties of the Oakes aquifer. Field work was initiated in August 1985 and completed in April 1986. Results of phase I of the artificial–recharge feasibility study are described in North Dakota State Water Commission Water–Resources Investigations Nos. 5 and 6. Investigation No. 5 (Shaver and Schuh, 1989) describes the hydrogeology of the Oakes aquifer. Investigation No. 6

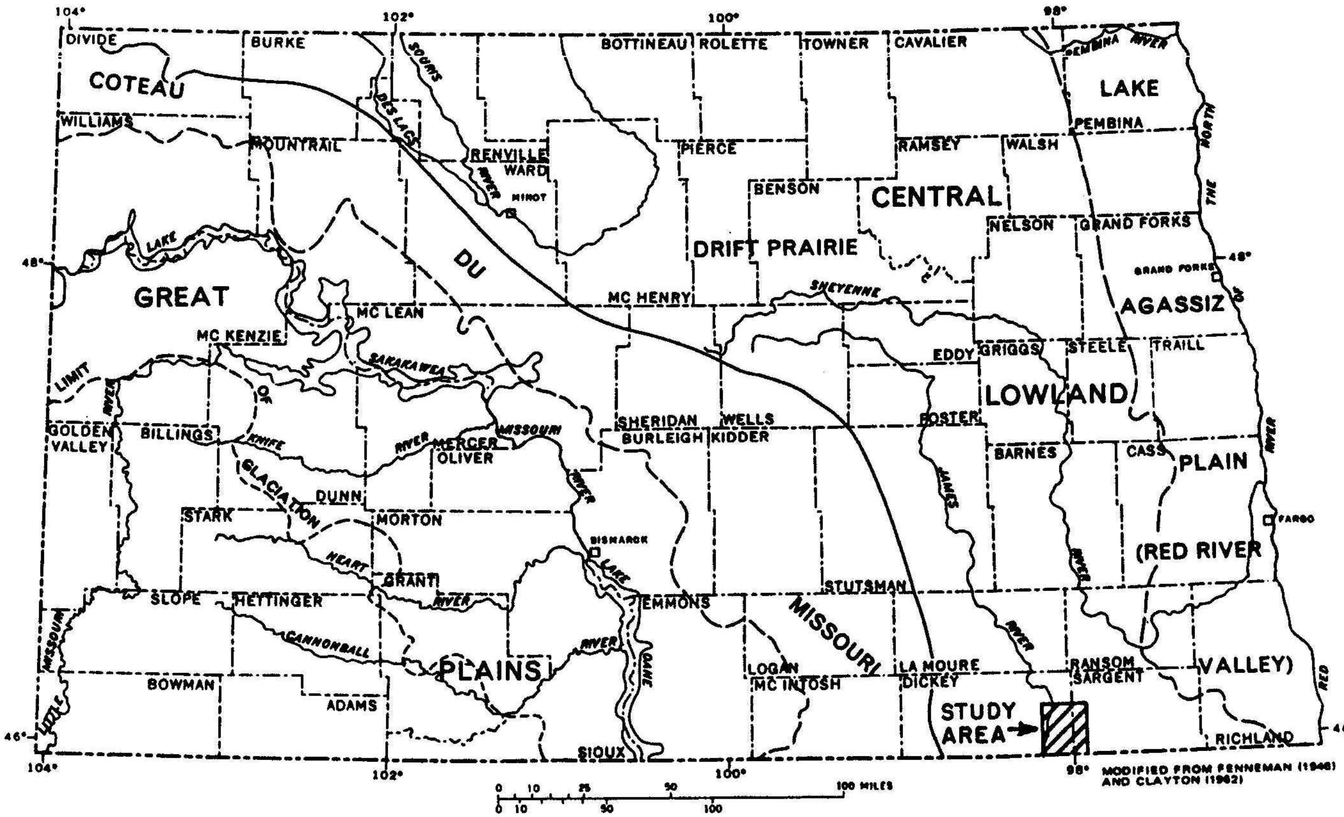

Figure 1.--Physiographic divisions in North Dakota and location of study area

(Shaver and Hove, 1989) presents the ground–water data, which consists of lithologic logs of test holes and wells (volume 1), water–level measurements (volume 2), and water–quality analyses (volume 2).

Phase II of the artificial–recharge feasibility study describes the selection, construction, maintenance, and performance evaluation of surface–recharge test facilities in the Oakes aquifer. Water used to perform the recharge tests was diverted from the James River. Field work was initiated in May 1986 and completed in November 1987. Results of phase II of the artificial–recharge feasibility study are presented in North Dakota State Water Commission Water–Resources Investigation No. 7 (Schuh and Shaver, 1988), and a U.S Geological Survey report. Investigation No. 7 describes infiltration through recharge basins, physical processes that affected infiltration, and operational and maintenance techniques used to enhance infiltration rates. A report by the U.S. Geological Survey (in preparation) describes the chemical and biological processes operative during basin recharge.

Phase III (this report) of the artificial–recharge feasibility study describes a preliminary design and cost–estimate analysis of a full project–scale and pilot–scale well field and artificial–recharge facilities for the Oakes aquifer. The Phase III study was prepared by the North Dakota State Water Commission.

PURPOSE AND SCOPE

The purpose of Phase III is to provide a cost estimate analysis for both a project–scale and pilot–scale well field and artificial–recharge facilities in the Oakes aquifer. Requirements for project–scale development are (1) the continuous withdrawal of 70 cubic feet per second for 60 days (8,330 acre–feet) from a well field, and (2) an annual artificial recharge rate of 35 cubic feet per second for 120 days (60 days in April and May; 60 days in September and October). In addition, the aquifer must accommodate a maximum discharge rate of 100 cubic feet per second for 60 days (11,900 acre–feet) during periods of peak irrigation demand. The requirement for pilot–scale development is the continuous withdrawal of 60 cubic feet per second for 34 days (4,057 acre–feet) from a well field.

A previously developed finite–difference computer model of the Oakes aquifer was used to determine an acceptable well–field configuration in the proposed project area near Section 13, Township 129 North, Range 59 West. In addition, the model was used to simulate the effects on the water table of operating a pilot–scale well field and operating a project–scale well field in conjunction with artificial recharge. A cost–estimate analysis was prepared for selected project– and pilot–scale well field and recharge facilities.

LOCATION–NUMBERING SYSTEM

The location–numbering system used in this report is based on the public land classification system used by the U.S. Bureau of Land Management. The system is illustrated in figure 2. The first number denotes the township north of a base line; the second number denotes the range west of the fifth principal meridian; and the third number denotes the section in which the well or test hole is located. The letters A, B, C, and D designate, respectively, the northeast, northwest, southwest, and southeast quarter section, quarter–quarter section, and quarter–quarter–quarter section (10–acre tract). For example, well 130–059–15DAA is located in the NE1/4 NE1/4 SE1/4 Sec. 15, T. 130 N., R. 59 W. Consecutive terminal numerals are added if more than one well or test hole is located within a 10–acre tract.

WELL FIELD

Aquifer Properties in the Project Area

The area of the Oakes aquifer most feasible for a project–scale well field is located in the channel–fill sand and gravel deposits near Section 13, Township 129 North, Range 59 West (fig. 3). In this area, the aquifer generally is unconfined, anisotropic, and nonhomogeneous with the coarsest deposits comprising the bottom one–half of the aquifer.

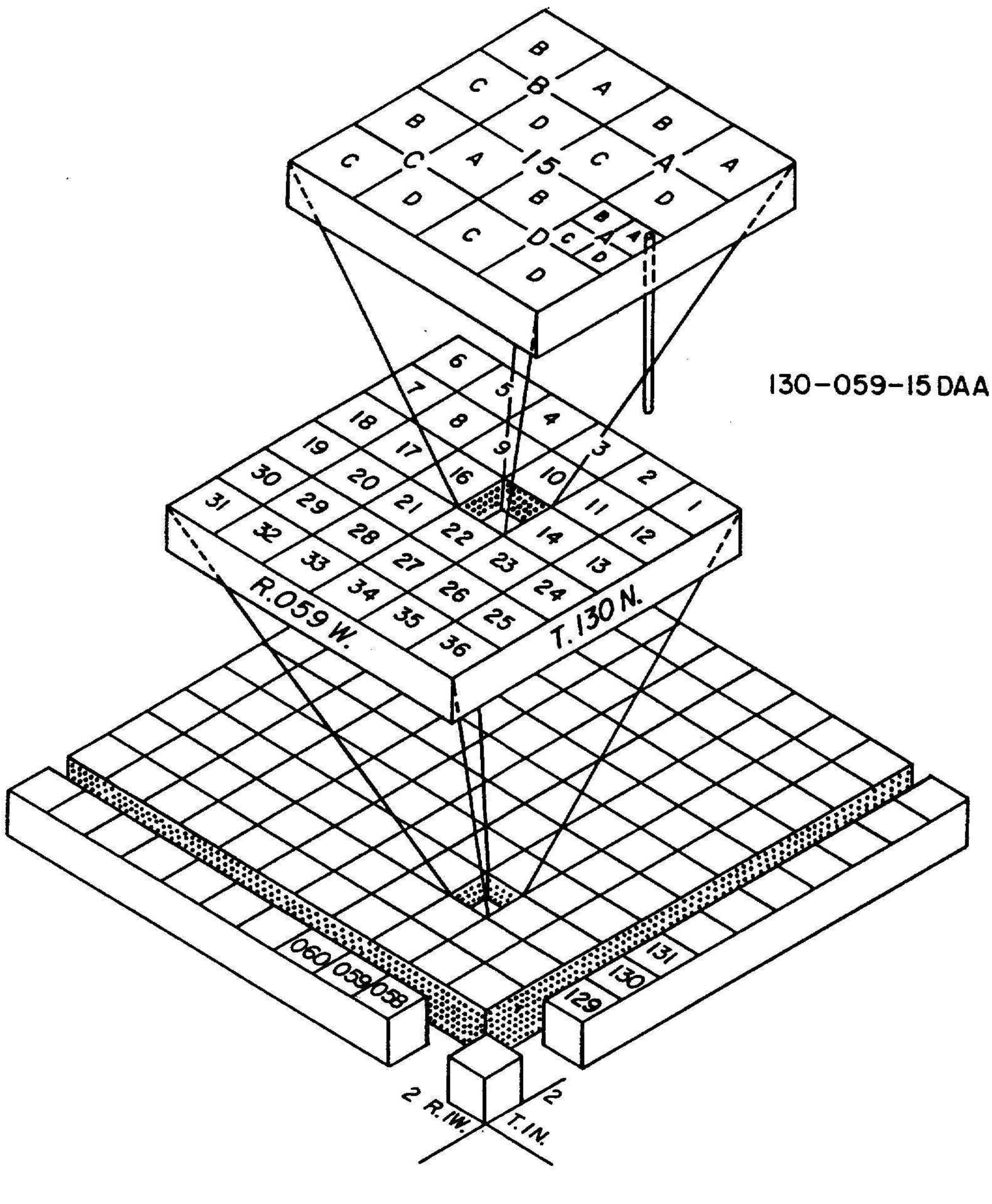

Figure 2.--Location-numbering system

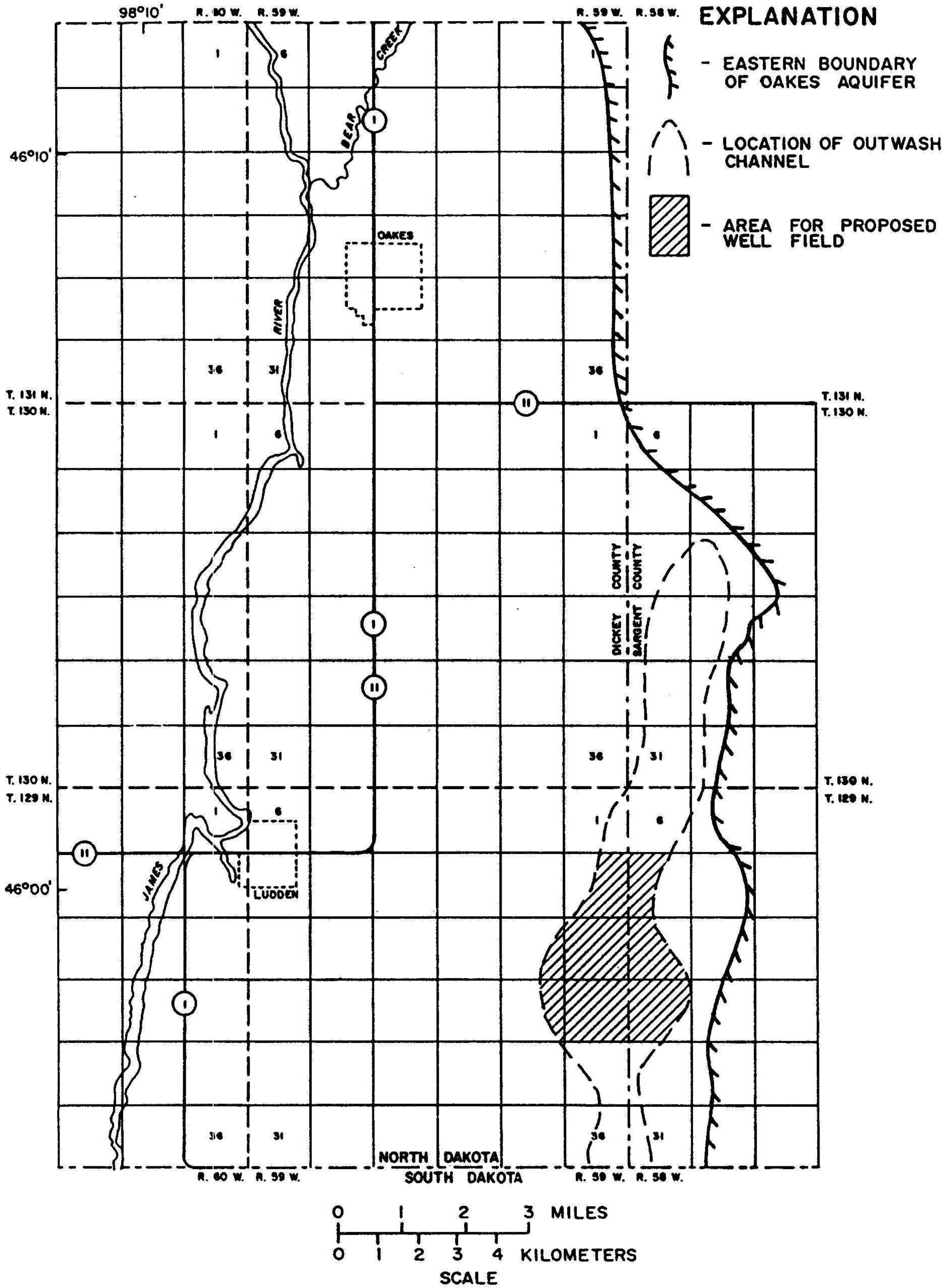

Figure 3.--Location of proposed well field

Based on available data, the estimated average saturated thickness of the aquifer is about 120 feet with an average depth to the water table of about 5 feet.

The State Water Commission conducted an aquifer test using an irrigation well located at 129–059–13ADD (AT–3, fig. 4) (Shaver and Schuh, 1989). The irrigation well was constructed using 16–inch diameter steel casing from land surface to a depth of 95 feet. A 14–inch diameter, telescopic, stainless–steel screen was installed from 95 to 115 feet below land surface. Slot size was 0.150 inch from 95 to 100 feet and 0.125 inch from 100 to 115 feet below land surface. The driller's log indicated gravel with occasional sand lenses throughout the screened interval.

The specific capacity of the irrigation well, after continuously pumping 780 gallons per minute for 6,000 minutes, was 215 gallons per minute per foot of drawdown. An average transmissivity of 94,000 feet squared per day and an average hydraulic conductivity of 775 feet per day were calculated from the aquifer–test data. Storativity was not calculated because (1) the aquifer underwent conversion from confined to unconfined conditions during the test; and (2) late–time drawdown data was affected by one or more barrier boundaries. A storativity of 0.20 was estimated.

The irrigation well was not completed to the bottom of the aquifer. A State Water Commission observation well, located 200 feet north of the irrigation well, indicated sand, gravel, and cobbles from 120 to 135 feet below land surface. The total depth to the base of the aquifer at this observation well site was 135 feet. Additional test–drilling is required in this area to determine an accurate cross–sectional profile of the outwash channel.

For preliminary planning purposes, the following average aquifer parameters for the channel–fill deposits are estimated for the project area:

1) Hydraulic conductivity – 775 feet per day
2) Storativity – 0.20
3) Saturated thickness – 120 feet
4) Transmissivity – 93,000 feet squared per day

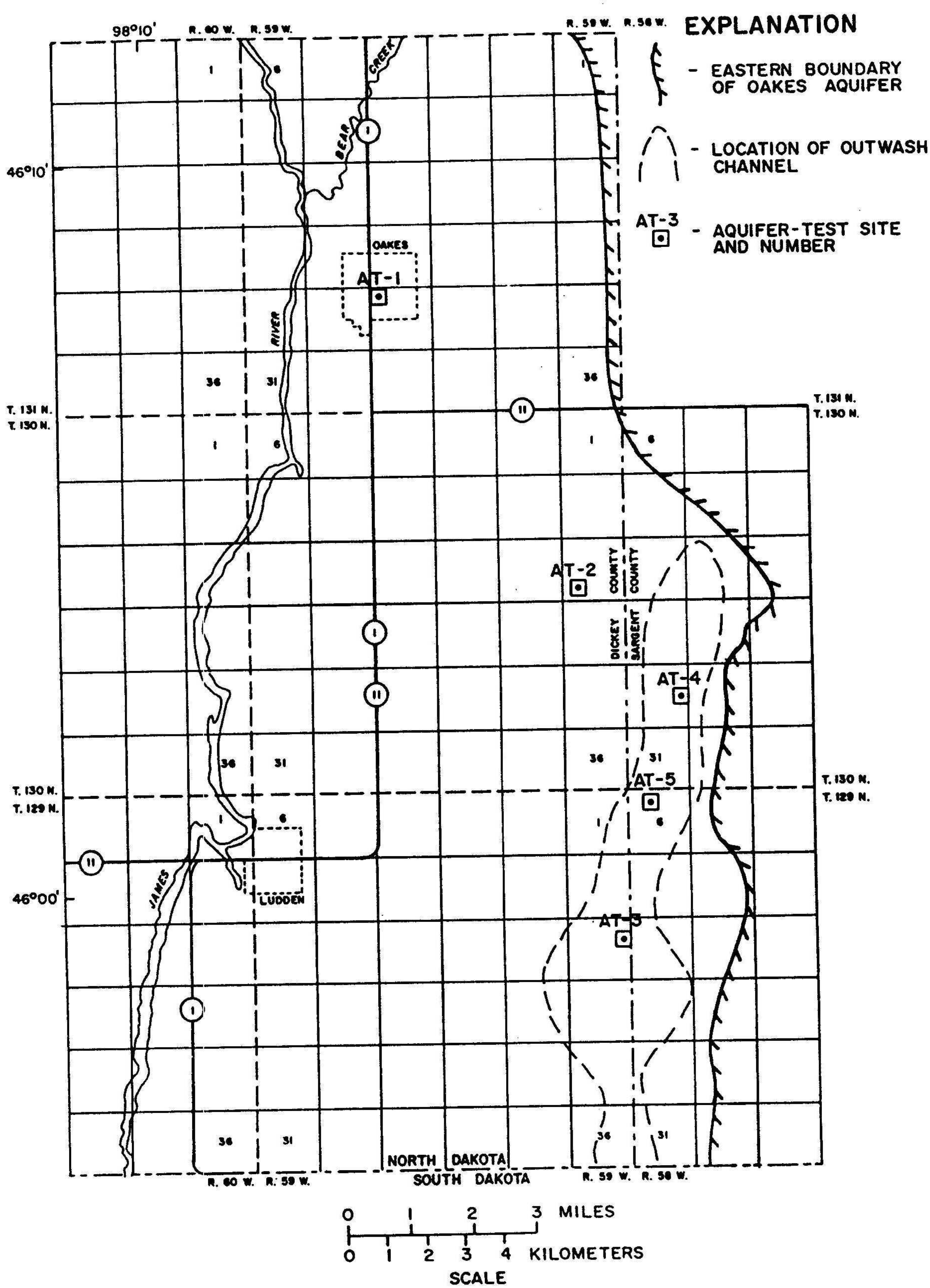

Figure 4.--Location of aquifer-test sites

Construction of Existing Wells in the Project Area

A preliminary well design and cost analysis requires examination of available data from existing wells in the proposed project area. There are four irrigation wells completed in the outwash channel sand and gravel deposits within the proposed project area (fig. 5). The irrigation wells are located at 129–059–12ADD, 129–059–12DAD, 129–059–13AAD6, and 129–059–13DAC. Another irrigation well is completed in the outwash channel sand and gravel deposits about three–quarters of a mile north of the proposed project area at 129–058–06BAD5. A summary of the construction, geologic, and hydraulic data for the above irrigation wells is presented in table 1.

Preliminary Well–Design Requirements in the Project Area

Selection of casing, screen, and pumps are determined, in part, by the desired discharge rate. Based on the aforementioned aquifer properties in the project area, long–term well yields of about 3,000 gallons per minute are possible from properly constructed wells.

Optimum well design requires screening of the bottom one–third to one–half of a homogeneous unconfined aquifer less than 150 feet thick. This also applies to nonhomogeneous unconfined aquifers less than 150 feet thick where the coarsest deposits comprise the bottom one–third to one–half of the aquifer. In addition, the drawdown in a well should not exceed 67 percent of the initial saturated thickness of the aquifer (fig. 6). Figure 6 shows that, at 67 percent of maximum drawdown (initial saturated thickness), 90 percent of the maximum well yield is obtained.

Within the project area, the aquifer is unconfined and nonhomogeneous with the coarsest deposits comprising the bottom one–half of the aquifer. The estimated average saturated thickness of the aquifer is about 120 feet. The estimated average depth to the water table is about 5 feet. Screening the bottom one–third of the aquifer would require 40 feet of screen set from 85 to 125 feet below land surface. This leaves 80 feet of available head above the top of the screen. As a general rule, in relatively thin, unconfined aquifers

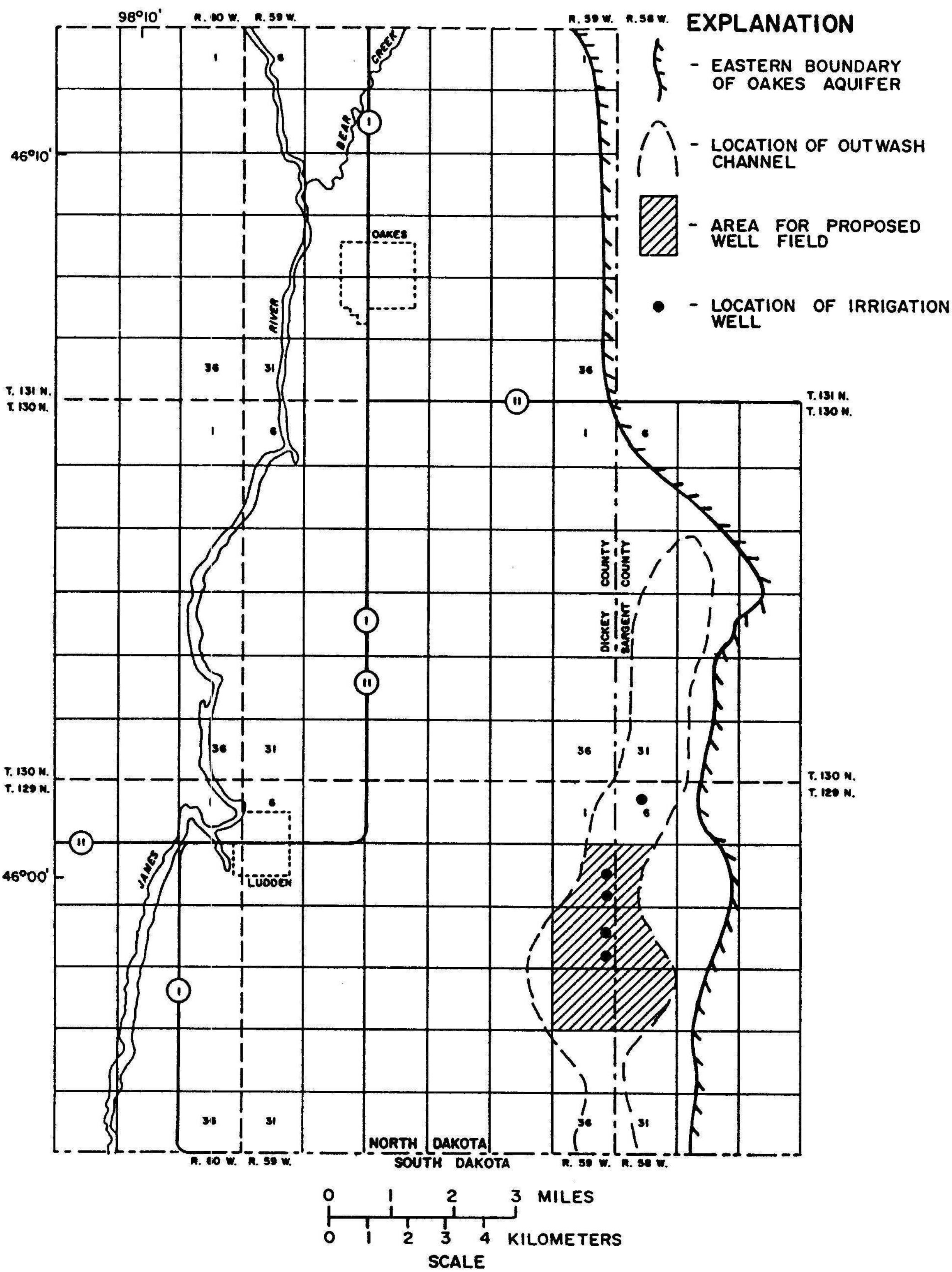

Figure 5.--Location of existing irrigation wells in the project area

Table 1.--Existing well data in the project area

Location of Irrigation Well	Depth of Well (feet)	Casing Diameter (Inches)	Casing Type	Screened Interval (feet)	Screen Slot Size (Inches)	Screen Diameter (Inches)	Screen Type	Pump Type	Power Supply and Horsepower	Water Level Below Land Surface at Time of Well Construction (feet)	Producing Interval Lithologies	Specific Capacity (gpm per foot)
129-059-12ADD	105	16	Plastic	85- 105	0.125	16	Plastic	Turbine	Electric motor 75 H.P.	4 (6/76)	Medium to coarse sand	Not reported
129-059-12DAD	105	16	Plastic	85- 105	0.100	16	Plastic	Turbine	Electric motor 75 H.P.	Not reported	Coarse sand	Not reported
129-059-13AAD6*	115	16	Steel	95- 100 100-115	0.150 0.125	14 14	Stainless Stainless	Turbine	Electric motor 125 H.P.	1 (7/75)	Gravel with occasional coarse sand lenses	215
129-059-13DAC	125	16	Steel	95- 100 110-125	0.150 0.125	14 14	Stainless Stainless	Turbine	Electric motor 125 H.P.	1(7/75)	Coarse sand and gravel	Not reported
129-058-06BAD5*	158	12	Steel	126-138 138-152 152-158	0.100 0.120 0.140	10.75 10.75 10.75	Stainless Stainless Stainless	Turbine	Electric motor 125 H.P.	8	Sand and gravel	102

*Aquifer test conducted by N.D.S.W.C. using this well

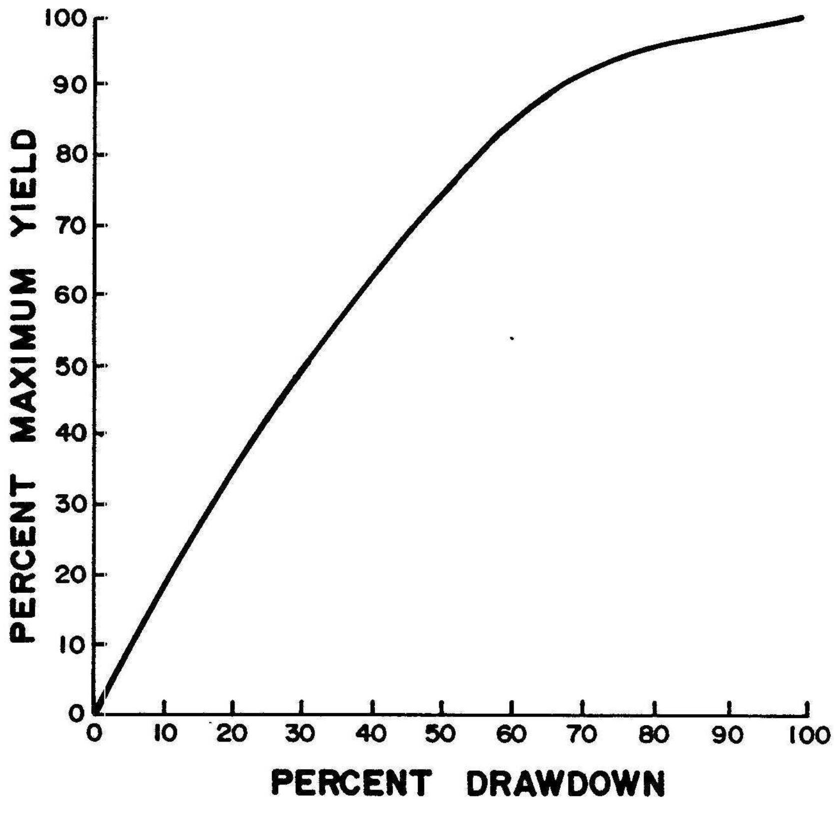

Figure 6.--Comparison of yield with drawdown in an ideal unconfined aquifer that is fully penetrated and open to the well (from Driscoll, 1984)

of limited areal extent, the North Dakota State Water Commission reserves about one–third of the available head above the well screen for potential well interference from other appropriators. Applying this guideline to the project area would leave about 53 feet (2/3 · 80) available drawdown at each well.

Preliminary design of the well casing, screen, and pumps is based on the following:

1) A discharge rate of 3,000 gallons per minute per well.
2) A saturated thickness of 120 feet.
3) A screen length of 40 feet (bottom one–third of the aquifer).
4) A pumping level not to exceed two–thirds of the available head (53 feet) above the top of the screen.

A finite–difference model of the Oakes aquifer (Shaver and Schuh, 1989) was used to select an appropriate well spacing based on a maximum drawdown at each well of 53 feet. A discussion of the model and its utility in the selection of an appropriate well–spacing requirement is presented in a later section of this report.

Well Design

The principal objectives of good well design should ensure the following (Driscoll, 1986):

1) The highest yield with minimum drawdown consistent with aquifer capability.
2) Good quality water with proper protection from contamination.
3) Water that remains sand free.
4) A well that has a long life (25 years or more).
5. Reasonable short–term and long–term costs.

There are three basic components of a water well:

1) Casing
2) Intake area (screen)
3) Pump

Casing design elements include diameter, wall thickness, type of material, and length. Well–screen design elements include diameter, length, slot size, open area, and type of material. Pump design elements include type, size, and power supply requirements.

Casing Design

Based on (1) an estimated average total depth to the base of the aquifer of 125 feet, (2) an estimated average saturated thickness of 120 feet, and (3) screening the bottom one–third of the saturated thickness of the aquifer (40 feet), 85 feet of casing will be required for each well. The selection of casing materials is based on corrosive potential of the ground water and strength requirements. Corrosion of casing can reduce strength, causing failure and, in certain cases, can allow inflow of poor quality water. The potential for casing corrosion can be assessed by the following chemical parameters and constituents (Driscoll, 1986):

1) pH – If the pH is less than 7, the water is acidic and corrosion is indicated.
2) Dissolved oxygen – If dissolved oxygen exceeds 2 mg/L, corrosive water is indicated.
3) Hydrogen sulfide – Hydrogen sulfide in ground water can be detected by its characteristic rotten–egg odor. Less than 1 mg/L can cause severe corrosion, and this amount can be detected by odor and taste.
4) Total dissolved solids – If total dissolved solids exceed 1,000 mg/L, electrical conductivity of the water is great enough to cause serious electrolyte corrosion.
5) Carbon dioxide – If the amount of this gas exceeds 50 mg/L, corrosive water is indicated.
6) Chlorides – If the chloride content of the water exceeds 500 mg/L, corrosion can be expected.

The range and mean of the above chemical parameters and constituents (less CO_2

gas) for ground water in and near the outwash channel, including the project area, are shown in table 2. For the most part, ground water in the outwash channel is noncorrosive. As a result, a low–carbon, steel well casing can be used.

Casing diameter is based on two requirements: 1) the casing must be large enough to accommodate the pump, with enough clearance for installation and efficient operation, and (2) the diameter of the casing must be sufficient to assure that the uphole velocity does not exceed 5 feet per second (Driscoll, 1986). Based on an estimated well yield of 3,000 gallons per minute, a casing diameter of not less than 18–inch O.D. (17.25–inch I.D.) with a standard wall thickness (0.375 inches) is recommended. To insure adequate strength requirements, a low–carbon steel is recommended.

Screen Design

Well–screen length is based in part on the effective open area of the screen and an optimum screen entrance velocity (Walton, 1962). On the average, about one–half of the open area of the screen will be blocked by aquifer material. Thus, the effective open area averages about 50 percent of the actual open area of the screen.

A relationship exists between hydraulic conductivity and optimum screen entrance velocities (table 3). The length of screen can be selected using table 3 and the following equation (Walton, 1962):

$$S_L = \frac{Q}{7.48\ A_o V_c}$$

where S_L = optimum length of screen, in feet

Q = discharge of production well, in gallons per minute

A_o = effective open area per foot of screen, in square feet, and

V_c = optimum screen entrance velocity, in feet per minute.

Based on a hydraulic conductivity of 775 feet per day, an optimum screen entrance velocity of about 11 feet per minute is selected from table 3. Available data from existing irrigation

Table 2. -- Chemical parameters and constituents used to assess the potential for corrosion in the project area

	Chemical Parameter or Constituent				
	pH	Dissolved Oxygen (mg/L)	Total Dissolved Solids	Chloride (mg/L)	Hydrogen Sulfide (mg/L)
Sum of values (Σ)	355.4	73.76	20814	692	*
Mean (X)	7.25	1.64	425	14.1	*
Maximum value	7.62	3.8	912	160	*
Minimum value	6.25	0.7	231	0	*

*No odor detected during sample collection, therefore H_2S assumed negligible

Explanation

X = Arithmetic mean
Σ = Sum of values
N = Number of samples = 49 except for dissolved oxygen where N = 45

Table 3. - - Optimum screen entrance velocities
(from Walton, 1962)

Hydraulic Conductivity (feet per day)	Optimum screen entrance velocities (feet per day)
>800	12
800	11
670	10
535	9
400	8
335	7
270	6
200	5
135	4
67	3
<67	2

wells and test holes in the project area indicate a screen slot size ranging from 0.100 to 0.150 inch. For 18–inch diameter, 100–slot, telescopic Johnson[1] stainless–steel screen with an effective open area of 0.83 square feet, 43.9 feet of screen would be required to transmit 3,000 gallons per minute. For 18–inch diameter, 150–slot, telescopic Johnson stainless–steel screen with an effective area of 1.04 square feet, 35.1 feet of screen would be required to transmit 3,000 gallons per minute. These screen lengths represent about one–third of the estimated average saturated thickness of the aquifer in the project area and are within the range of optimum well design (i.e., screening the bottom one–third to one–half of a homogeneous unconfined aquifer less than 150 feet thick).

Three factors govern the choice of material used to fabricate well screens (Driscoll, 1986). These are: (1) water quality, (2) potential presence of iron bacteria, and (3) strength requirements of the screen. Both corrosion and incrustation can result in well–screen failure. Corrosion can cause enlargement of screen openings allowing sediment to enter the well. Incrustation can reduce screen openings causing a reduction in well yield. As with well casing, the potential for corrosion and incrustation of well screen can be estimated by analyzing selected chemical parameters and constituents. As previously stated, ground water in the project area is non–corrosive.

Indicators of incrusting ground water are (Driscoll, 1986):

1) pH – If the pH value is above 7.5, the water will tend to be incrusting.
2) Carbonate hardness – If carbonate hardness of the ground water exceeds 300 mg/L, incrustation of calcium carbonate (lime scale) is likely.
3) Iron – If the iron content of the water exceeds 0.5 mg/L, precipitation of iron is likely, although some precipitation may begin at concentrations as low as 0.25 mg/L.

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[1]Product and manufacturer names are given for information purposes only, and do not imply endorsement by the North Dakota State Water Commission.

4) Manganese – If the manganese content of the water exceeds 0.2 mg/L and the pH value is high, precipitation of manganese is likely if oxygen is present.

The range and mean values of the above chemical parameters and constituents for ground water in and near the outwash channel, including the project area, are shown in table 4. For the most part, ground water in the outwash channel is incrusting as indicated by high carbonate hardness and high iron and manganese concentrations. Mineral deposits from incrusting–type ground water can often be removed from well screens by acidizing techniques. In incrusting environments, the well screen should consist of a non–corrosive material to withstand the corrosive effect of acid treatment.

Iron bacteria form gelatinous masses that block well–screen openings thereby reducing well yield. Remedial techniques include chemical treatment (chlorination) or pasteurization (steam injection). These techniques are corrosive and, therefore, non–corrosive well screen should be used in aquifers where iron bacteria growth is anticipated.

The Bureau of Reclamation reports iron–bacteria growth in the drain complex within the 5,000–acre test plot south of Oakes (Mathison, verbal communication). The author is also aware of iron bacteria reported in domestic and stock wells completed in the Oakes aquifer. Because of the potential for both incrustation and iron–bacteria growth in wells completed in the Oakes aquifer, production wells should be constructed with a non–corrosive, stainless–steel type well screen.

The final design requirement for well screen is strength. Loads (forces) imposed on a well screen are column load (vertical compression), tensile load (extending forces), and collapse pressure (horizontal forces). The strength of the well screen is based on the type of material used to construct the screen, dimensions of the screen components, and the slot configuration (Driscoll, 1986). The strength of most well screen is satisfactory for large–diameter wells less than 300 feet deep. However, it is advisable to consult with the well–screen manufacturer to assess strength characteristics and requirements.

Table 4. -- Chemical parameters and constituents used to assess the potential for incrustation in the project area

	pH	Carbonate Hardness (mg/L)	Iron (mg/L)	Manganese (mg/L)
Sum of values (Σ)	355.4	15,271	35.02	25.67
Mean (X)	7.25	312	0.71	0.52
Maximum Value	7.62	647	2.90	1.40
Minimum Value	6.25	185	0.03	0.17

Explanation

$\bar{X}$ = arithmetic mean
Σ = sum of values
N = number of samples = 49

Pump Design

Pumps are classified generally into two groups: (1) shallow–well pumps, and (2) deep–well pumps (Driscoll, 1986). A shallow–well pump is mounted at land surface and removes water from the well by suction lift. Shallow–well pumps are used when pumping levels are less than about 20 to 25 feet below land surface. A deep–well pump is installed within the well casing, with the pump intake submerged below the pumping level. It is anticipated that pumping levels in wells within the project area will average about 60 feet below land surface. Therefore, deep–well pumps will be used in the project area.

The most widely used pump for high–capacity, large diameter wells is the deep–well turbine (centrifugal) pump. The pumping assembly of a vertical turbine pump consists of one or more impellers housed in a single– or multi–stage unit called a bowl assembly. The impellers are suspended on a vertical line shaft (drive shaft) housed within the pump column which conducts water to the surface. The size of the outer column is selected on the basis of a pumping rate. The head losses in the column should not exceed 5 feet per 100 feet of column at the designed capacity. The amount of column is based on the pumping level in the well. The pump column is attached at the surface to a discharge head which serves several purposes. It provides a base for a driver (electric motor or right–angle gear coupled to a reciprocating engine); creates a packing box around the shaft, preventing water from entering the motor; acts as an elbow to divert the discharge into the above–ground piping system; and also supports the column pipe in the well (Driscoll, 1986).

Pump design parameters include (Driscoll, 1986):

1) Well diameter
2) Desired yield
3) Total dynamic head
    a) Pumping water level
    b) Above–ground head

c) All friction losses

4) Horsepower requirements

a) Brake horsepower

b) Horsepower required to offset shaft losses (vertical turbine)

c) Motor efficiency without thrust load

d) Losses caused by friction in the thrust bearing

e) Horsepower curve for varied discharge rates

5) Power source

a) RPM preferred or required

6) Pumping deviation: system–head–curve parameters

7) Sand pumping potential

8) NPSH (net positive suction head)

9) Water quality

10) Short– and long–term costs

a) Initial capital costs

b) Amortization of investment

c) Power costs

d) Supervision and maintenance

e) Cost of down time and standby equipment

The selection of a pump for large–capacity production wells in the project area will ultimately be based on pumping–test data. Pumping levels, well yields, and lift requirements will vary from well to well due to spatial variability of aquifer hydraulic properties and differences in well efficiency and well interference. Therefore, it is not possible to fully incorporate the above list of pump design parameters in a preliminary design analysis.

The following factors provide the basis for preliminary pump design in the project area:

1) The average depth to the water table in the project area is about 5 feet.

2) The average depth to the bottom of the aquifer is about 125 feet.

3) Anticipated well yield: 3,000 gallons per minute.

4) Casing diameter: 20 inch O.D. (19.25 inch I.D.).

5) Screened interval: 85 to 125 feet.

6) Maximum estimated lift: 85 feet.

Based on the above, each single–well pump system will consist of the following:

1) Pump type: deep–well vertical turbine pump.

2) Bowl assembly: 15–inch O.D. single– or two–stage bowl assembly,(number of stages, impeller type, and trim to be determined after test pumping each well).

3) Pump column: 80 feet of 12–inch diameter (O.D.) with 1.5 inch diameter lineshaft. Because static water levels in the project area are less than 30 feet below land surface (average is about 5 feet below land surface), a water–lubricated line shaft is preferred.

4) Power supply: 75 horsepower hollow–shaft electric motor.

5) Discharge head: 12–inch diameter (minimum) steel.

Well–Drilling and Construction Methods

The selection of a particular well–drilling and construction method is based primarily on the hydrogeologic setting. A geologic log of test hole 129–059–13AAD4 representative of the project area is shown in table 5. This test hole did not penetrate to the top of the bedrock formation (Niobrara Formation) underlying the till. It is estimated that the Niobrara Formation (shale) occurs at about 150 to 165 feet below land surface at this site. This test hole and others completed in the channel–fill deposits were drilled using a forward mud–rotary rig. The channel–fill deposits consist of unconsolidated stratified sand, gravel, and cobbles. The basal 20 to 30 feet of the aquifer contains the coarsest gravel and cobble deposits. This part of the aquifer was very difficult to drill.

Table 5. -- Geologic log of test hole 129-059-13AAD4

Location: 129-59-13AAD$_4$

Owner and number: SWC 11674

Depth drilled (ft.): 140

Screened interval (ft.): 105-110

Casing diameter: 1½-inch pvc

Date completed: 9/17/85

Use of well: Observation

Principal aquifer: Oakes

Altitude of land surface (ft., msl): 1311.2

Lithologic log from: SWC

Comments: East well of pair, 205 feet north of irrigation well 129-059-13AAD$_6$

Lithologic Log

Unit description	Thickness (ft.)	Depth (ft.)
Clay, silty, yellow-gray-brown, soft, oxidized	7	7
Sand, v. fine to v. coarse, sl. gravelly, fine, subangular to well-rounded, composed of detrital shale, quartz, carbonates, silicates, yellow-stained, oxidized	4	11
Sand, as above, gray, unoxidized	21	32
Clay, silty, greenish gray, soft	2	34
Sand, as above, gray	3	37
Clay, silty, greenish gray, soft	1	38
Sand, as above, more gravelly, drills as stratified, gravel is fine to medium, composition as above, caving, mixed bentonite mud to prevent caving	10	48
Sand (80%), v. fine to v. coarse, predom. medium to coarse, and gravel (20%) fine to medium, less detrital shale, more carbonates and silicates	19	67
Sand and gravel, coarser section than above, gravel is fine to coarse, composition as above, subangular to well rounded, mixed bentonite mud to prevent caving, takes water	43	110
Gravel, sandy, composition as above, drills as stratified, strong bit chatter	21	131
Gravel and cobbles, very hard drilling, many carbonate and silicate chips	4	135
Clay, silty, sandy, pebbly, olive gray (till)	5	140

Penetration rate was significantly slower in the basal part of the aquifer as compared to the overlying deposits. A large amount of bentonite was used to increase the viscosity of the drilling fluid in order to maintain circulation and to prevent the drill hole from collapsing. The increased viscosity of the drilling fluid caused most of the sand fraction to remain in suspension. Therefore, it was not possible to estimate (1) the percentage of sand, gravel, and cobbles, (2) the grain–size range of the sand, and (3) the degree of stratification in the bottom 40 to 60 feet of the channel–fill deposits.

Exploratory Test Drilling

An exploratory test–drilling program is recommended for the project area to define in detail the geometry of the outwash channel. Additional test drilling also will provide a generalized description of the channel–fill deposits. Forward, mud–rotary drilling techniques will be suitable for this exploratory phase of the project.

Using the above data, production–well sites will be selected. Sampling by forward–rotary methods using clay–based drilling muds is not recommended as a basis for well design and screen selection in the project area. Drive–core sampling is recommended for sampling the upper part of the aquifer at each proposed production–well site. Drive–core sampling, however, may not be practical in the bottom part of the aquifer that contains coarse gravel and cobbles. An alternative approach would be to drill the pilot hole with a cable–tool rig and collect samples using a bailing procedure.

A mechanical (grain–size) analysis will be performed on samples recovered over the interval of the aquifer considered for screening. Standard analytical techniques will be used in conjunction with mechanical analyses to select slot size of well screen. Design criteria for each production well will be based on data obtained from drilling the pilot hole.

Production–Well Drilling Methods

There are four methods that can be used for production–well drilling and screen installation in the proposed project area. These include:

1) Conventional rotary.
2) Conventional rotary and cable–tool drilling using the pull–back method to install screen.
3) Conventional rotary using a bail– or wash–down method to install screen.
4) Reverse rotary.

Conventional Rotary Drilling

The basal 20 to 30 feet of the channel–fill deposits in the project area consist of very fine to coarse sand, gravel, and cobbles. Previous experience using conventional (forward mud–rotary) rotary drilling methods in the project area indicates large amounts of bentonite will be required to increase the viscosity of the drilling fluid to maintain circulation and to prevent the drill hole from collapsing. Deep penetration of clay–based drilling fluid into the aquifer is anticipated during the drilling process. Well–development techniques may not sufficiently remove drilling mud from the aquifer. This could result in unacceptably low well efficiencies. Although conventional rotary drilling is cost effective in terms of drilling time, it may not be cost effective in terms of well–development time and possible decreased well efficiency.

Conventional Rotary and Cable–Tool Drilling Using the Pull–Back Screen Installation Method

One approach to avoid clay–based drilling fluids is to drill to the top of the proposed screened interval using a conventional rotary rig and then install the well screen with a cable–tool rig using the pull–back method. Assuming a screened interval from 85 to 125 feet below land surface, the following drilling protocol is recommended:

1) Drill a 24–inch diameter hole to a depth of 85 feet using a conventional rotary method.

2) Set 20–inch O.D. (19.25 inch I.D.) steel casing to a depth of 85 feet.

3) Place a cement grout in the annular area between the well casing and formation.

4) Drill the remainder of the hole using a cable–tool method.

5) Set 18–inch O.D. (17.25 inch I.D.) steel casing below the 20–inch O.D. steel casing to the bottom of the aquifer.

6) Attach a lead packer to the top of the screen assembly.

7) Insert a 40–foot length of 18–inch telescopic screen (16.125–inch O.D.) through the 18–inch O.D. (17.25–inch I.D.) steel casing to the bottom of the hole.

8) Pull back the 18–inch O.D. steel casing installed during the drilling process with the cable–tool rig.

9) Develop the well, using jetting and surging techniques. It is possible that the screened interval will consist of coarse sediments of relatively uniform grain size. Therefore, gravel packing should not be required.

10) Insert a swedge block through the casing to expand the lead packer and seal the casing–screen joint.

11) Install deep–well turbine pump.

Conventional Rotary Drilling Using a Bail– or Wash–Down Method to Install Well Screen

Under some conditions, it may be impossible or undesirable to pull back the well casing to expose the screen. For example, sidewall friction on casing by subsurface materials may require too much pulling force, or movement of the casing may disturb the sanitary seal around it (Driscoll, 1986). Alternatives to the pull–back method for screen installation are bail–down and wash–down methods. Both methods are the same as described for the pull–back method except a cable–tool rig is not used to set casing and screen. Instead either a bail–down shoe and nipple or a wash–down fitting is attached to the bottom of the screen.

If a bail–down shoe is used with special connection fittings, the screen is suspended on a string of pipe called bailing pipe. The screen assembly is worked into the formation below the well casing by operating the bailer or drilling tools through the bailing pipe. When the screen has been bailed down to the desired depth, a plug is lowered or dropped through the bailing pipe to seat in the special extra–heavy nipple above the bail–down shoe. Occasionally, cement is used to plug the bottom of the screen. The string of bailing pipe is then disconnected leaving the plug or cement and nipple to seal the bottom of the screen. After removing the bailing pipe, the lead packer at the top of the screen is expanded with a swedging tool (Driscoll, 1986).

For the wash–down method, a self–closing, bottom–fitting, or back–pressure valve is mounted in the bottom of the screen and connected by a left–hand thread to a string of pipe (usually drill pipe) used as the wash line. The screen is lowered to the bottom of the casing and lightweight drilling fluid or water is then pumped through the wash line. The jetting action loosens and removes the sediment and allows the screen to sink (Driscoll, 1986).

Reverse Rotary Drilling

In reverse rotary drilling, flow of the drilling fluid is reversed when compared to the conventional rotary method. The suction end of the centrifugal pump, rather than the discharge end, is connected through the swivel to the kelly and drill pipe. The drilling fluid and its load of cuttings move upward inside the drill pipe and are discharged by the pump into the settling pit. The fluid returns to the borehole by gravity flow. It moves down the annular space between the drill pipe and borehole wall to the bottom of the hole, picks up the cuttings, and reenters the drill pipe through ports in the drill bit (Driscoll, 1986).

Drilling fluid additives are seldom mixed with the water to make a viscous fluid. The hydrostatic pressure of the water column, plus the velocity head (inertia of the water moving downward) outside the drill pipe, support the borehole wall (Driscoll, 1986).

Depth to water table in the project area is generally less than 5 feet. Therefore, to develop enough hydrostatic pressure to prevent hole collapse, minor earthwork at each drill site probably will be required to elevate the drill rig and settling pit. In addition, drilling by reverse rotary methods requires large water supplies. It would probably be more practical to complete a supply well at each drilling site rather than hauling water by truck.

Based on preliminary hydrogeologic data in the project area, the reverse–rotary drilling method appears to be the most efficient method of production well drilling. The drill holes can be drilled quickly and economically, and no casing is required during the drilling operation. Clay–based drilling additives are not used. Well screens can be set easily as part of the casing installation (Driscoll, 1986).

Well–field Maintenance and Rehabilitation

To evaluate changes in well or pump performance, basic data on the well and pump should be collected immediately after construction. An as–built construction diagram of the well–formation log and mechanical analysis of the aquifer and gravel pack (if used) should be part of the record (USBR, 1977). After each well is completed, a step–drawdown test should be conducted to determine drawdown–yield relationships and to select a suitable pumping rate. In addition, the step–drawdown test may be used to measure well efficiency. During the step–drawdown test, water samples should be collected to measure sand content and selected chemical constituents and parameters.

After the permanent pump is installed and adjusted, it should be tested for wire–to–water efficiency (USBR, 1977). Wire–to–water efficiency is the actual discharge of the pump compared to the theoretical discharge considering the amount of energy used.

Project production wells will be operated during the summer months (June through August). Prior to each operational season, static water levels in each well should be measured and recorded. At the beginning of each operational season, pumping level, discharge rate, and specific capacity of each well should be measured and recorded. In addition, water samples should be collected to measure sand content and selected chemical constituents and parameters. Pumping level, discharge rate, specific capacity, sand content, and selected water quality parameters and constituents should also be measured and recorded just prior to the end of each summer pumping period.

Periodic evaluation of the above data will provide a basis for assessing well deterioration. A decrease in specific capacity, without a proportional decline in the static water level, may indicate blockage of the screen by accumulated sediment in the bottom of the well, blockage of the screen by encrustation, or collapse of casing or screen (USBR, 1977). An increase in sand content of the discharge may indicate (1) enlargement of screen slots due to corrosion; (2) settlement of gravel pack (if used) beneath a bridge, leaving an unpacked zone opposite part of the screened interval; (3) a break in the casing or screen, usually at a joint; or (4) failure of a packer seal.

Decline in pump discharge and head may be due to deterioration of the pump or both the well and pump. Pump problems include (1) improper adjustment of the impeller due to wear or other causes, (2) a hole in the column pipe, and (3) erosion or corrosion of the impeller or bowls (USBR, 1977).

Significant decreases in well yield, detected using the previously described monitoring program, may be remedied by a number of actions. These include:

1) Well re–development using jetting and surging techniques.
2) Removal of accumulated sand from the bottom of the well by bailing techniques.
3) Removal of scale (incrustation) on well screen by acidizing techniques.
4) Removal of iron–bacteria slime by pasteurization (steam injection) or acidizing techniques.
5) Replacement of well screen that is severely corroded or collapsed.
6) Adjustment or replacement of impellers.
7) Replacement of pump bowls and pump column.

It is difficult to predict the life of a production well. Most production wells will, with continuous heavy pumping, eventually become partially clogged. Thus, periodic maintenance is a requirement. With the use of modern materials and construction techniques and appropriate maintenance, most production wells could last for 40 years or more (Walton, 1970)

ARTIFICIAL RECHARGE

Methods of Artificial Recharge

There are six general methods of artificial recharge: 1) basins, 2) ditches and furrows, 3) surface spreading, 4) use of natural stream channels, 5) pits and shafts, and 6) injection wells (Bianchi and Muckel, 1970). Basins are the most common method of artificial recharge. If surface or near surface impeding layers (clay and silt) are absent, basins are formed by the construction of levees. If surface or near surface impeding layers occur, basins are formed by shallow excavations generally less than 5 feet deep. The objective of the basin–type project is to obtain the maximum ratio of wetted area to gross land area, commensurate with efficient operation and maintenance (Bianchi and Muckel, 1970). Standby basins commonly are used for continuous recharge projects when other basins are removed from operation for maintenance and rehabilitation. Natural desiccation, scarifying, discing, and excavation are common maintenance methods used to rehabilitate basins.

Advantages of basins include (Richter and Chun, 1959):

1) Basins utilize the maximum area for spreading, with only the tops of the levees being unproductive. This is particularly important where suitable locations are scarce, or land values are extemely high.
2) Irregular and gullied surfaces can be used with a minimum of preparation.
3) Silt–laden waters can be used, particularly if the upper basins are utilized for

desilting and are periodically cleaned.

4) Considerable surface storage capacity is available in basins, which can be used to store a portion of the water from flash floods for later slow percolation into the ground–water reservoir.

5) In general, local materials can be used for construction of dikes and levees.

There are three basic types of ditch and furrow recharge methods: 1) contour, where the ditch follows the ground contour; 2) tree–shaped, where the main canal successively branches into smaller canals and ditches; and 3) lateral, where a series of small ditches extend laterally from the main canal. The ratio of wetted to gross area is usually low, averaging about 10 percent. This method may combine well with the basin method where the natural ground slopes are too steep for economical stepped–basin construction. The width of ditches generally range from 1 to 6 feet, depending on the terrain and flow velocity. An advantage of the ditch system is that the ratio of the perimeter to wetted area is large, thereby permitting more lateral flow than in a basin system. Where infiltration is retarded by substrata of lower hydraulic conductivity than the surface soils, the same total recharge to the ground water may be obtained with a system of ditches, which would occupy far less surface area than a basin system occupying 100 percent of the surface (Bianchi and Muckel, 1970).

Water may be diverted to spread evenly over a large area of relatively flat topography (surface spreading). Canals or ditches are used to release water into the surface spreading area. It is desirable to form a thin sheet of water over the land, moving at low velocity to avoid soil erosion. Highest infiltration rates occur on areas with undisturbed vegetation and soil covering (Todd, 1980). In comparison to other recharge methods, surface spreading costs are low in terms of both land preparation and operation.

Stream channels offer another method of artificial recharge. Infiltration through stream channels is enhanced by increasing 1) the period of time water is available for seepage, and 2) the wetted area of the stream channel (Bianchi and Muckel, 1970). The period of time available is increased by the construction of dams for reservoirs along the stream channels. Increasing the wetted area of a stream channel is accomplished by widening, scarifying, or ditching. Advantages of using stream channels are 1) low land acquisition costs, and 2) recharge occurs over a long, narrow strip, which is an efficient recharge method in areas where shallow layers of low hydraulic conductivity occur.

Pits and shafts are commonly used recharge methods in areas where low hydraulic conductivity surface deposits occur. Because excavation costs can be high, abandoned excavations such as gravel pits are used. Infiltration is enhanced by steep–walled excavations, because sediment clogging is much greater along pit floors than along sidewalls. Shafts are applicable in areas where silt–free water is available and where biologic clogging is minimal. If sediment and biologic clogging are significant, rehabilitation may be prohibitive.

Injection wells are practical where deep, confined aquifers must be recharged, or where economy of space, such as in urban areas, is an important consideration (Todd, 1980). Recharge rates are difficult to maintain because of sediment clogging, bacterial and algae growths, air entrainment, rearrangement of soil particles, and deflocculation caused by reaction of high sodium water with the aquifer matrix. Successful injection wells require water treatment to reduce suspended loads and bacteria and algae growth. In addition, periodic well redevelopment is required.

Three methods of recharge are practical for the Oakes aquifer. These include basins, surface spreading, and canals. The importation of sediment–laden water from the James River to recharge facilities in the Oakes aquifer will require special operation and rehabilitation techniques to maintain adequate infiltration rates. Basins, surface spreading, and canals are best suited for periodic removal of accumulated sediment (filter cake). Depending on long–term infiltration rates, basins and surface spreading can require large land areas. The land in the proposed project area is agricultural and primarily is used for pasture.

Canals may be practical on a limited scale in areas overlying the outwash channel where low hydraulic conductivity surface deposits are between 5 and 10 feet thick. In these areas, perched ground–water mounding may control infiltration rates. To minimize the height of perched ground–water mounds, lateral flow components should be maximized. This is achieved with canals because the ratio of the outside perimeter to the wetted area is large.

The James River is not in direct hydraulic connection with the Oakes aquifer. In addition, there are no intermittent streams overlying the Oakes aquifer. Therefore, stream–channel infiltration is not a practical artificial recharge method for the Oakes aquifer.

There are no major abandoned pits or shafts in the Oakes aquifer study area. Pits and shafts are used to penetrate surficial deposits of low hydraulic conductivity. Low hydraulic conductivity deposits below 5 feet are not widespread in the proposed project area. Maintenance and rehabilitation techniques for pits and shafts generally are cost prohibitive, particularly if sediment–laden water is used for recharge. Therefore, pits and shafts are not practical methods of artificial recharge in the Oakes aquifer.

Basin–Recharge Testing in the Oakes Aquifer

Since 1) basins are the most common method of artificial recharge, 2) basins are practical in the Oakes aquifer, and 3) time and economic constraints precluded an adequate investigation of all practical methods of artifical recharge to the Oakes aquifer, a comprehensive investigation of basin recharge was initiated in August 1986 (Schuh and Shaver, 1988). The purpose of the basin recharge tests was to 1) measure temporal changes in infiltration rate in recharge basins, 2) determine processes that control infiltration rate, and 3) evaluate selected design criteria and operational procedures that enhance infiltration rate.

Five recharge tests were conducted in a 4–foot deep, 50 by 50 foot square basin, located about one–half mile south of Oakes in the SE1/4 SE1/4 of Section 29, Township

131 North, Range 59 West. One recharge test was conducted in a 4–foot deep, 10 by 20 foot rectangular basin, located about 400 feet southeast of the large test basin. James River water was conveyed to the recharge basins for each test using a surface pipeline.

For each test, discharge into the basin and basin stage were measured. Discharge was periodically adjusted to maintain a constant basin stage. Discharge and stage measurements were used to calculate temporal infiltration rates within each basin.

Double–ring infiltrometers and tensiometer nests were installed at selected sites along the basin floor. Prior to basin flooding, short–term infiltration tests were conducted at these sites to develop functional relationships between unsaturated–hydraulic conductivity and moisture content, and to determine saturated–hydraulic conductivity at selected depths below the basin floor. Tensiometric data was periodically collected at the tensiometer nests throughout each recharge test. The tensiometric data and pre–test, saturated– and unsaturated–hydraulic conductivity measurements were used to measure temporal fluctuations in infiltration rate and characterize the growth and extent of clogging.

Physical properties were also measured at selected depths below the basin floor to assess the depth of clogging. USDA texture, wet combustion organic carbon, bulk density, and moisture retention curves were determined for selected tests before and after recharge.

Various operational and maintenance techniques were used to enhance infiltration rate within the 50 by 50 foot square test basin. These included:

1) Desiccation of basin floor.
2) Changing basin stage.
3) Excavation of the clogged surface layer on the basin floor.
4) Placement of an organic mat (partially decomposed sunflower seed hulls) over the basin floor.

The following general conclusions for basin recharge to the Oakes aquifer were determined from the basin–recharge tests.

1) Under conditions of turbid–water (James River) infiltration through a natural sand filter (Oakes aquifer), most of the clogging will occur in the top 3 inches. Within the top 3 inches, the greatest degree of clogging will occur in the surface filter cake that is about .004 to .008 inch thick.

2) Some sediment penetration and attenuation of hydraulic conductivity will occur as deep as 15 inches.

3) Partial and temporary recovery of infiltration capacities can be obtained by allowing the basin surface to dry and crack for periods of about two weeks or more. Initial recoveries of up to 73 percent of the fully renovated infiltration rate may be effected. The rate of basin resealing, however, will be much greater than for a fully renovated basin and, within about 10 days, the basin will require additional renovation.

4) An organic–mat filter can substantially increase the total recharge accomplished through a basin within a single operational period. However, deeper penetration of clay into the subbasin sand profile will occur using organic mats.

5) Generally, the infiltration rate response to ponded head depth will be least during the early phases of infiltration where the largest rates occur, and will be greatest following the formation of the surface crust, when infiltration is the slowest. At later times, after the formation of the filter–cake layer, infiltration increases of 60 percent or more may be achieved by doubling the ponded depth for a 2–foot deep ponded basin.

6) During turbid–water infiltration in shallow basins in the Oakes aquifer, perched, ground–water mounds may be the principal infiltration–rate control during early time (24 to 48 hours). After this time, the impedance layer developed along the basin floor from turbid–water infiltration will probably become the principal infiltration–rate control.

Proposed Location of Surface Recharge Facilities

Infiltration tests have been conducted at five sites in the Oakes aquifer study area (fig. 7) (Shaver and Schuh, 1989). Three sites are located along the eastern part of the Oakes aquifer overlying the glacial–outwash channel. The other two sites are located in the central part of the Oakes aquifer overlying lacustrine sand deposits. Basins were excavated to a depth of about 5 feet at sites I–1, I–2, I–3, and I–4. Infiltration rates using non–turbid ground water were determined with double–ring infiltrometers installed along the basin floors. Surface–infiltration rates, using non–turbid ground water, were also determined with double–ring infiltrometers at sites I–4, and I–5. Surface and basin steady–state infiltration rates measured at all sites are shown in table 6.

In aquifers characterized by small saturated thicknesses (less than 150 feet), artificial recharge facilities should be located near the recovery wells to minimize dewatering and maintain adequate well yields. It is apparent from the infiltration–test data that the area with the highest basin infiltration rate is in the central part of the lake plain overlying the lacustrine sands (T. 130 N., R. 59 W., less Sections 1 through 6). In this area, individual well yields are, for the most part, less than 500 gallons per minute (fig. 8). Average initial saturated thickness is about 35 feet. A project–scale irrigation and recharge development in this area would require about 90 recovery wells and a large number of small–scale, surface–recharge facilities located near the wells. Thus, a project–scale irrigation and recharge project in the central part of the Oakes aquifer is not practical.

The area most feasible for the development of a project–scale well field is near Section 13, Township 129 North, Range 59 West (fig. 3). This area overlies a glacial outwash channel. The estimated average saturated thickness of the channel–fill deposits in this area is 120 feet. Because of the relatively small saturated thickness (less than 150 feet thick), surface–recharge facilities must be located near the well field.

A surface/near surface fluvial silt and clay deposit overlies part of the glacial

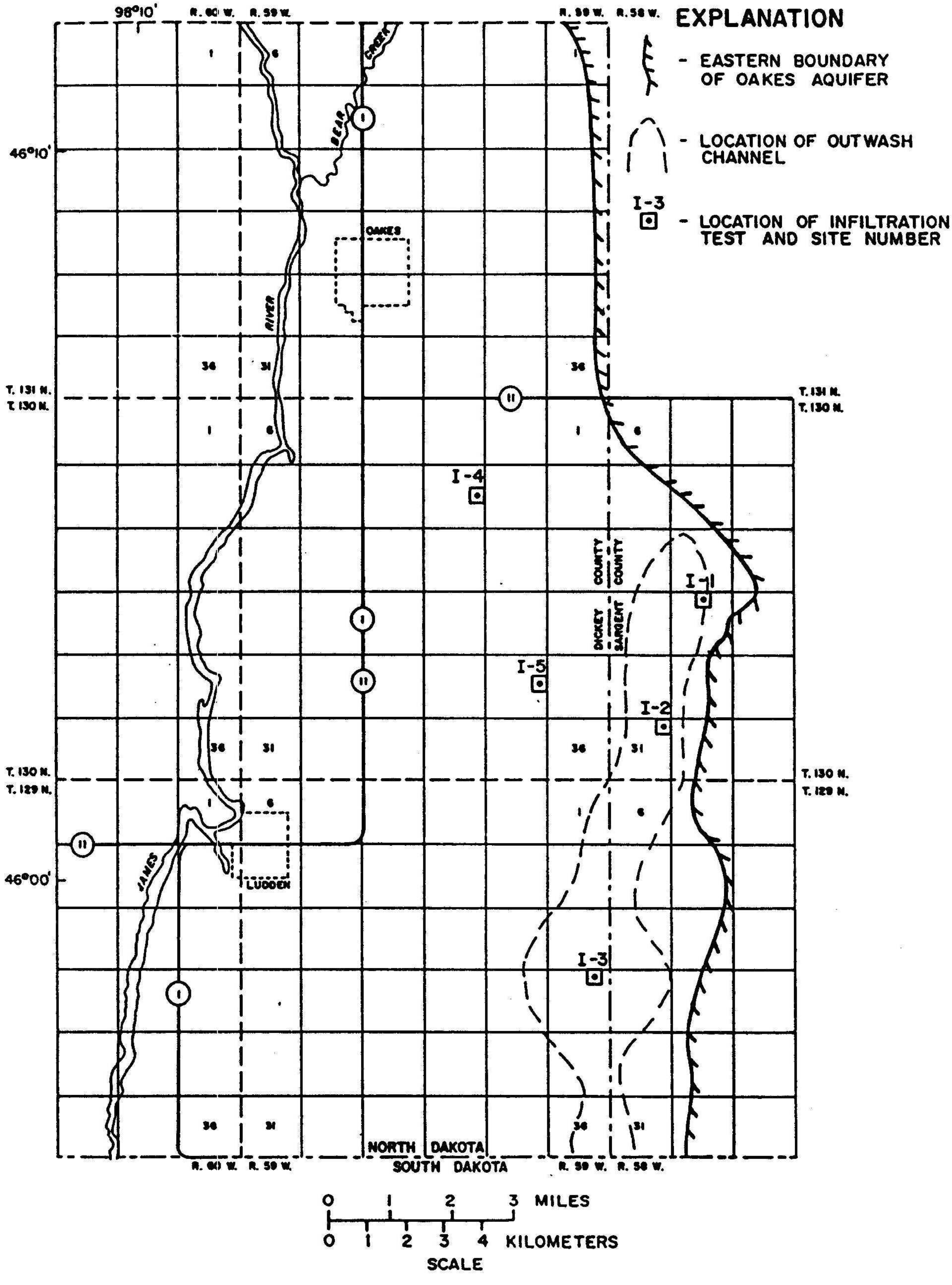

Figure 7.--Location of infiltration-test sites

Table 6. - - Surface and basin steady–state infiltration rates measured at selected sites in the Oakes aquifer study area

Site number			Infiltration rate, in feet per day
IR–1			20
IR–2			7
IR–3			2.5
IR–4	Surface		
		Hecla*	10
		Ulen*	3.6
		Arveson*	26
	Basin		
		Hecla*	57
		Ulen *	67
		Arveson*	26
IR–5	Surface		
		Hecla*	15
		Hecla*	8.3
		Hamar*	12.9

*Soil Association

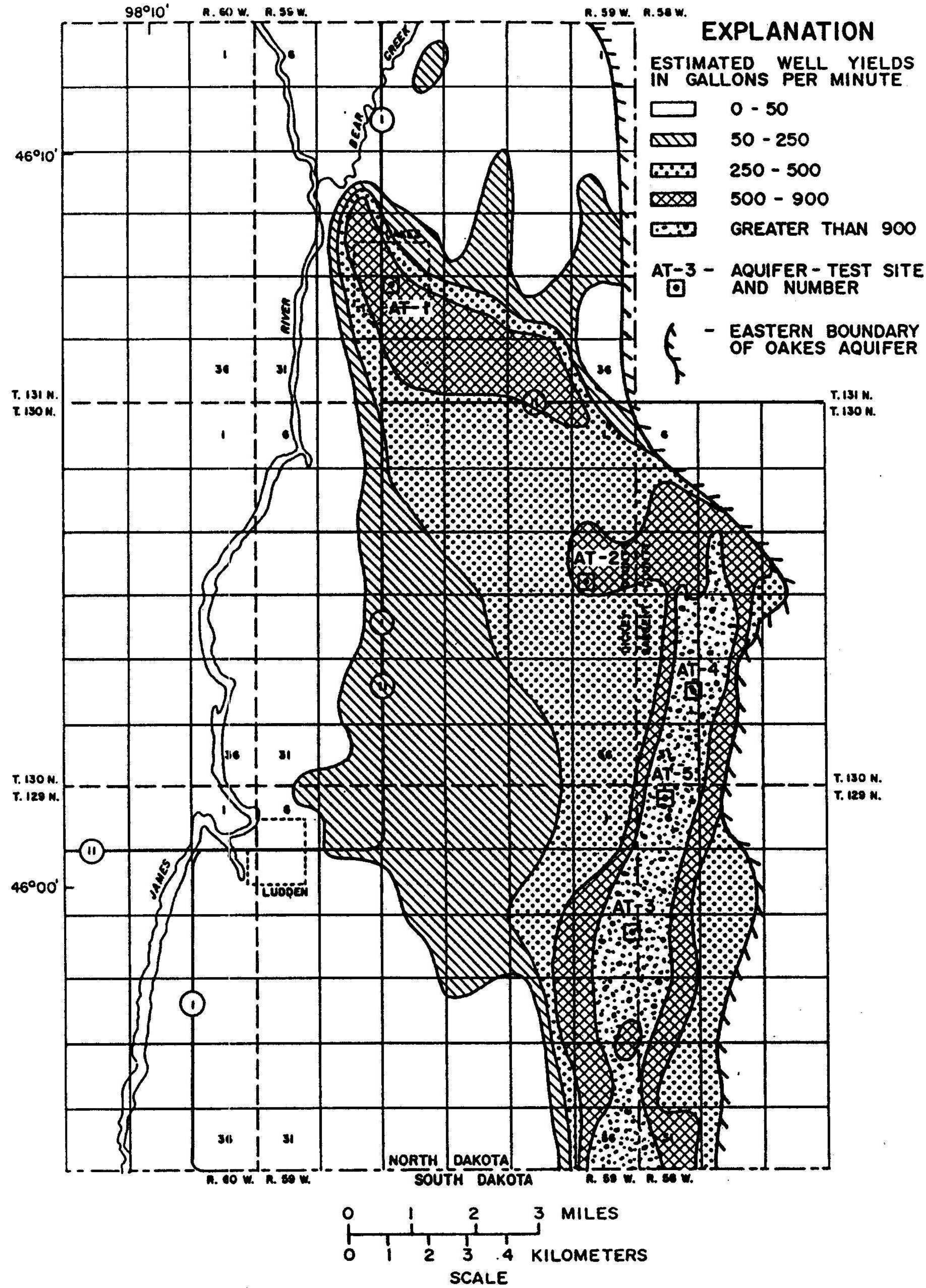

Figure 8.--Estimated well yields in the Oakes aquifer

outwash channel (fig. 9). Near the area of the proposed well field, the thickness of the surficial silt and clay layer ranges from less than 1 foot to about 30 feet. Based on excavation costs, basins probably will not exceed a depth of 5 feet. Therefore, recharge basins and/or surface–spreading sites will not be located at the center of the well field. Preliminary test–drilling data indicates that the recharge facilities will be located near the southern part of the well field where the surficial silt and clay layer is less than 5 feet thick (fig. 10).

FINITE–DIFFERENCE MODEL OF GROUND–WATER FLOW

Previous Work

A finite–difference model of ground–water flow in the Oakes aquifer was developed by the North Dakota State Water Commission in 1981 (R.B. Shaver, written communication, 1986). The model was developed for use as a management tool to allocate ground water primarily for irrigation. A U.S. Geological Survey two–dimensional, finite–difference model (Trescott, Pinder, and Larson, 1976) was utilized. Various combinations of recharge and evapotranspiration rates produced very similar water–table configurations during steady–state calibration. Steady–state simulations were insensitive to changes in hydraulic conductivity.

The model was calibrated against water levels measured by the U.S. Bureau of Reclamation from 1967 to 1981. Potential ground–water evapotranspiration was computed externally from the model by subtracting monthly precipitation from monthly potential evapotranspiration calculated by a modified Jensen–Haise method. An assumption of this approach is that most summer precipitation events do not contribute to ground–water recharge. Minor adjustments to the above monthly ground–water evapotranspiration calculations were made in the model during calibration to account for occasional summer recharge events. The average annual potential ground–water evapotranspiration, calculated for the calibration period, was 13 inches. Potential ground–water

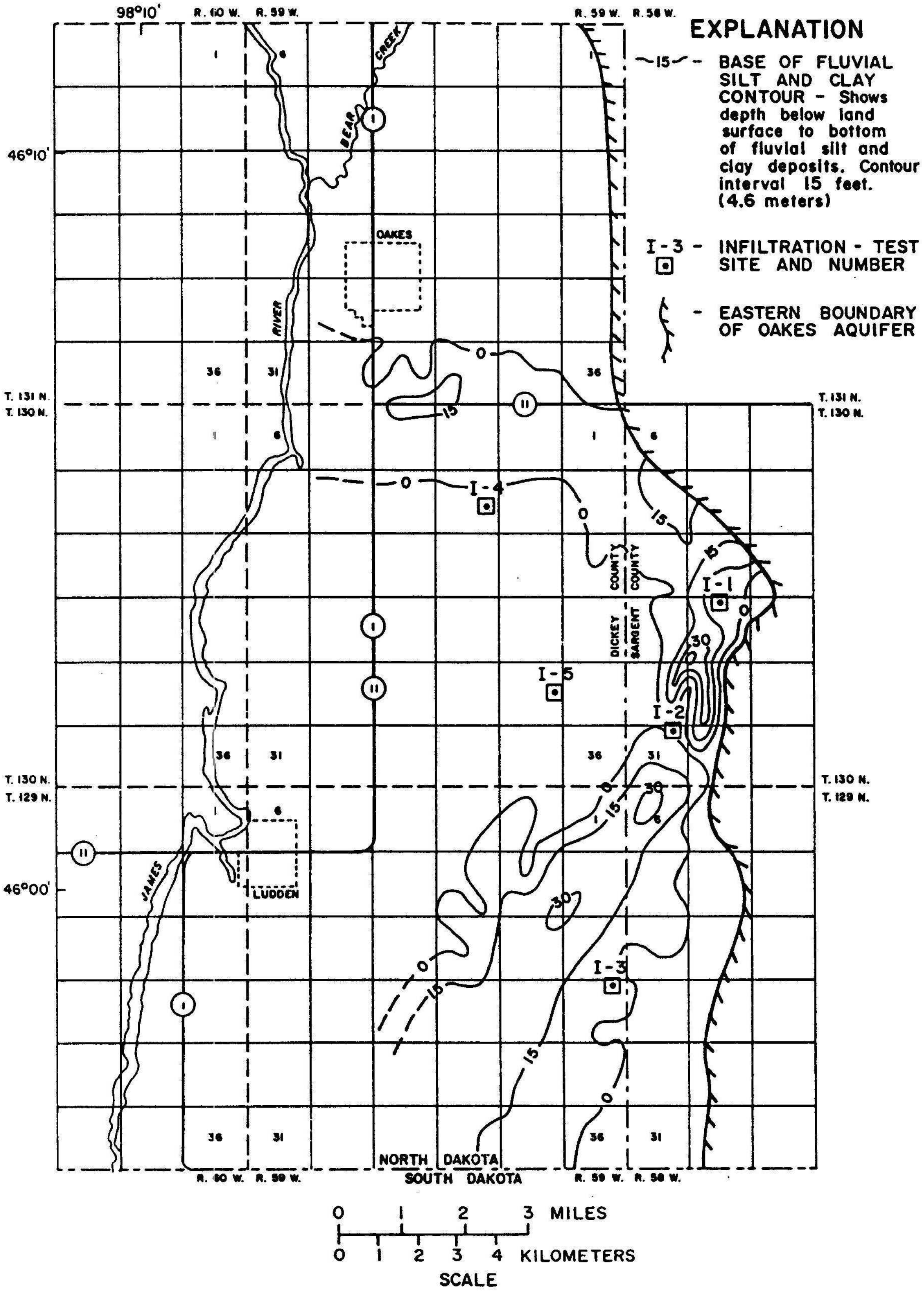

Figure 9.--Depth below land surface to bottom of surficial fluvial silt and clay deposits

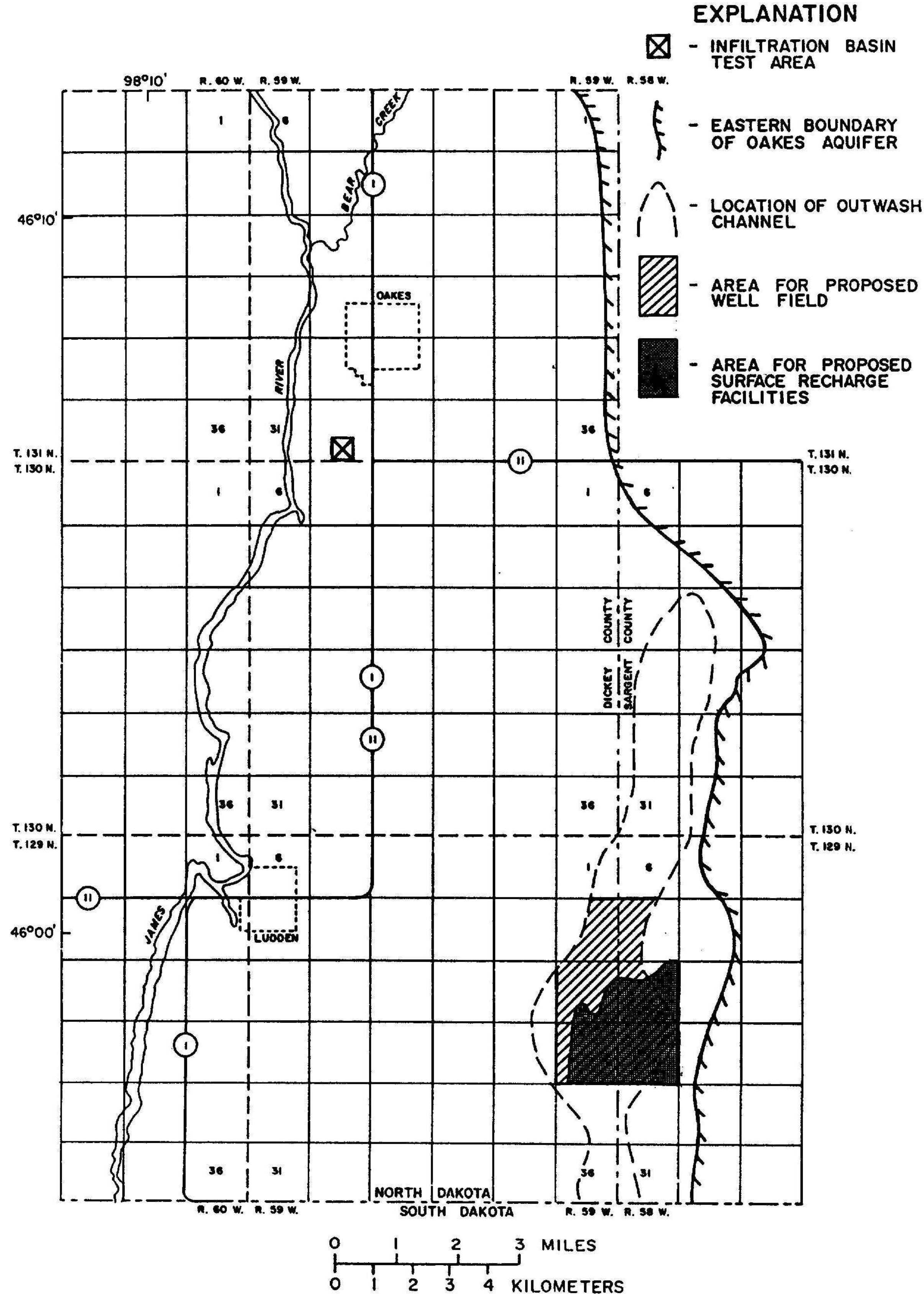

Figure 10.--Location of proposed well field and surface-recharge facilities

evapotranspiration was 100 percent of the maximum specified rate at land surface and was assumed to decrease linearly to zero at a depth of 8 feet below land surface.

Recharge was calculated within the model as the product of an assumed specific yield and the amount of water required to replicate observed change in storage. The average annual recharge rate calculated for the calibration period was 4.4 inches.

Various combinations of recharge, evapotranspiration, and specific yield produced equal water–level fluctuations. The model could not be utilized to calculate annual recharge and evapotranspiration rates because of this nonuniqueness. Both recharge and evapotranspiration must be determined externally from the model. As a result, the model proved inadequate as a long–term predictive management tool.

Model Description

The finite–difference ground–water flow model developed by the U.S. Geological Survey was utilized in this study (McDonald and Harbaugh, 1984). The model determines the approximate solution, in two dimensions, to the following partial differential equation for ground–water flow:

$$\frac{\partial}{\partial x}\left(Kxx\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(Kyy\frac{\partial h}{\partial y}\right)-W = Sy\frac{\partial h}{\partial t} \qquad (1)$$

where,

x and y are cartesian coordinates aligned along the major axes of hydraulic conductivity Kxx, Kyy;

h is the potentiometric head (L);

W is a volumetric flux per unit volume and represents sources or sinks of water or both (t^{-1});

Sy is the specific yield of the porous material; and

t is time (t).

Ground–water flow within the aquifer is simulated using a block–centered, finite–difference approach. The continuous system described by equation 1 is replaced by a finite set of discrete points in space and time, and the partial derivatives are approximated by differences between functional values at these points. The process leads to systems of simultaneous non–linear algebraic difference equations. The solutions to the systems of simultaneous equations yield values of head at specific points and time. The finite–difference equations can be solved using either the Strongly Implicit Procedure or Slice–Successive Overrelaxation (McDonald and Harbaugh, 1984). The Strongly Implicit Procedure was selected for this investigation.

Model Development

Boundary Conditions

The Oakes aquifer was discretized into 6,592 blocks (103 rows by 64 columns; Shaver and Schuh, 1989). The grid is variably spaced and the block dimensions range from 500 by 500 feet to 1,000 by 1,000 feet. Input for the steady–state simulation consisted of areally nonuniform and uniform parameters. For the areally nonuniform parameters, the average value within each block was assigned to that block. Nonuniform parameters were:

1) Starting head
2) Altitude of base of aquifer
3) Altitude of top of aquifer
4) Hydraulic conductivity
5) Land–surface altitude
6) Recharge rate

The starting head array consists of water levels measured during May 1984 in 212 U.S. Bureau of Reclamation and North Dakota State Water Commission observation wells completed in the Oakes aquifer. The array for the altitude of the base of the aquifer was determined by subtracting aquifer thickness from land–surface altitude. The array for the altitude of the top of the aquifer was determined by subtracting the thickness of the surface–near surface fluvial silt and clay layer from land–surface altitude. The hydraulic conductivity array was developed from aquifer–test, specific–capacity, and grain–size analysis data. A generalized land–surface altitude map was used to develop the land–surface altitude array. Based on the finite–difference model of the Oakes aquifer, developed by the North Dakota State Water Commission in 1981, a conservative annual average recharge rate of 3 inches was selected for the steady–state simulation. Recharge was set at 3 inches in areas where the surficial fluvial silt and clay layer was absent. Recharge was set at zero where the surficial fluvial silt and clay layer occurred.

Areally uniform parameters were:

1) Evapotranspiration rate

2) Evapotranspiration extinction depth

Based on the finite–difference model of the Oakes aquifer, developed by the North Dakota State Water Commission in 1981, an annual potential ground–water evapotranspiration rate of 13 inches, with an extinction depth of 8 feet, were selected for the steady–state simulation.

The model simulates evapotranspiration by a simple linear decay function. The evapotranspiration rate is 100 percent of the maximum specified rate at land surface and decreases linearly to zero at a specified evapotranspiration extinction depth.

The eastern margin of the aquifer was treated as a no–flow boundary. The no–flow boundary appears to be a valid assumption because the moraine consists of small–transmissivity deposits. The northern and southern boundaries of the model also are treated as no–flow boundaries. The Oakes aquifer extends beyond the northern and southern boundaries of the model. Under steady–state conditions, ground–water flow is, for the most part, parallel to the northern and southern boundaries of the model and, as a result, the no–flow boundary assumption appears to be valid. The northern and southern no–flow boundaries are located far enough from projected stress areas to avoid image–well effects on the drawdown distribution. The James River comprises most of the western boundary of the model and was treated as a constant head boundary.

From November, 1986 through February, 1987, the U.S. Bureau of Reclamation completed 60 test holes in the southeast part of the study area to determine the thickness and areal extent of the surface fluvial silt and clay layer that overlies the Oakes aquifer. A truck–mounted, solid–stem, spiral auger was used to drill all test holes. The maximum depth of the test holes was 33 feet. The areal extent and depth to bottom of the surficial silt and clay layer is shown in figure 9.

In areas where water levels are above the base of the silt and clay layer, the model treated the aquifer as confined with a storativity of .0004. In areas where the water levels dropped below the base of the silt and clay layer and in areas where the silt and clay layer is absent, the model treated the aquifer as unconfined with a storativity of 0.20.

The surficial silt and clay layer is a leaky confined layer as determined from aquifer testing completed in Phase 1 of this artificial recharge feasibility study (Shaver and Schuh, 1989). However, it was not possible to determine the hydraulic properties (hydraulic conductivity, storativity) of the silt and clay from the aquifer–test data. Because the hydraulic properties of the silt and clay layer are unknown and because the amount of water in storage in the silt and clay layer is small, relative to the amount of water in storage in the underlying aquifer, the silt and clay layer was treated as non–leaky.

Steady–State Simulation

A volumetric water budget for the steady–state simulation is shown in table 7. Ground–water discharge is primarily from evapotranspiration and discharge to the James River is minor.

Water levels from the steady–state simulation were compared to water levels measured during May 1984 in 212 U.S. Bureau of Reclamation and North Dakota State Water Commission observation wells completed in the Oakes aquifer. The average absolute difference between simulated and measured water levels in the 212 wells is 2.2 feet. In the northern and north–central parts of the model area, simulated water levels

Table 7. - - Volumetric water budget for the Oakes aquifer steady-state computer simulation

Volumetric budget for entire model at end of time step 1 in stress period 1

Cumulative volumes	Cubic feet	Rates for this time step	Cubic feet per day
<u>IN:</u>		<u>IN:</u>	
Storage =	0.00000E+00	Storage =	0.00000E+00
Constant head =	4,380.7	Constant head =	4,380.7
Recharge =	.23528E+07	Recharge =	.23528E+07
Evapotranspiration =	.00000E+00	Evapotranspiration	.00000E+00
Total IN =	.23572E+07	Total IN =	.23572E+07
<u>OUT:</u>		<u>OUT:</u>	
Storage =	.00000E+00	Storage =	0.00000E+00
Constant head =	7,655.3	Constant head =	7,655.3
Recharge =	.00000E+00	Recharge =	.00000E+00
Evapotranspiration =	.23509E+07	Evapotranspiration =	.23509E+07
Total OUT =	.23586E+07	Total OUT =	.23586E+07
IN - OUT =	-1,346.0	IN - OUT =	-1,346.0
Percent discrepancy =	-.06	Percent discrepancy	-.06

were higher than measured water levels. The U.S. Bureau of Reclamation installed a pilot drain adjacent to the northern boundary of the model area in 1969. In 1983, the U.S. Bureau of Reclamation began construction of a drain network in the central part of the study area. Since the mid 1970s, ground–water withdrawals for irrigation in the north–central part of the model area have increased significantly. The area of influence of the drains and most irrigation development is in the north–central part of the model area. The best potential for large–scale, ground–water withdrawals and artificial recharge is in the southeast part of the aquifer within the outwash channel. This area of the aquifer is outside the area of influence of the drains and irrigation development. As a result, ground–water discharge from the drains and irrigation development was ignored.

In the southeastern part of the model area, simulated water levels also were higher than actual measured water levels. Numerous topographic depressions occur in this part of the model area. The depressions were poorly approximated by the land–surface altitude array and grid size.

The steady–state model adequately approximates the geometry and hydraulic conductivity of the outwash channel near Sec. 13, T. 129 N., R. 59 W. Therefore, the model can be used as a short–term (1 to 3 years) predictive tool in this area of the Oakes aquifer. For these predictive simulations, long–term average recharge and evapotranspiration rates are not required.

The computer model of the Oakes aquifer was used in this study to:

1) Develop a preliminary design of a project–scale well field.
2) Estimate the effects on water levels in the aquifer of a continuous withdrawal of 100 cubic feet per second for 60 days from a project–scale well field.
3) Estimate the effects on water levels in the aquifer of a continuous withdrawal of 60 cubic feet per second for 34 days from a pilot–scale well field.
4) Estimate the effects on water levels in the aquifer of a continuous withdrawal of 70 cubic feet per second for 60 days from a project–scale well field,

operating in conjunction with artificial recharge facilities supplying a continuous rate of 35 cubic feet per second for 120 days (60 days in spring, 60 days in fall).

PILOT–SCALE WELL FIELD AND ARTIFICIAL–RECHARGE FACILITIES

Justification

Pilot–scale artificial ground–water recharge studies are prerequisite to the development of full–scale artificial recharge projects. According to Bouwer (1988), "There are hundreds of successful artificial ground–water recharge projects in the United States alone, and many more in the rest of the world. Recharge systems are site specific and what works well in one place may not be the best in another. Thus, when artificial recharge of ground water is considered in areas where there is no previous experience with such systems, it is always desirable to start with a small project to obtain local experience with artificial recharge of ground water and then develop design and management criteria for the full–scale project. This prevents costly mistakes and can save large amounts of money later on." Due to the lack of practical experience in the operation of artificial–recharge facilities in the Oakes aquifer project area, a pilot–scale well field and artificial–recharge test program are recommended.

The water table in the project area generally is less than about 5 feet below land surface. Computer simulations (to be described later in this report) indicate that, because of shallow water–table conditions, the aquifer will have to be evacuated in advance of pilot–scale artificial recharge operations. In addition, the pilot–scale well field must be designed to sufficiently dewater the aquifer during abnormally wet climatic cycles characterized by large annual spring recharge events. For example, in the spring of 1986, aquifer–wide water levels rose about 5 to 6 feet in response to abnormally large snowmelt and precipitation events. A water–table fluctuation of this magnitude should be prevented in the pilot recharge area to avoid intersection of ground–water mounds with the base of surface or basin recharge–test facilities. Based on historic climatic conditions, operation of the pilot–scale well field may be required for up to three consecutive years to sufficiently dewater the aquifer and begin artificial recharge testing. The pilot–scale well field should consist of enough wells to insure adequate dewatering of the aquifer if one or more wells becomes temporarily inoperable.

Discharge water from the well field can be diverted by pipeline or canal west to the James River. This water would not be applied to beneficial use. Another option would be to divert discharge water by pipeline or canal north to provide an interim supplemental irrigation supply for the 5,000–acre test plot in the West Oakes irrigation development tract south of Oakes. The Bureau of Reclamation does not have sufficient water available from the Jamestown reservoir to irrigate the entire 5,000–acre test plot, and a supplemental annual water supply of about 4,000 acre–feet to irrigate 3,500 acres in the 5,000–acre test plot is needed. The well field can be used to provide a supplemental water supply. This option is more favorable than disposal of discharge water to the James River because water would be applied to beneficial use. In addition, this allows the Bureau of Reclamation to proceed in a timely manner with full–scale operation of the 5,000–acre test plot as mandated by Congress.

The utility of the pilot–scale well field will not terminate after pilot–scale, artificial–recharge testing is completed. Additional wells will be incorporated into the pilot–scale well field to manage water–table depth for the full–scale irrigation and artificial recharge project.

It is important that the pilot–scale well field and recharge test facilities be completed and operated in a timely manner to avoid delays when Missouri River water is delivered to the project area. The pilot–scale, recharge–test facilities should be operated for about 5 to 6 years to evaluate the most efficient and cost effective method of artificial recharge and to identify how the Oakes aquifer recharge program would or would not fit into full project development in the Oakes area. To avoid delays, preliminary field studies, including aquifer and aquitard exploratory drilling, should be initiated during the fall of 1988 or spring of 1989.

The irrigators in the Garrision Diversion Unit must pay operation and maintenance costs of project facilities including canals, drains, wells, and recharge basins. It is necessary to establish operation and maintenance costs of these facilities so that irrigators can plan for the costs of using project water. The operation and evaluation of a pilot–scale well field and recharge facilities will be the basis for developing more accurate estimates of operation and maintenance costs.

Computer simulations of both pilot–and project–scale well field and artificial–recharge systems indicate the water table will be lowered in and around the project area. Adverse effects caused by the changed water–table condition may include loss of stock ponds, stock and/or domestic wells, sub–irrigation, and wetlands. Sub–irrigation occurs when plant roots extend downward to the capillary fringe or water table. There is a significant amount of crop and pasture land that is subirrigated in the project area. In wet years, particularly during the spring, parts of the project area are ponded with water and/or the soils are affected by water logging. These conditions reduce crop yield. A net benefit from lowering the water table in the project area will be a reduction in ponding and water logging of soils. The operation of a pilot–scale well field and artificial–recharge test facilities will provide the basis for evaluating and predicting effects on the water table resulting from project–scale development.

Pilot–Scale Well Field Computer Simulation

A pilot–scale well field was simulated in the SE1/4 of Section 13, T. 129 N., R. 59 W. The purpose of this simulation was to estimate the residual drawdown at the end of a 34–day peak irrigation period and the residual drawdown for the following spring after 214 days of recovery. Nine wells, spaced 1,000 feet apart, were placed along the central axis of the outwash channel (fig. 11). The model was used to simulate the annual withdrawal of 4,057 acre–feet of water at a continuous pumping rate of 60 cubic feet per second (27,000

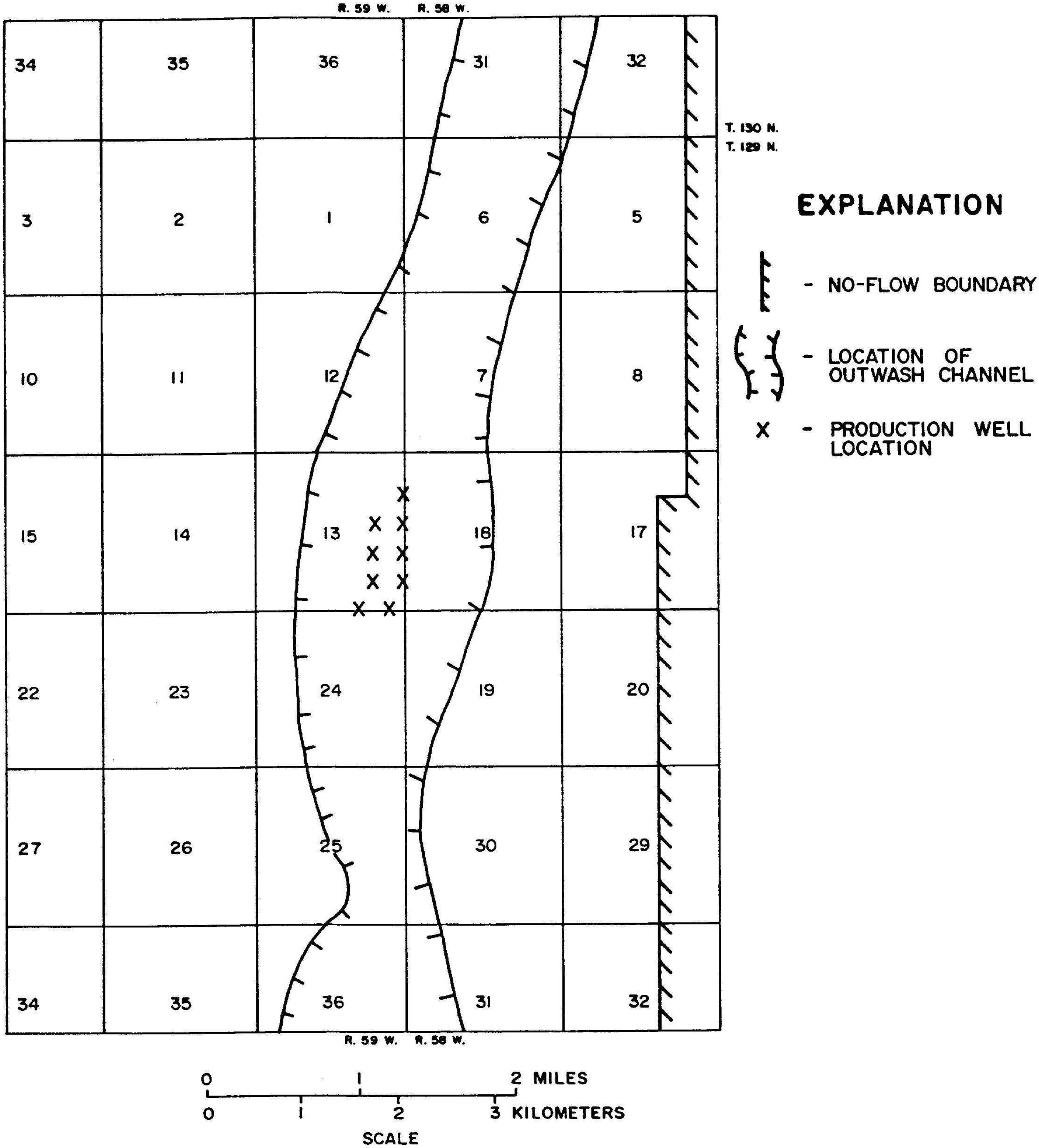

Figure 11.--Location of pilot-scale well field

gallons per minute/3,000 gallons per minute per well) for 34 days.

Based on the channel capacity of the James River, surplus water for artificial recharge during project–scale operation will only be available during the spring and fall. Therefore, pilot–scale surface and basin recharge–test facilities must be operated during the spring and fall to evaluate physical, chemical, and biologic conditions characteristic of these periods.

It is anticipated the well field may have to operate for up to three consecutive irrigation seasons before pilot–scale artificial recharge facilities are constructed, instrumented, and ready for operation. Therefore, the model simulated the annual withdrawal of 4,057 acre–feet of water at a continuous pumping rate of 27,000 gallons per minute for 34 days for three consecutive years with no artificial recharge.

Each year was divided into three periods. The first period was a pumping period in which nine wells were pumped continuously for 34 days at a rate of 3,000 gallons per minute per well (4,057 acre–feet). Ground–water evapotranspiration was 4.4 inches and the recharge rate was zero. The purpose of the first pumping period was to estimate the drawdown distribution at the end of each irrigation season prior to a potential fall–recharge period.

The second period was a 214–day recovery period that ended in the spring. Water levels were allowed to recover during this period with no pumping wells in operation. Ground water evapotranspiration was 2.7 inches and the recharge rate was zero. The purpose of this recovery period was to estimate the residual drawdown distribution prior to a potential spring–recharge period.

The third period was for 117 days and ended during early summer, prior to the next 34–day summer pumping period. Discharge from the pumping wells was zero. Ground–water evapotranspiration was 8.2 inches and the recharge rate was 3 inches. The purpose of this period was to simulate the natural spring–recharge event and to estimate residual drawdown prior to the next pumping period.

An operational summary of this simulation is shown in table 8. The drawdown distributions at the end of stress periods 1, 2, 4, 5, 7, and 8 are shown in figures 12–17. These figures can be used to provide a preliminary assessment of the effects of lowering the water table (due to pilot–scale development) on the other ground–water appropriators. The 2– and 4–foot drawdown contours are truncated at the northern boundary of the maps shown in figures 13, 15, and 17. These contours are oriented along the axis of the north–south trending surface to near surface silty–clay channel (fig. 9). Natural recharge was set to zero (worst case) in areas where the silty–clay channel is located. As a result, much of the drawdown north of the project area is the result of a lack of natural recharge and not strictly due to pumping from the well field.

Results of the pilot–scale well–field simulation indicate the Oakes aquifer near the SE1/4 of Section 13, T. 129 N., R. 59 W. can support the annual withdrawal of 4,057 acre–feet of water for at least three consecutive years without artificial recharge. It is also apparent the annual withdrawal of 4,057 acre–feet of water for three consecutive years may not sufficiently dewater the aquifer to accommodate full pilot–scale artificial recharge testing. The amount of residual drawdown (dewatering) may be substantially less than calculated if the pilot–scale well field is operated during abnormally wet years like 1986. Therefore, the operation of the pilot–scale well field should be flexible enough to offset large water–table fluctuations caused by abnormally wet climatic cycles.

Artificial–recharge facilities were not included in the previous simulation because it is premature to determine all artificial–recharge options that may be tested during the pilot study. It is not possible to accurately estimate long–term average infiltration rates characteristic of each recharge option. In addition, for the pilot study, it is not necessary or desirable to replace the exact amount of water withdrawn for irrigation during the two recharge periods immediately following in the fall and spring. As seen in figure 13, the residual drawdown in the spring, resulting from pumping during the previous summer, is less than 6 feet at the well field. Water–table mounds would probably intersect the base of

Table 8. - - Operational summary of pilot–scale well field computer simulation

Stress Period Number	Length of Stress Period, in Days	Operation	Volume of Water Pumped, in Acre–Feet
1	34	pump wells	4,057
2	214	recovery	0
3	117	recovery	0
4	34	pump wells	4,057
5	214	recovery	0
6	117	recovery	0
7	34	pump wells	4,057
8	214	recovery	0

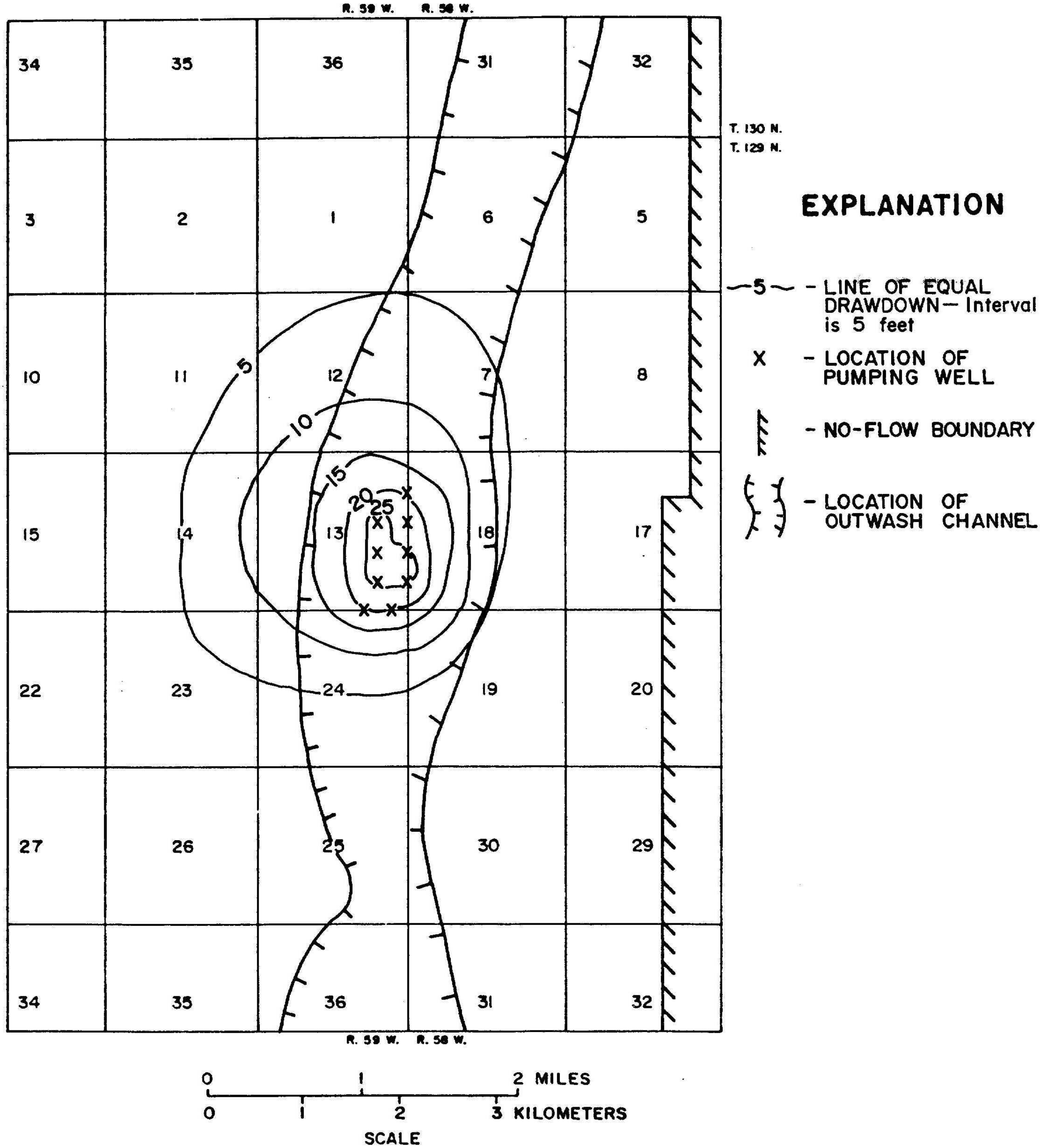

Figure 12.--Pilot-scale well field computer simulation: drawdown distribution at the end of stress period #1

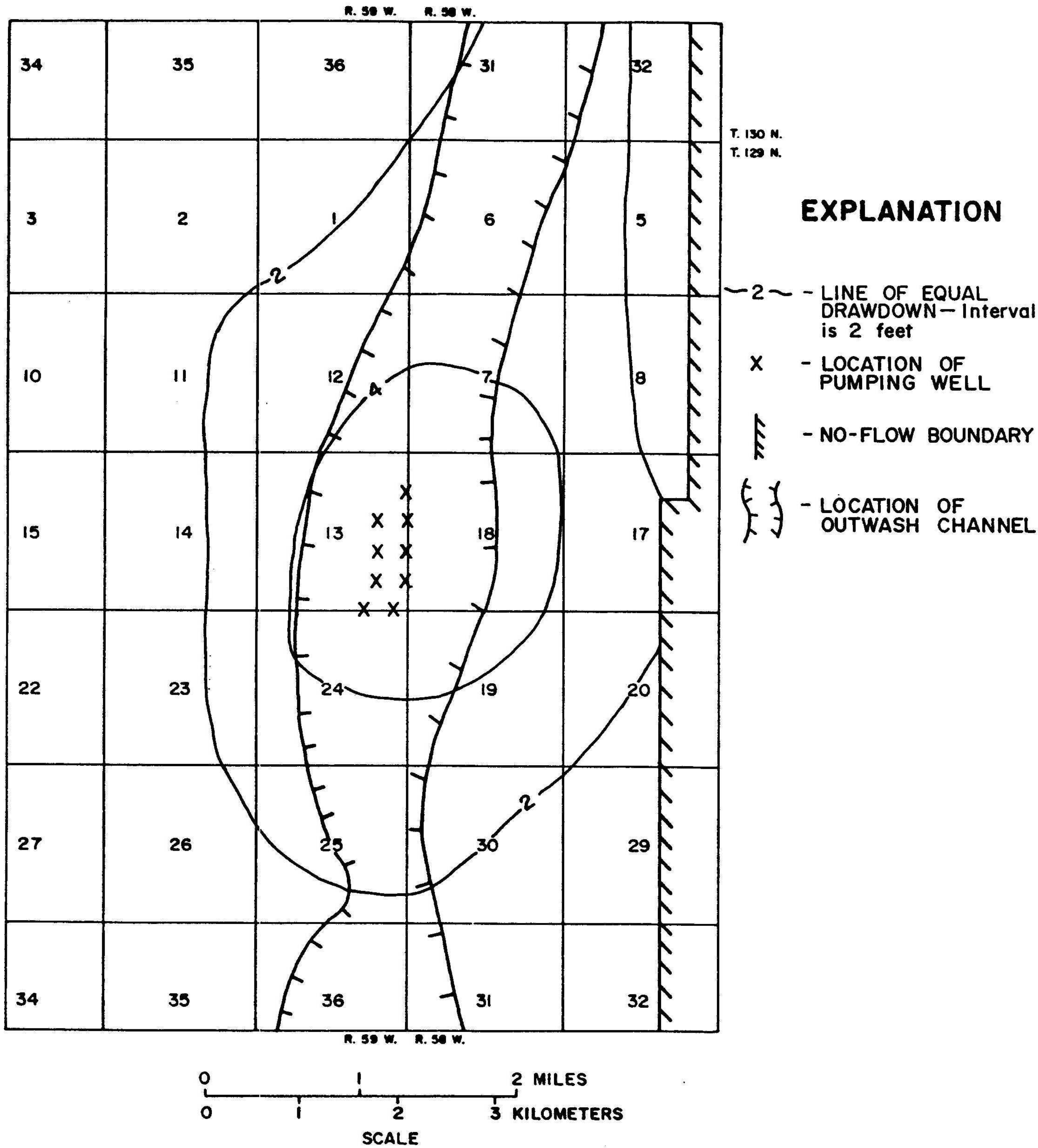

Figure 13.--Pilot-scale well field computer simulation: drawdown distribution at the end of stress period #2

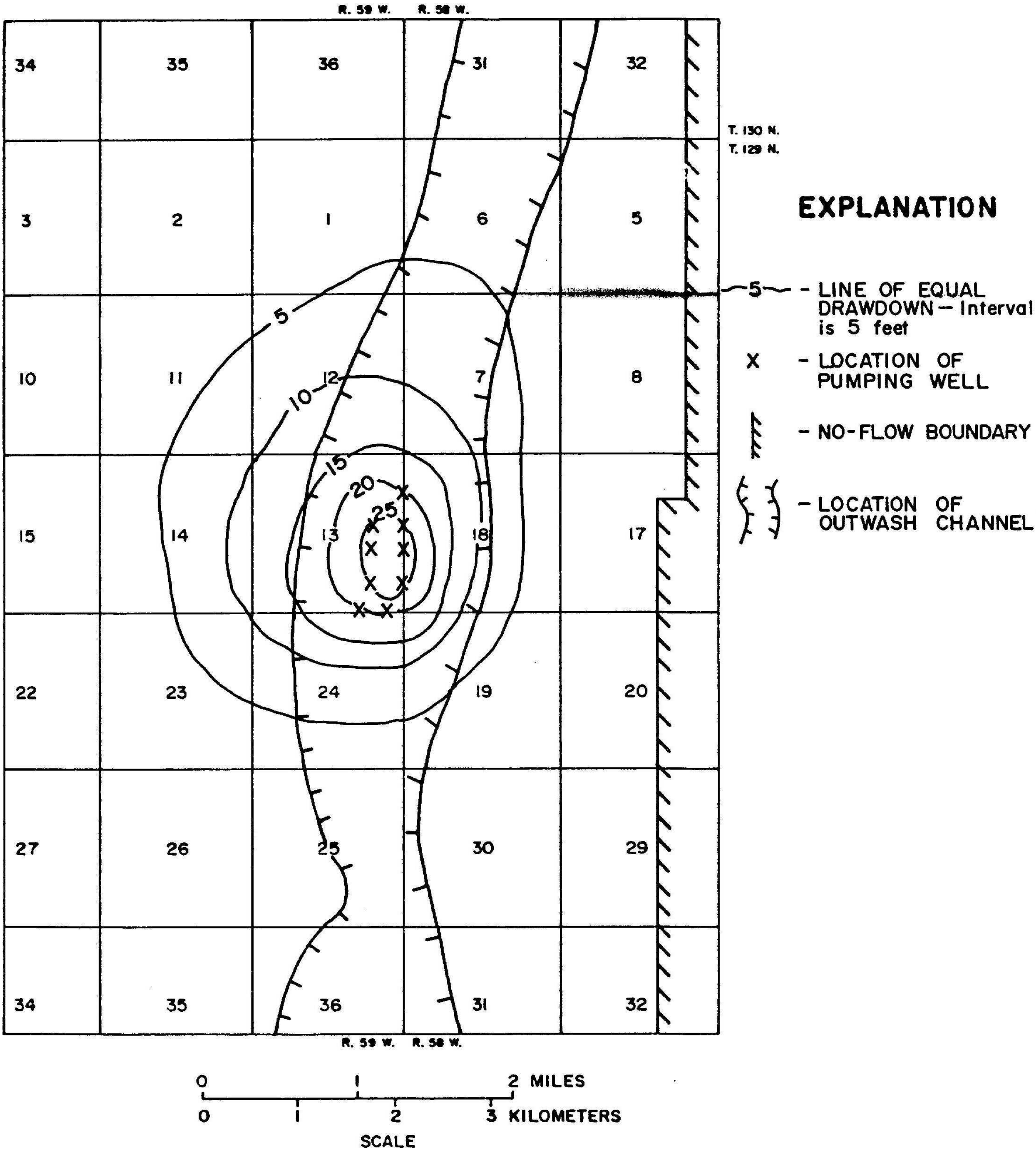

Figure 14.--Pilot-scale well field computer simulation: drawdown distribution at the end of stress #4

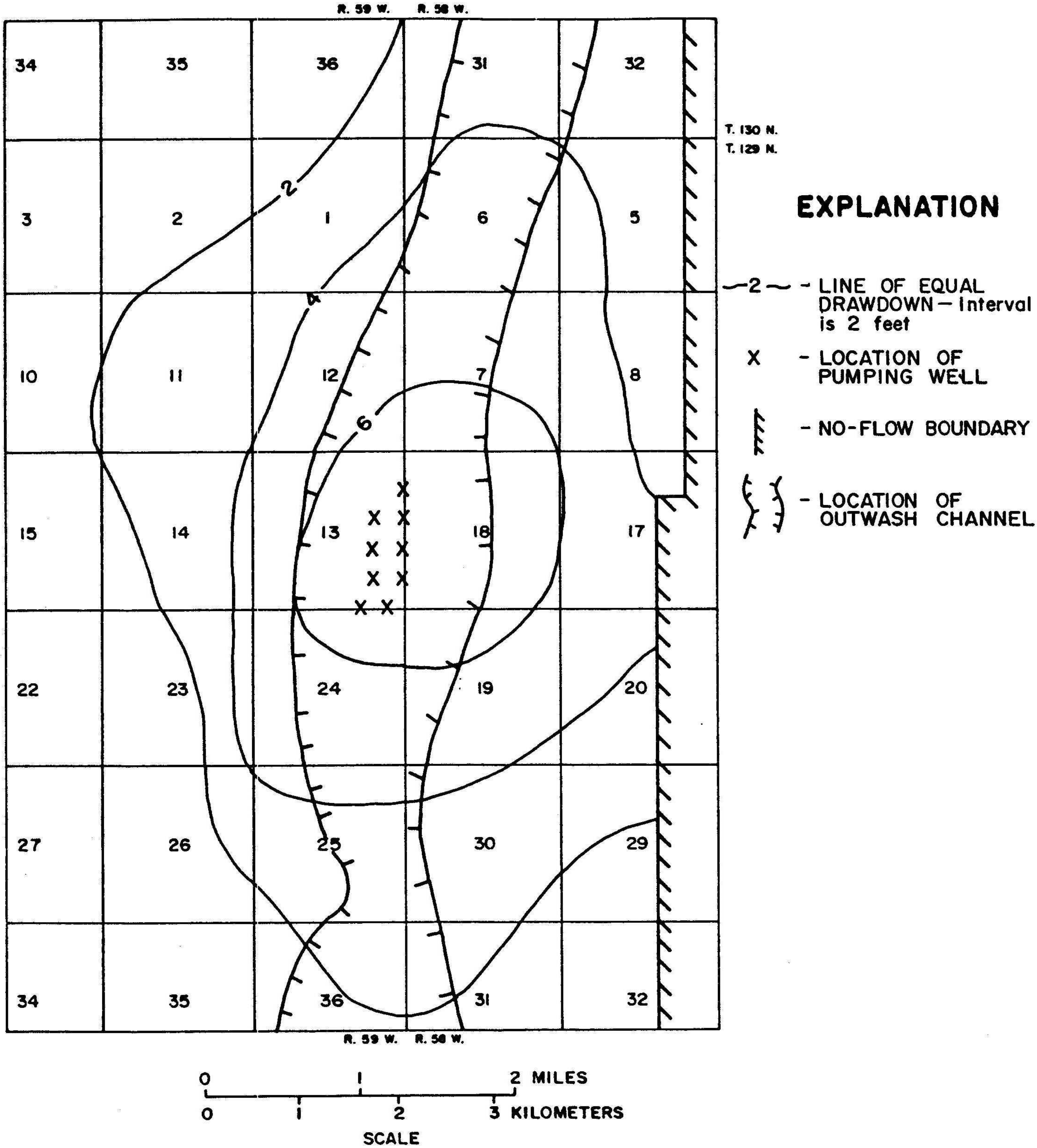

Figure 15.--Pilot-scale well field computer simulation: drawdown distribution at the end of stress period #5

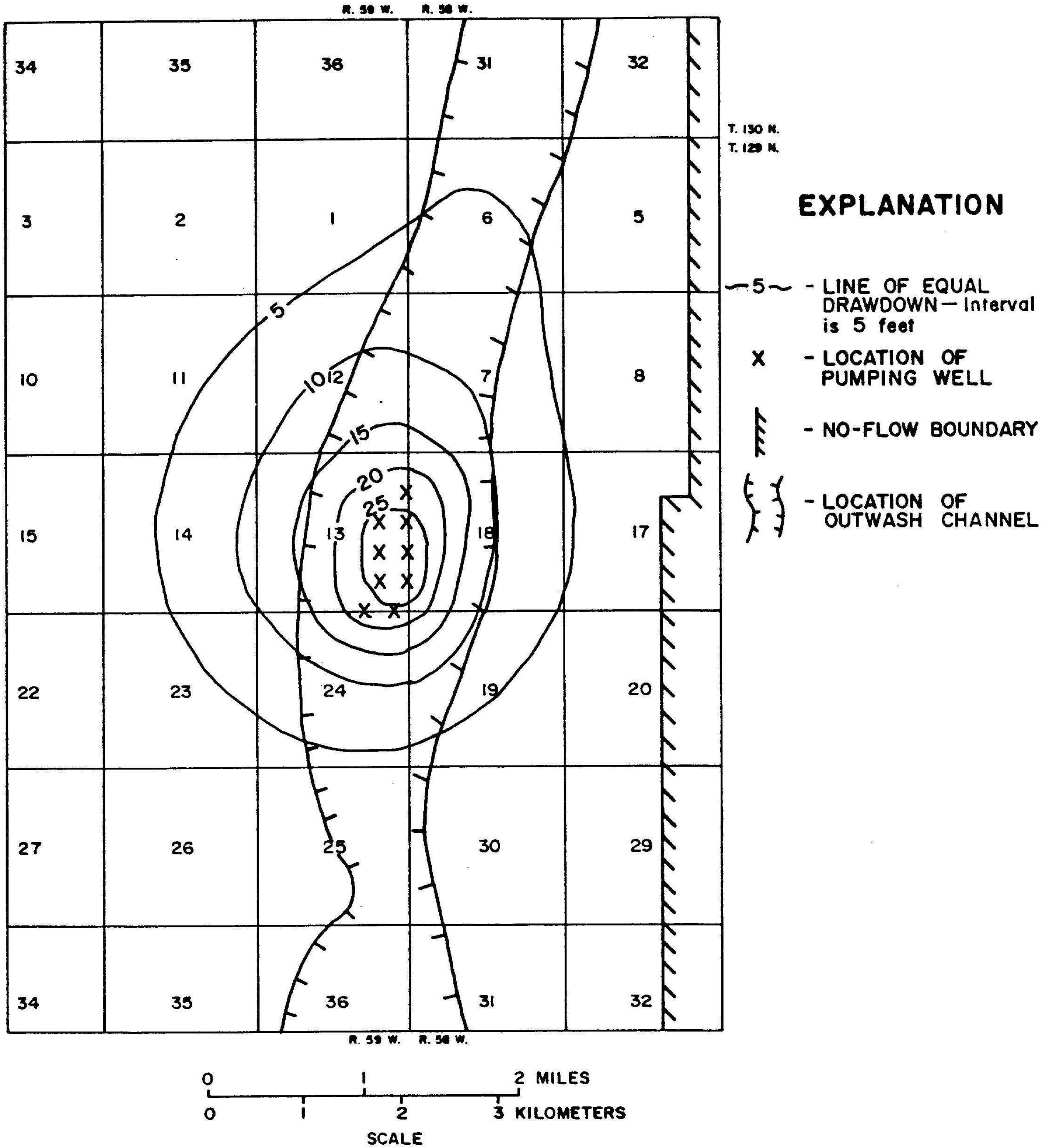

Figure 16.--Pilot-scale well field computer simulation: drawdown distribution at the end of stress #7

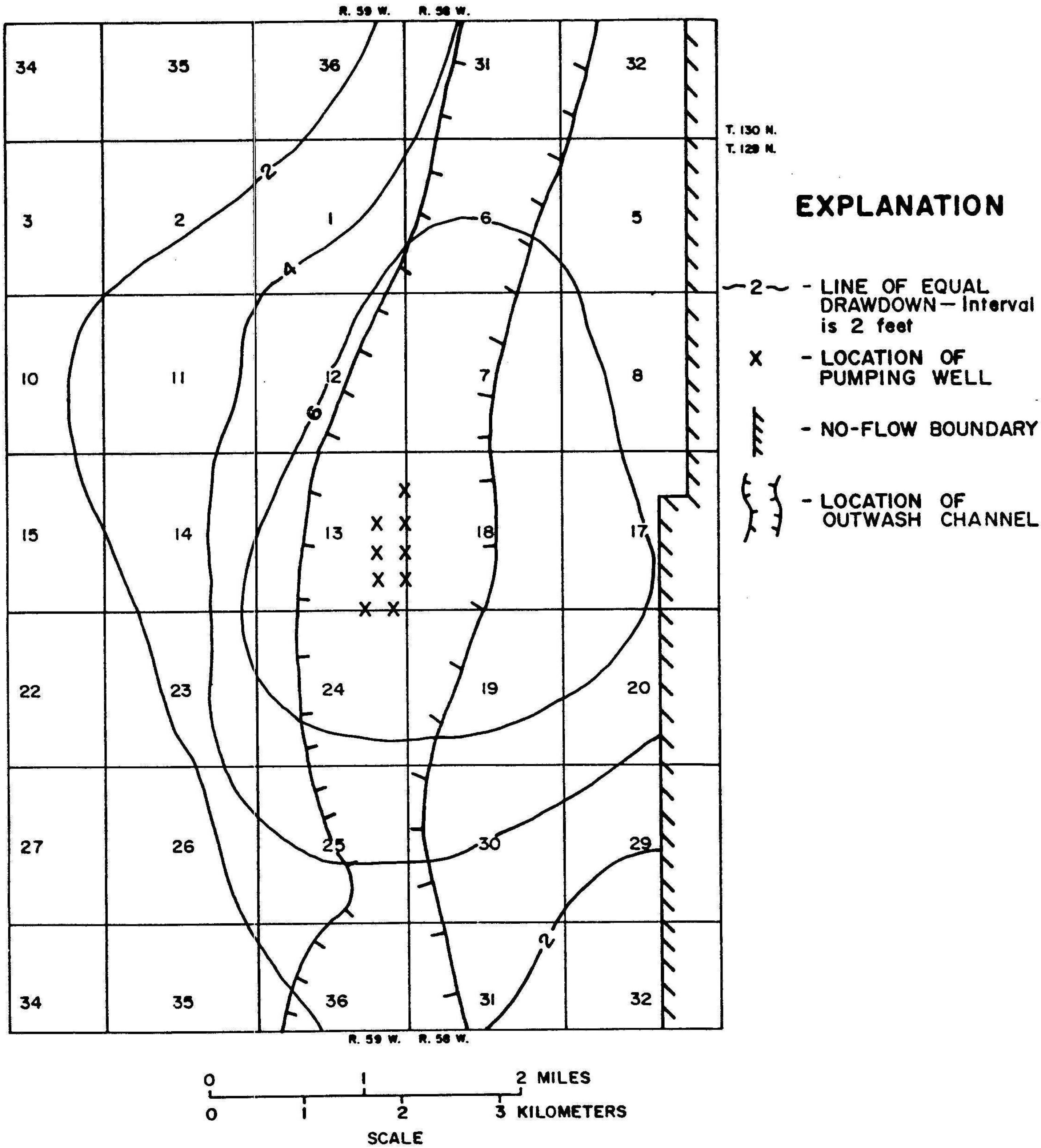

Figure 17.--Pilot-scale well field computer simulation: drawdown distribution at the end of stress period #8

large–scale surface recharge facilities in the test area, thereby significantly reducing infiltration rates. Artificial recharge would be even further restricted during the spring of the first year of operation if the aquifer was artificially recharged during the previous fall.

Pilot–Scale, Well–field Cost Analysis

The pilot–scale, well–field cost analysis is based on an average well depth of 125 feet. Based on previously described design parameters, each well will consist of 85 feet of 20–inch O.D., standard–wall thickness (0.375 inch), low carbon steel casing and 40 feet of 18–inch telescopic stainless–steel screen.

A 26–inch diameter hole will be drilled to a depth of 85 feet. To protect the casing from corrosion, the 3–inch annular area will be grouted with cement. The well screen will be installed and developed by employing jetting and surging techniques. After well development is completed, a test pump will be installed to perform step tests and determine well efficiency. After well–efficiency testing is completed, the test pump will be removed and the new pump and electric–drive motor will be installed. The electric drive motor will then be connected to the control box. The cost per well and the complete 9–well, pilot–scale well field are presented in table 9. The computation of annual electrical power costs per well is shown in appendix 1.

Design and Operation of Pilot–Scale Recharge Facilities

The primary purpose of the pilot–scale testing program is to determine the most efficient method of surface and near–surface artificial recharge that is cost effective in terms of construction, operation, and maintenance. The following artificial recharge tests are recommended for the pilot–scale investigation:

1) Basin recharge using raw, turbid James River water.
2) Basin recharge using James River water pretreated with chemical flocculants to remove suspended solids.
3) Basin recharge using an organic mat.
4) Surface spreading using raw, turbid James River water.

Table 9. - - Pilot-scale well field cost analysis

Component	Unit Cost	Cost Per Well	Cost of Pilot-Scale Well Field (9 wells)
Drill hole	$ 65.00/ft.	$ 5,600	$ 50,400
Casing	$ 32.86/ft.	$ 2,800	$ 25,200
Casing grouting	---	$ 2,000	$ 18,000
Well screen[1]	---	$ 6,000	$ 54,000
Screen installation	$120.00/Hr.	$ 6,000	$ 54,000
Well Development	$100.00/Hr.	$ 4,000	$ 36,000
Test pumping			
Install and remove pump	---	$ 1,000	$ 9,000
Step testing	$ 80.00/Hr.	$ 1,200	$ 10,800
Pump[2]	---	$10,000	$ 90,000
Electrical control box[3]	---	$ 5,000	$ 45,000
Annual well electrical power costs[4]	---	$ 300	$ 16,200
Annual well monitoring[5]	---	$ 200	$ 10,800[6]
Preparation of specifications, construction supervision, and overhead	--	$13,000	$117,000[6]
Contingencies	---	$ 7,000	$ 63,000
Total cost		$64,100	$599,400

[1]Includes packer and bottom plate

[2]Includes bowl assembly, column, discharge head, electric motor, and installation

[3]Includes installation

[4]Based on $0.0025 per kilowatt hour; does not include wheeling and maintenance costs - (see appendix 1)

[5]Includes water-level measurements, water quality sampling, and specific capacity determinations

[6]Based on a 6-year pilot testing program

5) Contingency testing.

Basin Recharge: Turbid James River Water

One pair of recharge basins will be used to evaluate basin recharge using turbid James River water. The depth of the basins will be based on local stratigraphic controls (spatial distribution of low hydraulic conductivity layers) and will not exceed 5 feet. Basins will be rectangular in shape with a width to length ratio of about 4 to 1 to minimize the height of perched ground–water mounds. Sidewall slopes will be 4 to 1 to prevent collapse and to control erosion. The dimensions of the two basins, including 4 to 1 sidewall slopes will be 1140 feet (length) by 320 feet (width). Depending on the spatial variability of hydraulic properties in the test area, the above basin dimensions may be scaled down 50 percent.

One basin of the pair will be in operation while the other basin of the pair is being renovated. Renovation will consist of natural drying and desiccation of the basin floor. During selected renovation periods after drying, the surface crust deposited along the basin floor will be removed by scraping. Length of recharge and renovation periods will vary according to physical, chemical, and biologic parameters associated with the recharge water and the infiltration media. The overall objective of these tests will be to maximize infiltration rate while minimizing operation and maintenance costs.

Variable–head tests will be conducted at selected times during operation of the recharge basins. Varying basin stage is an effective technique used to enhance infiltration rate (Schuh and Shaver, 1988).

Basin Recharge: Pretreated James River Water

A pretreatment flocculation–settling basin will be operated to deliver sediment–free James River water to a recharge basin. The depth of the recharge basin will be based on local stratigraphic controls (spatial distribution of low hydraulic conductivity layers) and will not exceed 5 feet. The basin will be rectangular in shape with a width to length ratio of 4 to 1 to minimize the height of perched ground–water mounds. Sidewall slopes will be 4 to 1 to prevent collapse and to control erosion. The dimensions of the recharge basin including 4 to 1 sidewall slopes will be 815 feet (length) by 240 feet (width). Depending on spatial variability of hydraulic properties in the test area, the above basin dimensions may be scaled down 50 percent.

The dimensions of the flocculation–settling basin are based on a water retention time of 3 hours to effectively remove suspended solids before delivery to the recharge basin. Sidewall slopes of the settling basin will be 2 to 1 to prevent collapse and to control erosion. The dimensions of the settling basin, including 2 to 1 sidewall slopes, will be 220 feet (length) by 70 feet (width) by 5 feet (depth). Variable–head tests will be conducted at selected times during the operation of the recharge basin to evaluate the effect on infiltration rate.

Basin Recharge: Organic Mats

Based on the results of basin–recharge tests conducted in phase II of the Oakes aquifer artificial recharge feasibility study, organic mats enhance average infiltration rates in basins using turbid water. The use of an organic mat also resulted in the deeper penetration of clay–sized particles into the subsoil beneath the basin floor. Long–term operation may require removal of up to 2 feet of basin subsoil. Therefore, long–term infiltration rate gains may be offset by maintenance costs associated with excavation of up to 2 feet of adulterated basin subsoil. It is recommended that organic–mat testing be conducted during the entire pilot–testing period to evaluate deep penetration of clay–sized particles.

A 6–inch thick organic mat, consisting of composted sunflower–seed hulls, will be placed along the floor of a single recharge basin. The depth of the recharge basin will be based on local stratigraphic controls and will not exceed 5 feet. The basin will be rectangular in shape with a width to length ratio of 4 to 1 to minimize the height of perched ground–water mounds. Sidewall slopes will be 4 to 1 to prevent collapse and to control erosion. The dimensions of the recharge basin including sidewall slopes will be 400 feet (length) by 100 feet (width).

Surface Spreading: Turbid James River Water

In terms of construction costs, surface spreading is an economical means of artificial recharge. However, low surface–infiltration rates often make surface spreading impractical. Available soils and shallow auger drill–hole data in the proposed test area indicate initial surface–infiltration rates of up to about 2.5 feet per day are possible. Therefore, pilot–scale surface–spreading testing is warranted.

The area selected for surface spreading will be based on local land–surface topography and stratigraphic controls. The size of the surface–spreading test area will be at least 10 acres. Attempts will be made to select rectangular–shaped areas with a width to length ratio of 4 to 1 to minimize the height of perched ground–water mounds. Dikes consisting of locally derived lacustrine sediment will be constructed along the perimeter of the surface area. Sidewall slopes of the dikes will be 4 to 1 to prevent collapse and to control erosion. Maximum height of the dikes will be 5 feet. Previous work in the southwestern United States indicates that infiltration rates are enhanced by planting grass in spreading areas. The grass acts as a filtering media to reduce suspended solids. In addition, the grass provides root channels that enhance infiltration rates. It is anticipated that the surface spreading area will be divided in half with one–half planted in canary grass and the other half left undisturbed.

Contingency Recharge Testing

It is envisioned that the pilot–scale well field and recharge facilities will be operated during the spring and fall for up to 6 years to provide necessary design, operation, and maintenance data to develop full–scale project facilities. As pilot–scale testing proceeds, additional physical, chemical and/or biological processes may require innovative design, operation, and maintenance procedures not described in the aforementioned pilot–scale tests. It is important that the pilot–scale test program be flexible enough to accommodate modification of recharge testing protocol as more site–specific data is obtained and evaluated.

Data Collection and Instrumentation of Pilot–Scale Recharge Facilities

Data collection and instrumentation for the various pilot–scale recharge facilities essentially will be the same as described by Shaver, and others (July 1986). Briefly, prior to basin and surface spreading, short–term infiltration tests will be conducted at selected sites along basin floors and at land surface using double–ring infiltrometers. The purpose of these tests will be to develop functional relationships between unsaturated–hydraulic conductivity and moisture content and to determine saturated–hydraulic conductivity at selected depths below the bottom of the test basins and below land surface. A simplified functions approach for determining in–situ hydraulic conductivity will be used, requiring only steady–state infiltration and field tensiometric profiles versus time for the draining profile (Ahuja, 1980). The double–ring infiltrometers will be left in place during the long–term recharge tests. Tensiometric data will be collected at each tensiometer nest throughout the duration of each recharge test. The tensiometric data and pre–test saturated/unsaturated hydraulic conductivity measurements will be used to drive the Darcy equation to characterize the growth and extent of clogging. A schematic of a typical tensiometer nest is shown in figure 18.

In addition to hydraulic properties, physical properties will be determined to assess depth of clogging. USDA texture, wet combustion organic carbon, bulk density, and moisture retention curves may be determined at selected depths before and after recharge.

Bulk mineralogy will be determined for samples at selected depths from land surface to the top of the saturated zone. This data will provide the basis for assessing chemical reactions between the recharge water and the aquifer matrix.

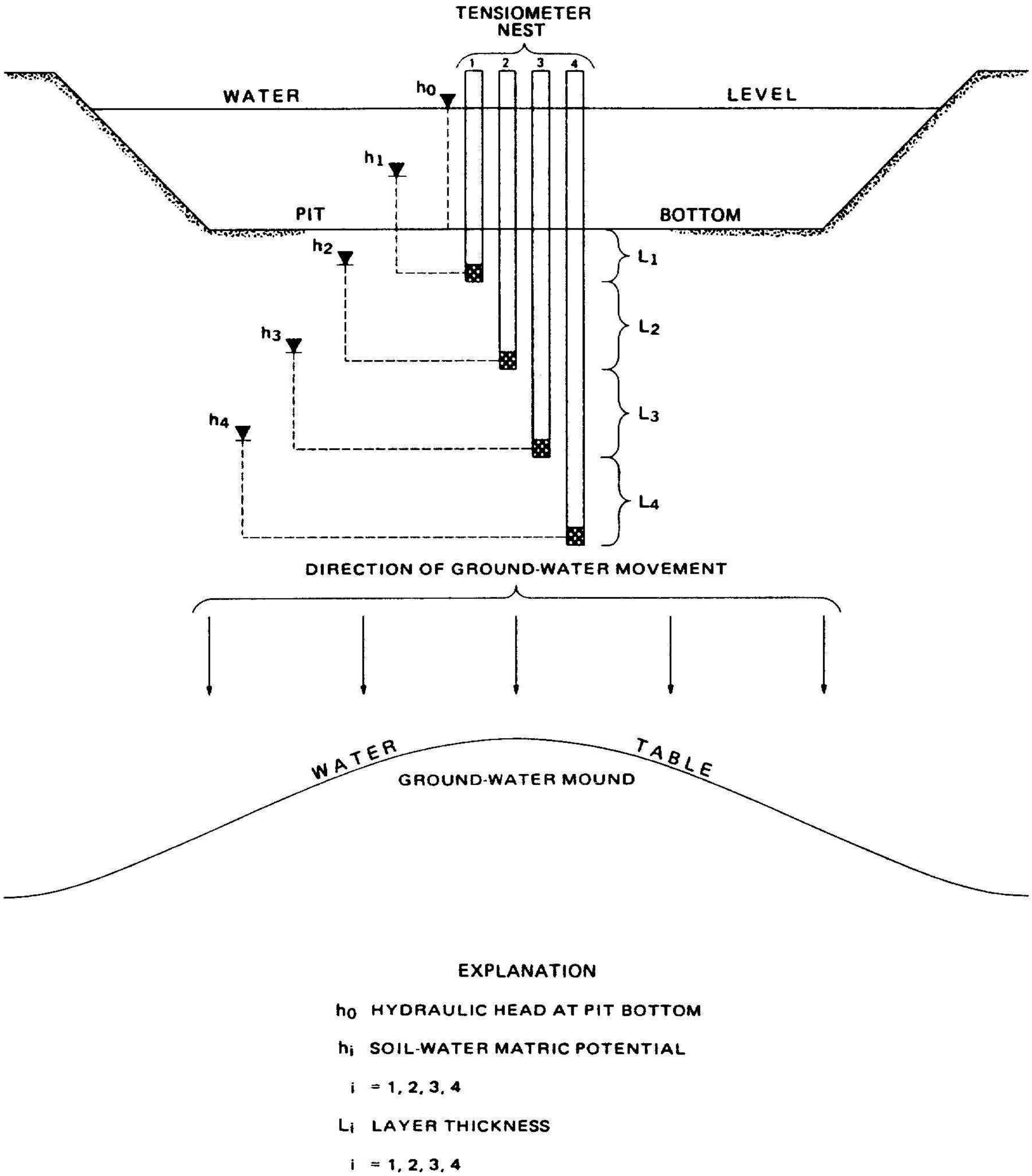

Figure 18.--Schematic of a typical tensiometer nest

Raw James River water will be analyzed for specific chemical parameters and constituents at selected times during the pilot–scale recharge tests. In addition, unsaturated and saturated zone water samples will be collected at various depths below recharge facilities and analyzed for specific chemical parameters and constituents. The range of chemical parameters and constituents is described in Shaver, and others (July 1986, p. 30).

Raw James River water samples will be collected in the Oakes area and analyzed for various trace organic compounds that relate to agricultural practices (pesticides and herbicides). Results of these analyses will provide a basis for developing a trace–organics sampling protocol in the pilot–scale test area.

Basin–recharge testing in Phase II of the artificial–recharge feasibility study indicated that sediment clogging was the primary infiltration–rate control. It is anticipated that pretreated (flocculation) sediment–free water will be imported to a recharge basin as part of the pilot–scale study. For this test case, biologic clogging may be the primary infiltration–rate control. It is recommended that bioassays be performed on recharge water for this test and others initiated in the pilot study. Suggested bioassays include algal biomass, chlorophyll, and bacterial population.

Piezometer nests will be installed at the center of recharge basins and surface–spreading areas to monitor growth and dissipation of water–table and perched ground–water mounds. In addition, piezometers will be installed at selected sites throughout the recharge area to monitor the growth and dissipation of water–table mounds.

Discharge into the recharge facilities and stage will be continuously monitored throughout each recharge test. This data will be used to measure infiltration rate with time and provide the basis for terminating each recharge test.

<u>Pilot–Scale Recharge–Test Facilities Cost Analysis</u>

Five types of artificial recharge tests are recommended for the pilot–scale investigation. These include:

1) Basin recharge using raw turbid James River water.
2) Basin recharge using James River water that has been pretreated with chemical flocculants to remove suspended solids.
3) Basin recharge using an organic mat.
4) Surface spreading, using raw, turbid James River water.
5) Contingency testing.

Basin Recharge: Turbid James River Water

Two basins will be operated to evaluate attenuation of infiltration rate and renovation by natural dessication and scraping. One basin will be operational while the other basin is being renovated. Excavation costs are based on removing 1 foot of topsoil and 4 feet of subsoil to construct each basin. Topsoil removal is based on a cost of $2.00 per cubic yard and subsoil removal is based on a cost of $1.25 per cubic yard. Basin dimensions are 1,140 feet (length) by 320 feet (width) by 5 feet (depth). Basin appurtenances are based on a cost of $10,000 per basin (Abe, 1986). A basin monitoring network is based on a cost of $5,000 per basin. Land easement costs are based on $300.00 per acre. Land easement cost for two basins is more than twice that of a single basin because additional land will be needed between the basins for access and water–distribution facilities. A cost analysis of this test is presented in table 10.

Basin Recharge: Pretreated James River Water

A pretreatment flocculation–settling basin will be operated to deliver sediment–free James River water to a recharge basin. Excavation and land easement costs are based on unit costs described above. Maximum dimensions of the pretreatment basin will be 220 feet (length) by 70 feet (width) by 5 feet (depth), and those of the recharge basin will be 815 feet (length) by 240 feet (width) by 5 feet (depth). A cost analysis of this test is

Table 10. -- Cost analysis of pilot-scale basin recharge tests using turbid James River water

Component	Cost per Basin	Cost for Two Basins
Topsoil excavation[1]	$27,000	$54,000
Subsoil excavation[2]	58,500	117,000
Basin appurtenances	10,000	20,000
Basin monitoring	5,000	60,000[4]
Land easements[3]	2,250	5,100
Basin renovation	200	2,400[4]
	Total	$258,500

1 13,291 yd^3 of topsoil per basin at a cost of $2.00/$yd^3$

2 46,471 yd^3 of subsoil per basin at a cost of $1.25/$yd^3$

3 Based on $300.00 per acre

4 Based on a 6-year pilot-scale testing program.

presented in table 11.

Basin Recharge: Organic Mat

The effectiveness of organic–mat filters on infiltration–rate enhancement in basins will be investigated in the pilot–scale testing program. Excavation and land easement costs are based on unit costs previously described. Maximum dimensions of the recharge basin are 400 feet (length) by 100 feet (width) by 5 feet (depth). The organic mat will consist of composted sunflower seed hulls. It is estimated that 270 tons of sunflower seed hulls will be required for the pilot–scale recharge tests. The cost per ton is $13.00. A cost analysis of this test is presented in table 12.

Surface Spreading: Turbid James River Water

A surface–spreading area of at least 10 acres will be operated as part of the pilot–scale testing program. Dikes consisting of locally derived lacustrine sediment will be constructed along the perimeter of the surface area. The maximum height of the dikes will be 5 feet. Dike excavation costs are based on a unit cost of $1.25 per cubic yard of subsoil. A cost analysis of this test is presented in table 13.

Contingency Recharge Testing

As previously mentioned, the pilot–scale test program must be flexible enough to accomodate modification of recharge–testing protocol as more site–specific data is obtained and evaluated. It is estimated the cost of contingency testing will be $100,000, which is about 20 percent of the total cost of the four previously described pilot–scale recharge tests.

Total Cost of Pilot–Scale, Well–Field and Recharge Tests

A 20 percent cost overrun, which amounts to $295,000, is estimated for the

Table 11. -- Cost analyses of pilot-scale basin recharge tests using pretreated James River water

Component	Cost per Basin
Pretreatment basin	
Topsoil excavation[1]	$1,100
Subsoil excavation[2]	2,500
Basin appurtenances	10,000
Recharge Basin	
Topsoil excavation[3]	14,200
Subsoil excavation[4]	29,500
Basin appurtenances	10,000
Basin monitoring[5]	30,000
Land Easements[6]	2,100
Total	$99,400

1 548 yd^3 of topsoil at a cost of $2.00/$yd^3$

2 1,942 yd^3 of subsoil at a cost of $1/25/$yd^3$

3 7,085 yd^3 of topsoil at a cost of $2.00/$yd^3$

4 23,564 yd^3 of subsoil at a cost of $1.25/$yd^3$

5 Based on an annual cost of $5,000 for a 6-year pilot-scale test period

6 Based on $300.00 per acre

Table 12. -- Cost analysis of pilot-scale basin recharge tests using an organic mat

Component		Cost
Topsoil excavation[1]		$2,900
Subsoil excavation[2]		5,400
Basin appurtenances		10,000
Basin monitoring[3]		30,000
Organic mat[4]		3,500
Land easements[5]		600
	Total	$52,400

[1] 1,413 yd^3 of topsoil at a cost of $2.00/$yd^3$

[2] 4,271 yd^3 of subsoil removal at a cost of $1.25/$yd^3$

[3] Based on an annual cost of $5,000 for a 6-year pilot-scale test period

[4] 270 tons of sunflower seed hulls at a cost of $13.00 per ton

[5] Based on $300.00 per acre

Table 13. - - Cost analysis of pilot-scale surface-spreading tests

Component		Cost
Dike construction[1]		$23,000
Surface-flooding area appurtenances		10,000
Surface-flooding area monitoring[2]		30,000
Surface-flooding area renovation[3]		1,200
Land easements[4]		3,000
	Total	$67,200

1 18,400 yd^3 of subsoil at a cost of $1.25/$yd^3$

2 Based on an annual cost of $5,000 for a 6-year pilot-scale test period

3 Includes seeding (canary grass), discing, and scraping

4 Based on $300.00 per acre

pilot–scale, artificial–recharge testing program. Three full–time professionals will be required at an annual cost of $36,000 per person. This amounts to a total cost of $648,000 over a 6–year test period. Two full–time technicians will be required at an annual cost of $18,000 per person. This amounts to a total cost of $216,000 over a 6–year test period. The total cost of the complete pilot–scale, well–field and recharge–test program is presented in table 14.

PROJECT–SCALE WELL FIELD AND ARTIFICIAL–RECHARGE FACILITIES

General Statement

The previously described computer model of the Oakes aquifer was used to aid in designing a project–scale well field. Preliminary evaluation and selection of artificial recharge facilities is more difficult because until pilot–scale testing is completed, it will not be possible to determine the most efficient and cost effective artificial–recharge method. For preliminary planning purposes, basin recharge and surface spreading were evaluated for the proposed project area. Total recharge area was based on average infiltration rates of 1, 2, and 3 feet per day.

Project–Scale, Well–Field Computer Simulation

Two objectives of the modeling study were to 1) develop a preliminary design of a project–scale well field, and 2) estimate the effects on aquifer water levels of a continuous withdrawal of 100 cubic feet per second for 60 days (11,900 acre–feet) from a full project–scale well field. This withdrawal rate is the maximum rate anticipated for years of peak irrigation demand. Available data indicate that the best potential for the above withdrawal scenario occurs within the channel–fill deposits that occupy the outwash channel along the eastern margin of the study area near Sec. 13, T. 129 N., R. 59 W. This area of the Oakes aquifer was selected based on the following criteria:

1) The channel–fill deposits have the largest transmissivity in comparison to other depositional facies of the Oakes aquifer. Individual well yields of about

Table 14. -- Cost analysis of complete pilot-scale well field and artificial recharge test program

Component	Cost
Basin recharge - turbid James River water	$258,500
Basin recharge - pretreated James River water	99,400
Basin recharge - organic mat	52,400
Surface spreading	67,200
Contingency testing	100,000
Professional and technical services	864,000
20 percent cost overrun	295,000
Well field	599,400
Total	$2,335,900 [1]

[1] Does not include costs for the supply system to convey water to and from the well field and artificial recharge areas.

3,000 gallons per minute are possible.

2) The width of the outwash channel is at a maximum in this area. Therefore, the amount of water in storage is greater in this area as compared to other areas of the outwash channel.

3) Overlying fluvial silt and clay confining beds are thin or absent, which is conducive to the development of surface–recharge facilities (basins, surface flooding).

4) Chemical analyses of water samples collected from the channel–fill deposits in this area pose no limitations for irrigation use.

The preliminary well–field design was based on 1) continuous pumping at a rate of 44,880 gallons per minute (100 cubic feet per second) for 60 days, and 2) pumping levels that do not exceed about two–thirds of the available head above the top of the screen. Various well–field configurations were simulated using the finite–difference ground–water flow model. The most favorable well–field configuration is shown in figure 19. The well field consists of 15 production wells in two parallel lines located along the principal axis of the outwash channel. The wells generally are spaced 1,000 feet apart. The pumping rate of each well was 3,000 gallons per minute.

Data used to estimate drawdown in each production well is shown in table 15. Production–well drawdown computed by the model is shown in the column labelled S_m. This drawdown is not corrected for well loss, partial penetration, and a real–well radius. For this study, well loss was assumed to be 20 percent. To calculate additional drawdown based on a 20 percent well loss, the following formula was used:

$$S_e = [(0.20)s(\text{Theis})]$$

where S_e = additional drawdown, in feet, due to a 20 percent well loss (table 15)

s(Theis) = Drawdown, in feet, in production well after pumping 3,000 gallons per minute continuously for 60 days. Calculated analytically based on Theis assumptions.

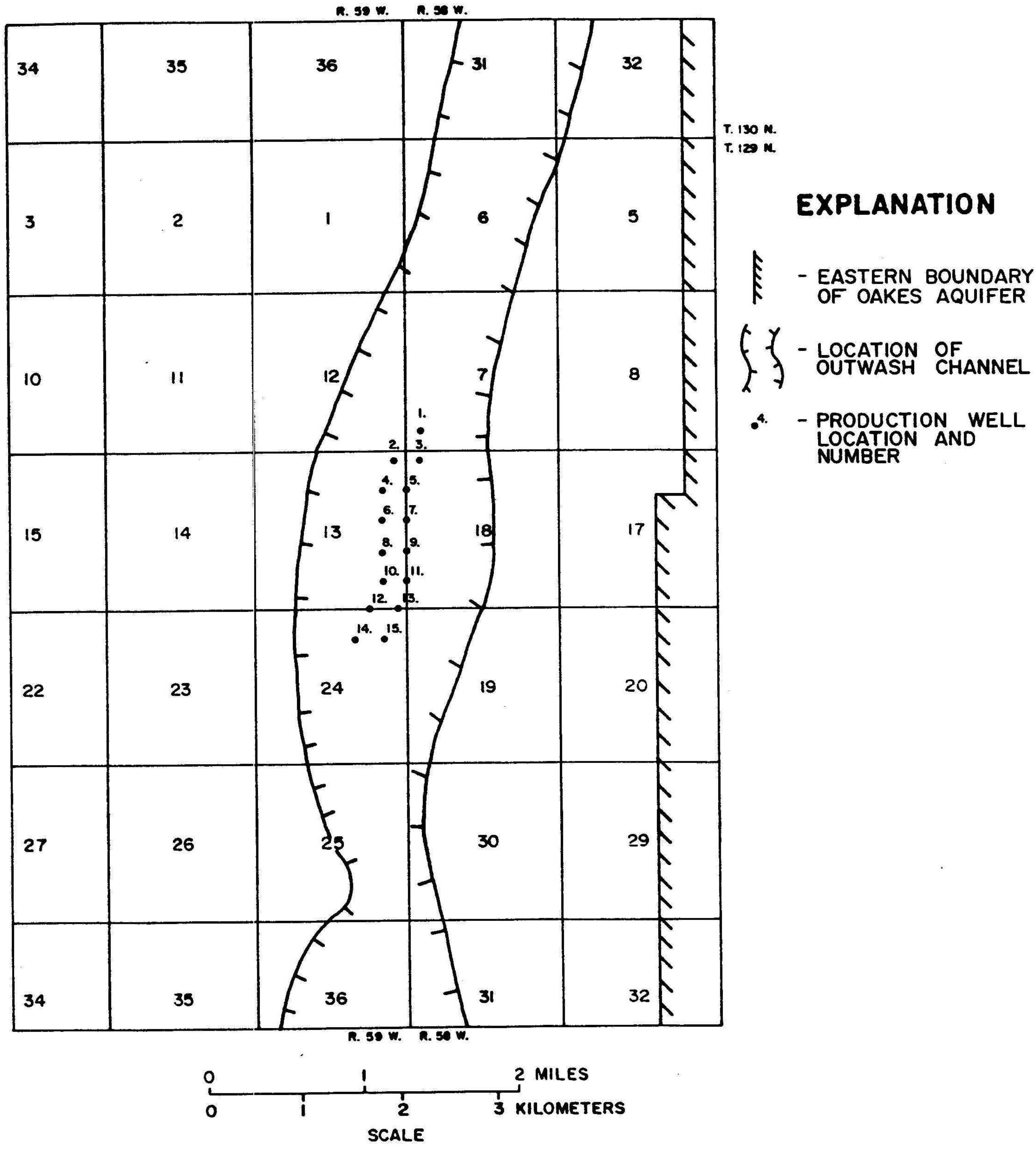

Figure 19.--Location of project-scale well field

Table 15. -- Estimated project-scale production-well drawdown and corrections for well loss, partial penetration and real-well radius

Well Number (see fig. 19)	Starting Head (feet above msl)	Elevation of Base of Aquifer (feet above msl)	h_w	K	T $(K \cdot h_w)$	(S_m)	h_s	p (h_s/h_w)	r_{pw}	(S_t)	S_e	S_p	S_{rw}
1	1304.9	1175	130	775	100,750	33.3	40	.31	1.5	7.86	1.57	1.8	3.9
2	1304.7	1190	115	775	89,125	42.1	40	.35	1.5	8.82	1.76	1.9	4.7
3	1304.8	1175	130	775	100,750	39.6	40	.31	1.5	7.86	1.57	1.8	3.9
4	1304.5	1190	115	775	89,125	47.2	40	.35	1.5	8.82	1.76	1.9	4.4
5	1304.6	1175	130	775	100,750	45.5	40	.31	1.5	7.86	1.57	1.8	3.9
6	1304.3	1190	115	775	89,125	49.4	40	.35	1.5	8.82	1.76	1.9	4.4
7	1304.4	1165	140	775	108,500	47.6	40	.29	1.5	7.33	1.47	1.8	3.6
8	1304.2	1170	134	775	103,850	48.8	40	.30	1.5	7.64	1.52	1.8	3.8
9	1304.3	1170	134	775	103,850	48.4	40	.30	1.5	7.64	1.52	1.8	3.8
10	1304.1	1185	119	775	92,225	47.6	40	.34	1.5	8.54	1.71	1.9	4.2
11	1304.2	1210	94	775	72,850	47.6	40	.43	1.5	10.66	2.13	1.8	5.5
12	1303.8	1200	104	775	80,600	42.4	40	.39	1.5	9.69	1.94	1.9	4.8
13	1303.9	1210	94	775	72,850	44.2	40	.43	1.5	10.66	2.13	1.8	5.5
14	1303.6	1200	104	775	80,600	34.0	40	.39	1.5	9.69	1.94	1.9	4.8
15	1303.7	1210	94	775	72,850	36.9	40	.43	1.5	10.66	2.13	1.8	5.5

EXPLANATION

h_w – Saturated thickness at production well, in feet

K – Hydraulic conductivity, in feet per day

T – Transmissivity, in feet squared per day

S_m – Drawdown, in feet, in production well after pumping 3,000 gallons per minute continuously for 60 days (computed by model)

h_s – Screened interval, in feet

r_{pw} – Production–well radius, in feet

S_t – Drawdown, in feet, in production well after pumping 3,000 gallons per minute continuously for 60 days. Calculated analytically based on Theis assumptions (used to calculate A).

S_e – Additional drawdown in production well based on a 20% well loss [(0.20) s(Theis)]

S_p – Additional drawdown in production well due to partial penetration

$$\left[\frac{\frac{Qp}{\pi K} \cdot \frac{1-p}{p} \cdot \ln \frac{(1-p)\,h_s}{r_{pw}}}{h_w}\right]$$

where Q = discharge rate, in cubic feet per day

$p = h_s/h_w$

S_{rw} – Additional drawdown in production well based on a real–well radius

$$\left[\frac{Q}{2\pi K h_w} \cdot \ln\left(\frac{a}{4.81\, r_{pw}}\right)\right]$$

where a = length of model block in feet

Additional drawdown in the production well due to partial penetration was estimated using the following formula (Todd, 1980):

$$S_p = \left[\frac{\frac{Qp}{\pi K} \cdot \frac{1-p}{p} \cdot \frac{\ln (1-p) h}{r_{pw}}}{2h_w} \right]$$

where

S_p = additional drawdown, in feet, due to partial penetration (table 15)

Q = discharge rate, in cubic feet per day

K = hydraulic conductivity, in feet per day

h_s = screened interval, in feet

r_{pw} = production–well radius, in feet

h_w = saturated thickness, in feet

$p = (h_s/h_w)$

The drawdown calculated by the model within each production well cell or block is based on an effective–well radius (r_e) that is much larger than the proposed real–well radius. The effective radius is determined by the following formula: (Prickett, 1971):

$$r_e = \frac{a}{4.81}$$

where

a = cell or block dimension, in feet

The production–well cell dimensions are 500 feet by 500 feet. The drawdown computed by the model at each production well cell (S_m) is for an effective radius of 104 feet (500/4.81). The real–well radius of the proposed production wells is estimated at 1.5 feet. The following modification of the Theim equation is used to calculate additional drawdown (S_{rw}) that must be added to the model drawdown (S_m) to estimate drawdown for a real–well radius of 1.5 feet.

$$S_{rw} = \frac{Q}{2\pi K h_w} \ln \frac{a}{4.81 r_{pw}}$$

where

S_{rw} = additional drawdown, in feet, based on a real–well radius of 1.5 feet

The hydraulic properties, starting heads, base of aquifer elevations, and saturated thicknesses shown in table 15 for each production well cell are interpolated values based on limited field data collected in the proposed project area. Each production well will have a unique specific capacity due to spatial variability in aquifer geometric and hydraulic properties and variations in both well efficiency and interference. It is not practical to estimate minor variations in production–well drawdown for a preliminary well–field design and cost analysis. Therefore, an estimated maximum constant–drawdown value applicable to all wells was calculated using the following mean values derived from table 15.

1) Saturated thickness (h_w) = 117 feet.
2) Screened interval (h_s) = 40 feet.
3) Available head above top of screen (117–40) = 77 feet.
4) Drawdown in production–well cells (S_m) = 43.7 feet.
5) 20% well loss drawdown (S_e) = 1.8 feet.
6) Partial penetration drawdown (S_p) = 1.8 feet.
7) Real–well radius drawdown (S_{rw}) = 4.4 feet.

Using the above values, the estimated total drawdown in all production wells shown in figure 19, with each well pumping continuously at a rate of 3,000 gallons per minute for 60 days, is 52 feet ($S_m + S_e + S_p + S_{rw}$). The available head above the top of the screen minus total drawdown (77–52) is 25 feet. Thus, the total drawdown is about two–thirds the initial available head above the top of the screen, which is consistent with good management practices.

The computer–simulated drawdown distribution resulting from pumping 15 wells continuously, each at a rate of 3,000 gallons per minute for 60 days (100 cfs – 11,900 acre–feet), is shown in figure 20. The shape of the area of influence is elongated in a roughly north–south direction along the principal axis of the outwash channel. The westward elongation of the area of influence reflects a westward gradient in the natural ground–water flow system.

The development of a three–dimensional ground–water flow model in the project area may be warranted in the future to provide a better approximation of the drawdown distribution from the proposed artificial–recharge irrigation system. The model could be used to simulate anisotropy and nonhomogeneity that characterize the outwash–channel deposits in the project area. Additional aquifer hydraulic parameters would be required to define anisotropy and nonhomogeneity. For a preliminary feasibility study, the development of a three–dimensional ground–water flow model is not required.

Project–Scale Well Field Cost Analysis

The project–scale well field (15 wells) cost analysis is based on previously described design criteria and is presented in table 16. Long–term well and pump maintenance costs such as well redevelopment (acidizing well screen) and replacement of worn pump components are not included in the cost analysis.

Preliminary Project–Scale Recharge Basin Design

Preliminary basin design criteria in the project area include the following:

1) Each system of basins must accommodate an annual recharge volume of 8,330 acre–feet of water (4,165 acre–feet over a 60–day period in the spring and 4,165 acre–feet over a 60–day period in the fall). Peak irrigation demand for full development of the west Oakes area (23,660 acres) requires that 11,900 acre–feet of water (100 cubic feet per second for 60 days) be pumped from the Oakes aquifer. It is estimated that average annual irrigation demand would be 70 percent of peak demand, which amounts to 8,330 acre–feet of water (70 cubic feet per second for 60 days).

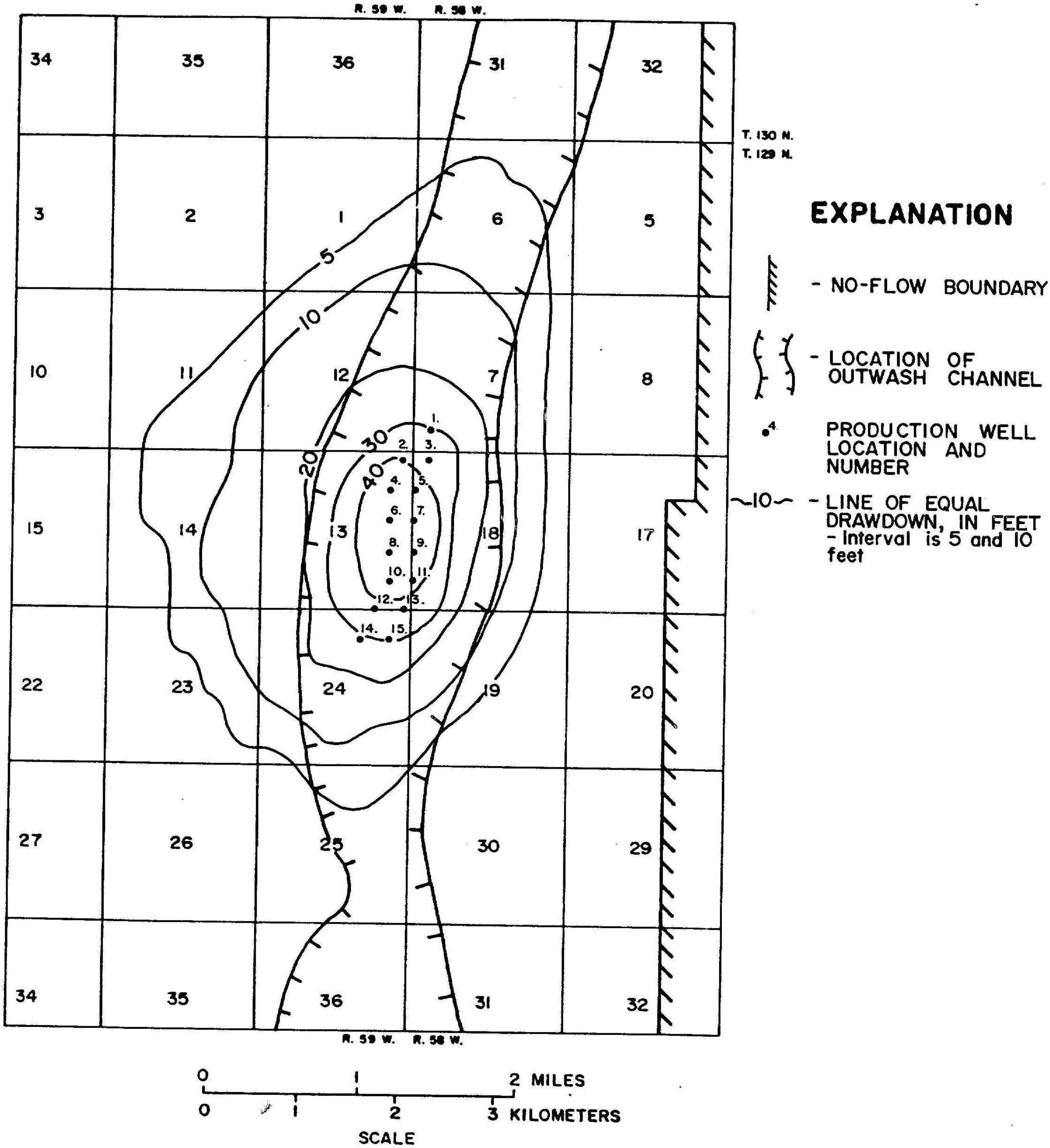

Figure 20.--Project-scale well field computer simulation: drawdown distribution after pumping 100 cubic feet per second for 60 days (11,900 acre-feet)

Table 16. - - Project-scale well field cost analysis

Component	Cost per well	Cost of project-scale well field (15-wells)
Drill hole	$5,600	$84,000
Casing	2,800	42,000
Casing grouting	2,000	30,000
Well screen[1]	6,000	90,000
Screen installation	6,000	90,000
Well development	4,000	60,000
Test pumping		
Install and remove pump	1,000	15,000
Step testing	1,200	18,000
Pump[2]	10,000	150,000
Electrical control box[3]	5,000	75,000
Annual well electrical power costs[4]	300	180,000[6]
Annual well monitoring[5]	200	120,000[6]
Preparation of specifications, construction supervision, and overhead	13,000	195,000
Contingencies	7,000	105,000
Total cost	$64,100	$1,254,000

[1] Includes packer and bottom plate

[2] Includes bowl assembly, column, discharge head, electric motor, and installation

[3] Includes installation

[4] Based on $0.0025 per kilowatt hour, does not include wheeling and maintenance costs (see appendix I)

[5] Includes water-level measurements, water quality sampling, and specific capacity determinations

[6] Based on a 40-year well life

2) Raw (turbid) water from the James River will be imported to the recharge basins without pretreatment.

3) Basins will be constructed in pairs with one basin of each pair operational for 15 days while the other basin of the pair is renovated.

4) Renovation will consist of natural dessication and removal of the surficial impeding layer on the basin floor.

5) Basin depth will be based on site–specific criteria (distribution of low–hydraulic conductivity layers) and will not exceed 5 feet.

6) Sidewalls of the basins will be constructed with at least 4 to 1 slopes to prevent collapse and to control erosion.

7) Basin stage will vary between 1 and 4 feet.

8) To minimize the development of perched ground–water mounds, the ratio of width to length of each basin will be about 4 to 1.

9) Average 15–day infiltration rates are estimated at 1 to 3 feet per day.

Based on an average 15–day infiltration rate of 1 foot per day, about 70 acres of basin area will be required to recharge 4,165 acre–feet of water to the aquifer in 60 days. Applying a basin width–to–length ratio of about 4 to 1 and using 10 pairs of basins (one operational, one undergoing renovation) will require basin dimensions of 1140 feet (length) by 320 feet (width). A basin spacing of 100 feet is recommended to allow for conveyance facilities and access. The configuration of the recharge basins and well field is shown in figure 21.

Based on an average 15–day infiltration rate of 2 feet per day, about 35 acres of basin area will be required to recharge 4,165 acre–feet of water to the aquifer in 60 days. Applying a basin width–to–length ratio of about 4 to 1 and using 10 pairs of basins (one operational, one undergoing renovation) will require individual basin dimensions of 815 feet (length) by 240 feet (width). A basin spacing of 100 feet is recommended to allow for conveyance facilities and access. The configuration of the recharge basins and well field is

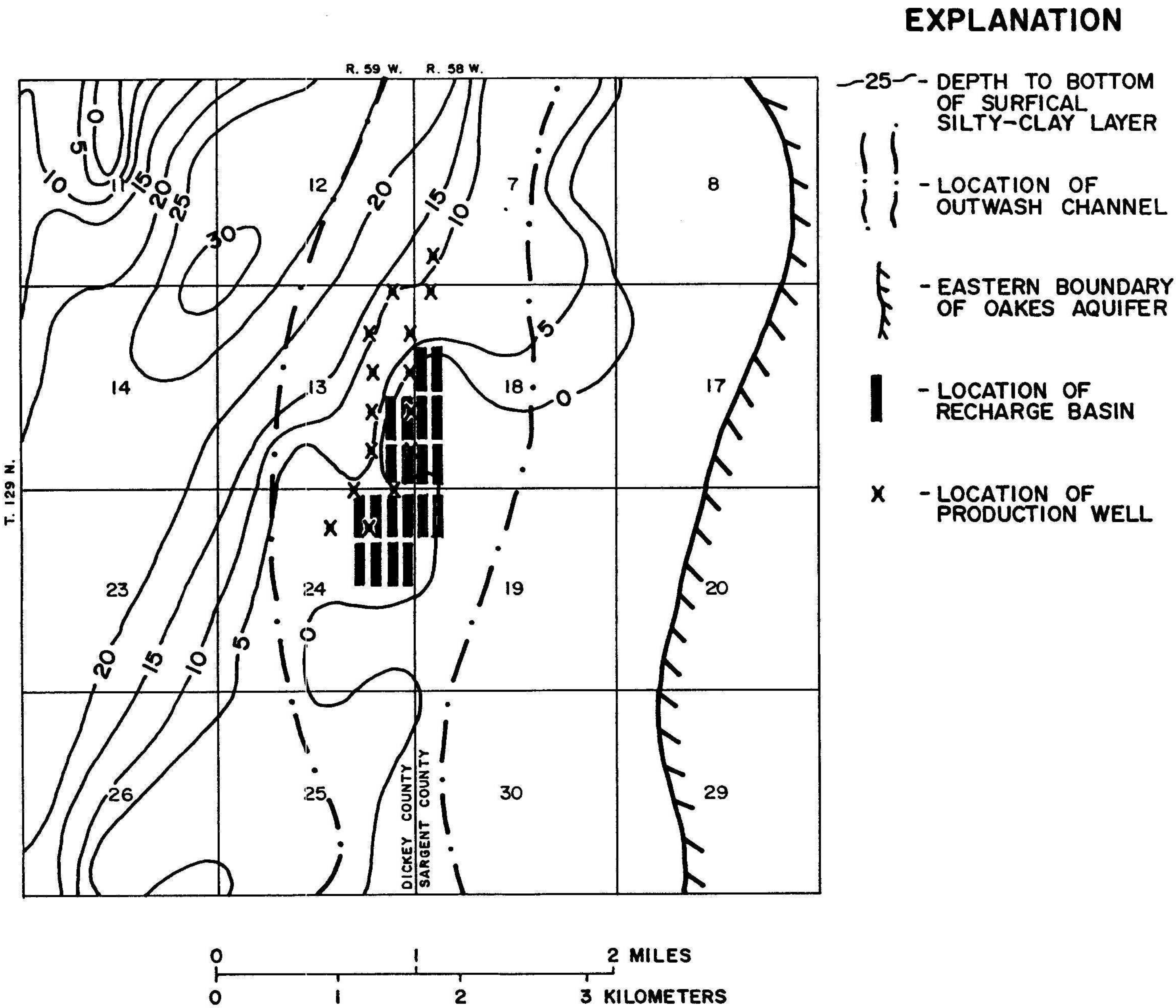

Figure 21.--Location of project-scale recharge basins, based on an infiltration rate of 1 foot per day

shown in figure 22.

Based on an average 15–day infiltration rate of 3 feet per day, about 23.3 acres of basin area will be required to recharge 4,165 acre–feet of water to the aquifer in 60 days. Applying a basin width–to–length ratio of about 4 to 1 and using 10 pairs of basins (one operational, one undergoing renovation) will require individual basin dimensions of 690 feet (length) by 190 feet (width). A basin spacing of 100 feet is recommended to allow for conveyance facilities and access. The configuration of the recharge basins and well field is shown in figure 23.

Project–Scale Recharge Basin Cost Analysis

Based on an average infiltration rate of 1 foot per day, a total basin area of 70 acres will be required to recharge 4,165 acre–feet of water to the aquifer in 60 days (8,330 acre–feet in 120 days). Applying a basin width–to–length ratio of about 4 to 1 and using 10 pairs of basins (one operational and one undergoing renovation) with 4 to 1 sidewall slopes will require individual basin dimensions of 1140 feet by 320 feet. Basin depth is estimated at 5 feet. Excavation costs are based on removing 1 foot of topsoil and 4 feet of subsoil to construct each basin. Topsoil removal is based on a cost of \$2.00 per cubic yard and subsoil removal is based on a cost of \$1.25 per cubic yard. Land easement costs are based on \$300 per acre. Basin appurtenances are based on a cost of \$10,000 per basin. A basin–monitoring network is based on a cost of \$5000 per basin. Preparation of specifications, construction supervision, and overhead costs are based on 30 percent of the total costs and amount to \$606,000. Contingency costs are based on 15 percent of the total costs and amount to \$394,000. Operation and maintenance costs are not calculated because they are poorly defined at this stage of the feasibility study. A cost analysis of this recharge basin option (Case A) is presented in table 17.

Based on an average infiltration rate of 2 feet per day, a total basin area of 35 acres will be required to recharge 4,165 acre–feet of water to the aquifer in 60 days (8,330

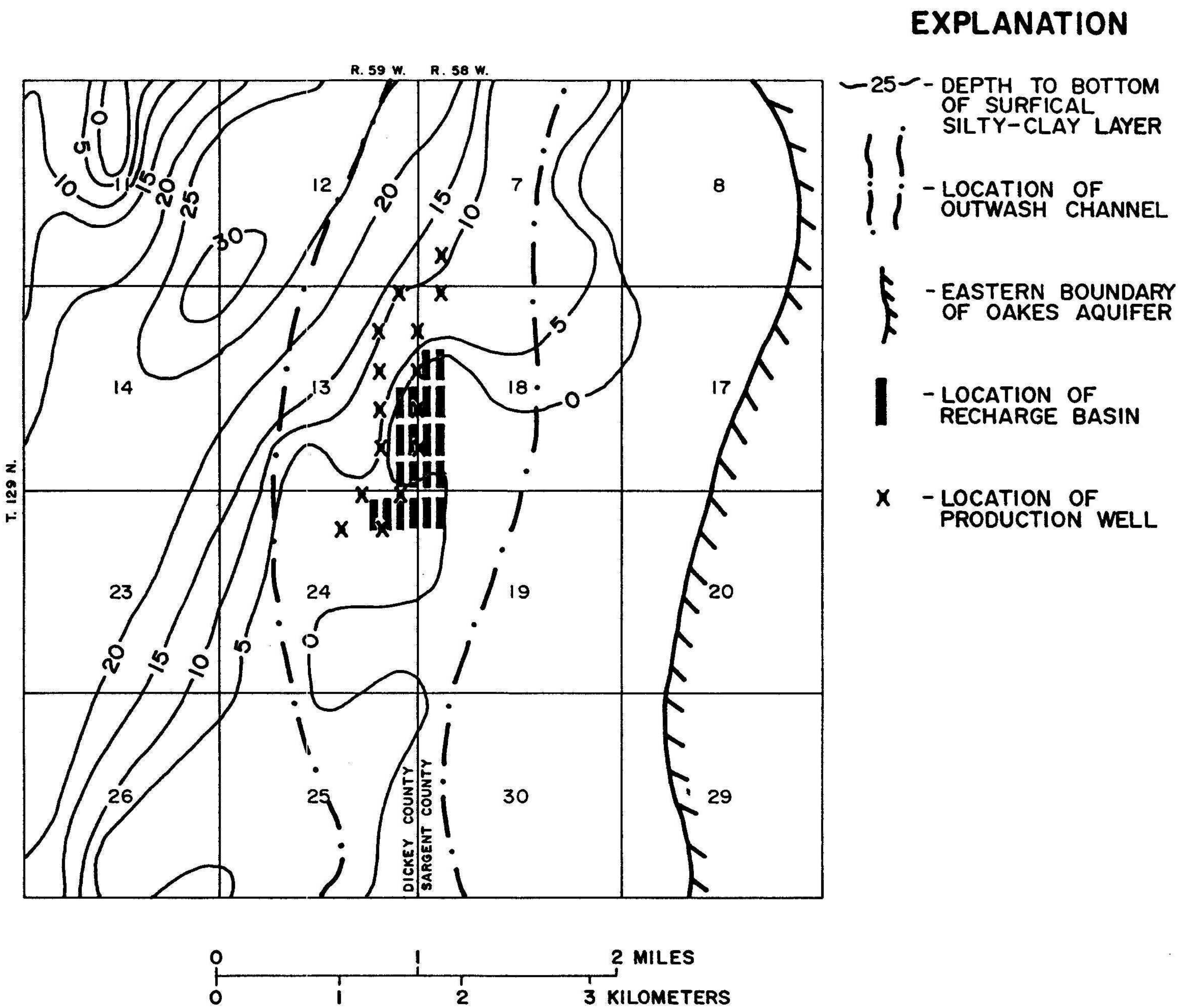

Figure 22.--Location of project-scale recharge basins, based on an infiltration rate of 2 feet per day

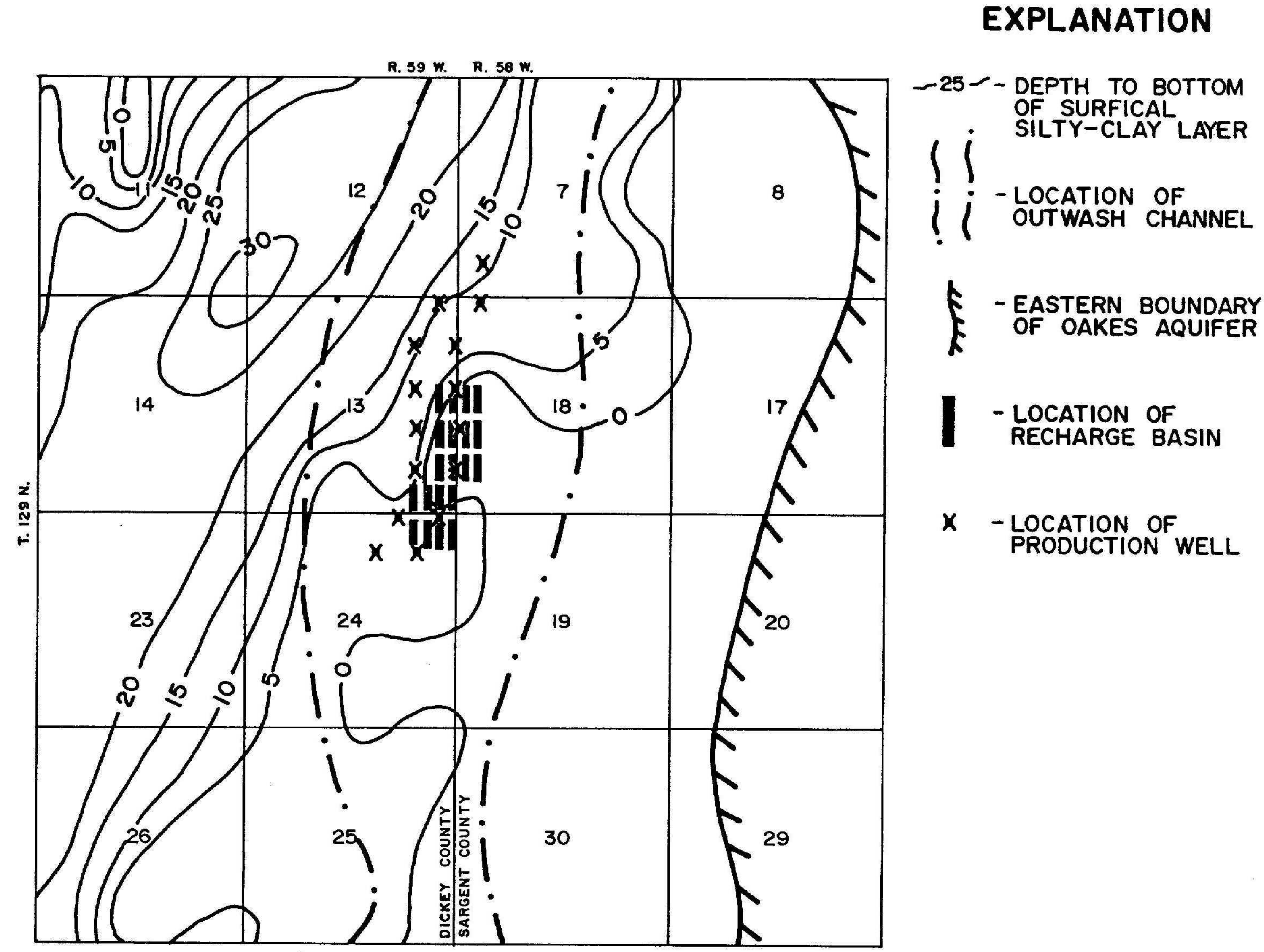

Figure 23.--Location of project-scale recharge basins, based on an infiltration rate of 3 feet per day

Table 17. -- Cost analysis of project-scale recharge basins based on average infiltration rates of 1, 2, and 3 feet per day

Component		Cost	
	Case A[1]	Case B[2]	Case C[3]
Topsoil excavation	$532,000	$284,000	$189,000
Subsoil excavation	1,162,000	590,000	371,000
Basin appurtenances	200,000	200,000	200,000
Basin monitoring network	100,000	100,000	100,000
Land easements	27,000	14,000	10,000
Preparation of specifications, construction supervision, and overhead	606,000	356,000	261,000
Contingencies	394,000	232,000	170,000
Total	$3,021,000	$1,776,000	$1,301,000

[1] Based on an average infiltration rate of 1 foot per day

[2] Based on an average infiltration rate of 2 feet per day

[3] Based on an average infiltration rate of 3 feet per day

acre–feet in 120 days). Applying a basin width–to–length ratio of about 4 to 1 and using 10 pairs of basins (one operational and one undergoing renovation) with 4 to 1 sidewall slopes will require individual basin dimensions of 815 feet by 240 feet. Basin depth is estimated at 5 feet. Excavation, appurtenances, monitoring networks, and land easement costs are the same as previously described for Case A. Preparation of specifications, construction supervision, overhead, and contingency costs are also based on the same percentages of total costs as previously described. A cost analysis of this recharge option (Case B) is presented in table 17.

Based on an average infiltration rate of 3 feet per day, a total basin area of 23.3 acres will be required to recharge 4,165 acre–feet of water to the aquifer in 60 days (8,330 acre–feet in 120 days). Applying a basin width–to–length ratio of about 4 to 1 and using 10 pairs of basins (one operational and one undergoing renovation) with 4 to 1 sidewall slopes will require individual basin dimensions of 690 feet by 190 feet. Basin depth is estimated at 5 feet. Excavation, appurtenances, monitoring networks, and land easement costs are the same as previously described for Case A. Preparation of specifications, construction supervision, overhead, and contingency costs are also based on the same percentages of total costs as previously described. A cost analysis of this recharge option (Case C) is presented in table 17.

Preliminary Project–Scale Surface Spreading Facilities Design

Preliminary surface–spreading design criteria in the project area include the following:

1) Total surface–spreading area must accommodate an annual recharge volume of 8,330 acre–feet of water (4,165 acre–feet over a 60–day period in the spring and 4,165 acre–feet over a 60–day period in the fall).
2) Raw (turbid) water from the James River will be imported to surface–spreading areas without pretreatment.

3) Surface–spreading areas will be operated continuously during the 60–day recharge periods.

4) Renovation will consist of natural desiccation and periodic plowing to disrupt the continuity of the surface–impeding layer.

5) Grass will be planted on the surface–spreading areas to aid in filtering suspended solids and to maintain macropores in the form of root channels.

6) Earth dikes will be constructed around the perimeter of the surface–spreading areas. Maximum height of the dikes will not exceed 5 feet. Sidewall slopes will be 4 to 1 to prevent collapse and to control erosion.

7) Surface–spreading area stage will vary between 1 and 4 feet.

8) To minimize the development of perched ground–water mounds, the ratio of width to length of the surface–spreading areas should be about 4 to 1. However, natural land–surface topography in the project area will be an important factor in determining the shape and dimensions of spreading areas.

9) Average 60–day infiltration rates are estimated at 1 to 3 feet per day.

Based on average 60–day infiltration rates of 1, 2, and 3 feet per day, surface–spreading areas of 70 acres, 35 acres, and 23.3 acres, respectively, will be required to recharge 4,165 acre–feet of water to the aquifer in 60 days. It is premature to select surface–spreading area locations given the lack of detailed (1–foot contour interval) land–surface topographic maps for the project area. For this reason, preliminary computer simulations of artificial recharge, using surface–spreading areas, were not conducted.

Project–Scale Surface Spreading Cost Analysis

Based on average infiltration rates of 1, 2, and 3 feet per day, total surface–spreading areas of 70, 35, and 23.3 acres, respectively, will be required to recharge 4,165 acre–feet of water to the aquifer in 60 days (8,330 acre–feet in 120 days). Preliminary dimensions of the spreading areas are the same as previously described for basins. Dikes will be 5 feet high and have 4 to 1 sidewall slopes to prevent collapse and erosion. Locally derived lacustrine sediment will be used to construct dikes. Excavation for diking material is based on a unit cost of $1.25 per cubic yard. Land easement costs are based on $300.00 per acre. Surface–spreading area appurtenances are based on a cost of $10,000 per spreading area. A surface–spreading monitoring network is based on a cost of $5,000 per spreading area. Preparation of specifications, construction supervision, and overhead costs are based on 30 percent of the total costs. Contingency costs are based on 15 percent of the total costs. Operation and maintenance costs are not calculated because they are poorly defined at this stage of the feasibility study. A cost analysis of project–scale surface spreading, based on average infiltration rates of 1, 2, and 3 feet per day, is presented in table 18.

Project Scale Well Field and Recharge Basin Computer Simulation

Another objective of the modelling study was to simulate the short–term (2 years) operation of a project–scale well field and recharge basin system and estimate water–level response. Water levels in the project area are generally about 5 feet below land surface. Before project water can be imported to basin facilities for ground–water recharge, water levels in the aquifer must be lowered about 10 to 20 feet.

To estimate aquifer response to operation of a project scale irrigation and artificial recharge system, the well–basin configuration shown in figure 21 was simulated for two years. Each of the 15 irrigation wells was pumped continuously at a rate of 2,094 gallons per minute for 60 days (July–August), giving a total discharge of 8,330 acre–feet.

Two artificial–recharge periods (spring and fall), consisting of 60 days each, were simulated by the model. Each 60–day recharge period was divided into four 15–day recharge periods to simulate recharge and renovation of basin pairs. Total basin area was based on an average 15–day infiltration rate of 1 foot per day. Total recharge rate for each 60–day period was 35 cubic feet per second(4,165 acre–feet). Recharge basins were

Table 18. -- Cost analysis of project-scale surface spreading based on average infiltration rates of 1, 2, and 3 feet per day

Component		Cost		
		Case A[1]	Case B[2]	Case C[3]
Dikes		61,000	$ 43,000	$ 36,000
Appurtenances		200,000	200,000	200,000
Monitoring network		100,000	100,000	100,000
Land easements		27,000	14,000	10,000
Preparation of specifications, construction supervision, and overhead		116,000	107,000	104,000
Contingencies		76,000	70,000	68,000
	Total	$580,000	$534,000	$518,000

[1] Based on an average infiltration rate of 1 foot per day

[2] Based on an average infiltration rate of 2 feet per day

[3] Based on an average infiltration rate of 3 feet per day

simulated using recharge wells.

Ground–water evapotranspiration was set at 13 inches per year with an extinction depth of 8 feet. Natural ground–water recharge was set at 3 inches per year.

An operational summary of the simulation is shown in table 19. Note that recharge was set at zero for the first two 60–day artificial recharge periods (stress periods 2–5 and 7–10). The aquifer was not sufficiently dewatered to accommodate full–scale artificial recharge during these two recharge periods. The aquifer became sufficiently dewatered after the second year pumping period (stress period 12), and artificial recharge was simulated during stress period 14–17. After 152 days of recovery (stress period 17), water levels were again too high to accommodate full–scale artificial recharge during the spring of the second year. It is possible that the well field could be operated for up to two years (8,330 acre–feet withdrawn per year) prior to artificial recharge. This means that well construction and operation should be completed in advance of recharge basin construction.

The drawdown distribution at the end of stress periods 1, 6, 11, 12, and 17 are shown in figures 24–28. The drawdown distributions shown in figures 24–28 are estimates of the spatial and temporal drawdown during the initial operation of a full project–scale irrigation and artificial recharge system. These figures can be used to provide a preliminary assessment of the effects of lowering the water table on other ground–water appropriators in the proposed project area.

Total Cost of Project–Scale Well Field and Recharge Facilities

The cost of recharge facilities is dependent on method of recharge and average infiltration rates. These costs were presented in tables 17 and 18. The total cost of the project–scale well field and recharge facilities can be computed as the sum of the cost of a specific recharge method as related to infiltration rate (tables 17 and 18) and the cost of the project–scale well field (table 16). The most expensive well field and recharge system ($4,275,000) consists of 15 wells and recharge basins with dimensions that are based on an

Table 19. - - Operational summary of pilot–scale well field and recharge basin computer simulation

Stress Period Number	Length of Stress Period, in Days in Acre–Feet	Operation	Volume of Water
1	60	pump wells	8,330
2	15	recharge basins	0
3	15	recharge basins	0
4	15	recharge basins	0
5	15	recharge basins	0
6	152	recovery	0
7	15	recharge basins	0
8	15	recharge basins	0
9	15	recharge basins	0
10	15	recharge basins	0
11	33	recovery	0
12	60	pump wells	8,330
13	15	recharge basins	1,041
14	15	recharge basins	1,041
15	15	recharge basins	1,041
16	15	recharge basins	1,041
17	152	recovery	0

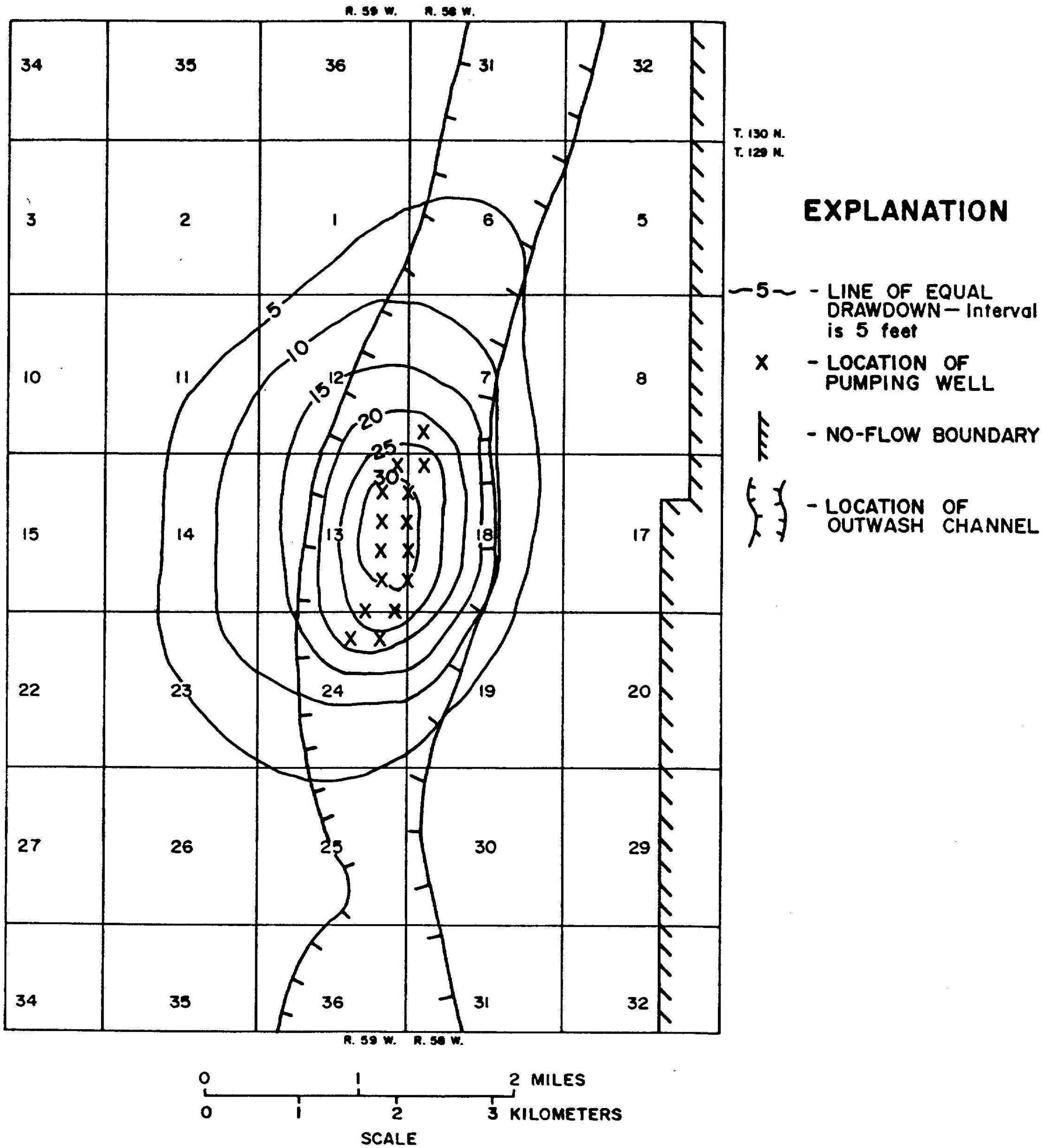

Figure 24.--Project-scale well field and basin recharge computer simulation: drawdown distribution at the end of stress period #1

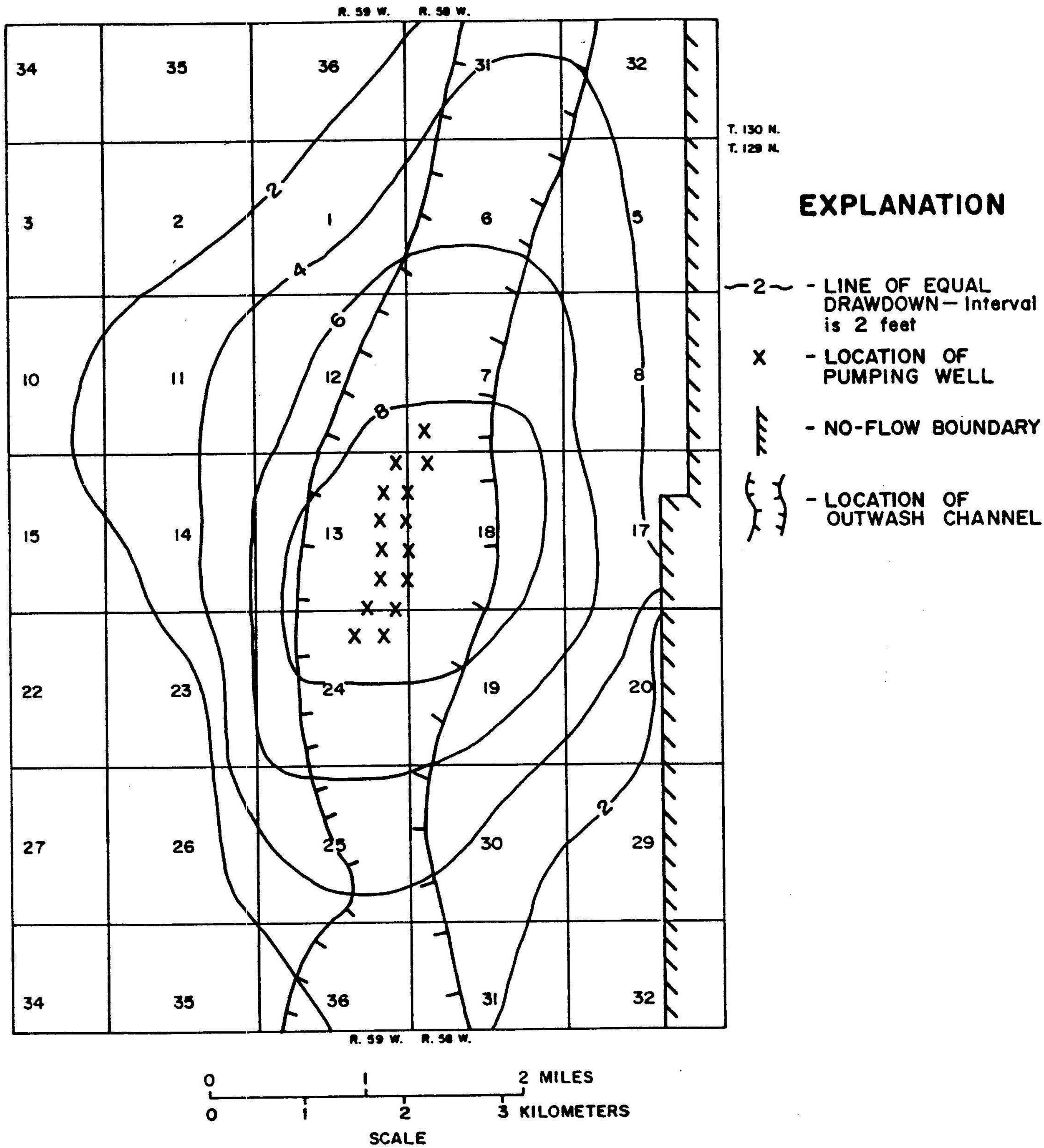

Figure 25.--Project-scale well field and basin recharge computer simulation: drawdown distribution at the end of stress period #6

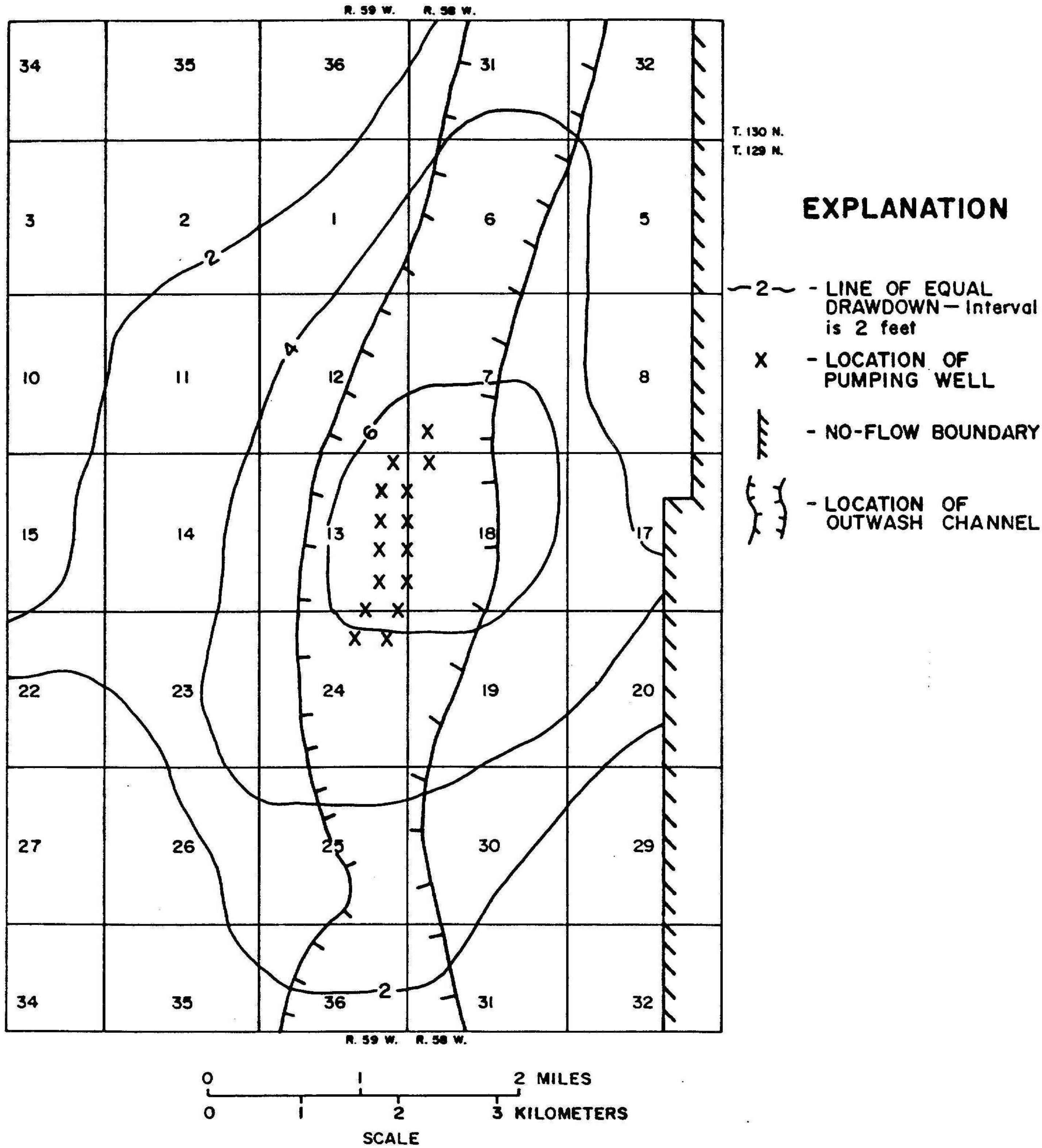

Figure 26.--Project-scale well field and basin recharge computer simulation: drawdown distribution at the end of stress period #11

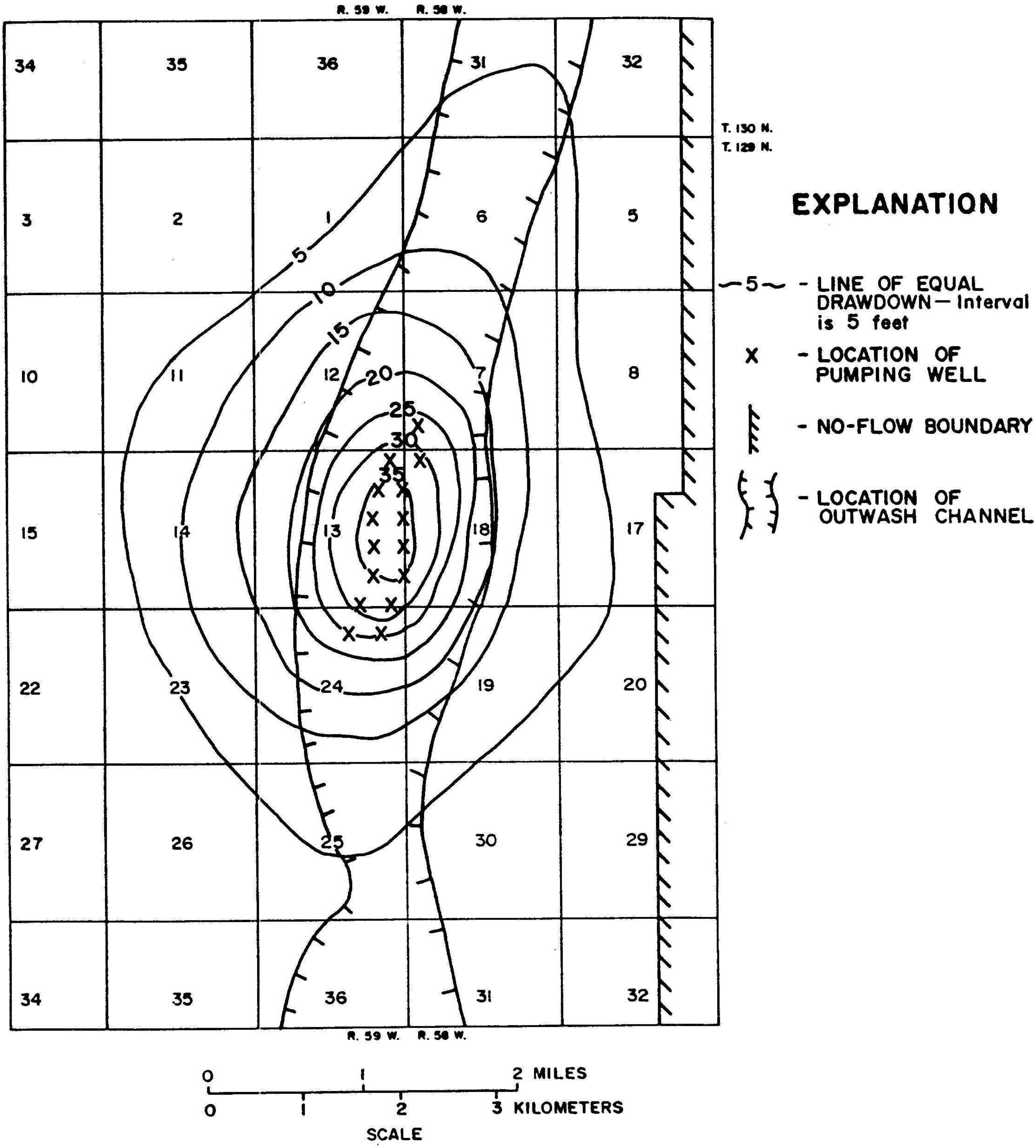

Figure 27.--Project-scale well field and basin recharge computer simulation: drawdown distribution at the end of stress period #12

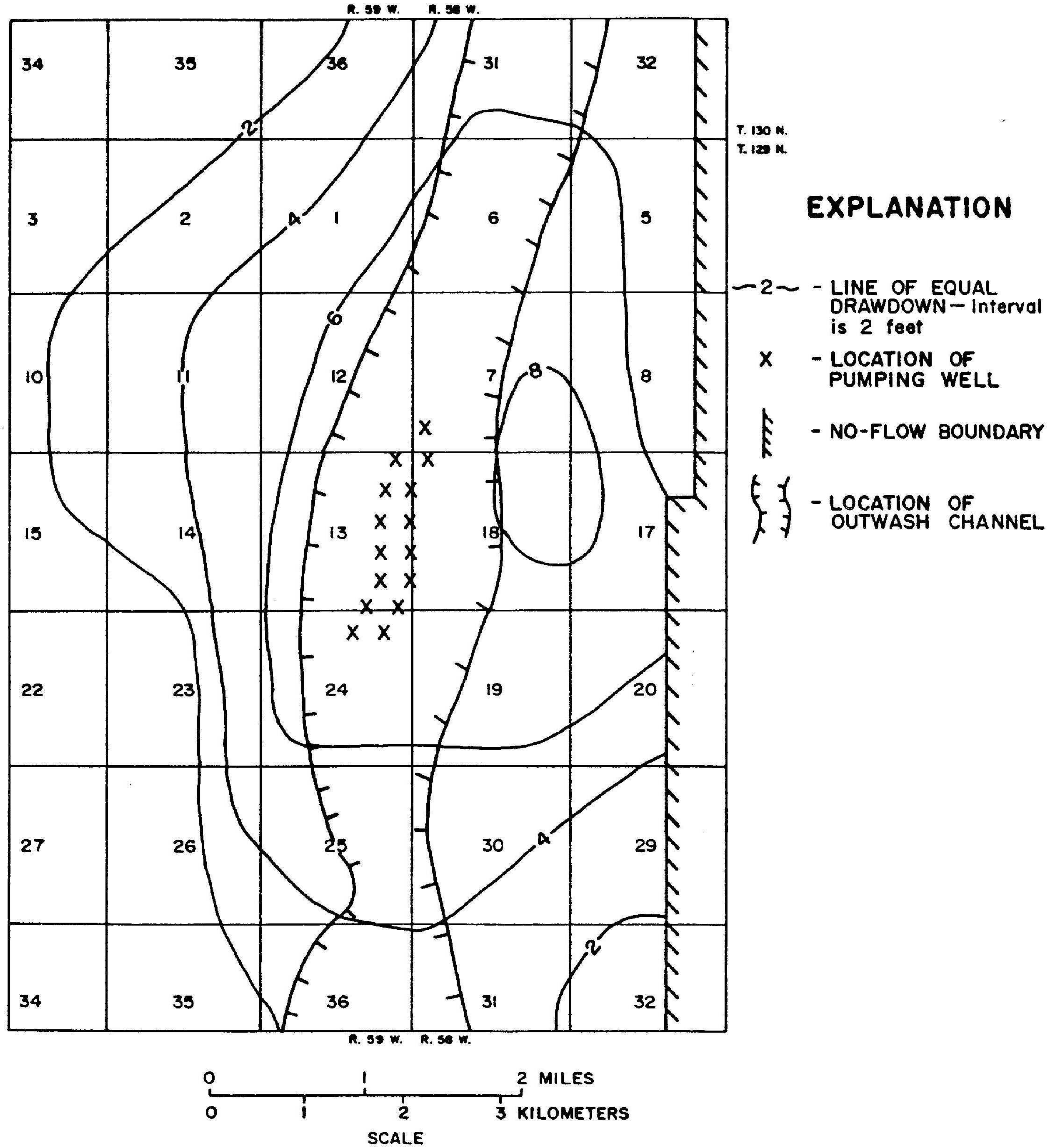

Figure 28.--Project-scale well field and basin recharge computer simulation: drawdown distribution at the end of stress period #17

average infiltration rate of 1 foot per day. The least expensive well field and recharge system ($1,772,000) consists of 15 wells and surface–spreading areas, with dimensions that are based on an average infiltration rate of 3 feet per day. Operation and maintenance costs for recharge facilities are not included in the above cost estimates.

SUMMARY AND CONCLUSIONS

The area of the Oakes aquifer most feasible for a project–scale well field and artificial recharge system is located in the channel–fill sand and gravel deposits near Section 13, Township 129 North, Range 59 West. In this area, the aquifer generally is unconfined, anisotropic, and nonhomogeneous with the coarsest deposits comprising the bottom one–half of the aquifer. Estimated average aquifer parameters for the channel–fill deposits in the project area are:

1) Hydraulic conductivity – 775 feet per day
2) Storativity – 0.20
3) Saturated thickness – 120 feet
4) Transmissivity – 93,000 feet squared per day

Preliminary well design is based on a discharge rate of 3,000 gallons per minute per well, screening the bottom one–third (40 feet) of the aquifer, and a pumping level not to exceed two–thirds of the available head (53 feet) above the top of the screen. A casing diameter of at least 18 inches (17.25–inch I.D.) is required to accommodate a pump of sufficient size to withdraw 3,000 gallons per minute. To allow enough clearance for installation and efficient operation, a casing diameter of 20 inches (19.25–inch I.D.), with a standard wall thickness of 0.375 inches, is recommended. Ground water in the project area is non–corrosive and as a result, a low–carbon steel well casing can be used.

Existing irrigation wells in the project area are completed with variable lengths of screen, ranging in slot size from 0.100 to 0.150 inch. Forty feet of 18–inch telescopic screen within this slot–size range is sufficient to transmit about 3,000 gallons per minute. Ground water in the project area is incrusting as indicated by high carbonate hardness and high iron and manganese concentrations. In addition, iron bacteria growth occurs in commercial wells and Bureau of Reclamation drains completed in the Oakes aquifer. Chemical treatment and pasteurization may be required to mitigate the effects of incrustation and iron bacteria growth. These remedial measures are corrosive and, therefore, non–corrosive stainless–steel well screen is preferred.

Deep–well vertical turbine pumps are recommended for production wells in the project area. Based on estimated well yield and lift requirements, a 15–inch O.D. single–or two–stage bowl assembly is required. The number of stages, impeller type, and trim will be determined after test pumping each well. The bowl assembly will be connected to 80 feet of 12–inch O.D. pump column with 1.5–inch diameter lineshaft. Because static water levels in the project area are about 5 feet below land surface, a water–lubricated lineshaft is preferred. The power supply will be a 75 horsepower hollow–shaft electric motor. The minimum size of the discharge head will be 12 inches.

Conventional forward, mud–rotary and drive–core drilling methods are recommended for exploratory test drilling. The conventional–rotary method will employ clay–based drilling fluids to maintain circulation and prevent hole collapse. This drilling method will be used to determine aquifer geometry and to select pilot–hole sites for production wells. Pilot holes will be drilled and sampled using a drive–core method. Sieve analyses will be performed on samples at selected intervals and will provide the basis for screen design.

There are four methods that can be used for production–well drilling and screen installation in the project area. These include:

1) Conventional rotary.
2) Conventional rotary and cable–tool drilling using the pull–back method to install screen.
3) Conventional rotary using a bail– or wash–down method to install screen.

4) Reverse rotary.

Based on preliminary hydrogeologic data, the reverse–rotary method appears to be the most efficient method of production–well drilling. The holes can be drilled quickly and economically and no casing is required during the drilling operation. Clay–based drilling additives are not used and well screens can be set easily as part of the casing installation.

Basins and surface–spreading methods of artificial recharge are practical in the project area. Initial--infiltration rates, measured at selected sites and coupled with soils data, indicate initial–infiltration rates in excess of 1 foot per day. Although surface spreading is more economical than basin recharge in the project area, surface spreading may be precluded because of the occurrence of a surficial fluvial deposit of silty clay and buried A soil horizons consisting of up to about 14 percent clay. Preliminary data indicates artificial–recharge facilities (basins or surface spreading) will be located in sections 13 and 24, T. 129 N., R. 59 W., and Sections 18 and 19, T. 129 N., R. 58 W.

A two–dimensional finite–difference model of ground–water flow in the Oakes aquifer was developed by the North Dakota State Water Commission in 1981. The model was inadequate as a long–term predictive management tool because annual recharge and evapotranspiration rates could not be calculated internally. The model was modified for this study and used to:

1) Develop a preliminary design of a pilot and project–scale well field.
2) Estimate the effects on aquifer water levels of a continuous withdrawal of 100 cubic feet per second for 60 days from a project–scale well field.
3) Estimate the effects on aquifer water levels of a continuous withdrawal of 60 cubic feet per second for 34 days from a pilot–scale well field.
4) Estimate the effects on aquifer water levels of a continuous withdrawal of 70 cubic feet per second for 60 days from a project–scale well field, operating in conjunction with artificial–recharge facilities supplying a continuous rate of 35 cubic feet per second for 120 days (60 days in spring, 60 days in fall).

Computer simulations indicate the pilot–scale well field will consist of nine wells each pumping at a rate of 3,000 gallons per minute. The wells will be spaced 1,000 feet apart and will form two roughly north–south trending parallel lines along the central axis of the outwash channel near Section 13, T. 129 N., R. 59 W. The project–scale well field will consist of the nine pilot wells plus six additional wells extending north and south from the pilot–well field spaced 1,000 feet apart.

The estimated cost of the pilot–scale well field (nine wells) is $599,400. The estimated cost of the project–scale well field (15 wells) is $1,254,000. Project–scale well field cost does not include well and pump maintenance and rehabilitation costs.

A pilot–scale, artificial–recharge study is prerequisite to the development of full–scale, artificial–recharge projects. Artificial–recharge systems are site–specific and require pilot studies to develop design, operation, and management criteria for full–scale projects. The irrigators in the Garrison Diversion Unit must pay operation and maintenance costs of project facilities, including canals, drains, wells, recharge basins, and spreading areas. Data from the pilot–scale recharge study will provide the basis for determining these costs.

It is imperative that the pilot–scale well field and recharge–test facilities be completed and operated in a timely manner to avoid delays when Missouri River water is delivered to the project area. The pilot–scale, recharge–test facilities should be operated for about 5 to 6 years.

Computer simulations of both pilot– and project–scale well field and artificial recharge systems indicate the water table will be lowered in and around the project area. For project–scale development, maximum drawdowns of about 40 feet within the well field are predicted at the end of each irrigation season prior to the fall recharge period. Drawdowns of less than about 2 feet are predicted in areas 2 to 3 miles from the well field. Adverse effects caused by the changed water–table condition may include loss of stock ponds, stock and/or domestic wells, sub–irrigation and wetlands. Net benefits from lowering the water table include a reduction in surface ponding and water logging of soils in wet years. The operation of a pilot–scale well field and artificial–recharge test facilities will provide the basis for evaluating and predicting effects on the water table resulting from project–scale development.

The following artificial recharge tests are recommended for the pilot–scale investigation:

1) Basin recharge using raw, turbid James River water.
2) Basin recharge using James River water pretreated with chemical flocculants to remove suspended solids.
3) Basin recharge using an organic mat.
4) Surface spreading using raw, turbid James River water.
5) Contingency testing.

Based on a 6–year test period, the total cost of the pilot–scale investigation is estimated at \$2,335,900. This \$2,335,900 does not include the cost of a supply system network to convey water to and from the well field and artificial recharge areas.

Due to the site–specific nature of artificial–recharge operations, it is premature to determine the most efficient and cost effective artificial–recharge method in the project area. For preliminary planning purposes, basin recharge and surface spreading was evaluated using raw James River water. Based on an average infiltration rate of 1, 2, and 3 feet per day, basin or surface–spreading areas of 70, 35, and 23.3 acres, respectively, are required to artifically recharge 8,330 acre–feet of water in 120 days (60 days spring, 60 days fall). The most expensive project–scale well field and artificial–recharge system (\$4,275,000) consists of 15 wells and 20 recharge basins, with basin dimensions based on an average infiltration rate of 1 foot per day. The least expensive well field and artificial–recharge system (\$1,772,000) consists of 15 wells and selected surface–spreading areas with spreading–area dimensions based on an average infiltration rate of 3 feet per day. Operation and maintenance costs for recharge facilities are not included in the above cost estimates.

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APPENDIX 1

Well–Electrical Power Costs

The following formula is used to estimate cost per hour of operation for each well in both the pilot– and project–scale well fields:

$$\text{Cost per hour of operation} = \frac{Q \cdot TDH \cdot 0.746 \cdot \text{cost per KWHR}}{3960 \cdot \text{overall pump efficiency} \cdot \text{motor efficiency}}$$

where

Q = pumping rate, in gallons per minute
TDH = total dynamic head, in feet
3960 = factor derived from the following expression:

$$\left(\frac{8.33 \text{ pounds of water}}{\text{gal.}}\right) \div \left(\frac{33{,}000 \text{ ft–lbs}}{\text{min.}}\right) \div \text{horsepower}$$

0.746 = factor for converting horsepower hours to kilowatt hours.

Cost per kilowatt hour is based on Pick–Sloan Missouri River Basin energy charges of \$.0025/Kwh. This does not include any wheeling or transmission line maintenance costs. An overall pump efficiency of 0.70 and an electric motor efficiency of 0.90 were selected and applied to the above formula. A pumping rate of 3,000 gallons per minute per well was also selected based on the yield capabilities of the aquifer in the proposed project area.

Total dynamic head consists of pump lift, pump column friction loss, friction loss through discharge head and fittings, and head of water above the discharge head. For this pumping plant cost analysis, head of water above the discharge head is zero. Maximum pump lift in the project area is estimated at 85 feet. Friction loss through both the pump column and discharge head is estimated at about 3 feet. Total dynamic head is estimated at 88 feet.

Using the above formula and data input, the estimated cost per hour of operation of each well is \$0.20/hr.

$$\text{Cost per hour of operation} = \frac{(3{,}000)(88)(0.746)(.0025)}{(3960)(0.70)(0.90)} = 0.20$$