FEASIBILITY OF ARTIFICIAL RECHARGE TO THE OAKES AQUIFER, SOUTHEASTERN NORTH DAKOTA: SUMMARY

By Robert B. Shaver
and William M. Schuh

Water Resources Investigation No. 9
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. 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 5 and 6. Investigation No. 5 (Shaver and Schuh, 1989) describes the hydrogeology of the Oakes aquifer. Investigation No. 6 (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 described in North Dakota State Water Commission Water-Resources Investigation No. 7 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 (Schuh and Shaver, 1988). The report prepared by the U.S. Geological Survey describes the chemical and biological processes operative during basin recharge.

Phase III of the artificial-recharge feasibility study (North Dakota State Water Commission Water-Resources Investigation No. 8) describes a preliminary design and cost-estimate analysis of a full project-scale and pilot-scale well field and artificial recharge system for the Oakes aquifer. The phase III study was prepared by the North Dakota State Water Commission (Shaver, 1989).

This report summarizes the phase I, II, and III reports prepared by the North Dakota State Water Commission.

DESCRIPTION OF THE STUDY AREA

Physiography and Climate

The study area is located in southeastern North Dakota (fig. 1). The study area occupies a flat lake plain. Relief generally is less than 10 feet per mile. Locally, the topography is hummocky because of scattered sand dunes and blowouts. Runoff from the lake plain is minor as indicated by the lack of surface drainage.

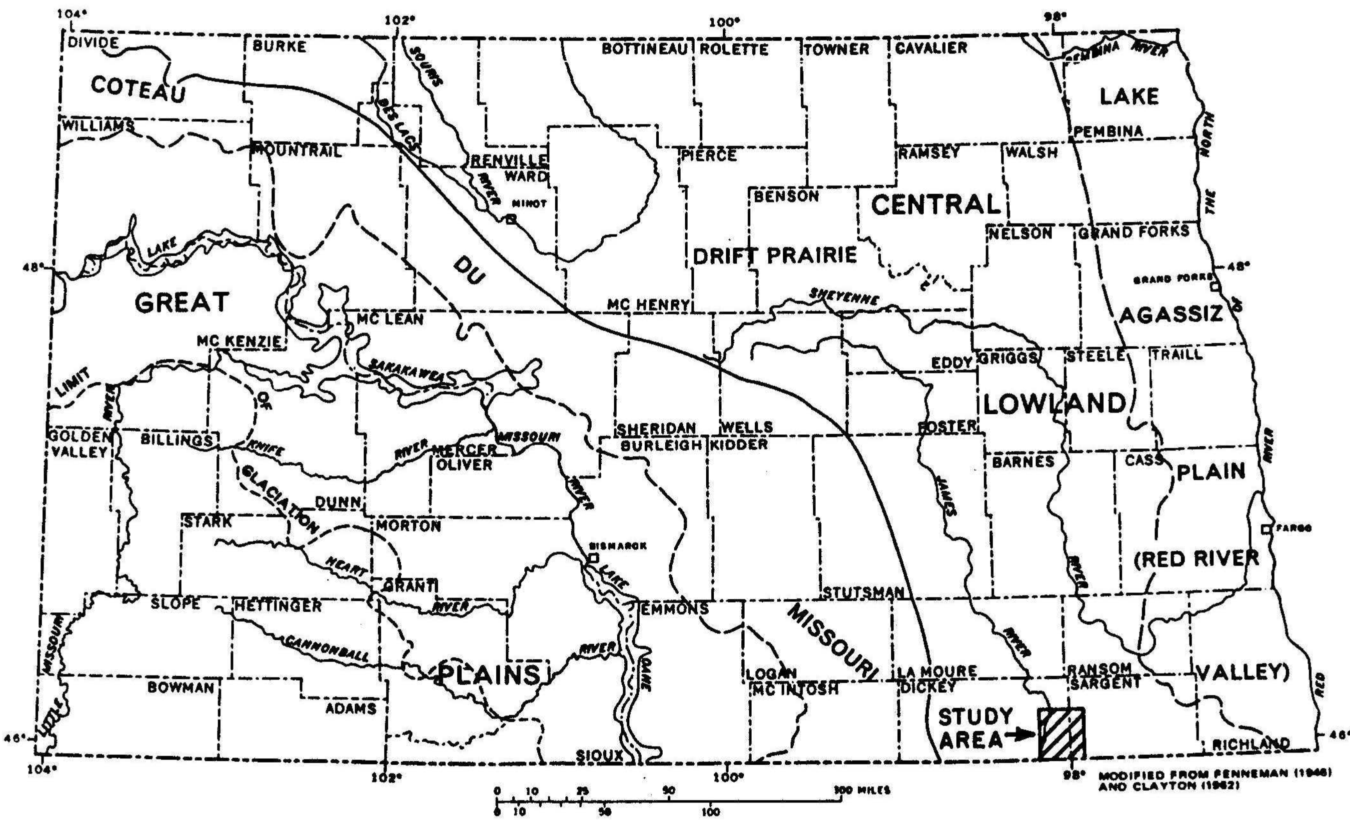

Figure 1.--Physiographic divisions in North Dakota and location of study area

Climate of the study area is semiarid to subhumid. Mean annual precipitation at Oakes from 1931 through 1974 is 19.21 inches. About 70 percent of the precipitation generally falls from April through August. Mean annual temperature at Oakes from 1951 through 1980 is 40.9^{O}F.

Geology

In descending order throughout most of the study area, the stratigraphic section consists of sand or sand and gravel, silt and clay, till, and bedrock shale (fig. 2). The Oakes aquifer consists predominately of sand or sand and gravel deposits of the Pleistocene Coleharbor Group (Bluemle, 1979).

Near Oakes, stratified sand and gravel deposits form a deltaic complex up to about 80 feet thick (fig. 3). The deltaic deposits grade into lacustrine sand south of Oakes. Medium sand predominates in the central part of the lake plain. South of Ludden, the medium sand grades into fine to very fine silty sand, clayey silt, and silty clay. Average thickness of the lacustrine deposits is about 35 feet.

Channel-fill deposits, consisting of stratified very fine sand to coarse, cobbly gravel up to 197 feet thick, occur in an outwash channel along the eastern margin of the lake plain (fig. 3). A surface to near-surface fluvial silt and clay deposit overlies the northern and central parts of the outwash channel

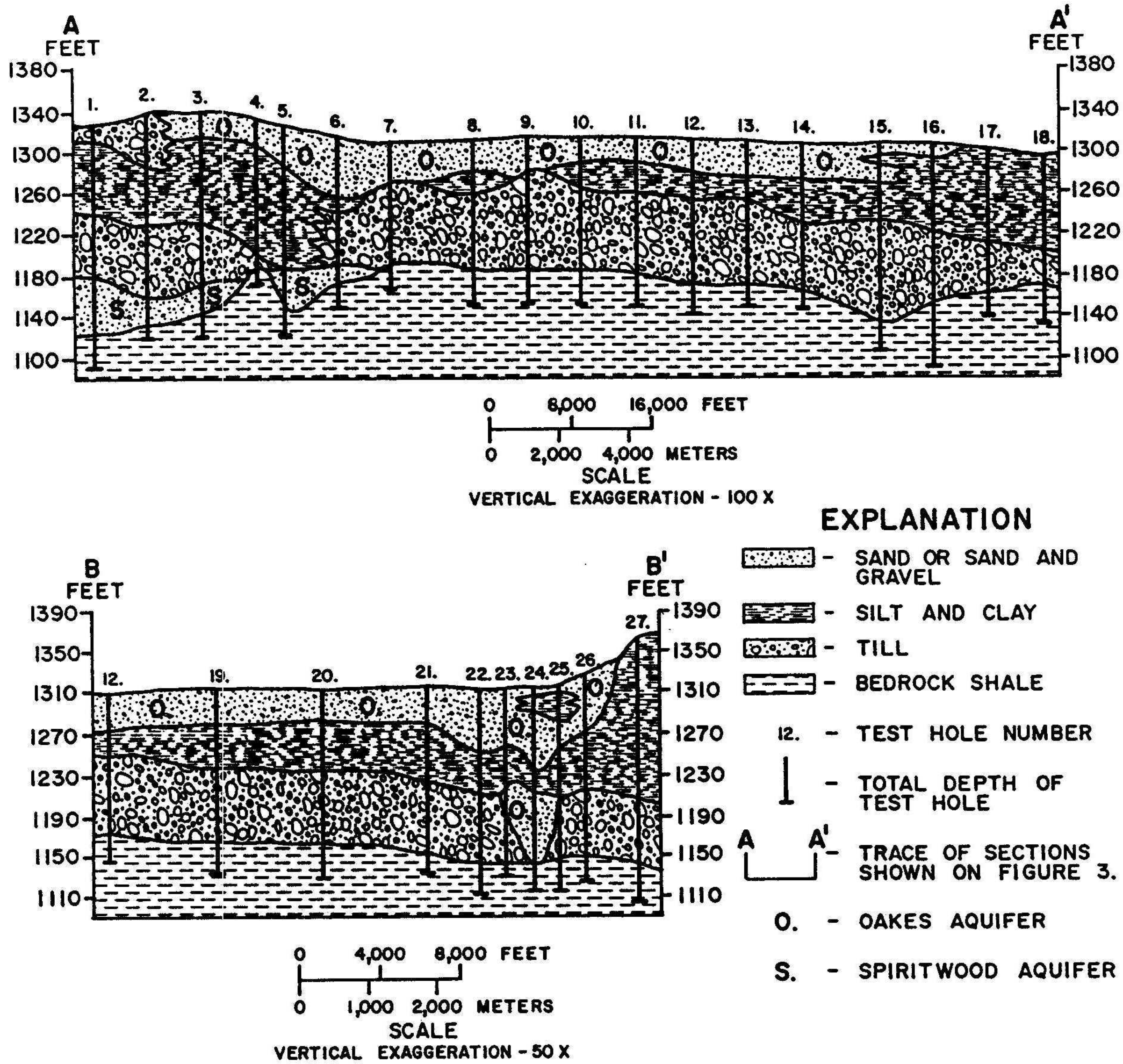

Figure 2.--Geologic sections A-A¹ and B-B¹ showing the Oakes aquifer

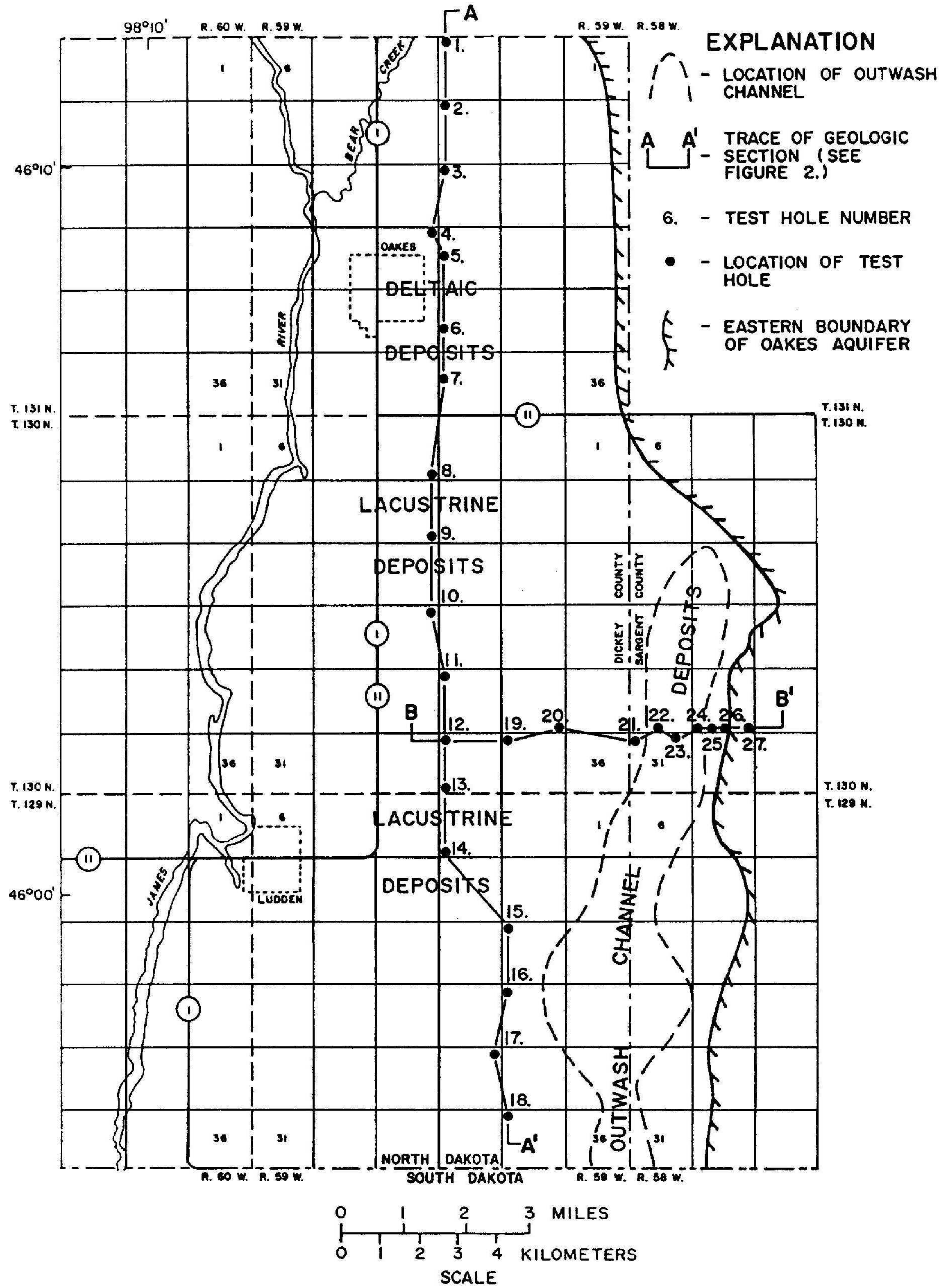

Figure 3.--Areal distribution of depositional facies in the Oakes aquifer and locations of geologic sections A-A' and B-B'

(fig. 4). The deltaic, lacustrine, and channel-fill deposits which comprise the Oakes aquifer are composed of quartz, shale, carbonates, Canadian Shield silicates and lignite fragments.

GROUND-WATER HYDROLOGY

Occurrence and Movement of Ground Water

For the most part, the Oakes aquifer is unconfined. Water occurs under leaky-confined conditions where fluvial silt and clay sequences overlie the lacustrine and channel-fill deposits (fig. 4).

In general, regional ground-water flow in the Oakes aquifer is from east to west toward the James River valley (fig. 5). Recent James River valley floodplain deposits, consisting of sandy silty clay, truncate the western flank of the Oakes aquifer. As a result, ground-water flow from the Oakes aquifer westward to the James River is negligible.

Ground-Water Recharge and Discharge

Depth to the water table in the Oakes aquifer generally is less than 8 feet below land surface. Scattered sand dunes and blowouts cause a hummocky land-surface topography. Therefore, the Oakes aquifer consists of numerous, localized flow systems. Within each local flow system, recharge is from direct infiltration of precipitation and local runoff that occurs primarily during the spring. Discharge primarily is from

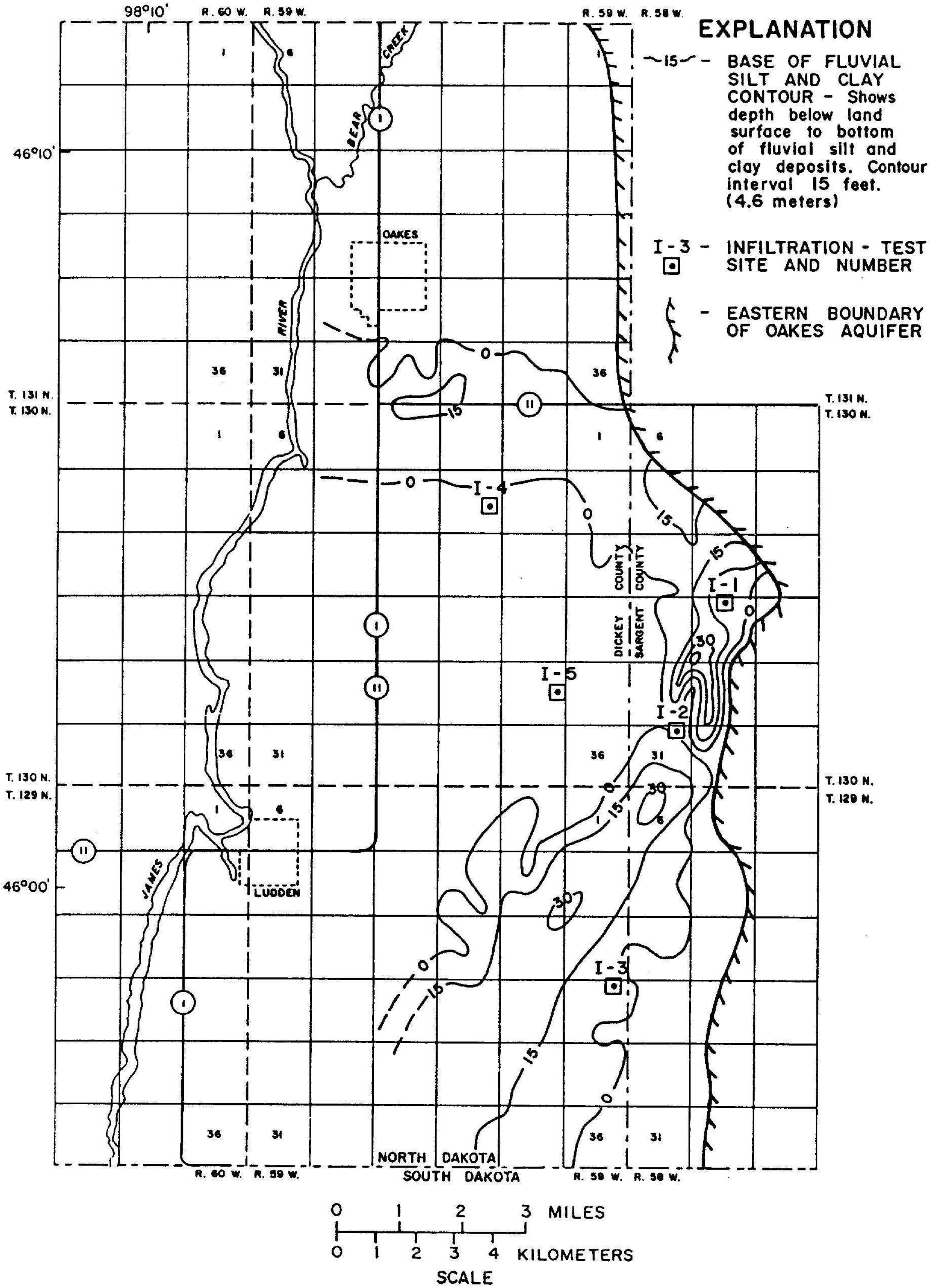

Figure 4.--Depth below land surface to bottom of fluvial silt and clay deposits

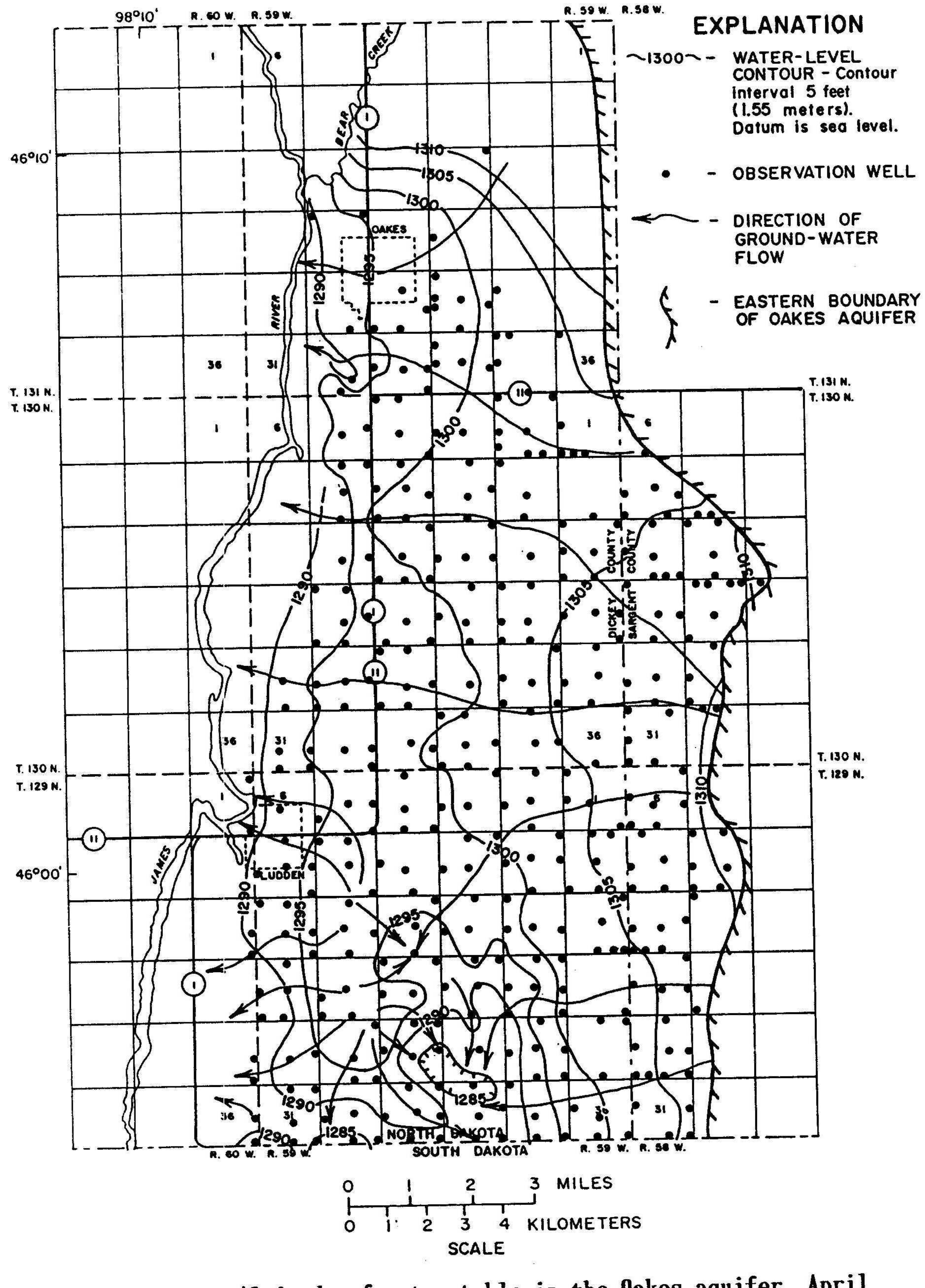

Figure 5.--Altitude of water table in the Oakes aquifer, April 1988

evapotranspiration that occurs during the summer. Determining recharge and natural discharge in the Oakes aquifer is virtually impossible because of the inability to describe spatial variation in precipitation, land-surface topography, soil physical properties, and the evapotranspiration process.

Aquifer Transmissivity and Well Yield

There are two large-transmissivity areas of the Oakes aquifer that can accommodate individual well yields of greater than 500 gallons per minute. (Shaver and Schuh, 1989). These areas are: (1) the northern part of the study area near Oakes (deltaic sand and gravel deposits); and (2) the eastern flank of the lake plain (channel-fill sand and gravel deposits) (fig. 6). As compared to the deltaic deposits, the channel-fill deposits have the largest transmissivity at 94,000 feet squared per day and will provide well yields of about 3,000 gallons per minute.

Surface Infiltration Rates

Surface and shallow basin infiltration tests conducted at five sites overlying the Oakes aquifer indicate initial-infiltration rates ranging from 2.5 to 67 feet per day (Shaver and Schuh, 1989). The locations of the infiltration-test sites are shown in figure 4. Initial infiltration rates are adequate for large-scale surface recharge facilities in sediments overlying most of the Oakes aquifer. Buried A soil horizons, containing up to about 14 percent clay and surficial fluvial silt and clay sequences (fig. 4), overlie parts of the lacustrine and

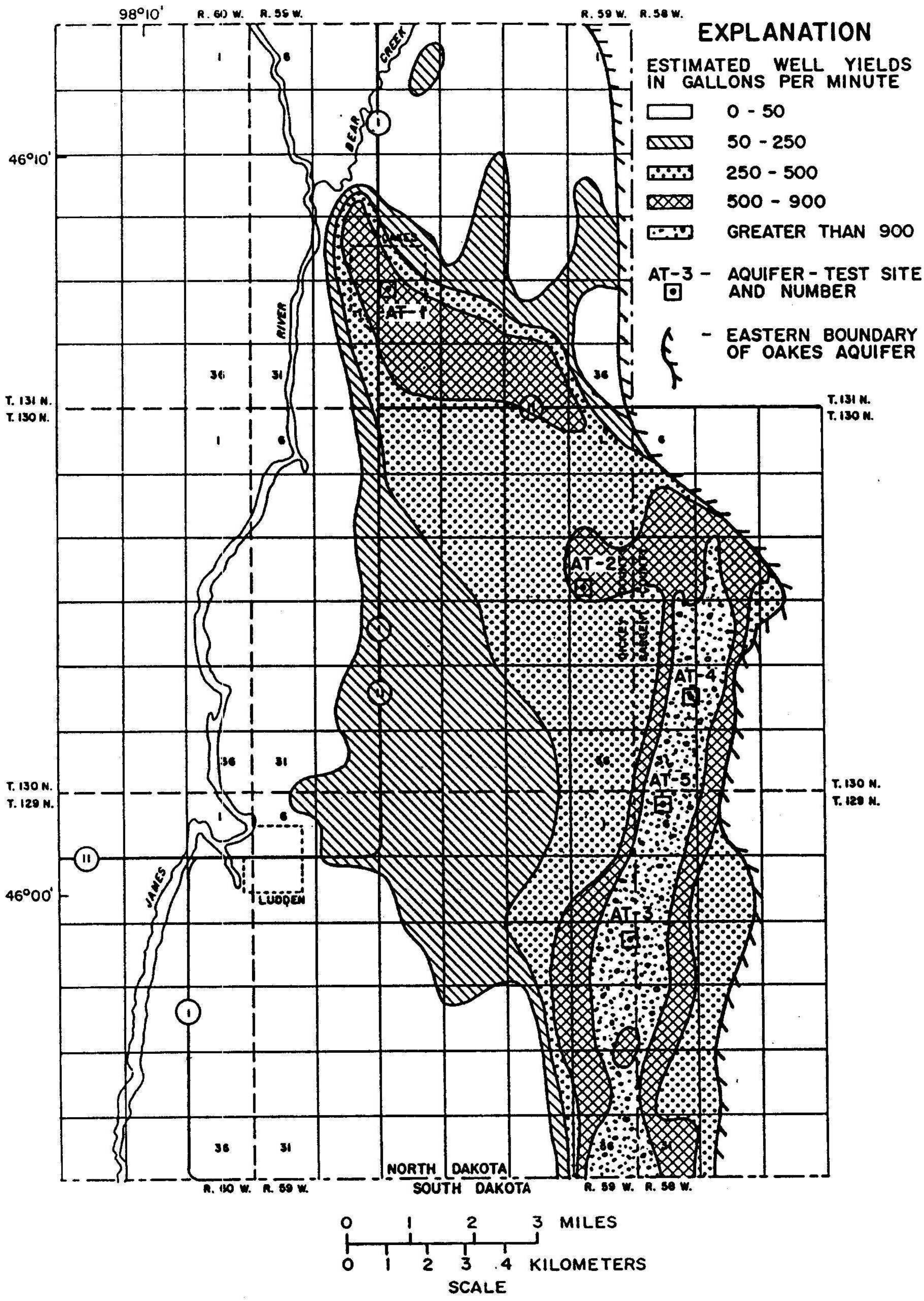

Figure 6.--Estimated well yields in the Oakes aquifer

outwash channel deposits. In these areas, surface-recharge facilities will be precluded because surface-infiltration rates are estimated to be significantly less than one foot per day.

Water Quality

Water quality in the Oakes aquifer is variable. Dissolved-solids concentrations range from less than 300 to more than 20,000 mg/L (fig. 7). Ground water with small (less than 650 mg/L) dissolved-solids concentrations is a calcium-magnesium bicarbonate type (fig. 8). The small-dissolved solids, calcium-magnesium bicarbonate type predominates in the Oakes aquifer and poses no limitations for irrigation use. Ground water with dissolved-solids concentrations greater than 2,000 mg/L is a sodium-magnesium sulfate or magnesium-sulfate type and occurs beneath closed land-surface depressions that represent net discharge areas. In these limited areas, ground water poses salinity, sodium, or boron hazards for irrigation use.

PROPOSED LOCATION OF A WELL-FIELD AND ARTIFICIAL-RECHARGE FACILITIES IN THE OAKES AQUIFER

The area of the Oakes aquifer most feasible for the development of a large-scale well-field and surface artificial-recharge facilities is located in the glacial-outwash channel near Section 13, Township 129 North, Range 59 West (fig. 9). This area was selected based on the following:

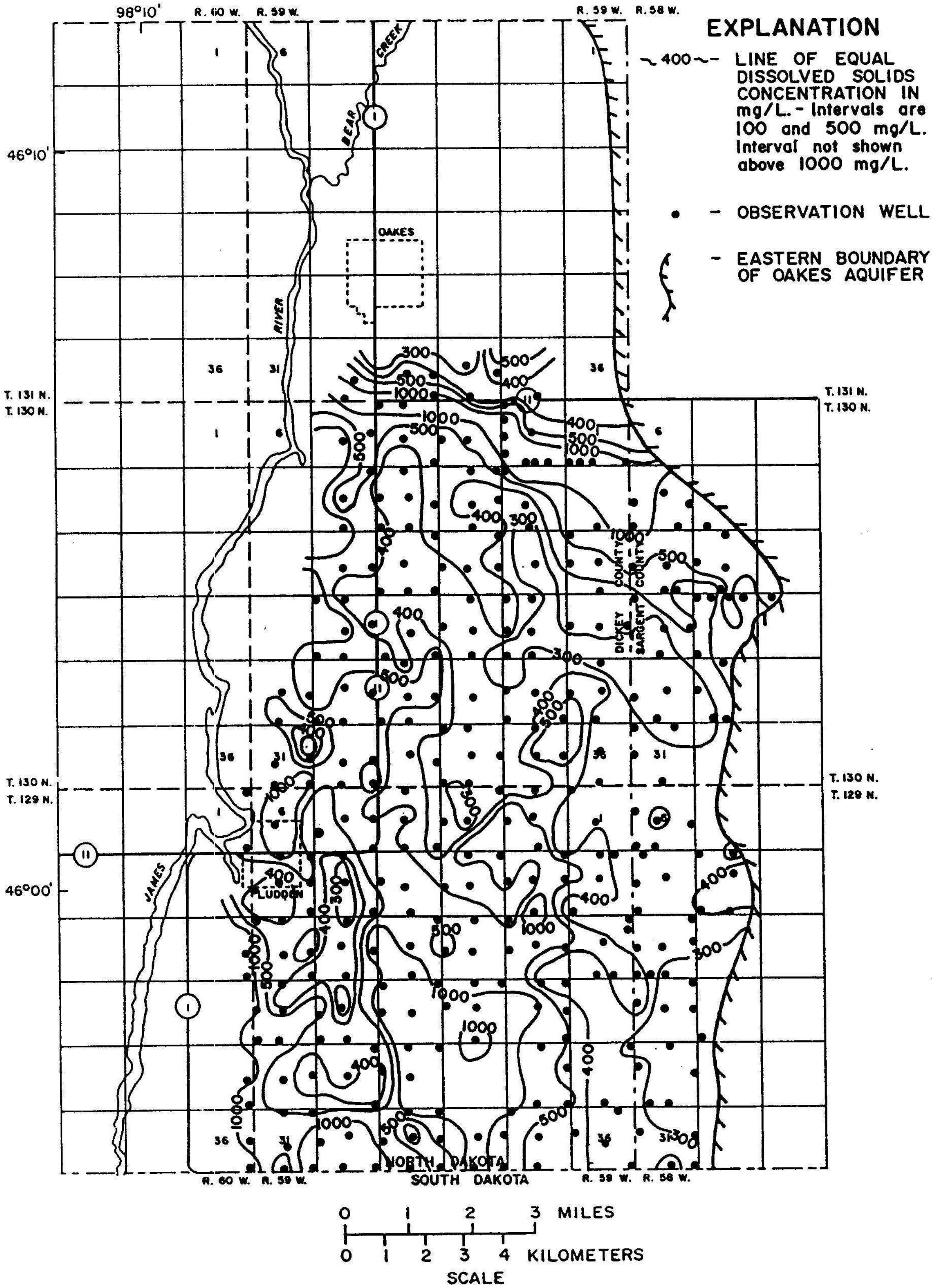

Figure 7.--Dissolved-solids concentrations in the Oakes aquifer

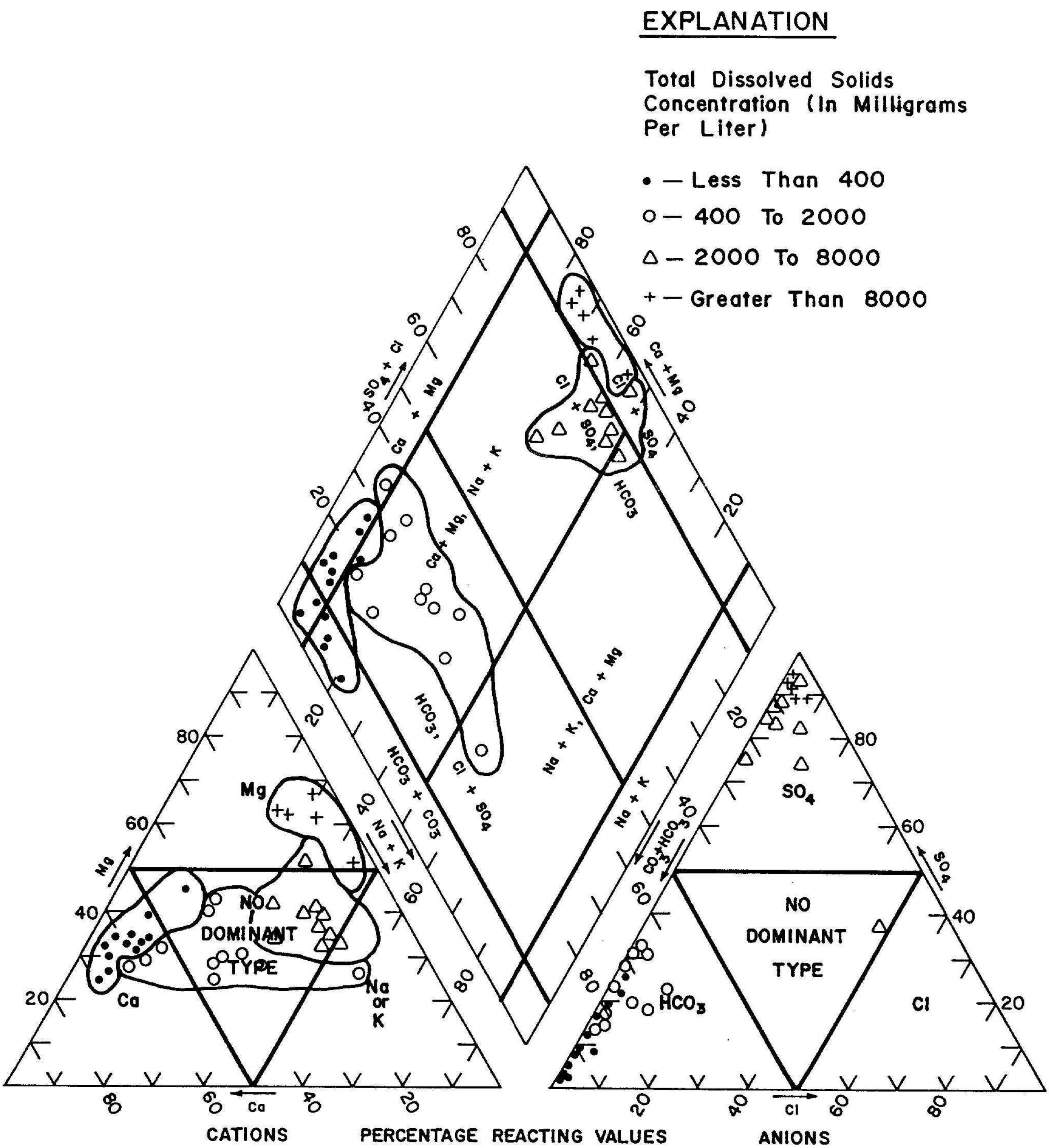

Figure 8.--Hydrochemical facies in the Oakes aquifer

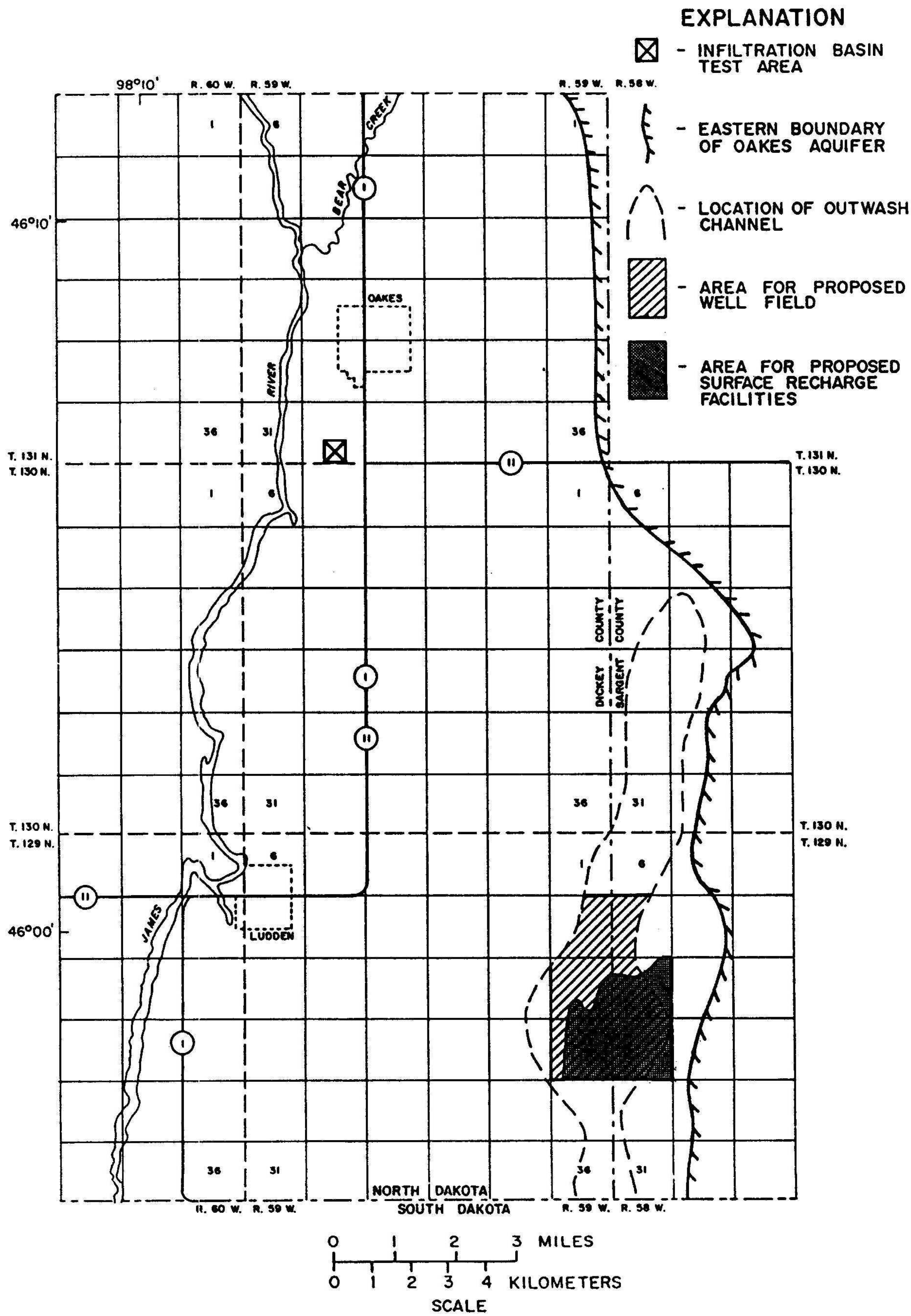

Figure 9.--Location of proposed well field and artificial-recharge facilities

1) The channel-fill deposits have the largest transmissivity in comparison to other depositional facies of the Oakes aquifer. Individual well yields of about 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, making the area conducive to the development of surface recharge facilities (basins and surface-spreading areas).

4) The chemical quality of the water collected from the channel-fill deposits in this area poses no limitations for irrigation use.

ARTIFICIAL GROUND-WATER RECHARGE TO THE OAKES AQUIFER

Based on a preliminary project area assessment (Shaver and Schuh, 1988), it was determined that abandoned pits, shafts, and well injection methods were not best suited for implementing artificial recharge in the Oakes aquifer. Three methods of recharge found to be most suited were: 1) basins, 2) surface spreading, and 3) canals (Schuh and Shaver, 1988).

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, basin recharge, and surface spreading can require large land areas. The land in the proposed project area is agricultural and is used primarily 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 feet and 10 feet thick. In those 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 outside perimeter to the wetted area is large.

ARTIFICIAL-RECHARGE TEST BASINS IN THE OAKES AQUIFER

In order to evaluate hydraulic interaction of turbid James River water with subsoils typical of the Oakes area, two small-scale artificial recharge test basins were operated periodically from September, 1986, to October, 1987. The two reharge test basins were located in the SE 1/4 of Section 29, Township 131 North, Range 59 West (fig. 9). Basin management practices examined included: 1) full renovation (F), which consisted of complete removal and replacement of the basin subsoil to an approximate depth of 2 feet; 2) desiccation (D), which consisted of a 10-day period of drying and desiccation of the basin surface followed by a period of basin operation; 3) use of an organic mat filter (OM), which consisted of a 4-inch thick surface mat of composted sunflower seed hulls; and 4) desiccation with an organic mat (OMD), which consisted of a 22-day desiccation period for the clogged organic mat followed by a period of basin operation. In addition, the effects of basin-ponded depth on infiltration rate were examined.

Areas of the two test basins were 200 square feet and 2500 square feet. The smaller basin was operated only once during the fall of 1986 under conditions of a clean, newly excavated basin floor. The larger basin was operated using each of the four listed management options. Operation schedules, basin-management regimes, and resulting total recharge for each of the basins are summarized on table 1. A detailed description and discussion of each of the test operations was presented by Schuh and Shaver (1988).

Effect of Turbid-Water Application on Infiltration Rate

The average-sediment composition of the James River for each of the test periods is summarized in table 2. Application of turbid water, which resulted in decreasing infiltration rates during basin operation, is illustrated in figure 10. The extent

Table 1 - Summary of recharge accomplished for each of the basin-management tests and projected equivalent required basin area for a project-scale (8,330 acre-feet per year) facility assuming operation and capabilities identical to those of each test. [Area projections are given for one operational period per year and two operational periods per year.]

Test Period	Basin[1] / Treatment[2]	Operation Time Per Period, in Days	Recharge, in Feet	Project-scale area required for 1 operation period per year, in acres	Project-scale area required for 2 operation periods per year, in acres
Fall 1986	S/F	25	196	43	21
Fall 1986	L/F	25	100	83	42
Spring 1987	L/F	25	81	103	51
Spring 1987	L/D	10	35	238	119
Spring 1987	L/F+D	35*	116	72	36
Fall 1987	L/OM	30	164	51	25
Fall 1987	L/OMD	10	46	181	91
Fall 1987	L/OM + OMD	40**	210	40	20

* not including 10 drying days

** not including 22 drying days

1 Location Key

S = small basin (10 x 20 ft.)
L = large basin (50 x 50 ft.)

2 Treatment Key

F = full renovation
D = desiccation

OM = organic material
OMD = organic material + desiccation

Table 2. Grain-size distributions for suspended solids and surface filter cake.

Test	Suspended solids, in milligrams per liter	Grain-size class	Suspended solids composition, in percent	Filter-cake composition, in percent
Fall 1986	50	Silt	23	71
		Clay	77	28
		Organic Carbon	-	2.8
Spring + Desiccation	68	Silt	32	53
		Clay	68	47
		Organic Carbon	-	4.0
Organic Mat + Desiccation	98	Silt	38	55
		Clay	62	45
		Organic Carbon	-	-
Average	69	Silt	31	60
		Clay	69	40

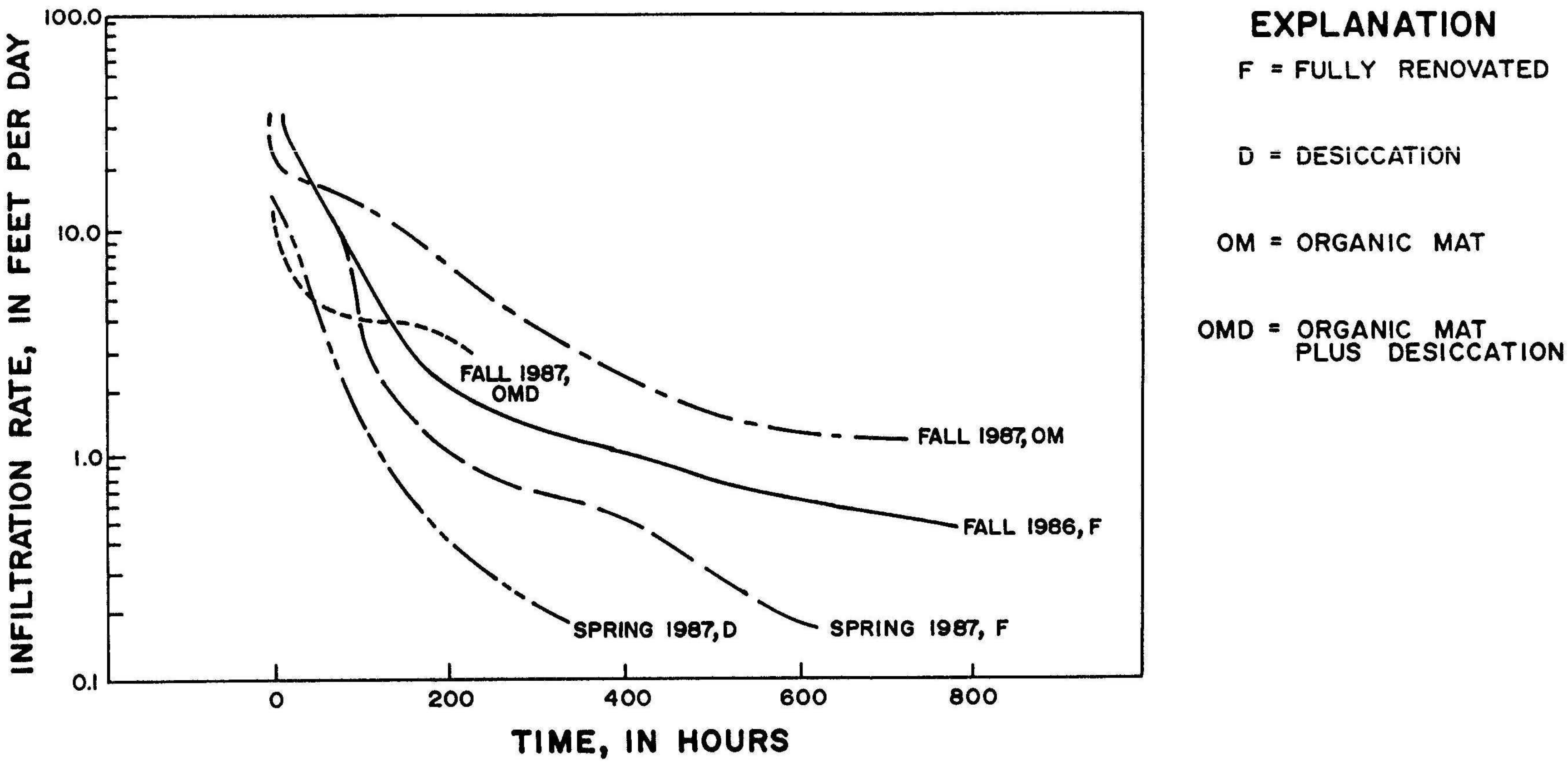

Figure 10.--Infiltration rate versus time for large test-basin operations

of basin infiltration-rate attenuation differed with management practices.

Full Renovation

* For the small basin (Fall 1986), the initial-infiltration rate was 78 feet per day and declined to 2.7 feet per day over 25 days of operation. Total recharge is listed on table 1. Full renovation was effected by the condition of initial construction.

* For the large basin (Fall 1986), the initial-infiltration rate was 31 feet per day and declined to 0.59 feet per day over 25 days of basin operation. Total recharge is listed on table 1. Full renovation was effected by the condition of initial construction.

* For the large basin (Spring 1987), the initial-infiltration rate was 34 feet per day and declined to 0.24 feet per day over 25 days of basin operation. Total recharge is listed on table 1. Full renovation was effected by removing approximately 2 feet of soil and replacing it with construction spoil. The replaced soil materials were slightly finer than those of the initial basin floor.

Desiccation

* The initial-infiltration rate, following 10 days of drying of the clogged-basin floor, was 23 feet per day (73 percent of

the Spring 1987 fully-renovated infiltration rate). The basin resealed quickly, however, and within 10 days reached the 0.24 feet per day rate measured for the clogged-basin soil before desiccation. Total recharge is shown on table 1.

Organic Mat

* The initial-infiltration rate was 27 feet per day, and declined to 1.3 feet per day over 30 days of basin operation. Total recharge is shown on table 1.

* The initial-infiltration rate, following 22 days of drying of the clogged organic mat, was 27 feet per day, but dropped almost immediately. After approximately 10 days of operation, the infiltration rate was still substantial at 3 feet per day.

Physical Changes of the Basin Subsoil Due to Turbid-Water Application.

The application of turbid James River water resulted in the formation of two fundamental levels of sediment clogging. These were: 1) a thin surficial filter-cake layer (averaging 0.04 to 0.08 inches in thickness), and 2) a layer of secondary clogging beneath the filter cake. During the early phases of basin operation, clay particles penetrated to varying depths within the basin subsoil. These are summarized in table 3. The length of the time during which deep penetration occurred varied from as little as 5 hours to approximately 200 hours. During this time,

Table 3. -- **Depth and degree of sediment penetration beneath the basin floor**

Test	Sediment grain-size class	Maximum depth of sediment penetration, in inches	Average grain-size fraction increase, in percent
Fall 1986	Clay	2.0	1.270
	Silt	1.0	0.620
	Organic carbon	1.0	0.054
Spring + Desiccation	Clay	3.0	0.900
	Silt	0.0	0.000
	Organic carbon	0.0	0.000
Organic Mat + Desiccation	Clay	9.0	1.270
	Silt	0.0	0.000
	Organic carbon	0.5	0.056

silt penetration into the basin subsoil was negligible. Silt particles were strained out on the surface, eventually forming a filter cake that intercepted clay particles and prevented further deep penetration. The preferential deep movement of clay particles is demonstrated by the differences between the silt and clay composition of the influent sediment and the filter-cake layer (table 2).

Full Renovation

* For the Fall 1986 test, significant clay penetration to 2 inches was detected in the combined large and small basins. Slight silt and organic carbon penetration to depths of 1 inch were also detected.

* For the Spring 1987 test, significant clay penetration to a depth of 3 inches was detected in the large basin. No increase in silt or organic carbon was detected. This included the combined full renovation and desiccation tests.

* During the Spring 1987 test, large quantities of algal colonies began to form early in the test. By the end of the test, little algae was visible, although the green color of chlorophyll was present within the filter cake. Large numbers of water beetles [Daphnia (sp.)] were observed, and it is speculated that consumption of algae by water beetles served to keep algal populations low. Algae effects on

infiltration rate, separate from sediment clogging, were not determined during these tests.

Organic Mat

* For large basin operations in which an organic-mat surface filter was used, deeper penetration of clay was observed (to 9 inches) than for previous tests without the organic mat. Silt and organic-carbon penetration were not detected. This comparison included the cumulative effect of consecutive fully-renovated, and post-desiccation tests.

Hydraulic Changes Due to Turbid-Water Application.

Hydraulic changes occurred as increases in soil impedance for the clogged subsoil layers.

Full Renovation

* For the spring and fall fully-renovated tests, a typical basin-subsoil profile indicated two to five orders of magnitude increase in impedance for the 0- to 3.2-inch layer, including the filter cake. Increases of zero to two orders of magnitude were indicated for the 3.2- to 9.1-inch layer. Increases of zero to one order of magnitude were observed to 15 inches. No increased impedance was observed below the 15-inch depth.

Desiccation

* During the 10-day desiccation period following the Spring 1987 basin operation, the impedance of the 0- to 3-inch layer decreased by one to two orders of magnitude, depending upon basin position.

* Following desiccation, no substantial recovery occurred below 3 inches.

* No further increases in basin-subsoil impedance occurred below 3 inches during the second operational period following desiccation. Thus, no further clogging was indicated.

* The surface 0 to 3 inches quickly resealed and reached an impedance higher than that of the first basin operation.

Organic Mat

* Impedance development at the 0.04- to 3-inch depth was significantly higher (two orders of magnitude) for basin areas without organic-mat cover than for those with organic-mat cover.

* Significant increases in impedance occurred to greater depths (15 inches) for areas with organic-mat cover than for those without the organic mat cover (9 inches).

* Transitory-clogging effects within the basin subsoil during basin operation indicated that gaseous clogging was likely. Carbon-dioxide evolution beneath the filter cake is considered to be most likely.

* Decreasing impedance for basin-subsoil layers below 3 inches during desiccation indicated that either organic matter or gaseous clogging had occurred during basin operation.

* Although changes in impedance were not measured within the organic-mat filter, it is likely that consolidation of the mat may have contributed to the decrease in long-term infiltration rate following desiccation.

MANAGEMENT CONSIDERATIONS FOR ARTIFICIAL-RECHARGE FACILITIES

For James River water delivered to the test basin, sediment load, rather than biological and chemical factors, was the primary infiltration-rate control during the limited duration of the recharge tests. Based on the documented depths of sediment penetration, the determined depth and degree of hydraulic impact, and the operational experience of other artificial-recharge facilities, the following considerations are presented:

1) Under conditions of infiltration through a natural sand filter, most of the clogging occurs in the top 3 inches. Within the top 3 inches, the greatest degree of clogging

occurs in the surface-filter cake, and in the fraction-of-an-inch underlying the filter cake. Substantial renovation should therefore be effected through shallow surface cleaning.

2) Some sediment penetration and attenuation of conducting capability is likely for soil layers below 3 inches. Increased long-term hydraulic impedance to depths of 15 inches have been documented. The effect of deeper-sediment penetration on recharge capacity is expected to be slight, however, and may be partially offset by increased porosity caused by tillage following cleaning of the basin surface.

3) Partial and temporary recovery of infiltration capacities can be obtained without cleaning by allowing the basin surface to dry and crack for at least 10 days. Initial recoveries of up to 73 percent of the fully-renovated infiltration rate may be effected. The rate of basin resealing, however, is much greater than for a fully renovated basin, and within 10 days the basin is fully incapacitated.

Desiccation and other forms of natural renovation would not likely be sufficient in themselves for maintaining adequate annual or long-term infiltration rates in recharge basins. They might be quite effective, however, in enhancing total infiltration rates when integrated into the cycle of basin operation. For example, given a 40-day total operational period, 20 days of initial fully-renovated operation, combined with 10 days of desiccation and 10 days of post-desiccation operation, would be more effective than a single 40-day operation following full renovation. More than one drying cycle may be required for more extended operations, but the effect of extended cycles of drying has not yet been investigated.

4) An organic-mat filter can substantially increase the total recharge accomplished through a basin within a single operational period. Work on other facilities has indicated long-term benefits can be effected. However, some potential drawbacks exist. Test-basin evidence indicates deeper sediment penetration into the basin subsoil can occur using an organic-mat filter. The long-term effects of such penetration are not certain. They may be small in relation to the infiltration-rate gains, or they may be substantial, necessitating deep renovation of the basin. The long-term gains and losses using an organic-mat filter are not fully understood.

Renovation of an organic-mat filter may or may not prove to be more difficult than for a natural-sand surface. It is known that a substantial decrease in infiltration rate occurs using an organic mat following an initial basin-operational period, and with renovation including 22 days of surface drying and shallow-surface raking to break up the filter cake. The surface of an organic mat is not conducive to a shallow thorough cleaning operation. Thus, the sediment must remain in the mat. Moreover, it is difficult to dry the entire filter due to the high water retention of organic materials.

On the other hand, a large portion of the infiltration-rate attenuation may be due to consolidation of the mat, rather than clogging. If so, large porosity, maintained by periodic tillage treatments to reloft the mat, and the soil aggregating properties of the organic mat might enable large long-term increases in sustained-infiltration rates, even with quantities of sediment added.

At present, the potential benefits from using an organic mat are promising, but uncertain. Further long-term investigation is needed to fully evaluate the effectiveness of organic mats in artificial-recharge basins.

5) The effectiveness of basin depth-control in enhancing total recharge is dependent on the local homogeneity of the soil at the basin site and on the depth at which the water table is managed. For surface clogging similar to that of the test basin, however, substantial gains (greater than 25 percent increase for doubling ponded

depth) in total recharge are expected from ponded-depth increases within a "moderate" range.

The term "moderate" is indefinite, but pertains to the maximum weight of water that can be held by the basin-subsoil materials without excessive consolidation and subsequent detrimental effects on infiltration rate. Moreover, the precise range defined by the term "moderate" is influenced by secondary effects on infiltration rate caused by increased detention times resulting from deeper-ponded conditions. For example, increased organic clogging, resulting from increased algal development, or from calcium-carbonate deposition due to increased pH from algal photosynthesis, has been shown to decrease recharge using deeper-ponded regimes on some facilities (Bouwer and Rice, 1984). Moreover, extended periods of deep flooding may not be compatible with certain management practices, like the use of grass- bottom filters.

A limiting depth of about 4 feet was suggested by one author (Baumann, 1965), but this may vary. With highly compressible materials, such as an organic mat, it is likely that lower ponded-depth regimes may prove to be more beneficial. It is suggested that ponded-depth capabilities of 4 to 6 feet would be desirable for an initial basin facility, but that careful experimental work and observation would be necessary to determine an optimal ponded-depth regime for each basin-management plan.

6) Other management options, not directly investigated, include reduction of suspended sediment prior to infiltration using detention basins, flocculation, or grass prefiltration basins. Tillage treatments, such as furrowing of the basin floor, might be beneficial. Sloped-basin design to facilitate runoff of surface filter-cake materials during natural rainfall has been tried in some basins, and the use of deep-rooted grasses to enhance basin macroporosity and infiltration has been used successfully in some studies. These are summarized in the full-basin test report (Schuh and Shaver, 1988), and should be considered as possible practices in a pilot-scale facility.

EVALUATION OF ARTIFICIAL-RECHARGE CAPABILITIES IN THE OAKES AQUIFER STUDY AREA

Table 1 summarizes the total recharge accomplished for each of the basin tests, and provides an evaluation of recharge quantities in terms of equivalent acreage requirements to meet the required average supplemental water for irrigation use in the West Oakes irrigation development tract of the Garrison Diversion Unit. These equivalent acreage figures apply to a scale expansion of the basins used in the feasibility test, with identical infiltration and recovery capacities. Depending on the management method used, equivalent acreages varied from as little as 26 acres to as much as 143 acres for two basin operational periods per year. Given proper management, it would not be unreasonable to expect a recharge volume equivalent to 75 feet of infiltration for each basin operational period on a sustained basis for such a "scale equivalent" facility.

The assumption of scale similarity must be viewed with extreme caution, however, in assessing broader area capabilities for artificial recharge as they apply to specific basin design. Numerous factors contribute to the need for a conservative and "stepwise" approach toward applying artificial recharge to the Oakes area. Some of the factors to be considered are as follows:

1) Spatial variability is always a major factor that must be considered when interpreting the direct applicability of test data to larger areas. The soil at the test basin site is classified in the Maddock series, compared with a composite of Hecla, Ulen, and Maddock soils for the area of the proposed project-scale facility (Schuh and Shaver, 1988). However, preliminary drilling and subsoil infiltration tests within the proposed project area (Shaver and Schuh, 1989) indicated the subsoil lithology includes discontinuous buried A horizons and clay-enriched strata that could potentially decrease infiltration rate. The areal extent of such layers is

not known, and they may be difficult to avoid for a project-scale facility.

2) Operation of project-scale artificial-recharge basins would increase the likelihood of infiltration-rate limitations caused by intersection of the basin floor with ground-water mounds. Control of ground-water mound formation can be effected by control of water-table depth through pumping during the irrigation season. However, some leeway for management of the combined pumping and recharge operations must be considered in any full-scale basin design.

3) A lower overall initial-infiltration rate can only be offset to a limited extent by longer operational periods, since progressive clogging renders the basin increasingly less effective with time within each operational period. It is likely that increased ponded acreage would have to be used to offset most of an underestimation of expected recharge.

While the small-scale basins do indicate that the operation of an artificial recharge basin in the Oakes area is physically feasible, the complexity of any field system is sufficient to warrant a cautious approach toward full-scale development. The likelihood and potential impact of some of the above constraints will be more thoroughly understood following further exploratory drilling in the area to better characterize the distribution of subsoil materials. However, the authors agree with Bianchi and

Muckel (1970) that there is no substitute for an on-site pilot project as a prerequisite to the implementation of large-scale facility design and operation.

RECOMMENDATIONS FOR FUTURE WORK

It is concluded from the phase I and phase II investigations that artificial recharge to the Oakes aquifer is physically feasible. Although a great deal has been determined concerning the hydraulic capabilities of area soils and their interaction with turbid James River water, further on-site work must be done to insure optimal design and management criteria for a project-scale facility. This can only be done through the operation of a smaller pilot-scale facility. Recommendations are as follows:

1) An exploratory program to better define the geometry and stratigraphy of the channel-fill deposits and the surficial silty-clay deposits in the proposed project area should be undertaken. Results should be used to refine design expectations for construction of a pilot-scale well field and recharge facility, and to determine specific locations for both pilot-scale and project-scale recharge facilities.
2) A smaller, pilot-scale artificial-recharge facility should be constructed and operated within the proposed project area for an extended period of time. The

pilot-scale facility should be used to integrate practices of well field and water-table management, basin-infiltration management, and conveyance facility design. Final capabilities determined from the pilot facility will provide a basis for the design of project-scale facilities.

3) The pilot-scale facility should incorporate a multiple-function design to allow for simultaneous and integrated assessment of best management practices. Some of the practices that should be evaluated are:

   a) Long-term use of an organic mat filter, with tillage.
   b) Long-term use of the unaltered natural sand filter, using combinations of periodic shallow renovation and tillage, combined with natural renovation (desiccation, weathering).
   c) Surface spreading.
   d) Pretreatment of turbid James River water using chemical flocculents.
   e) Grass prefiltration integrated with a pilot surface-spreading facility.

4) The pilot-scale facility will be monitored for hydraulic capability, water chemistry, and impact on the aquifer, water table and water supply impacts, effects on local wetlands, and the physical and economic viability of

water delivery to the recharge facility and to irrigators.

5) For ultimate cost efficiency, the pilot-scale facility should be designed for incorporation within the project-scale artificial-recharge plan, should expansion to project-scale be decided upon at a later time. Incorporation could occur either as a modular part of the full facility or as an expansion of the pilot-scale facility. Incorporation could be either as constructed, or with later modifications.

PRELIMINARY WELL DESIGN

Aquifer Hydraulic Properties

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 (fig. 9). In this area, the aquifer generally is unconfined, anisotropic, and nonhomogenous 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

Casing Design

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 O.D. (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.

Screen Design

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.

Pump Design

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.

EXPLORATORY WELL-DRILLING METHODS

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.

PRODUCTION WELL-DRILLING METHODS

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, conventional rotary using a bail-or wash-down method to install screen and the reverse-rotary method appear to be the most efficient methods of production-well drilling. The holes can be drilled quickly and economically. Clay-based drilling additives are not used with either drilling method.

FINITE-DIFFERENCE MODEL OF GROUND-WATER FLOW

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 to:

1) Develop a preliminary design of a pilot- and project-scale well field.
2) Estimate the effect 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.
3) 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.
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 Simulation

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 north-south trending parallel lines along the central axis of the outwash channel near Section 13, Township 129 North, Range 59 West (fig. 11).

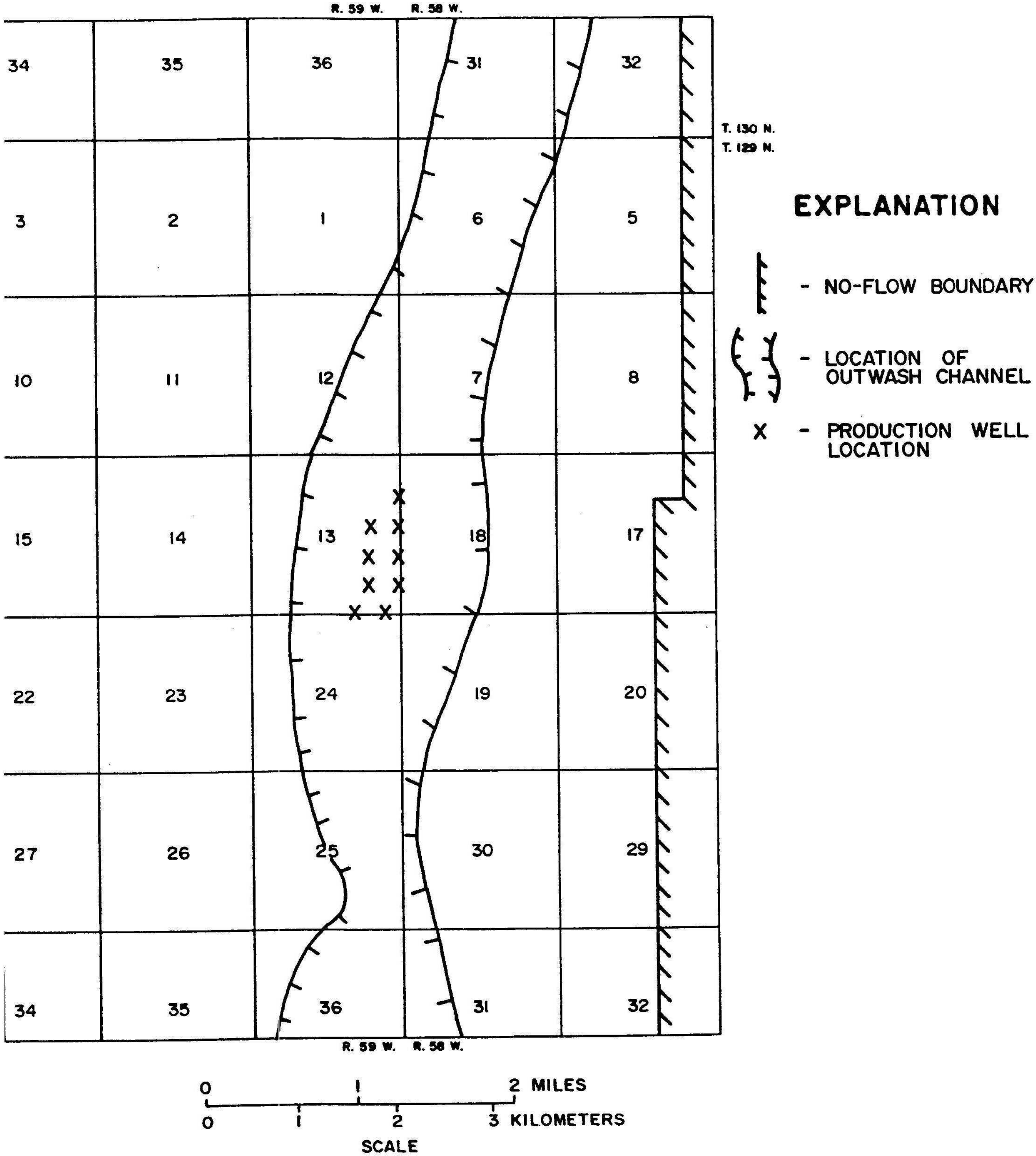

Figure 11.--Location of pilot-scale well field

The pilot-scale well-field simulation was used 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. 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. Therefore, the model was used to simulate the withdrawal of 4,057 acre-feet of water annually for 3 consecutive years, at a continuous pumping rate of 60 cubic feet per second (27,000 gallons per minute/3,000 gallons per minute per well) for 34 days with no artificial recharge. The drawdown distributions after the third year of pumping and recovery are shown in figures 12 and 13. These figures can be used to provide a preliminary assessment of the effects on the water table resulting from pilot-scale development.

Results of the pilot-scale well-field simulation indicate the Oakes aquifer near the SE1/4 of Section 13, Township 129 North, Range 59 West can support the annual withdrawal of 4,057 acre-feet of water for at least 3 consecutive years without artificial recharge. It is also apparent the annual withdrawal of 4,057 acre-feet of water for 3 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

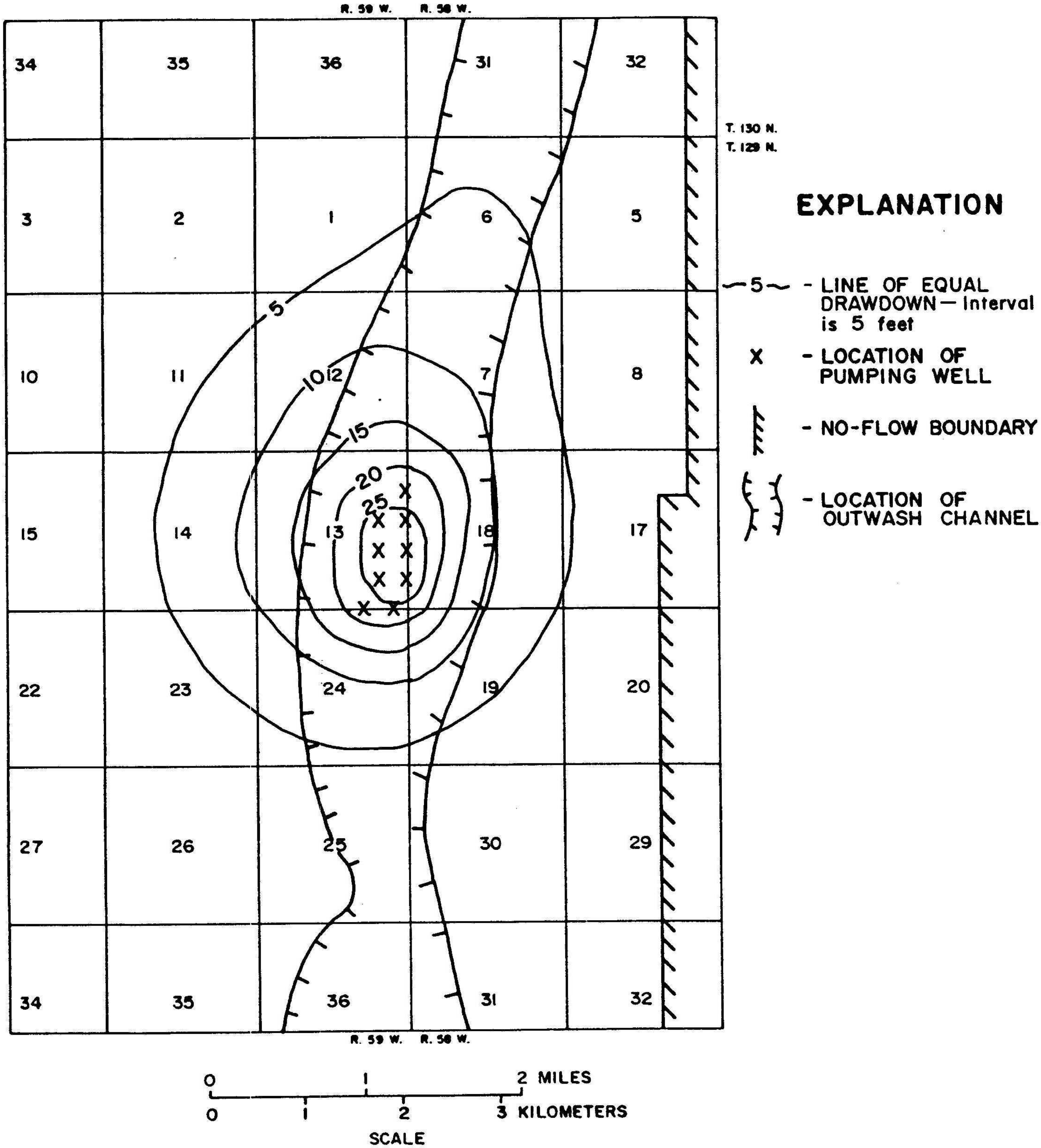

Figure 12.--Computer-simulated drawdown distribution after the third year of pumping an annual volume of 4,057 acre-feet

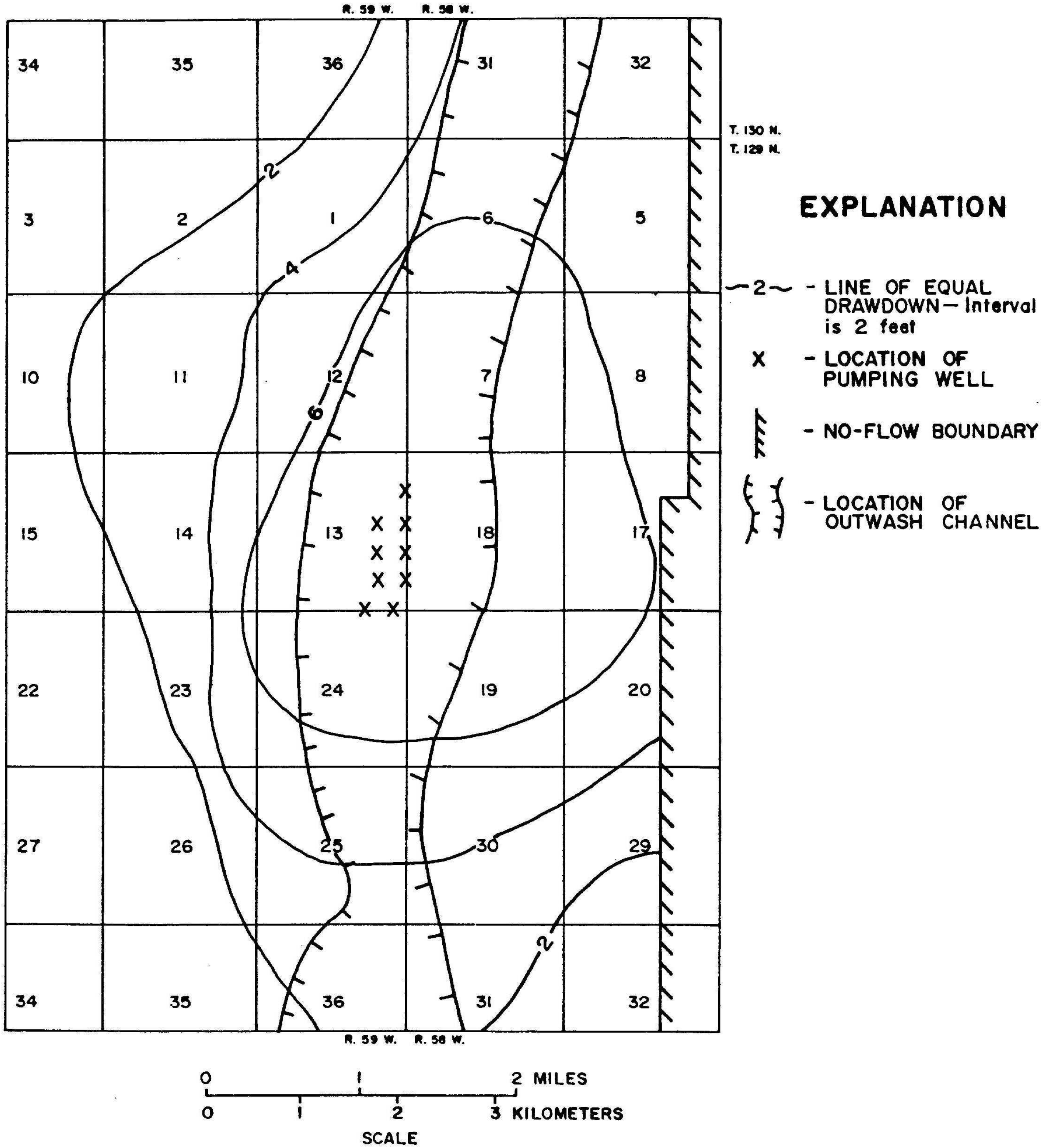

Figure 13.--Computer-simulated drawdown distribution after 214 days of recovery following the third year of pumping an annual volume of 4,057 acre-feet

field should be flexible enough to offset large water-table fluctuations caused by abnormally wet climatic cycles.

Project-Scale Well-Field Simulation

Other 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.

Various well-field configurations were simulated using the model. The most favorable well-field configuration is shown in figure 14. Note the pilot-scale well field is incorporated into the project-scale well field. The project-scale 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.

Total drawdown in each well was estimated by adding additional drawdown due to partial penetration, well loss, and real-well radius to drawdown computed by the model. Average estimated drawdown at each well is 52 feet. The available head above the top of the screen minus total drawdown (77-52) is 25 feet. Total drawdown is about two-thirds of the initial

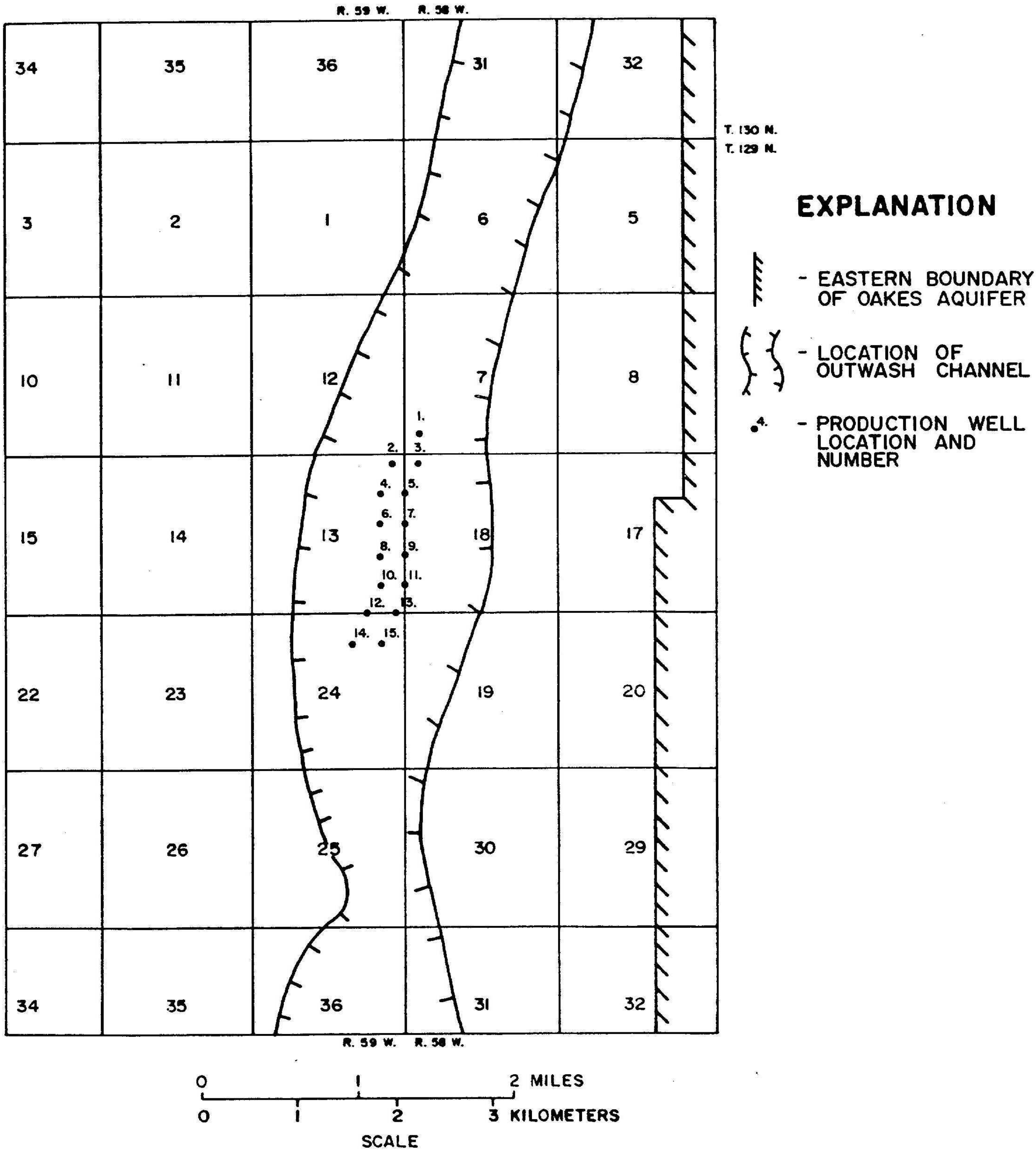

Figure 14.--Location of project-scale well field

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 (11,900 acre-feet) for 60 days, is shown in figure 15. 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.

Project-Scale Well-Field and Artificial-Recharge Simulation

The computer model was also used to simulate the short-term (2 years) operation of a project-scale well field and recharge basin system. The well and recharge-basin configuration is shown in figure 16. Each of the 15 wells was pumped continuously for 60 days (July-August) at a rate of 2,094 gallons per minute (total discharge = 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. Basin dimensions are 1,140 feet (length) by 320 feet (width) and were 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 simulated using recharge wells.

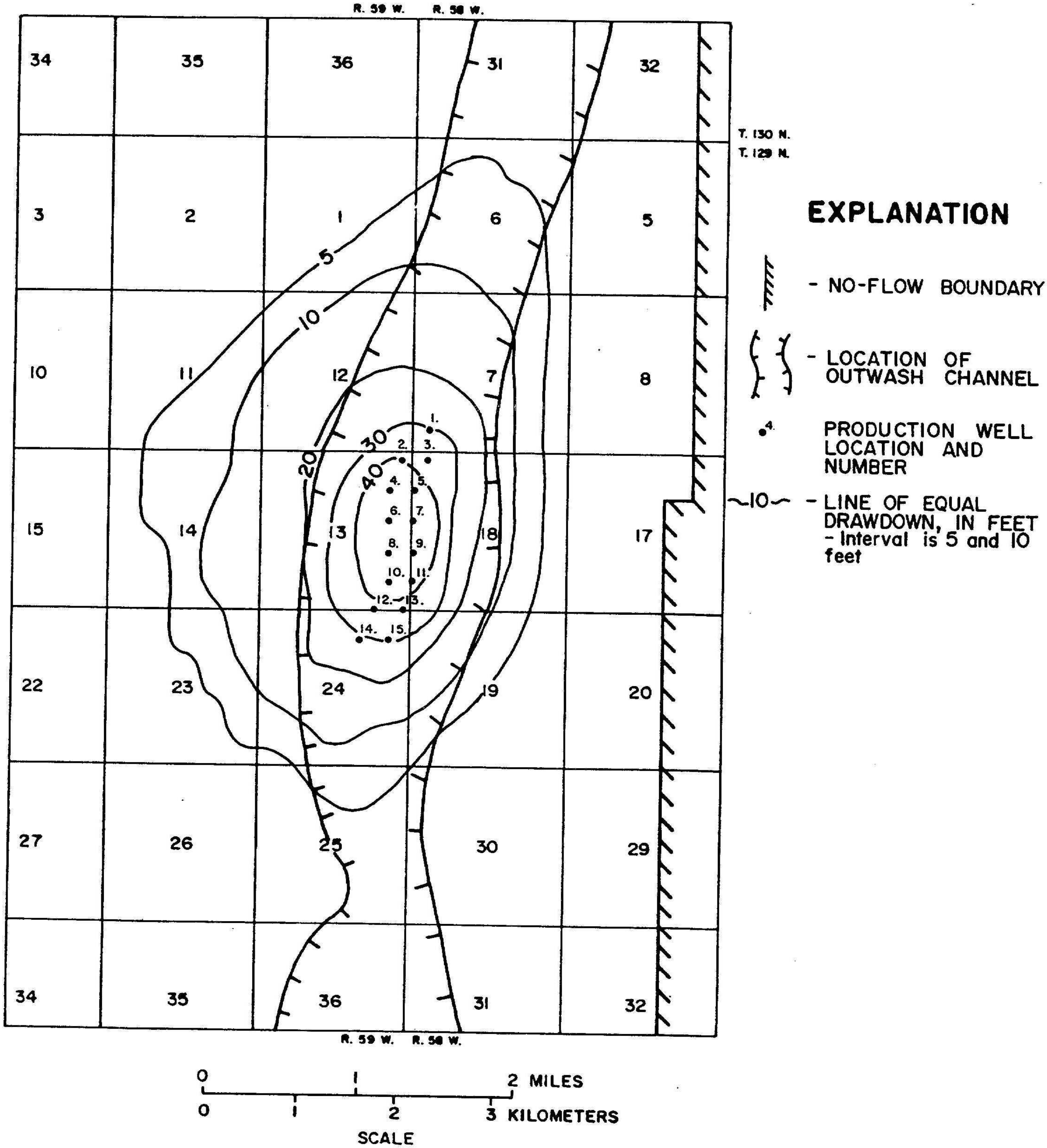

Figure 15.--Computer-simulated drawdown distribution after pumping 100 cubic feet per second for 60 days (11,900 acre-feet)

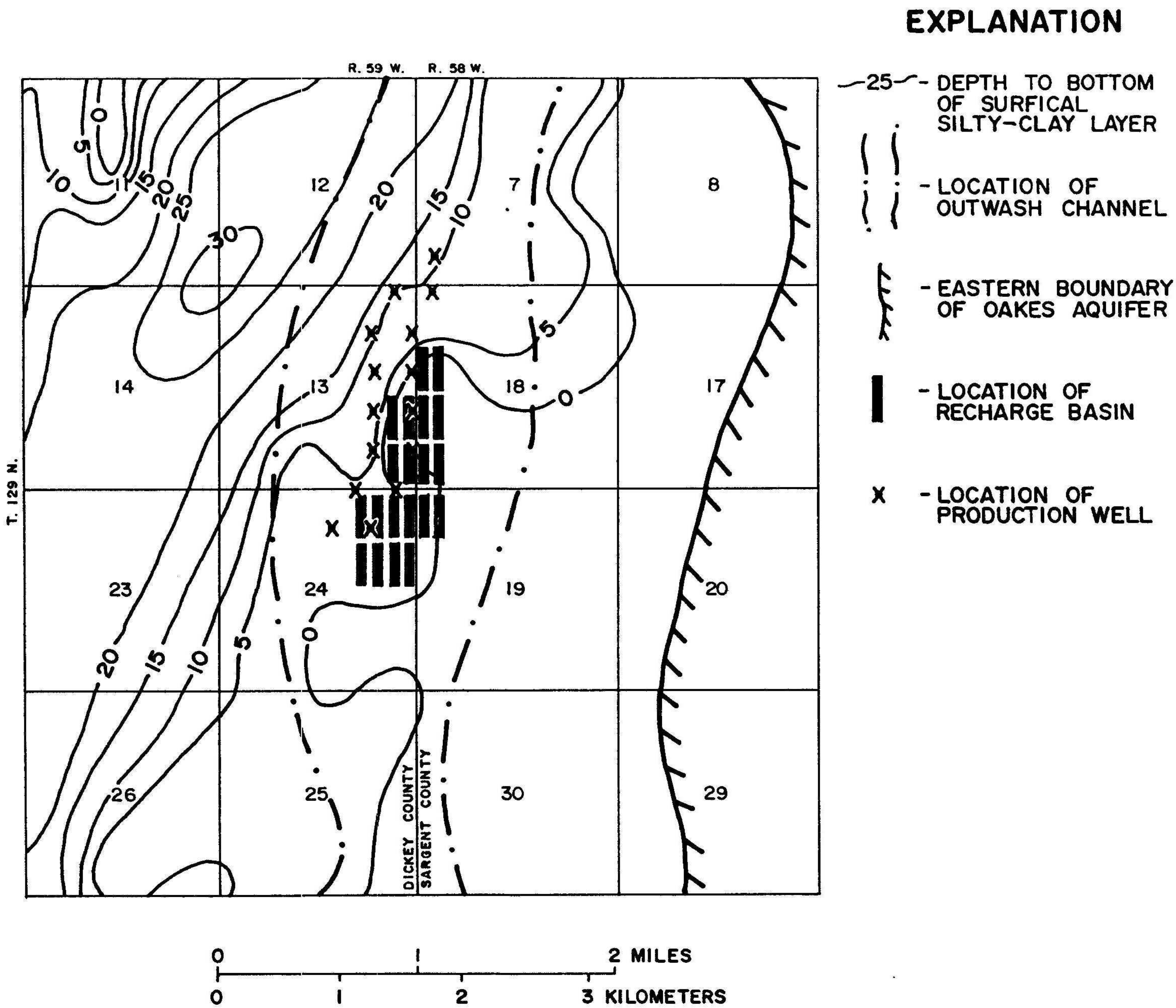

Figure 16.--Location of project-scale well field and recharge basins, based on an infiltration rate of 1 foot per day

Results of this simulation indicate full-scale artificial recharge is not practical during the spring and fall for the first 2 years of operation. The aquifer is not sufficiently dewatered to accommodate full-scale artificial recharge.

PILOT-SCALE WELL-FIELD AND ARTIFICIAL-RECHARGE FACILITIES

Justification

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 would provide the basis for developing more accurate estimates of these costs.

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. Pilot-scale testing would identify how the Oakes aquifer recharge program would or would not fit into full project development in the Oakes 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 facilities indicate the water table would be lowered in and around the project area. For project-scale development, a maximum drawdown of about 40 feet within the well field is predicted at the end of each irrigation season prior to the fall recharge period. A drawdown of less than about 2 feet is 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 supplies from shallow wells, sub-irrigation, and wetlands. In wet years, particularly during the spring, parts of the project area are ponded with water and/or the soils are affected by waterlogging. These conditions reduce crop yield. A net benefit from lowering the water table in the project area might be a reduction in ponding and waterlogging of soils. The operation of a pilot-scale well field and artificial-recharge test facilities would provide the basis for evaluating and predicting effects on the water table resulting from project-scale development.

Pilot-Scale Artificial-Recharge Tests and Cost Analysis

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 flocculents 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 and renovation by natural desiccation 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 $5,000.00 per basin. Land easement costs are based on $300.00 per acre. The 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 4.

Basin Recharge: Pretreated James River Water

A pretreated 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

Table 4. -- 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.

will be 220 feet (length) by 70 feet (width) by 5 feet (depth). A cost analysis of this test is presented in table 5.

Basin Recharge: Organic Mat

The effectiveness of organic-mat filters on infiltration 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 6.

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. 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. A cost analysis is presented in table 7.

Table 5.-- Cost analysis 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 6.-- Cost analysis of pilot-scale basin recharge test 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 7.-- 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

Contingency Recharge Testing

The pilot-scale test program must be flexible enough to accommodate 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.

Pilot-Scale Well-Field Cost Analysis

The pilot-scale, well-field cost analysis is based on an average well depth of 125 feet. Each well will consist of 85 feet of 20-inch O.D. standard-wall thickness, 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 space will be filled with cement grout. The well screen will be installed and will then be developed by employing jetting and surging techniques. After well development is completed, a test pump will be installed to perform step tests and to 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 cost per well and the complete nine-well pilot-scale well field are presented in table 8.

Table 8.-- 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 & remove pump	---	1,000	9,000
Step testing	$80.00/hr.	1,200	10,800
Pump[2]	---	10,000	90,000
Electric control box[3]	---	5,000	45,000
Annual well electrical power costs[4]	---	300	16,200[6]
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

Total Cost of Pilot-Scale Well-Field and Recharge Tests

A 20 percent cost contingency, which amounts to $295,000, is estimated for the 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 9.

PROJECT-SCALE WELL FIELD AND ARTIFICIAL-RECHARGE FACILITIES

Project-Scale Recharge Basin Design and Cost Analysis

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

1) The 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

Table 9.-- 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 fields and artificial-recharge areas.

to 8,330 acre-feet of water (70 cubic feet per second for 60 days).

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

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 desiccation 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 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, using 10 pairs of basins (one operational and one undergoing renovation) with 4 to 1 sidewall slopes, will require individual basin dimensions of 1,140 feet by 320 feet. Basin depth is estimated at 5 feet. The configuration of recharge basins and well field is shown in figure 16.

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 $300.00 per acre. Basin appurtenances are based on a cost of $10,000 per basin. A basin-monitoring network is based on a cost of $5,000 per basin. Preparation of specifications, construction supervision, and overhead costs, based on 30 percent of the total costs, amount to $606,000. Contingency costs based on 15 percent of the total costs, 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 10.

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 acre-feet in 120 days). Applying a basin width-to-length ratio of about 4 to 1, 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. The basin depth is

Table 10.-- 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

estimated at 5 feet. The configuration of the recharge basins and well field is shown in figure 17.

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 precentage of total costs as previously described. A cost analysis of this recharge basin option (Case B) is presented in table 10.

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, 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. The basin depth is estimated at 5 feet. The configuration of the recharge basins and well field is shown in figure 18.

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 percentage of total costs as previously described. A cost

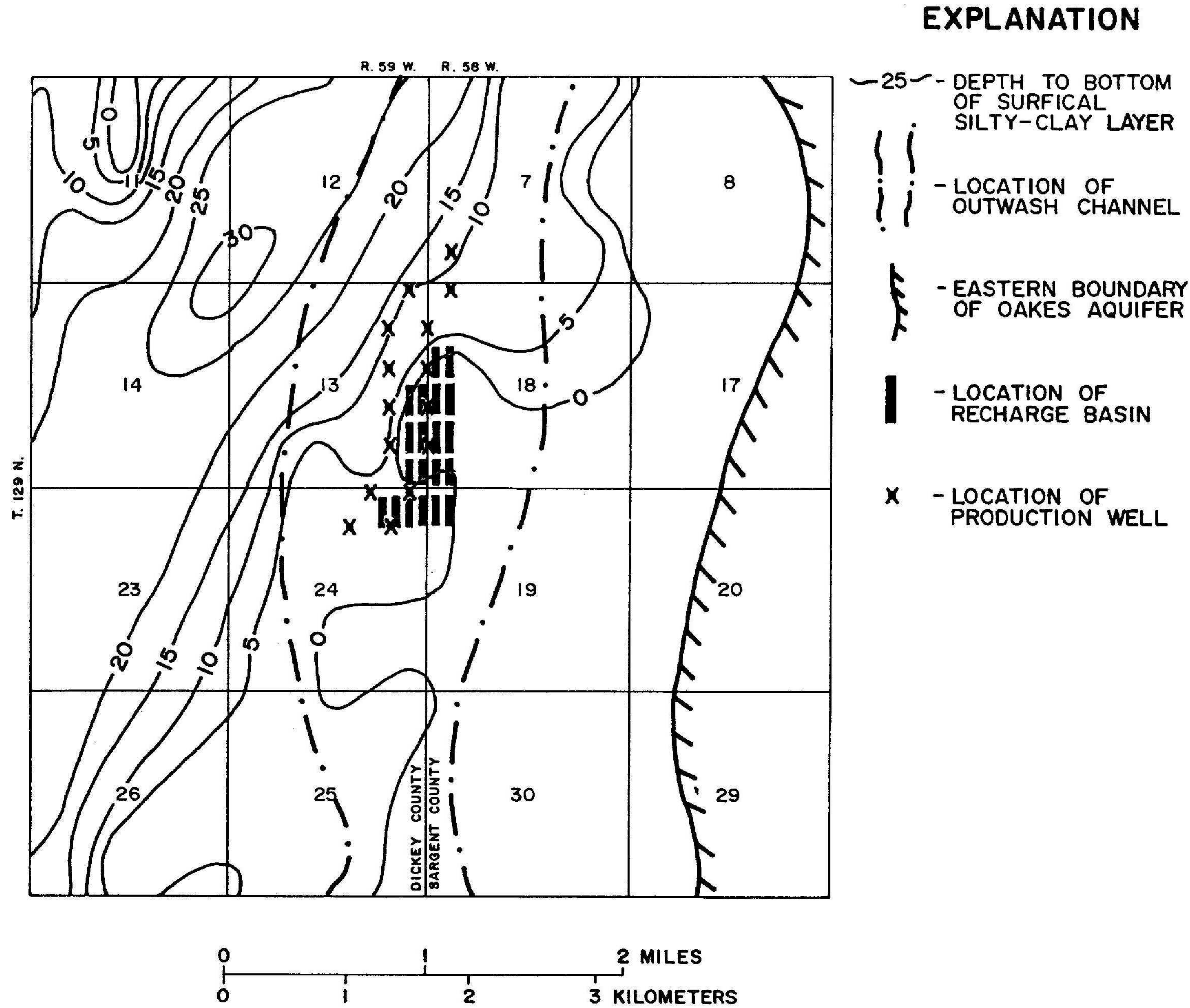

Figure 17.--Location of project-scale well field and recharge basins, based on an infiltration rate of 2 feet per day

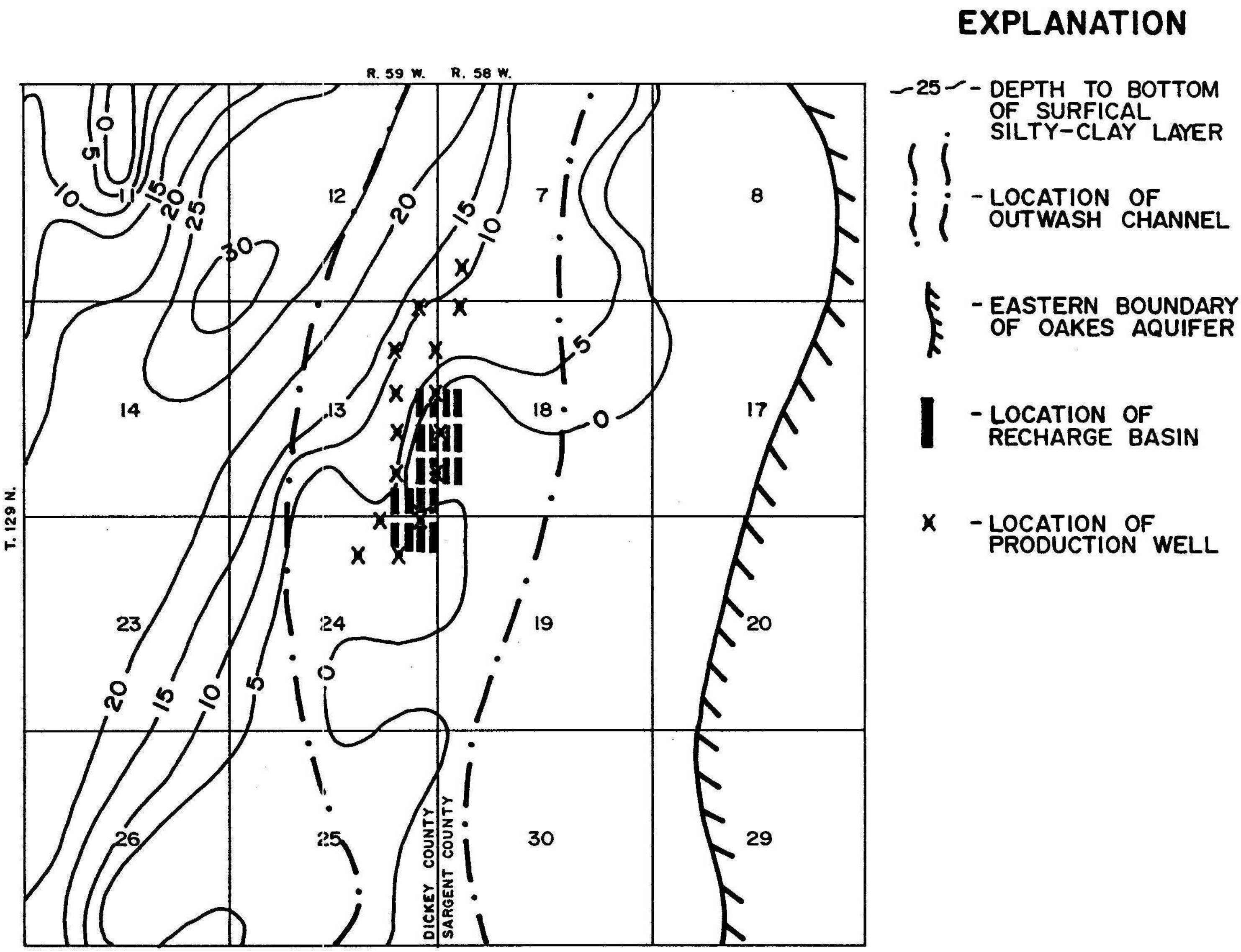

Figure 18.--Location of project-scale well field and recharge basins, based on an infiltration rate of 3 feet per day

analysis of this recharge option (Case C) is presented in table 10.

Project-Scale Surface-Spreading Facilities Design and Cost Analysis

Preliminary surface-spreading design criteria in the project area includes 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, the 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 per day.

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 the dikes. Excavation for diking material is based on a unit cost of $1.25 per cubic year. 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 1, 2, and 3 feet per day, is presented in table 11.

Project-Scale Well-Field Cost Analysis

The project-scale well field consists of 15 wells, nine of which comprise the pilot-scale well field. The cost analysis is based on previously described design criteria for the pilot-scale well field and is presented in table 12. Long-term well and pump maintenance costs, such as well redevelopment and replacement of worn pump components, are not included in the cost analysis.

Total Cost of Project-Scale Well-Field and Recharge Facilities

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 artificially recharge 8,330 acre-feet of water in 120 days (60 days spring-60 days fall). Using tables 10, 11, and 12, 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

Table 11.-- 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

[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

Table 12.-- 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 & 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

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.

REFERENCES CITED

Abe, J.M., 1986. Economic analysis of artificial recharge and recovery of water in Butler Valley, Arizona Water Resources Research Center, College of Engineering and Mines, University of Arizona, Tuscon, Arizona, 85721. 170 p.

Bluemle, J.P., 1979. Geology of Dickey and LaMoure Counties, North Dakota. North Dakota Geological Survey Bulletin No. 70, Part 1.

Baumann, P., 1965. Technical developments in ground-water recharge. In V.T. Chow (ed.). Advances in Hydroscience. 2:209-279 Academic Press, New York.

Bianchi, W.C., and D.C. Muckel, 1970. Ground-water recharge hydrology. U.S. Department of Agriculture, Agricultural Research Service. ARS 41-161.

Bouwer, H., and R.C. Rice, 1984. Renovation of wastewater at the 23rd avenue rapid-infiltration project, Phoenix, Arizona. J. Water Poll. Contr. Fed. 56:76-83.

Schuh, W.M., 1988. In-situ method for monitoring layered hydraulic impedance development during artificial recharge with turbid water. Journal of Hydrology. 101:173-189.

Schuh, W.M., and R.B. Shaver, 1988. Feasibility of artificial recharge to the Oakes Aquifer, southeastern North Dakota: Evaluation of experimental recharge basins. North Dakota State Water Commission Water-Resources Investigation No. 7, 248 p.

Shaver, R.B., and W.M. Schuh, 1989. Feasibility of artificial recharge to the Oakes Aquifer, southeastern North Dakota: Hydrogeology of the Oakes Aquifer. North Dakota State Water Commission Water-Resources Investigation No. 5. In preparation.

Shaver, R.B., and M.H. Hove, 1989. Feasibility of artificial recharge to the Oakes Aquifer, southeastern North Dakota: Ground-water data. North Dakota State Water Commission Water Resources Investigation No. 6, Vols. 1 and 2. In preparation.

Shaver, R.B., 1989. 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. North Dakota State Water Commission Water-Resources Investigation No. 8, 113 p.