FEASIBILITY OF STABILIZATION OF WATER LEVELS AND EXPANSION OF WATER USE FROM THE ENGLEVALE AQUIFER USING WATER CONSERVATION, WELL FIELD MODIFICATION, AND ARTIFICIAL RECHARGE

By

Royce Cline, Hydrologist,
Craig Odenbach, Water Resource Engineer,
Preston Schutt, Water Resource Planner,
and William Schuh, Hydrologist

Water Resource Investigation 23
North Dakota State Water Commission
David Sprynczynatyk, State Engineer

Prepared by the
North Dakota State Water Commission
In cooperation with the
Ransom County Water Resource District

ND State Water Commission

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Abstract

The northern part of the Englevale aquifer experienced declining water levels throughout the 1980s that resulted in the State Engineer eliminating 880 acres of irrigation for the 1991 irrigation season. Because of the limited available drawdown at many irrigation wells in the Englevale aquifer, significant reductions in well yields can occur if further water-level declines occur. If the climate of the 1980s persists, there is the possibility that future pumping restrictions will be required to stabilize aquifer water levels. This study was undertaken to evaluate the feasibility of pumping water from the Sheyenne River to artificially recharge the northern part of the Englevale aquifer as a means of sustaining the present level of development and to provide water to irrigate additional acreage.

The analysis of the Englevale aquifer and long-term climatic influences upon the aquifer indicate that the aquifer is subject to large fluctuations in water levels as a result of decade length variations in climate. The results indicate that artificial recharge does not provide an economically viable way of dealing with the problems related to low water levels during the long periods of drought observed in the climate record. The reliability of the water supply for the existing irrigated acreage can be improved by relocating wells to deep parts of the aquifer and improving water use efficiency.

The study shows that artificial recharge could be used to expand the acreage irrigated near Englevale. However, this is not practical with existing crops and economic circumstances. If specialty crops are incorporated into the irrigator's crop rotations, then artificial recharge could be an economically viable option.

Preface

The report is divided into two parts. The first part of the report describes the hydrogeology, water use, and water-level trends associated with the Englevale aquifer. In addition, the effect of long-term climate variability on the ability of the aquifer to sustain the present level of development is examined. Finally, the ability of artificial recharge to add stability to the water supply available from the aquifer is discussed. Part two of the study discusses the availability of water from the Sheyenne River and the construction and operation costs of an artificial recharge facility including pumping facilities, pipeline, and recharge basins. The suitability of additional land for irrigation and the cost of water delivery are considered. The possibility of improving water use efficiency through conservation practices is examined. Economic feasibility of artificial recharge for irrigation use is then evaluated.

PART I. WATER SUPPLY AVAILABILITY FROM THE ENGLEVALE AQUIFER IN RELATION TO LONG-TERM AND SHORT-TERM NATURAL RECHARGE

INTRODUCTION

The Englevale aquifer is located in western Ransom and Sargent Counties (Fig. I-1). Though the aquifer extends through Sargent county into South Dakota, this report only covers the aquifer as far south as Township 132 North, Range 58 West (Fig. I-2). The primary emphasis is on the area north of Englevale.

There are presently 10,800 acres irrigated by water from the Englevale aquifer (Fig. I-2). Most of the development occurred during the mid 1970s causing water-level declines of up to about 7 feet. In the area north of Englevale, much of the early development occurred along the Englevale road in Sections 7 and 18, Township 134 North, Range 57 West. Subsequent development and studies by the State Water Commission have shown this to be an area of thin saturated thickness with pre-development saturated thickness ranging from 23 to 30 feet. Many of the irrigation wells in this area have experienced a 25 to 35 percent decline in saturated thickness. The decline in saturated thickness has resulted in a significant reduction in well yields. Also, the decline in water levels has resulted in no irrigation in several years on land with water permits that are conditioned to a specified water level. In 1991, the invocation of the priority system eliminated 880 acres from irrigation in the area from Englevale north. Irrigation of these lands was resumed in 1992 after water levels rose about from 1.5 to 2 feet during the fall of 1991 and spring of 1992 due to increased recharge.

The purpose of this study was to evaluate the feasibility of pumping water from the Sheyenne River to artificially recharge the northern part of the Englevale aquifer as a means of sustaining the present level of development and to provide water to irrigate additional acreage. Water-level declines in the area north of Englevale indicated that the present level of development is not sustainable given the conditions of the 1980s. In evaluating the feasibility of artificial recharge, it was necessary to determine if the aquifer is over-appropriated in the long term. If the aquifer suffers from long-term over-appropriation, then water must be supplied to the aquifer at an average annual rate equal to the over-appropriation to maintain the present level of development. If the present problems represent drought effects and the development is sustainable under normal climatic conditions, then artificial recharge water must either be banked in the aquifer or artificial recharge used only during periods of drought. The ability to use artificial recharge to increase the reliability of the water supply during periods of drought is dependent on the ability to store water in the aquifer during wet periods for subsequent use during dry periods. Computer simulations were performed using Lisbon climate data from 1904 to 1989 to explore

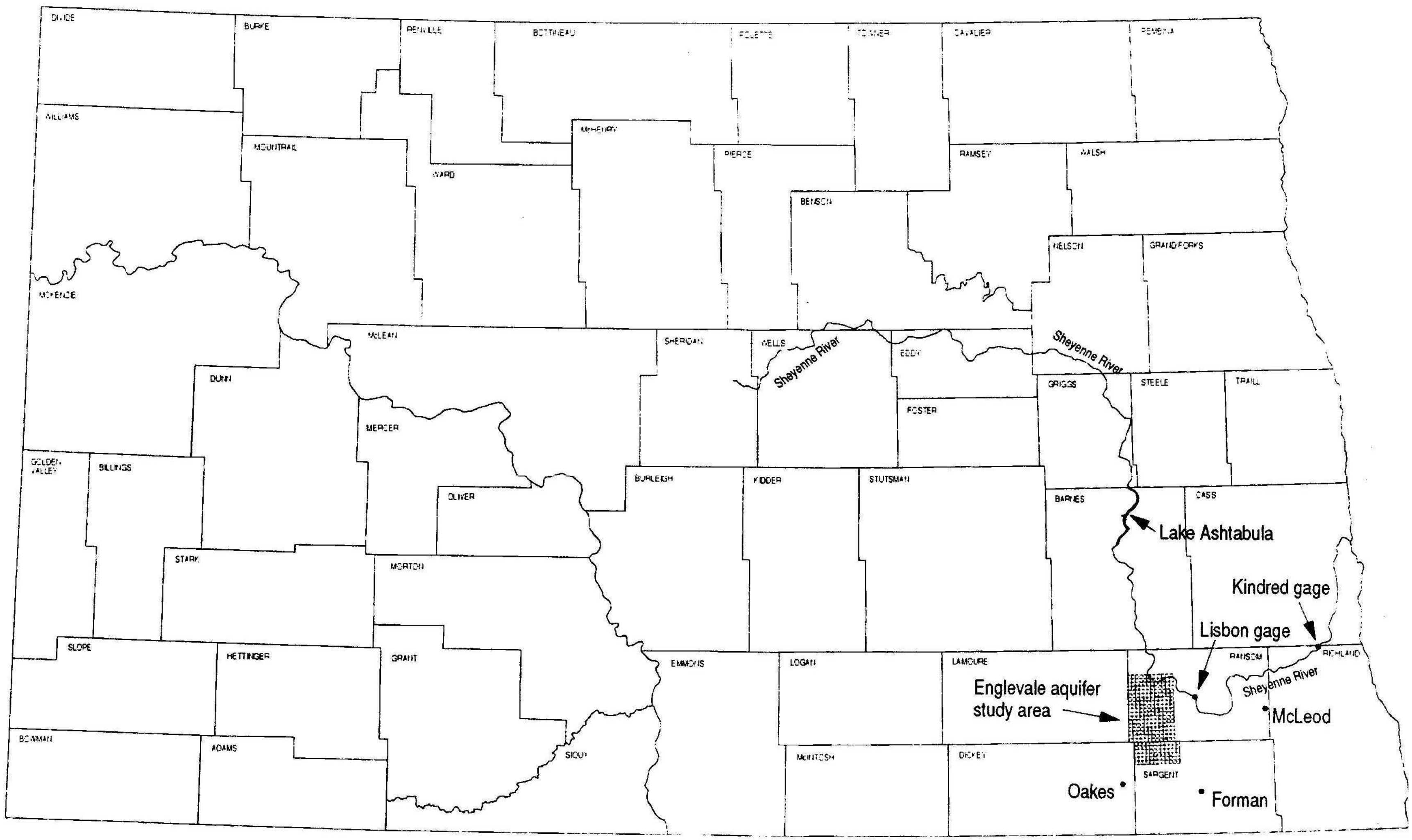

Fig. I-1. Map of North Dakota showing location of Englevale aquifer study area.

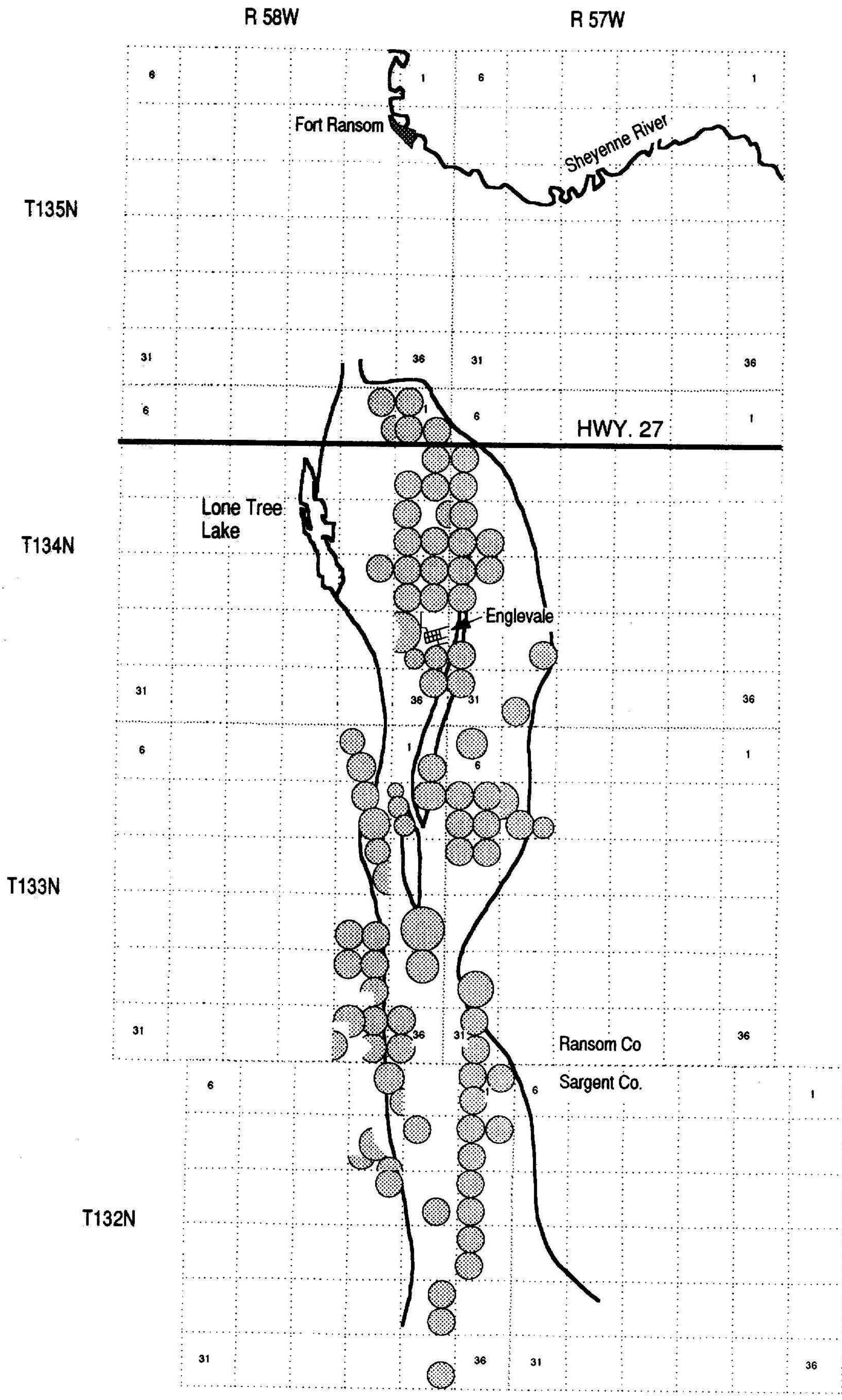

Fig. I-2. Map showing boundries of Englevale aquifer and location of irrigated land.

these issues. Moving wells to areas of greater saturated thickness was also examined in these simulations as a method to improve reliability of the water supply. The objective of moving wells would be to increase available drawdown which increases the amount of water that can be stored in the aquifer for irrigation. The effect of water conservation upon water levels is also examined. Potential water conservation methods are discussed in Part II.

WELL NUMBERING SYSTEM

The system for denoting the location of a test hole or observation well is based on the federal system of rectangular surveys of public land. The first and second numerals indicate Township North and Range West of the 5th Principal Meridian and base line (Fig. I-3). The third numeral indicates the section. The letters A, B, C, and D designate respectively the northeast, northwest, southwest, and southeast quarter section (160-acre tract), quarter-quarter section (40-acre tract) and quarter-quarter-quarter section (10-acre tract). Therefore a well denoted by 134-058-04ADD would be located in the $SE^1/_4SE^1/_4NE^1/_4$ of Section 4 , Township 134 North, Range 58 West. Consecutive terminal numerals are added if more than one well is located in a 10-acre tract, i.e., 134-058-04ADD1 and 134-058-04ADD2.

PREVIOUS INVESTIGATIONS

The Englevale aquifer was described by Armstrong (1982) as part of a ground-water study of Ransom and Sargent Counties. Ground-water data including well logs, water levels, and water quality analysis for the Englevale aquifer are presented by Armstrong (1979) as part of the data for Ransom and Sargent Counties. The geology of Ransom and Sargent Counties was described by Bluemle (1979). The State Water Commission (SWC) has conducted studies of the Englevale aquifer to aid in the management of the water resources of the area. Information on SWC test holes, water levels, and water quality are maintained by the SWC. Logs for privately drilled test holes and water wells are maintained by the Board of Water Well Contractors at the SWC offices.

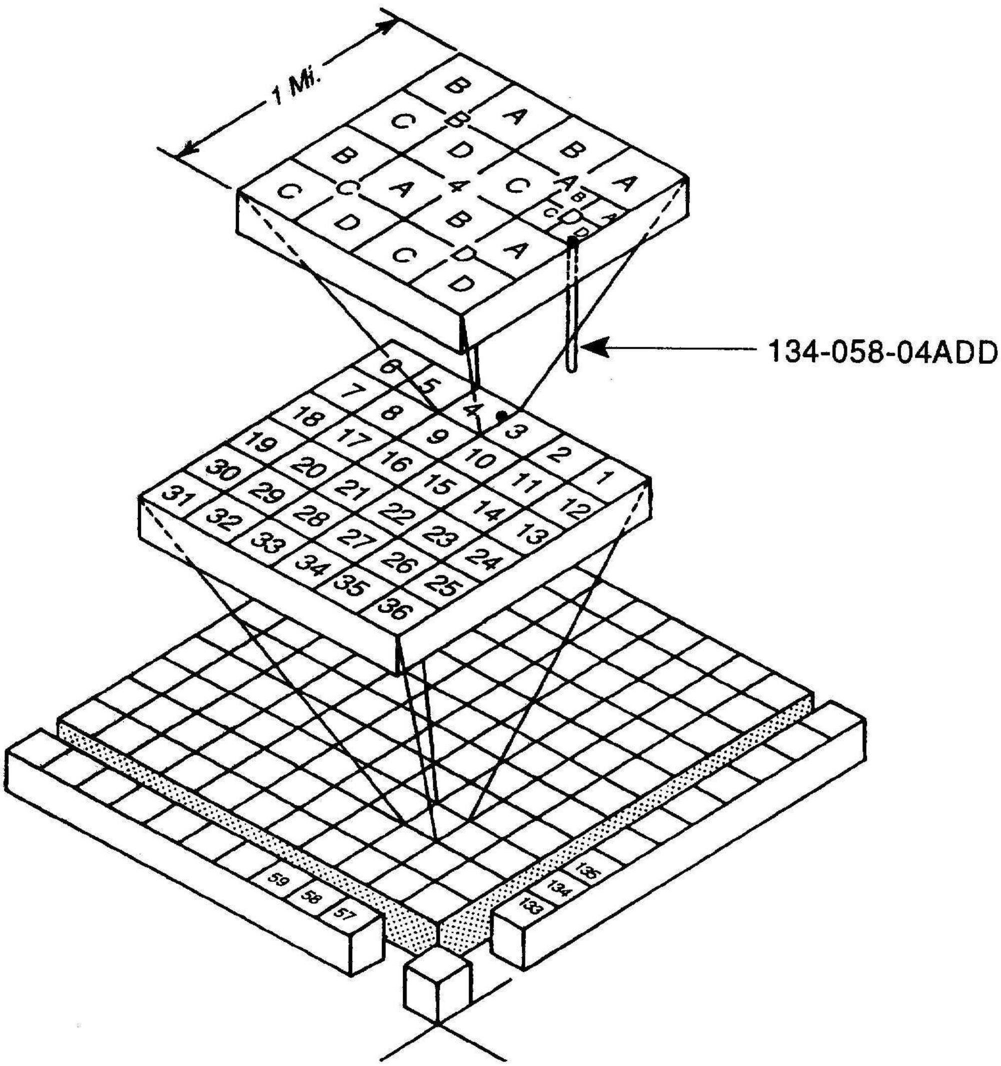

Fig. I-3 - Location-numbering system.

CLIMATE

The climate of the Englevale area is subhumid. The average annual temperature is 41.9°F and ranges from an average January temperature of 7.0°F to an average July temperature of 76.8°F (U.S. National Climatic Data Center, 1989). The mean annual precipitation at Lisbon is 20.05 inches and at Forman is 19.97 inches for the period 1951 to 1980. Approximately 55 to 60 percent of the precipitation occurs in the period from June through September. Annual water-year precipitation has been substantially below normal since 1979 (Fig. I-4). The water year starts October 1 and is used because it better represents hydrologic processes in that fall rains influence hydrologic response the following spring and summer. The available rain gage data for Englevale show similar precipitation to Lisbon although there are significant differences for some storms. During the 1980s drought period, the October to May precipitation with the exception of 1980, 1981, 1987, and 1988 has not been significantly below normal (Fig. I-5). Though the October to May precipitation shows considerable annual variation, there is little long-term variability shown by either the 5-year or 11-year moving average. Most of the long-term variability in precipitation occurs during the growing season (Fig. I-6).

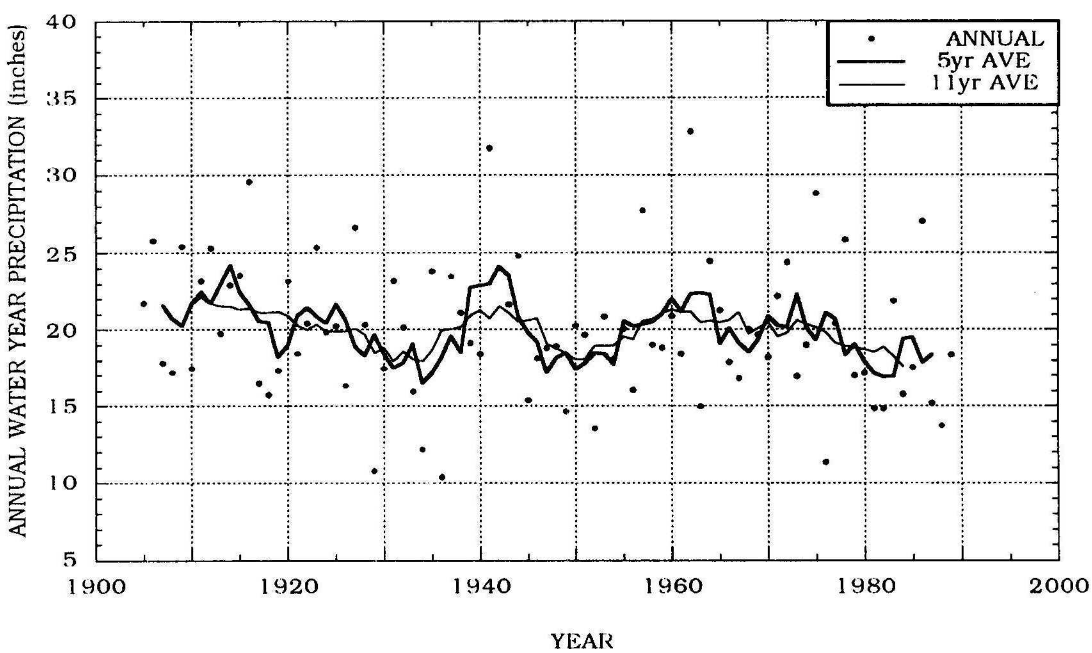

Fig. I-4. Lisbon annual water year precipitation with 5-year and 11-year moving averages

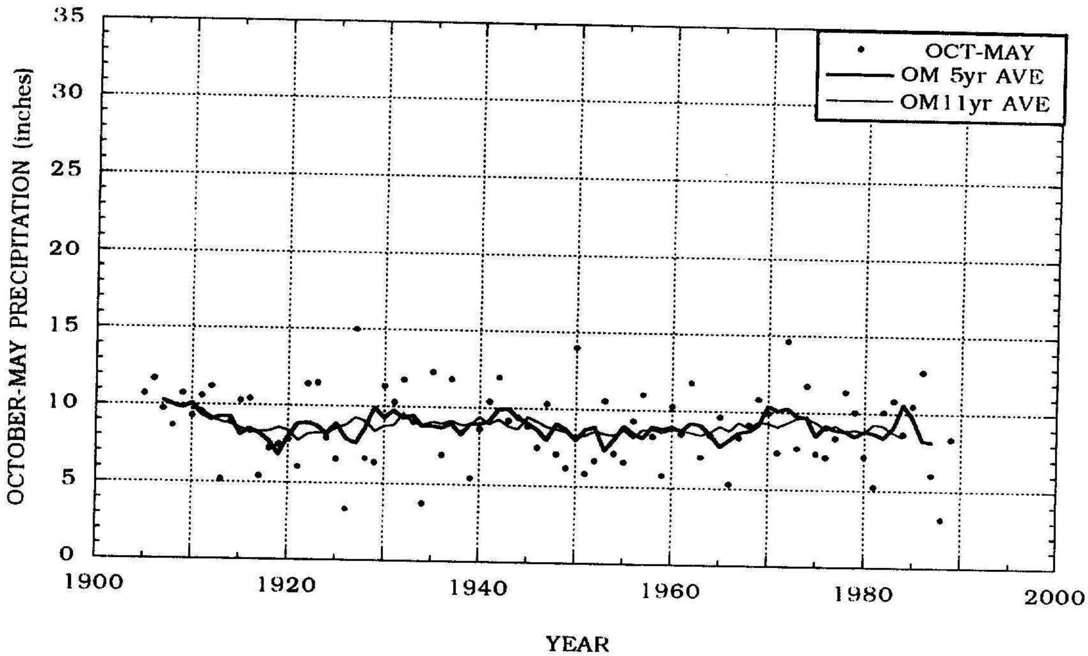

Fig. I-5. Lisbon October to May precipitation with 5-year and 11-year moving averages.

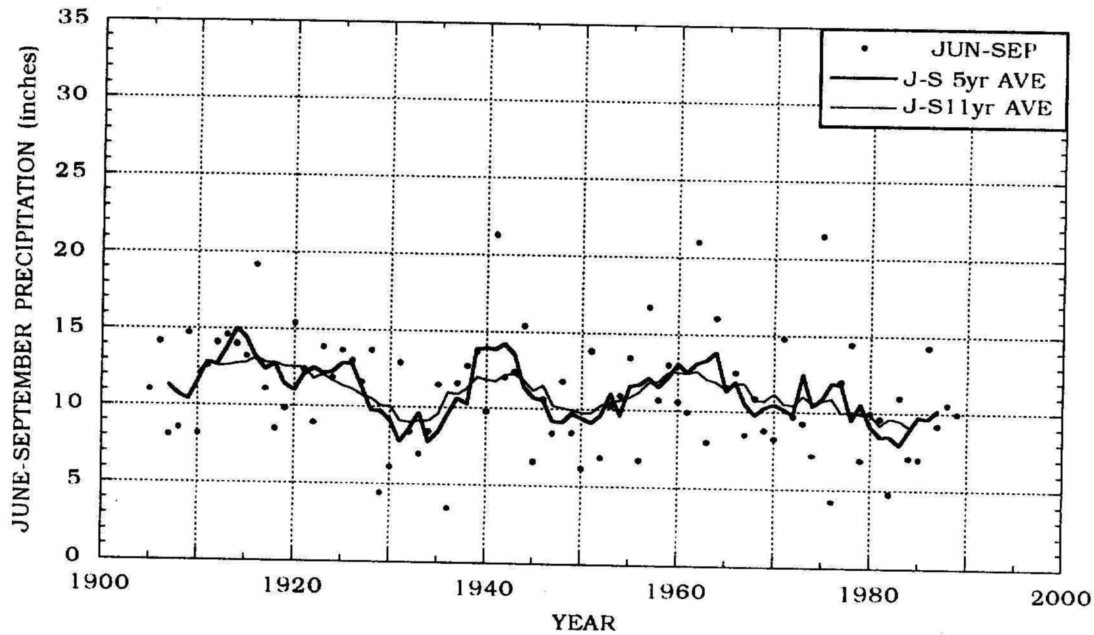

Fig. I-6. Lisbon June to September precipitation with 5-year and 11-year moving averages.

Though drier years occurred in the 1930s, the 1980s on average were almost as dry as the 1930s at Lisbon as shown by both the 5-year and 11-year moving average in Fig. I-4. The lowest 5-year average in the period of record was 16.48 inches of precipitation for the period 1932 through 1936. The 1980s were almost as dry with a 5-year average of 16.89 inches of precipitation in the period 1980 through 1984. However, other stations in the area show that the region was much drier during the 1930s than is indicated by the Lisbon data. Both Oakes and McLeod had long periods where the 5-year average remained under 16 inches of annual precipitation (Figs. I-7 and I-8).

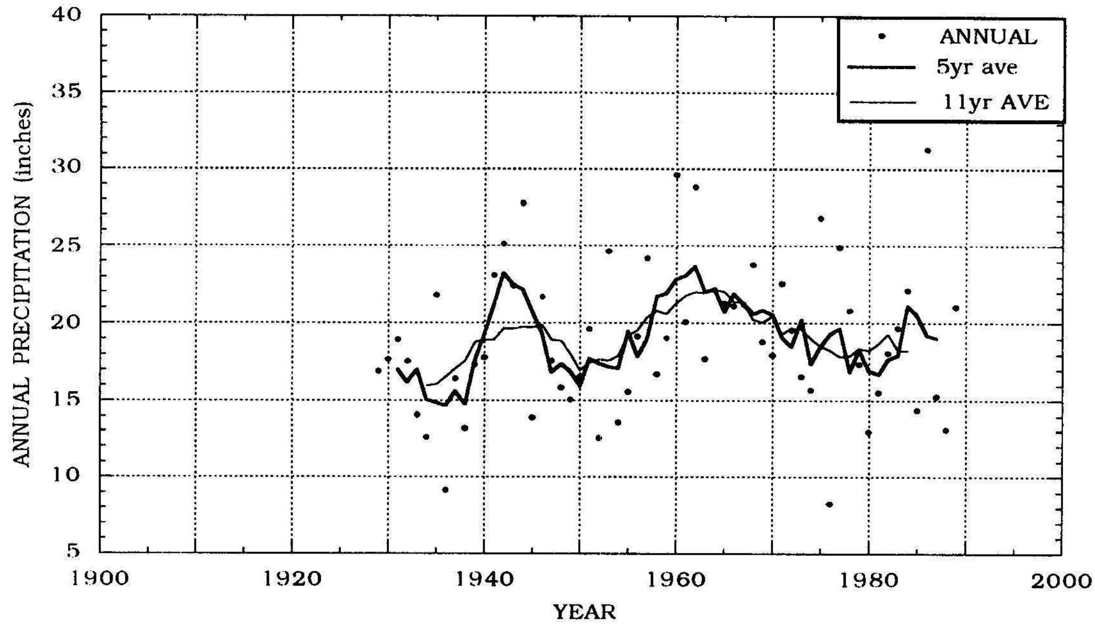

Fig. I-7 .Oakes composite annual precipitation with nearby stations used to fill in missing data.

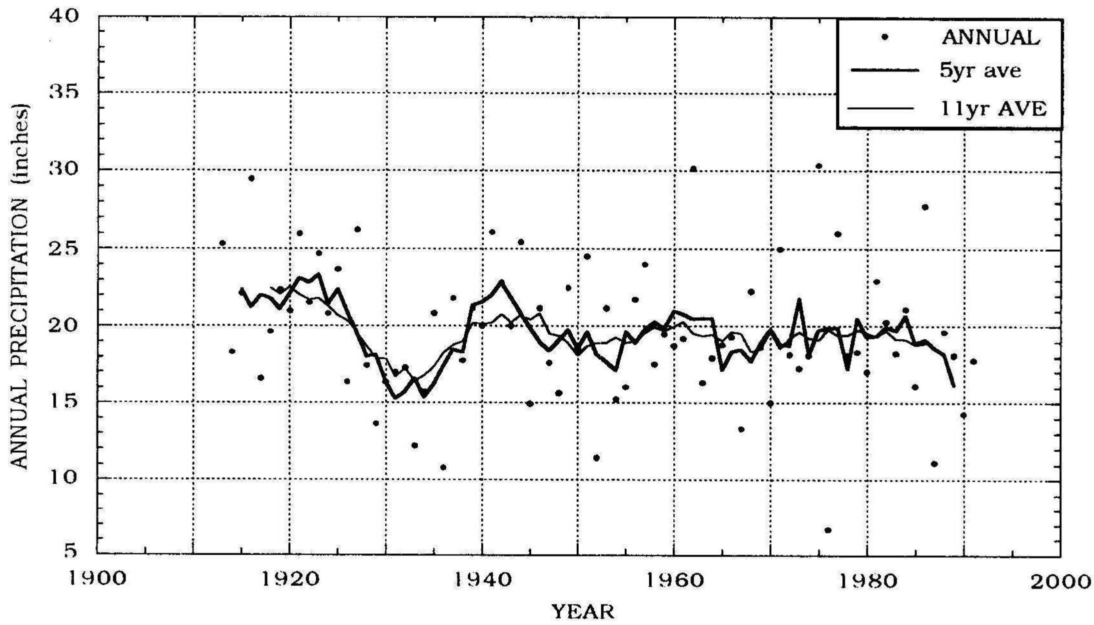

Fig. I-8. McLeod composite annual precipitation with nearby stations used to fill in missing data.

As can be seen in Figs. I-4, I-7, and I-8, there is considerable variation in long-term precipitation. The 11-year average precipitation at Lisbon for 1904 to 1989 varies from a minimum of 17.6 inches per year to a maximum of 22.5 inches per year. This indicates that the amount of water available from the aquifer system varies significantly over a period of decades.

Rainfall intensity and duration are probably more important than total annual rainfall in determining the hydrology of the Englevale aquifer. Large, but infrequent, rainfall events play an important part in the recharge of the Englevale aquifer. Heavy fall rains of over 5 inches during periods of less than 3 days have resulted in substantial recharge to the Englevale aquifer in the fall. These storms occurred in 1977, 1978, 1982, and 1984. Precipitation accumulations of over 6 inches occurred September-October of 1971 and 1973. In both of these years, the precipitation occurred as small storms distributed over the two month period. The lack of water table rise in either year indicates no fall recharge occurred. The amount of precipitation from the small storms was insufficient to fill the soil profile. This additional water in the soil profile was then lost to evapotranspiration before the next rainfall occurred.

Missing data are a problem in examining long periods of climate data. The three climate data sets used in this study are from Lisbon, Oakes, and McLeod. For some of the climate analysis and the model simulations, missing climate data were filled in with data from nearby stations. Both Lisbon and McLeod have little missing data and should be accurate. Oakes has long periods of missing data, particularly in the 1970s and 1980s. The Oakes data set has a large fraction of its data from Forman to fill in the missing records.

HYDROGEOLOGY

Origin of the Englevale aquifer

The Englevale aquifer is composed of sediments deposited along a former course of the Sheyenne River melt-water channel (Armstrong, 1982). Near the end of the last glaciation, the ancestral Sheyenne River flowed south from Fort Ransom and discharged into Lake Dakota along the North Dakota and South Dakota border. The history of the aquifer is complex as a result of the advance and retreat of glacial ice across the area now occupied by the aquifer. The Sheyenne River abandoned the channel to the south of Fort Ransom when the ice retreated to the northeast (Bluemle, 1979, p. 55).

Hydraulic Properties

The availability of water from an aquifer is partly determined by the hydraulic conductivity, transmissivity, saturated thickness, and specific yield of the aquifer sediments. Hydraulic conductivity reflects the ability of a material to conduct water. Well-sorted coarse sediments have higher conductivity than poorly sorted fine sediments. Silt, clay, and till generally have such low conductivity that they are not considered aquifers. The ability of an aquifer to conduct water is determined by the transmissivity which is the hydraulic conductivity multiplied by the saturated thickness. A larger transmissivity means that a well will derive its water from a larger area, thereby resulting in less drawdown at the well.

The water levels in the aquifer fluctuate with changes in recharge and discharge. The magnitude of the fluctuation is determined by the specific yield (S_y) of the aquifer. The S_y is the amount of water that will drain during a one foot decline in head from an area of one square foot. Aquifer tests indicate a S_y of 0.18 for the Englevale aquifer (Shaver, 1977 and Reiten, 1980). This means that 0.18 cubic feet of water per square foot will drain with a one foot decline in head. Actual specific yield values are generally higher than those indicated by an aquifer test because the duration of the test is not long enough to allow for complete drainage. The larger the specific yield, the smaller the drawdown will be at a well. This occurs because the aquifer yields more water for a given amount of drawdown.

Aquifer Description

The Englevale aquifer in the study area covers 44 square miles, 33 of which are in Ransom County (Fig. I-9). The aquifer is unconfined throughout the study area. However, the portion of the aquifer in Sargent County is a multi-layer system.

The northern part of the Englevale aquifer is divided into two channels separated by a till divide (Figs. I-9 and I-10). The divide is seen at the surface as a low oblong ridge shaped by flood

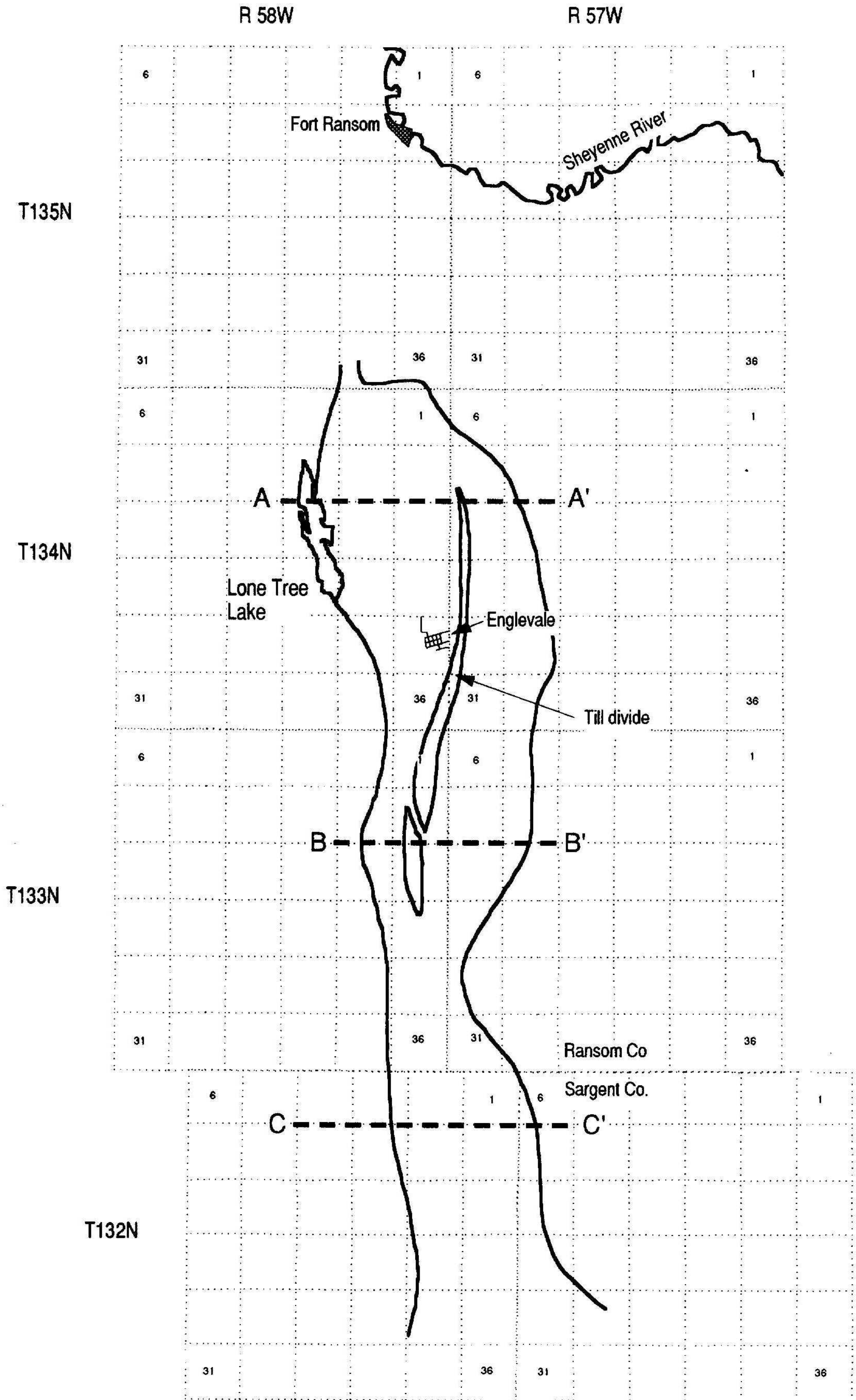

Figure I-9. Map showing location of generalized Englevale aquifer cross-sections.

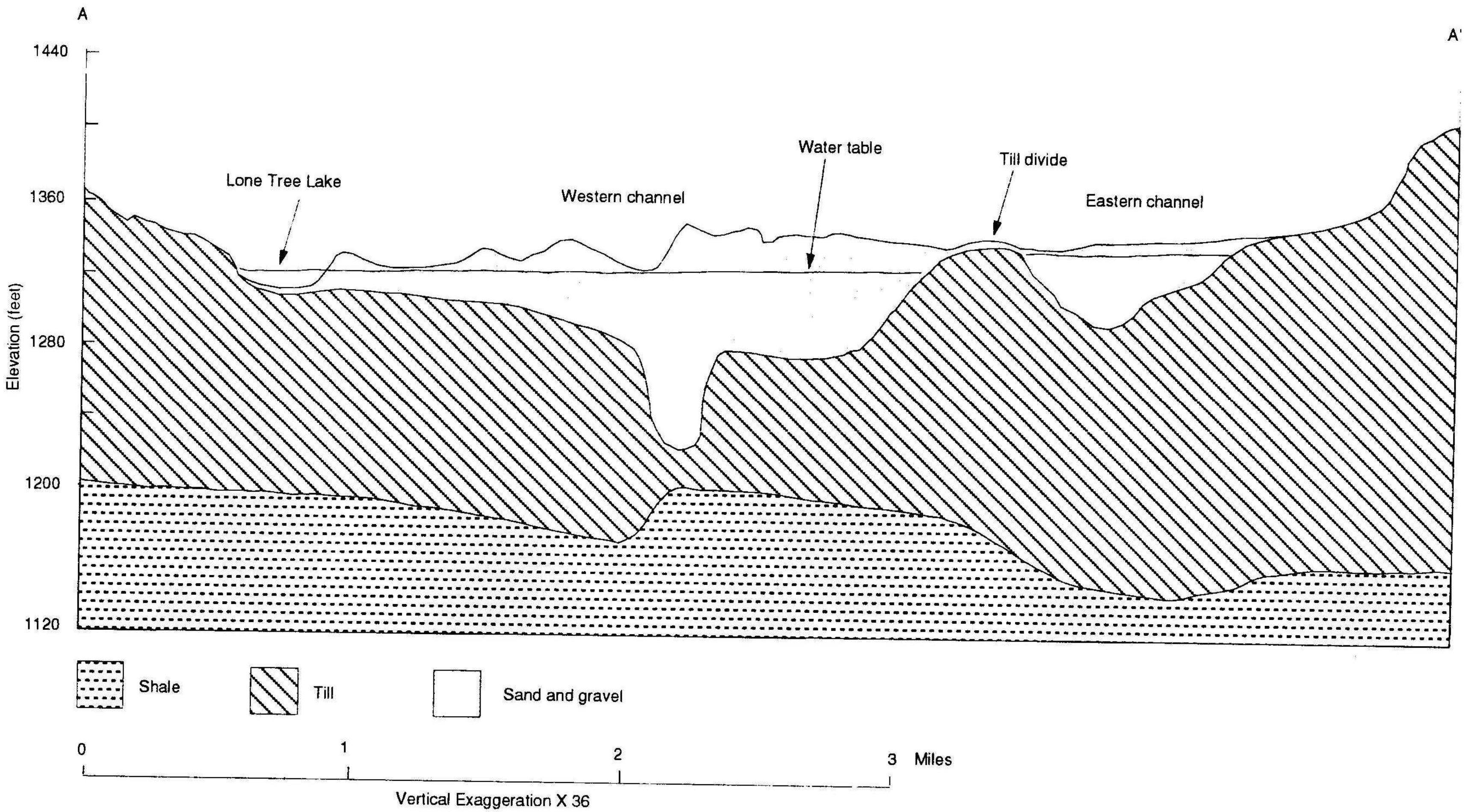

Fig. I-10. Generalized cross-section A-A' for northern part of Englevale aquifer study area.

waters in Sections 12 and 13, Township 133 North, Range 58 West. To the north, the ridge was either smaller or eroded, and covered by a thin mantel of fluvial sediments. To maintain the large water level difference across 134-057-18 requires the presence of a low permeability divide as shown in Fig. I-10. However, there is some flow across the divide north of Englevale. The divide is probably dissected by channels, and the overlying fluvial sediments may have enough saturated thickness to allow some flow across the divide.

The western channel is the principle channel of the aquifer and is the most productive for irrigation because both hydraulic conductivity and saturated thickness are larger. Saturated thickness ranges from less than 5 feet to over 120 feet. Saturated thicknesses greater than 30 to 40 feet occur primarily within a narrow inner channel extending from Section 1, Township 134 North, Range 58 West through Section 36, Township 132 North, Range 58 West. The channel appears to be less than 1/4 mile wide at the north end of the study area widening to approximately 3/4 mile at the south end. The channel is shown in the cross sections (Figs. I-10, I-11, and I-12).

There is no indication of the deeper part of the channel extending north of 134-058-01. The line of test holes across the southern edge of 135-058-35 and 135-058-36 indicate a base of aquifer elevation of approximately 1300 feet. A similar line of test holes across the northern edge of 135-058-26 shows the base of aquifer at approximately 1300 feet. At 135-58-35DDD and 36DCC the aquifer is overlain by 40 feet of till.

North of Englevale, the irrigation development occurs in the eastern part of the western channel which is characterized by large hydraulic conductivities. Hydraulic conductivities near the irrigation well at 134-057-18CBB were determined by an aquifer test to be 1100 feet per day (f/d) (Reiten, 1980). Another test at 134-058-25DCC indicated hydraulic conductivity of 600 f/d (Shaver, 1977). The problem in this area is with the irrigation wells located along the western edge of Township 134 North, Range 57 West. The wells are located close to the till ridge that divides the eastern and western aquifer units and initial saturated thicknesses ranged from 25 to 30 feet. Initial pumping rates were generally too high leaving these wells with no drawdown reserve if water levels declined. Most other wells in this area of the aquifer have adequate saturated thicknesses. A few wells, such as 134-058-01D, have as much as 60 feet of saturation.

The northern part of the eastern aquifer unit of the Englevale aquifer is thinner and generally of lower permeability fine to medium sand. Test drilling has never indicated an adequate section for the development of an irrigation well in the eastern aquifer unit north of Englevale. The eastern channel contributes some recharge to the western channel. Because of the contributed recharge and the serious water level declines in the western channel, no irrigation development should be allowed in the northern part of the eastern channel.

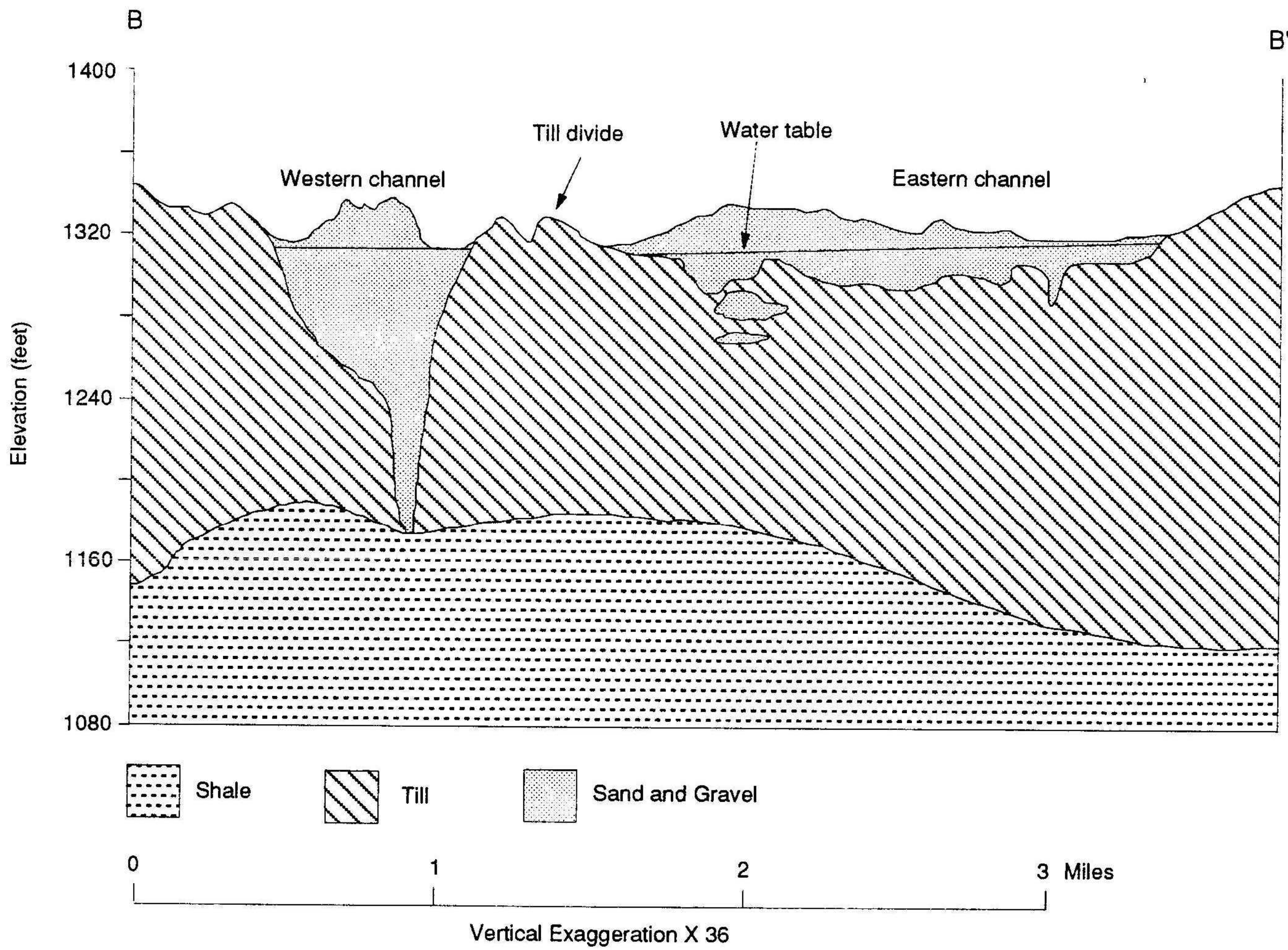

Fig. I-11. Generalized cross-section B-B' for middle part of Englevale aquifer study area.

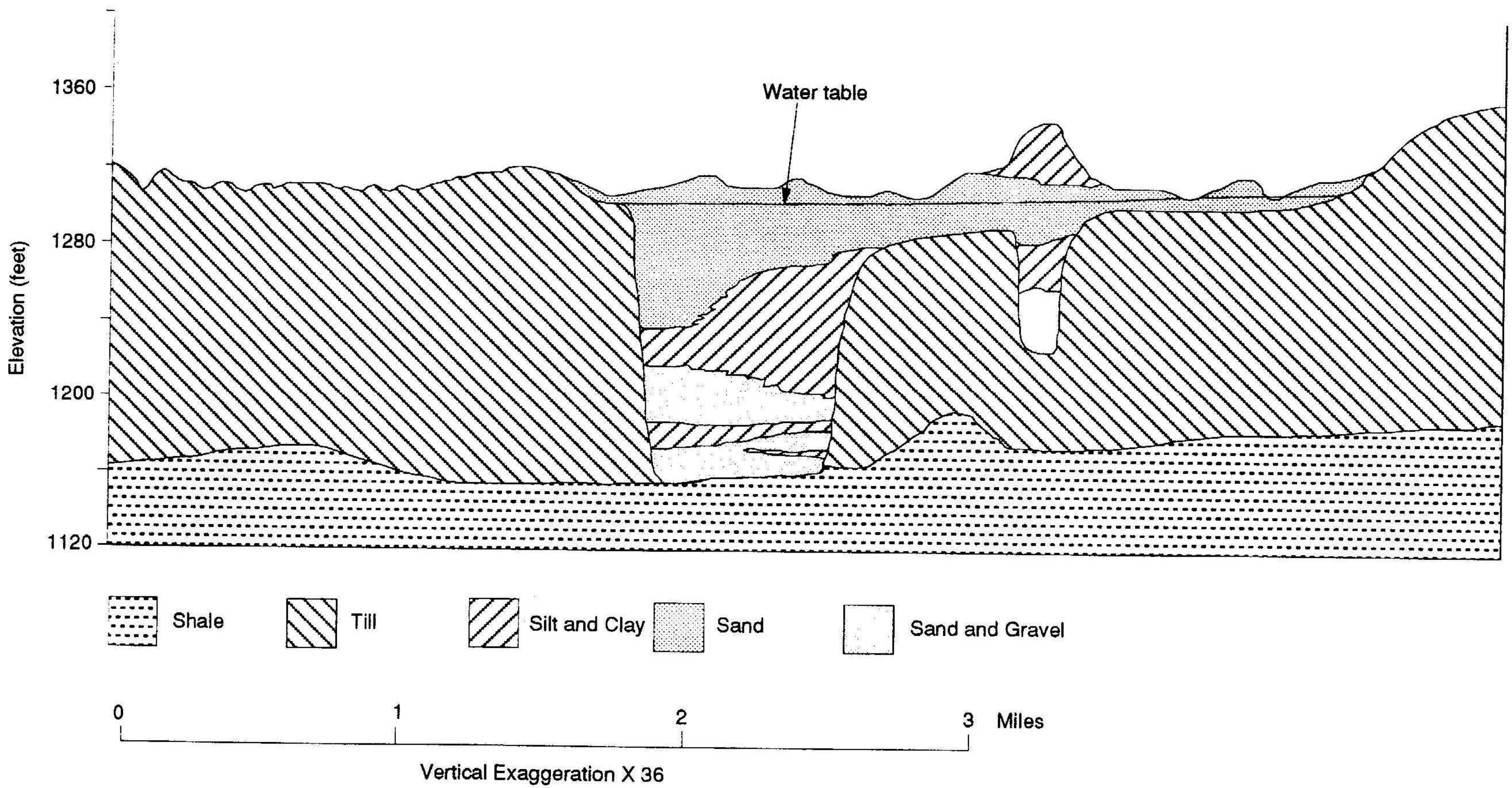

Fig. I-12. Generalized cross-section C-C' for southern part of Englevale aquifer study area.

At the central portion of the study area, shown by the generalized cross-section of Fig. I-11, the western channel is much narrower. The wells in the western channel range from 50 to 90 feet in depth. All of these wells should have large pumping capacities with good drawdown reserves.

The eastern channel aquifer is thin with generally less than 20 feet of saturated thickness. The area is dissected by narrow channels which are probably less than 300 feet wide with 30 to 40 feet of saturation. Most of the irrigation wells in this part of the eastern channel are developed in these deeper narrow channels. All of the irrigation in this area of the aquifer is from multiple well systems. Further water-level declines will significantly reduce well yields in this area. The eastern and western channels merge in the southern part of 133-058 (Fig I-9).

In the southern part of the study area, most of the irrigation wells are developed in the deeper confined aquifer units of either the main channel or the narrow eastern channel (Fig. I-12). Both of the confining units are very leaky. The thinner saturated thickness combined with the low hydraulic conductivity of the shallow water-table unit make it generally unsatisfactory for irrigation development. The lacustrine silts and clays of the confining units probably represent the periodic encroachment of Lake Dakota into the Englevale melt-water channel.

The narrow eastern channel is approximately 1000 feet wide (Fig. I-12). This channel appears to be a diversion of the main channel that occurred near 133-058-25. All of the irrigation wells located in 133-057-31, 132-058-01, 132-058-12 and 132-058-13B are in this unit. All irrigation wells in 132-058-11B, 132-058-14D, 132-058-13C, 132-058-24, and 132-058-26 are in the confined unit of the main channel. The remaining wells are located in the upper water-table unit. With the possible exception of the multiple-well system in 132-058-13D no serious effects from the present water level declines are foreseen.

Aquifer Recharge and Discharge

The ground water of the Englevale aquifer is derived from infiltration of precipitation. Part of precipitation is returned to the atmosphere by evaporation, part runs off to streams and the remainder infiltrates into the ground. Only part of the water that infiltrates reaches the zone of saturation. Part or all of the water is retained by the soil zone depending on its initial moisture content. The water retained by the soil zone is returned to the atmosphere by evaporation and transpiration. The combined process of evaporation and plant transpiration is referred to as evapotranspiration (ET). Because the soil is generally depleted of moisture by ET during the summer, very little recharge will occur to ground water during this time of year. Recharge occurs mostly in the spring as a result of snow melt and spring rains but may also occur in the fall when ET is less and when rainfall exceeds the soil moisture deficit that developed in the summer. Recharge will be greater on coarser textured soil than on fine textured soil because the moisture-holding capabilities of the coarser soils are less. Therefore less infiltration is required to make up the soil moisture deficit resulting from ET. Recharge rates for the Englevale aquifer are large because the soils overlying the Englevale aquifer are coarse textured with water-holding capacities often less than 4 inches in the root zone.

Long-term changes in aquifer water levels reflect changes in discharge and recharge. Many years of record are needed to establish long-term water-level trends because of the large annual variations in recharge, ET, and irrigation withdrawals. Discharge from the Englevale aquifer prior to irrigation was predominately by evapotranspiration from ground water (ET_{gw}). ET_{gw} occurs where the water table is within a few feet of land surface where plant roots are able to pull water from near the water table. Evaporation also occurs from Lone Tree Lake and other small surface water bodies that are expressions of the water table. As the water table has declined due to irrigation, the amount of ET_{gw} has declined. A new equilibrium water level is established when the decline in ET_{gw} is equal to the increase in irrigation withdrawals.

Occurrence and Movement of Ground Water

Ground water moves under the influence of the gravity from areas of recharge to areas of discharge. The rate of movement of ground water in the Englevale aquifer is less than 600 feet per year for the coarse gravel and considerably less for the finer sediments. The rate of movement is governed by the hydraulic conductivity and the slope of the water table (hydraulic gradient).

The waters of the Englevale aquifer generally flow to the south. There is some flow to the west toward Lone Tree Lake in the area north of Englevale. In the eastern channel north of Englevale, the water levels are much higher than in the western channel. There is some flow from the this region of the eastern to the western channel. There are some changes in flow direction between periods of high water levels such as 5/12/80 (Fig. I-13) and low water levels such as 12/21/82 (Fig. I-14). A significant change in the flow pattern between the two dates is the reduction of flow toward Lone Tree Lake that occurred by 12/21/82.

Water levels in the Englevale aquifer prior to irrigation were basically controlled by land surface. ET was the dominate discharge from the aquifer occurring near the many sloughs that covered the aquifer. Though Fig. I-13 indicates a single large flow system with water moving to the south, the flow system was actually composed of many local flow systems discharging at the sloughs. The water table gradient in Fig. I-13 generally mirrors the land surface gradient and does not reflect the quantity of water flowing to the south. A small amount of water flows out of the area to the south as underflow. It appears that there is no significant discharge from the aquifer to the north. The highest water levels in the western channel occur in 134-058-01 at the north end of the study area (Fig. I-13). Water levels here are approximately 190 feet above the elevation of the

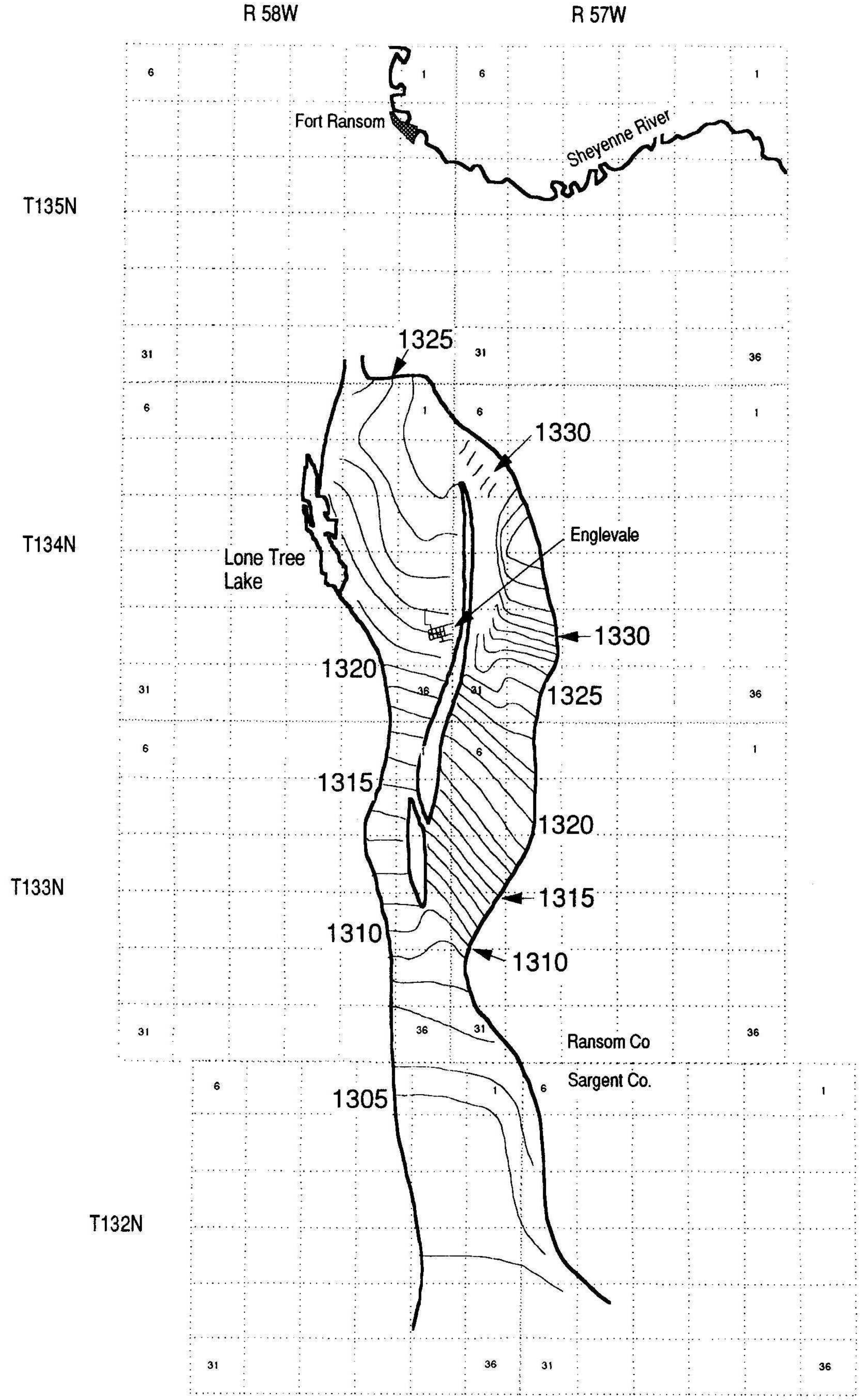

Figure I-13. Englevale aquifer water table contour map for 05/12/80. Elevation in feet above mean sea level..

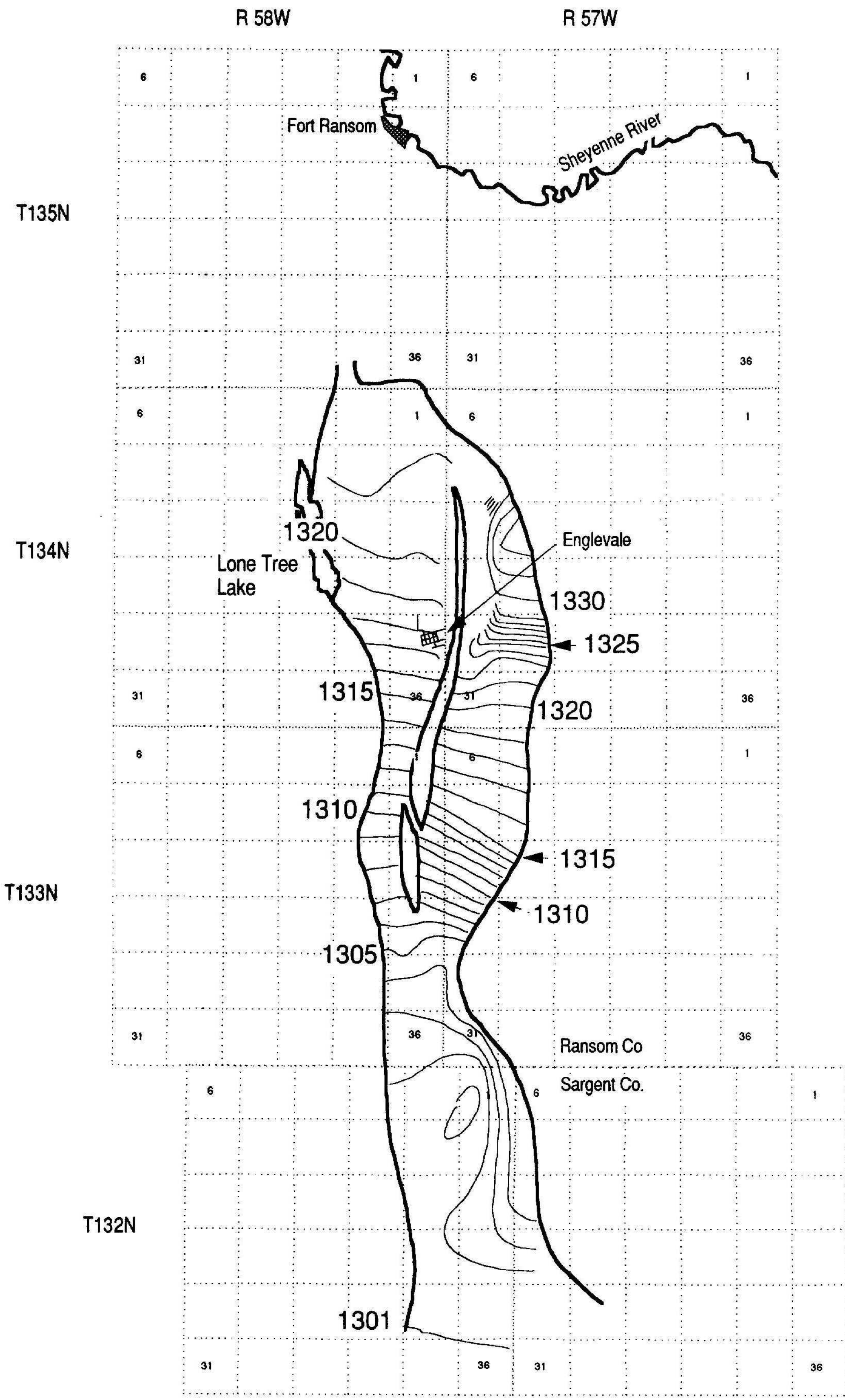

Figure I-14. Englevale aquifer water table contour map for 12/21/82. Elevation in feet above mean sea level..

Sheyenne River. If the deeper channel discharged to the north, then it would drain considerable water from the Englevale aquifer.

IRRIGATION

Large-scale irrigation development from the Englevale aquifer started in the mid 1970s. No effort was made to tabulate irrigated acres or water use before 1975. However, about 1800 acres were irrigated from the Englevale aquifer by 1975. Most of the previous development occurred in 1973 and 1974, though the earliest development was in 1958. Water-level data indicates irrigation had a minimal effect on water levels prior to 1976. The large rain storm of June 1975 resulted in abnormally high water levels and is assumed to have removed the effect of previous irrigation. Therefore, 1976 was chosen as the starting point for analysis of the aquifer and therefore tabulation of water use data. Another reason for ignoring the period prior to 1976 was the lack of adequate water use data. Also, most of the observation wells were installed after 1975.

The water use data were determined from reported water usage and kilowatt-hour data. A relationship between kilowatt-hours and water use was established by comparing kilowatt-hour data with water use data on years with reliable water use reports. The relationships were used to estimate water use on years with less reliable reports. The data for 1975 is reported water use only.

Figures I-15 and I-16 show acres irrigated and total acre-feet of water used respectively. At present there are 10,800 acres irrigated from the Englevale aquifer in the study area (Fig. I-2). This is from an aquifer with a surface area of less than 30,000 acres. Over one-third of the aquifer surface area is irrigated. Water use for 1981 to 1991 has ranged from 5,275 to 12,800 ac-ft/year. Water use for 1981 to 1991 from the Englevale aquifer has averaged 9,000 ac-ft/year. To supply the present average water use requires that at least 3.6 inches per acre in recharge be captured for irrigation.

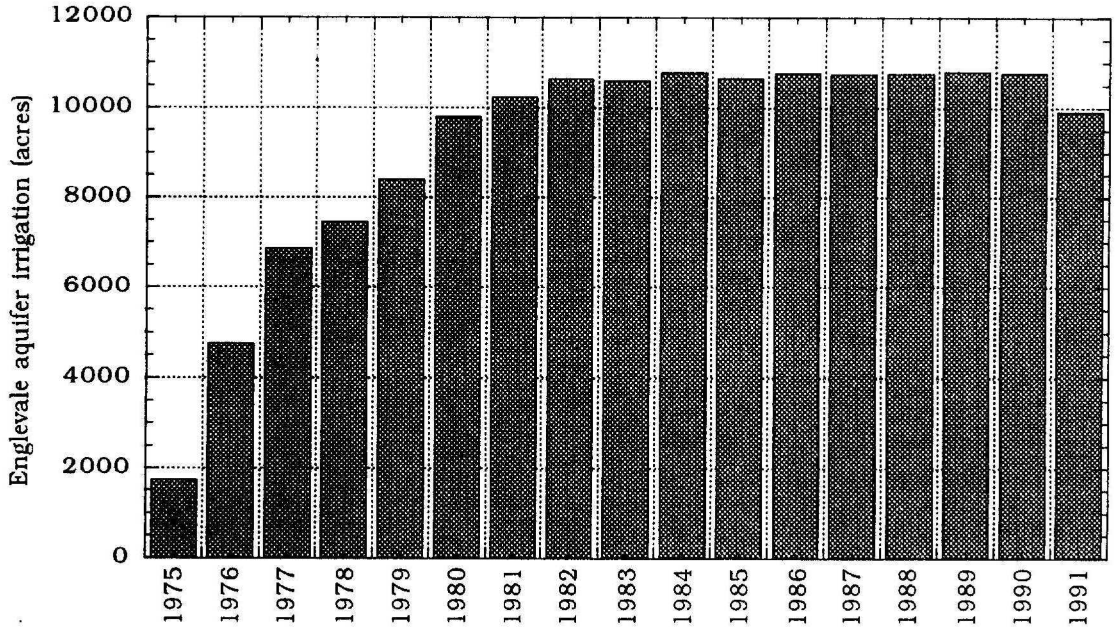

Fig. I-15. Total acres irrigated from the Englevale aquifer within the study area shown in Fig. I-2.

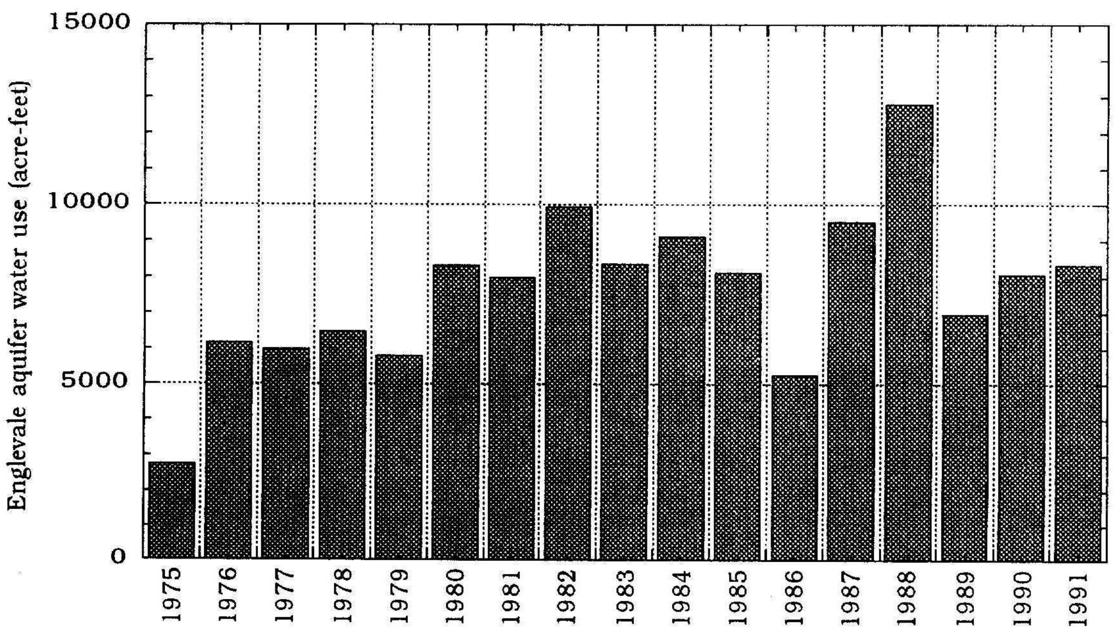

Fig. I-16. Total acre-feet of water pumped from the Englevale aquifer within the study area shown in Fig. I-2.

The average annual application of irrigation water is approximately 10 inches per year (Fig. I-17). The only large deviations were in 1976, 1986, and 1988 when 15.6 inches, 5.9 inches and 14.3 inches were applied, respectively. The years of 1976 and 1988 were years of extreme drought with irrigation starting earlier than normal. Though 1982, 1984, 1985 had below normal precipitation during the growing season, the water use was not significantly different than other years. This likely occurred due to good spring soil moisture conditions, increased irrigation of small grains due to the Federal Payment in Kind (PIK) program, and better water management practices encouraged by low grain prices and high power costs.

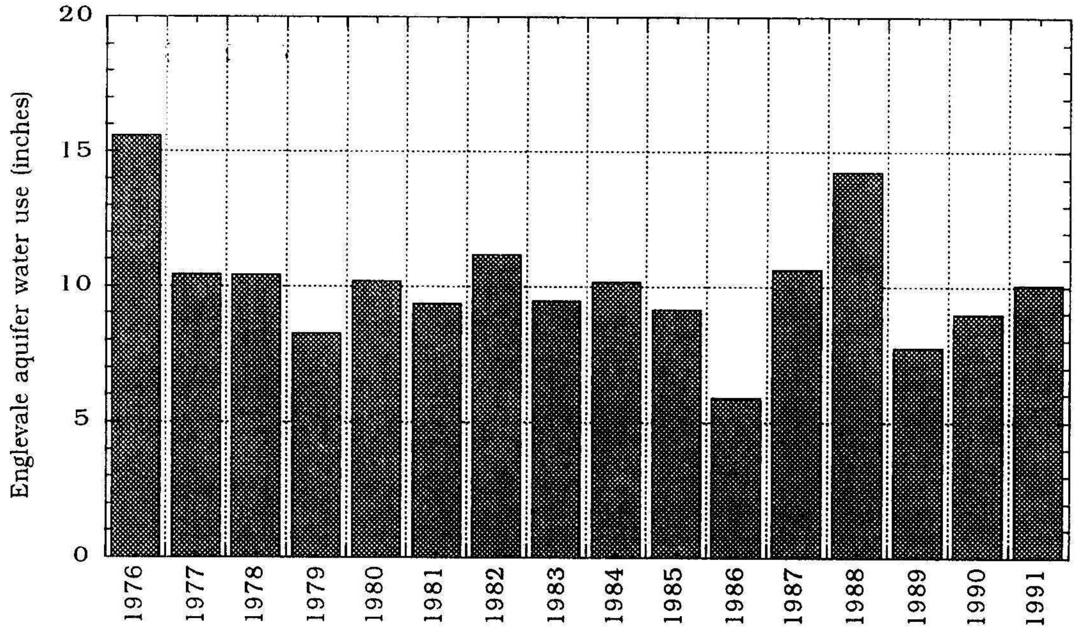

Fig. I-17. Average annual inches of irrigation water applied to land irrigated from the Englevale aquifer.

WATER LEVELS

The decline in water levels due to irrigation is determined by the hydrology of the wetland areas overlying the aquifer. Several factors will affect how far water levels must decline to reduce discharge by evapotranspiration. In wetlands, there may be free water where direct evaporation occurs. Evaporation also occurs where water is transferred from the water table to land surface by capillary rise. The major source of discharge around wetlands is evapotranspiration by plants. The amount of water discharged by evapotranspiration depends on the plant root depth, how the roots respond to a change in water level, movement of water from the water table to the bottom of the root zone by capillary rise, depth to the water table, and wetland topography. In aquifers without significant irrigation development, water levels often fluctuate 3 to 4 feet due to climatic variability. Therefore, to substantially eliminate evapotranspiration losses from an aquifer, the water level must be reduced by at least 3 or 4 feet.

There are two components to the developmental decline in water levels due to irrigation. The first is the decline required to reduce discharge due to evapotranspiration at the discharge area. This is shown by the decline in slough level in Fig. I-18. The size of the slough and size of the area around the slough where evapotranspiration occurs are reduced at the lower slough level. The slough will continue to shrink until evaporation from around the slough is reduced by the amount of water pumped for irrigation. The second component is due to the reduction in water

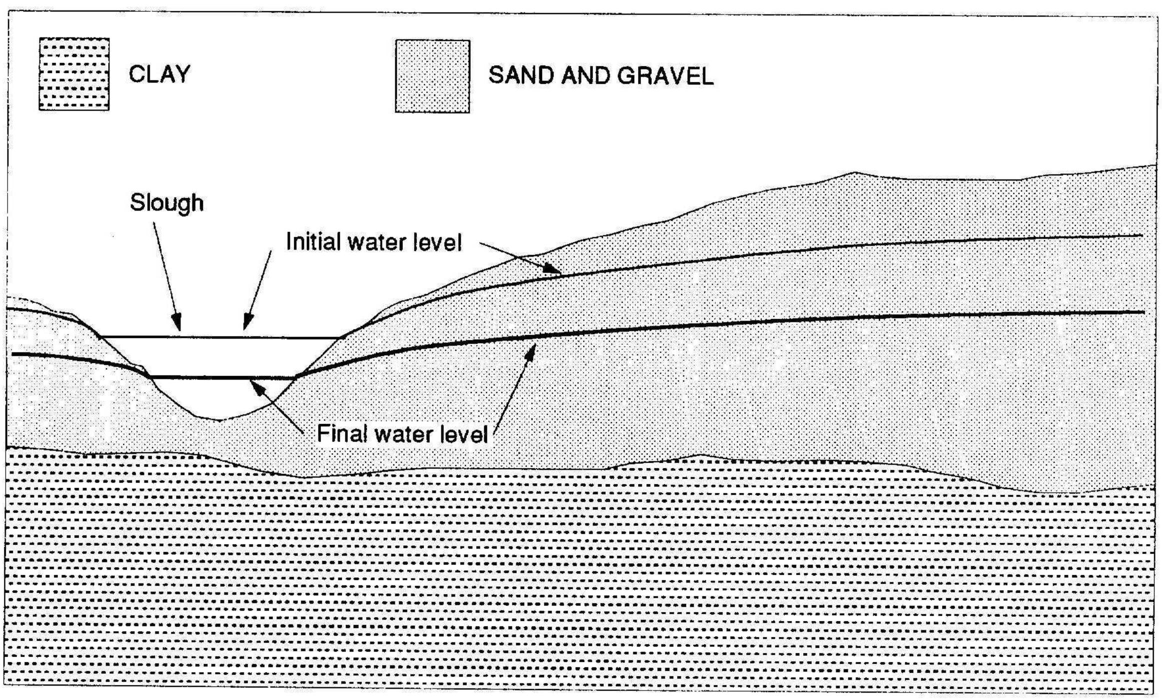

Fig. I-18. Diagram showing relationship between change in head at a discharge area (slough) and change in head and gradient in the aquifer assuming flow to the discharge area is reduced by irrigation.

table gradients resulting from less water flowing toward the discharge area when the irrigation wells are not at the natural discharge areas. In Fig. I-18 it is assumed that irrigation development has occurred to the right of the diagram. Therefore, less water flows toward the discharge area at the slough and results in a reduced water table gradient. The effect of this gradient reduction is a larger decline in water levels as the distance from the slough increases (Fig. I-18). The gradient reduction can be seen by comparing the area north of Englevale in Figs. I-13 and I-14. A new equilibrium water level is established when the amount of water pumped for irrigation equals the reduction in evapotranspiration. The amount of water that can be appropriated is determined by the amount of water that can be captured from evapotranspiration.

Developmental water level declines of at least 3 to 6 feet will occur when the amount of water pumped for irrigation is a large fraction of the water lost on average to evapotranspiration. Under natural conditions, prolonged periods of drought will cause water levels to decline to a minimum level where evapotranspiration is largely eliminated and water levels will stabilize at that level. Aquifer water levels will not stabilize during droughts when irrigation is introduced and the amount of water pumped is a large fraction of the water that was lost to evapotranspiration. With irrigation, the water lost from the system is independent of the water level in the aquifer and water levels will decline below the natural minimum levels. As long as the amount of water pumped for irrigation exceeds the amount of recharge to the aquifer, the water levels will continue to decline.

When the climate returns to "normal", then water levels will recover. Recharge will exceed discharge until evapotranspiration again becomes a factor. Therefore, the length of drought that can be tolerated without adverse effects will depend on the amount of available drawdown in the irrigation wells.

The water-level trends in the area north of Englevale for the 1980s indicate that the aquifer is over-appropriated under the climatic conditions that occurred during this period. The locations of observation wells discussed in this report are shown in Fig. I-19. Figure I-20 shows water levels in the area north of Englevale have declined over 6 feet since irrigation began. For the period 1976 to 1991, water levels have declined an average of 0.34 feet per year (ft/yr) at observation well 134-058-24CDC2. The rate of decline was only 0.14 ft/yr during the period 1982 to 1991. At observation well 134-057-18BBB the rate of decline was 0.19 ft/yr during the same period. The difference in rates of decline in water levels between the two periods partly reflects climatic differences. However, a major component is that the 1976 to 1991 period includes developmental declines while the 1982 to 1991 period does not. As can be seen in Fig. I-20, annual variations in water levels due to climate are much greater than the average annual long-term changes. Because of the large annual variability in water levels, long periods of water-level records are required to establish whether the aquifer is over-appropriated.

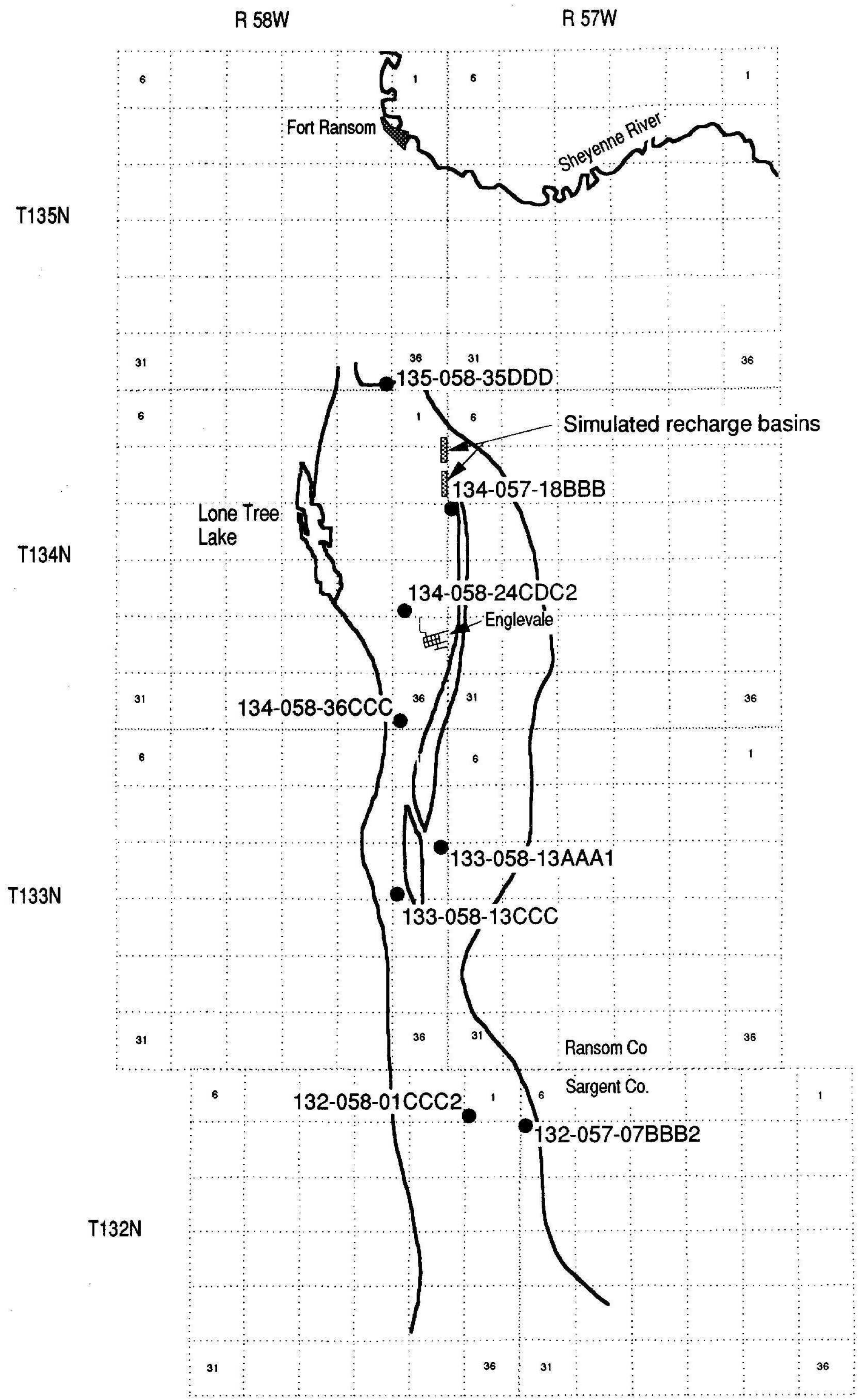

Figure I-19. Map showing location of selected Englevale observation wells.

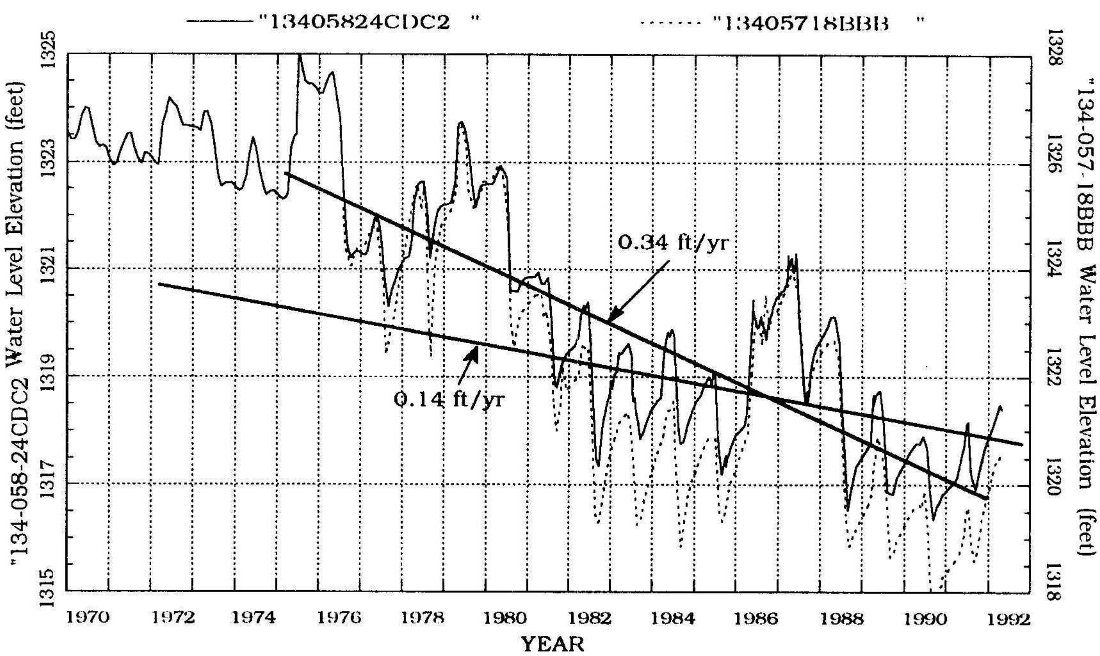

Fig. I-20. Water-level trends in observation wells 134-058-24CDC2 and 134-057-18BBB are shown for the area north of Englevale.

Figure I-21 compares water levels at observation well 134-058-24CDC2 north of Englevale and observation well 132-057-07BBB2 which is not affected by irrigation pumping. Observation well 132-057-07BBB2 provides a base line to evaluate the impact of irrigation on aquifer water levels. After early developmental declines, the difference in water level decline at the two sites was about 4 feet until 1989 Then the differential increased to about 6 feet. The larger differential was due to heavy rains in the vicinity of 132-057-07BBB2 resulting in a large increase in water levels during 1989 that did not occur at observation wells to the north of this site. The dry summer of 1991 in the vicinity of 132-057-07BBB2 has resulted in the reduction of the differential to 4 feet by the spring of 1992. This shows that the aquifer water levels are very sensitive to local climatic variability.

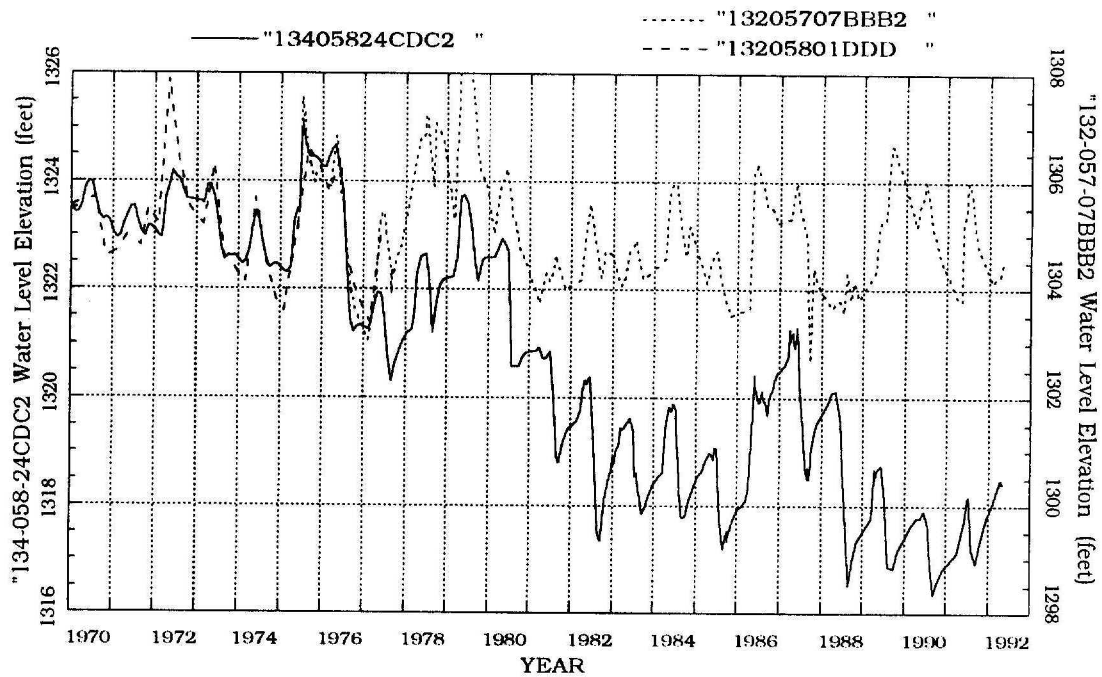

Fig. I-21. Comparison of water level trends at observation wells 134-058-24CDC2 and 132-057-07BBB2. Observation well 132-058-01DDD is a USBR well located near 132-057-07BBB2 and is used to extend its record.

Figures I-22 to I-24 show a comparison of water levels at 134-058-24CDC2 with other observation wells in the Englevale aquifer. In general water-level responses are similar throughout the study area. The large difference in water levels between the area near Englevale and the area south of the Ransom-Sargent county line results from heavy rains that occurred in the southern part of the study area during the summer of 1989. This rain resulted in a 2 foot differential in the amount of water level decline between the area north of Englevale and the area near the Ransom-Sargent county line. Because of the difference in 1989 precipitation the area near the Ransom-Sargent county line does not show the over-appropriation that the area north of Englevale does.

Future trends in water levels in the Englevale aquifer will depend on future climatic variability. Whether the aquifer is over-appropriated in the long term depends on how the climate of the last decade compares to long-term climatic averages. The next section examines long-term climatic trends and how the aquifer would have responded to this variability.

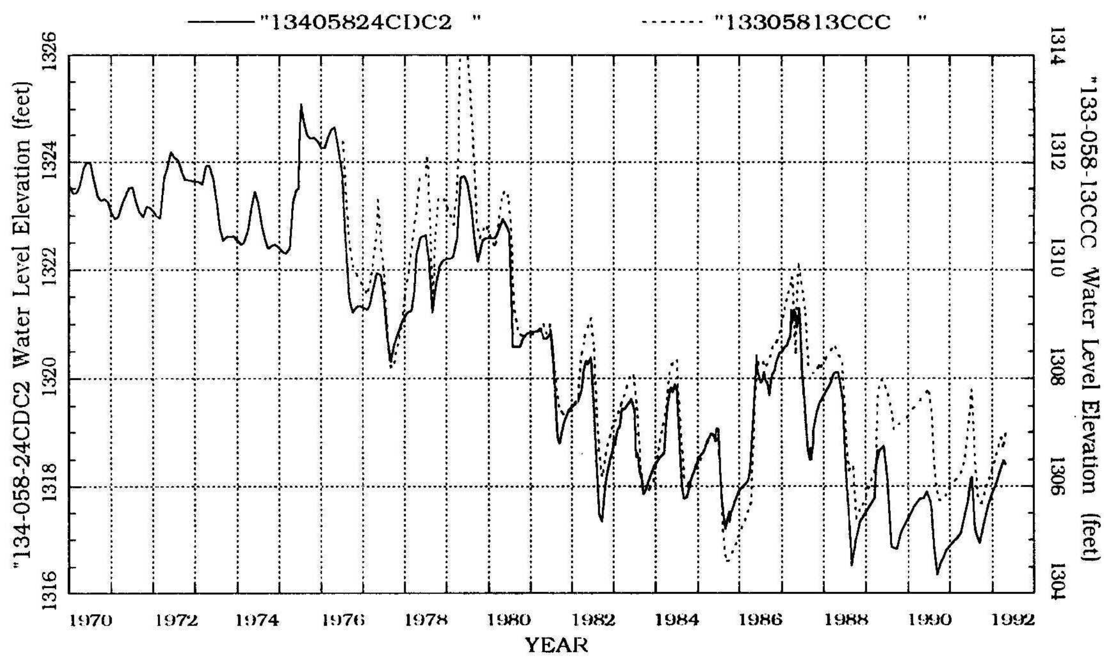

Fig. I-22. Comparison of water level trends at observation wells 134-058-24CDC2 and 133-058-13CCC.

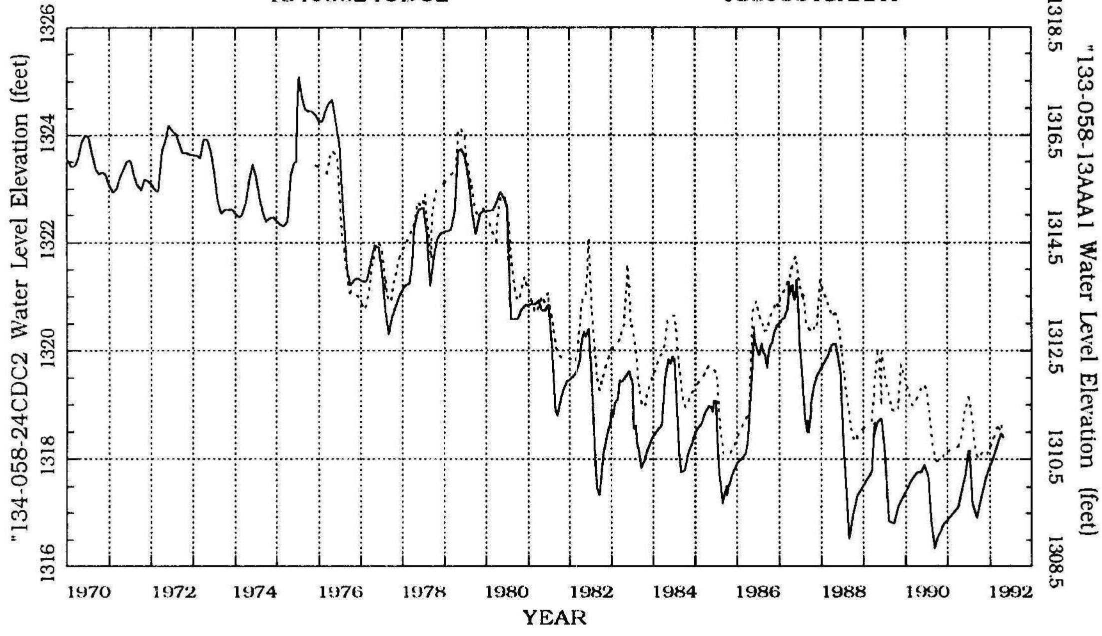

Fig. I-23. Comparison of water level trends at observation wells 134-058-24CDC2 and 133-058-13AAA1.

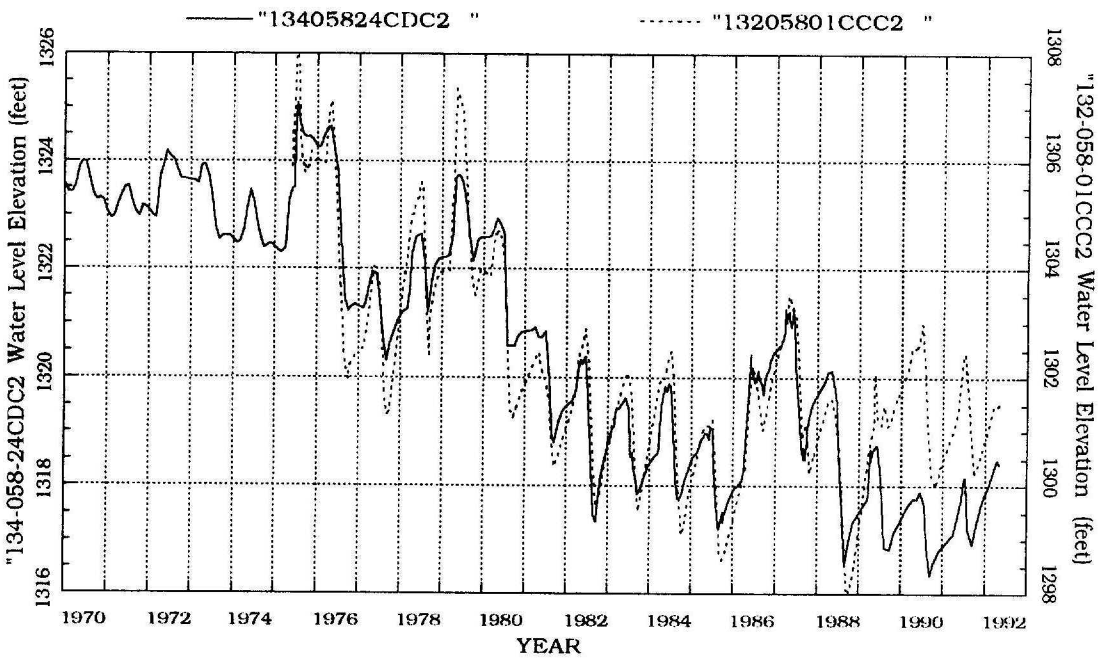

Fig. I-24. Comparison of water level trends at observation wells 134-058-24CDC2 and 132-058-01CCC2.

RELATIONSHIP BETWEEN CLIMATE AND WATER LEVELS

Finite-Difference Ground-Water Model

As a part of this study, computer simulations where used extensively to evaluate the conditions under which the aquifer can sustain the present level of development and estimate the ability of the aquifer to store artificial recharge water during wet periods for use during dry periods. Prior to this study, the Englevale aquifer model had been calibrated using the period from 1976 to 1980. The area of the aquifer modeled extends from the northern end of the aquifer approximately one mile north of Highway 27 to one mile south of the Ransom-Sargent County line.

To aid aquifer management, a mathematical model of the Englevale aquifer was developed in the late 1970s using the USGS "Finite-difference model for aquifer simulation in two dimensions" by Trescott et al. (1976). The data sets were converted in 1985 to use the USGS "Modular three-dimensional finite-difference ground-water flow model" (MODFLOW) developed by McDonald et al. (1984).

The model of the Englevale aquifer provided useful insights into the hydrology of the aquifer. The model works well within the calibrated range of stress. The period from 1981 to 1985 was used to verify the original model calibration. Ground-water evapotranspiration was calculated by the Jensen-Haise method (Stegman, et al., 1972) from Oakes climate data and adjusted for summer precipitation to maintain a correct water budget. Recharge was calculated from winter precipitation data by assuming the fall was entered with a maximum soil moisture deficit of 4". Rainfall data were primarily from Lisbon with adjustments for Englevale data where available. The spatially varied pattern of recharge was considered constant in all stress periods. Higher rates of recharge were considered to occur near the sloughs due to runoff. Each year of the simulation was divided into four stress periods consisting of spring recharge, summer irrigation, fall recharge, and winter.

No attempt was made to accurately reproduce water-level elevations. Because of uncertainties of several feet in land surface elevation, extinction depth of evapotranspiration, and scale problems with the evapotranspiration function any attempt to match water-level elevations to within more than a few feet was considered as likely to add more errors than it removed in the ability of the model to respond to changes in stress. The emphasis in calibration was placed upon creating the same pattern of water-level change in the model as was actually observed.

Soil-Moisture Budget Model

A soil-moisture budget model was developed to provide a better means of estimating recharge and ground-water evapotranspiration which were then used as input for the Englevale ground-water model. Inputs to the soil-moisture budget model are daily precipitation, daily maximum and minimum temperatures, crop type, soil water holding characteristics, and maximum root depth. The model is capable of simulating a two-layered soil and is based on an irrigation scheduling method developed by Stegman et al. (1972, 1977). The soil-moisture budget model uses the crop coefficients and root depth functions developed for the irrigation scheduling method. The initial soil-moisture budget model uses the Jensen-Haise method of calculating potential evapotranspiration which requires mean daily air temperature and solar radiation data. The model assumes only one crop and soil type.

As part of this study, the relation between climate and water level in the Englevale aquifer needed to be examined to determine if the aquifer could sustain its present level of development and determine the range of natural water-level fluctuations due to long-term climate variability. The soil-moisture budget model and the long-term climate records for Lisbon were used to generate input for the Englevale ground-water model to examine how the aquifer would respond to irrigation under past climate regimes. The data required for the soil-moisture budget model were minimized so the model could be used with long-term climate records where only precipitation and temperature data were available. The Baier and Robertson (1965) potential evapotranspiration model was used because it required only temperature and precipitation data. Comparison tests of the Baier and Robertson method using data from the North Dakota State University (NDSU) Oakes weather station indicate that it adequately reproduces long term averages of evapotranspiration. However, there are large errors in daily values compared to the more sophisticated Penman method discussed in Jensen (1973).

The model was calibrated against the period 1976 to 1989 by adjusting soil parameters within the soil-moisture budget model. This simple soil-moisture budget model appears to work reasonably well for soils with low soil water holding capacities such as those over most of the Englevale aquifer.

Development and Calibration of Aquifer-Climate Interaction Model

Climate is the dominant factor controlling water levels in the Englevale aquifer. It controls both the system inputs (ground-water recharge) and discharges (evapotranspiration and irrigation water use). To understand the ability of the Englevale aquifer to support the present level of irrigation requires relating water-level changes to climatic changes. In this section, long-term climate data for the area and the Englevale ground-water model were used to examine the relation between climate and aquifer water levels. There were many simplifying assumptions used to generate the scenarios, but they provide a conceptual basis for evaluating long-term water level trends.

Ground-water recharge processes are complex depending on many factors including antecedent soil moisture, evapotranspiration rates, size and duration of precipitation events, and whether precipitation occurred as rain or snow. Therefore, annual precipitation can only be considered a crude proxy for recharge. Though many simplifying assumptions must be made, on coarse textured soils a soil-moisture budget model can give reasonable estimates of ground-water recharge. Some of the processes that are not simulated in the budget model are snow melt, frost zone effects during the winter, and surface runoff. In addition, it does not consider areas surrounding the aquifer that contribute surface runoff during snow melt or large rain storms on the aquifer. The model assumes uniform soil and crops for the study area.

Water levels in the aquifer respond not only to changes in recharge but also to other factors which are also dependent on variations in climate such as changes in evapotranspiration and amounts of water pumped for irrigation. A simple soil-moisture budget model was used to evaluate the relationship of climate variability to response of water level in the Englevale aquifer. The model uses daily precipitation and temperature data as input. The budget model is used to estimate ground-water recharge, ET from ground water, and irrigation water use from Lisbon climate data.

To test the validity of the soil-moisture budget model, the output was used as input to the Englevale ground-water model. The resulting estimated water levels were compared to observed water levels (Figs. I-25 to I-28). The model does a good job of estimating water level changes

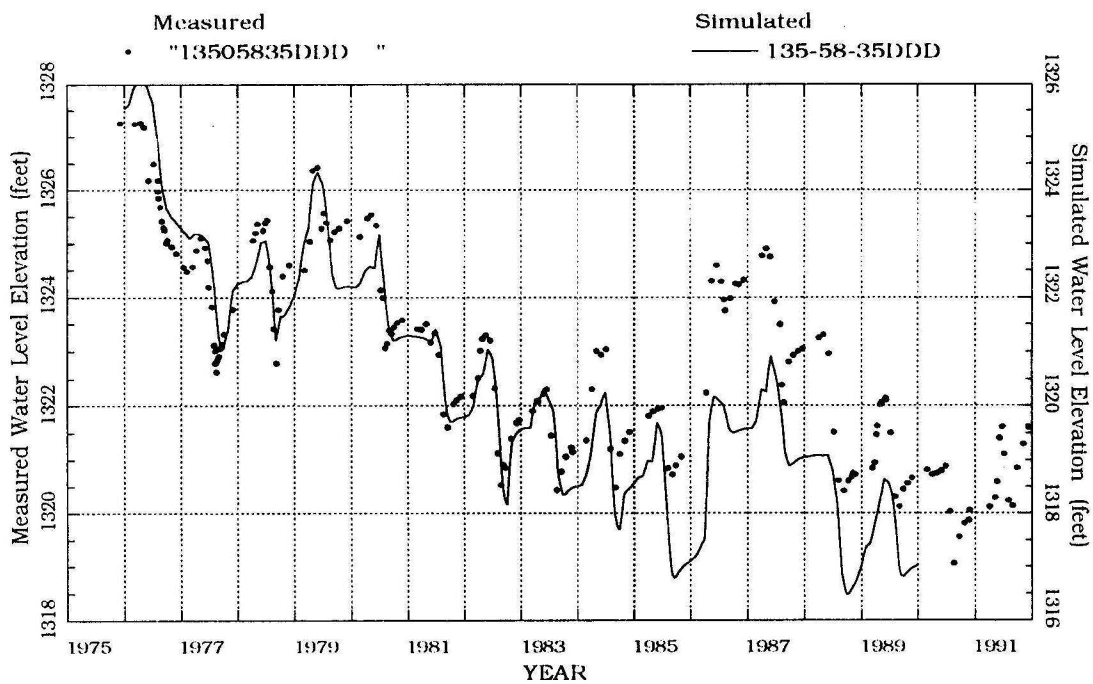

Fig. I-25. Comparison of simulated to measured water levels at observation well 135-058-35DDD

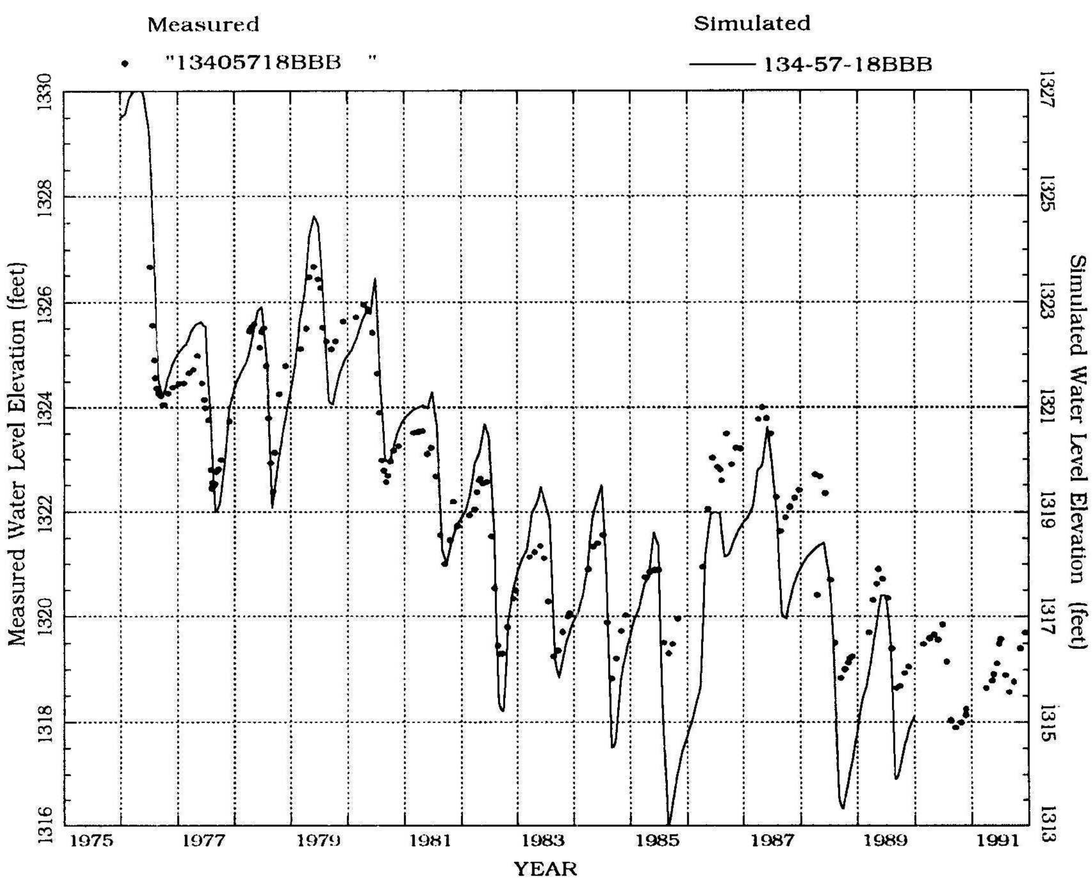

Fig. I-26. Comparison of simulated to measured water levels at observation well 134-057-18BBB

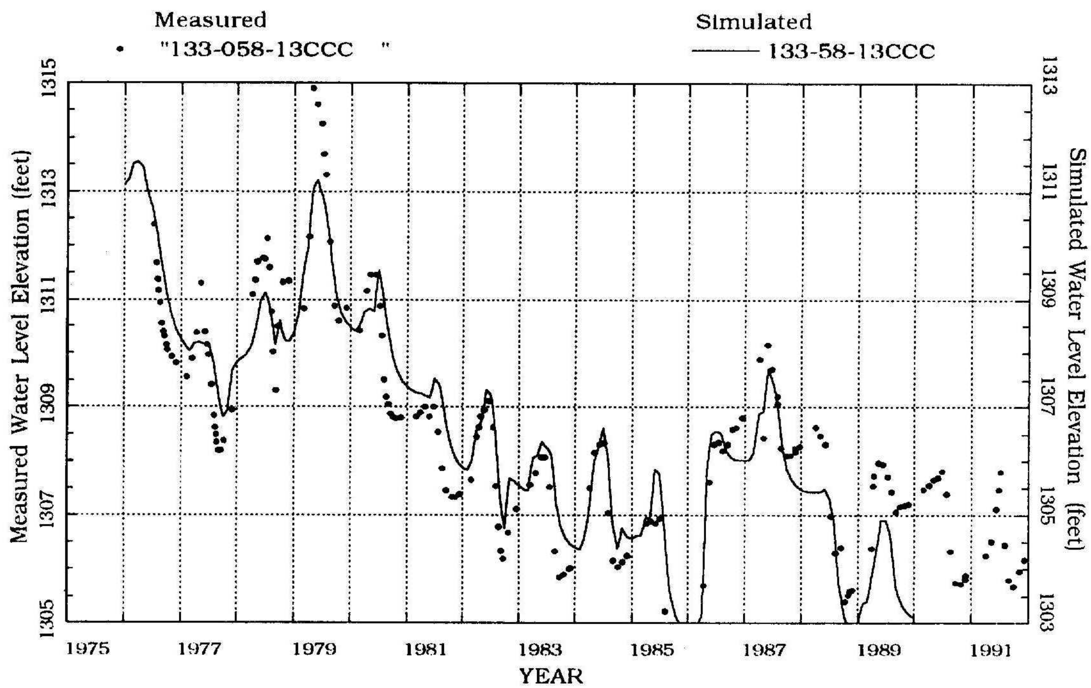

Fig. I-27. Comparison of simulated to measured water levels at observation well 133-058-13CCC

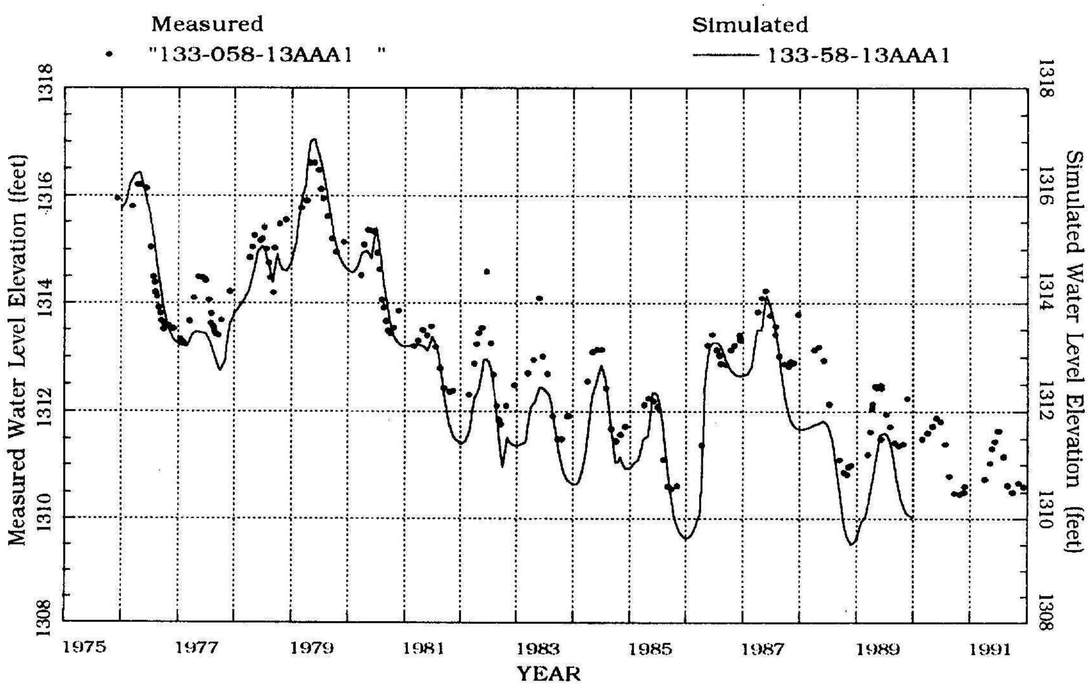

Fig. I-28. Comparison of simulated to measured water levels at observation well 133-058-13AAA1

considering all the simplifying assumptions in the model. There are significant differences between actual and estimated water levels for some years. However, the overall trend is good even though the model tends to overestimate the long-term decline near the irrigated parts of the aquifer in the area north of Englevale. The largest discrepancy occurs in 1985 in the northern part of the aquifer, when the model significantly over-estimates drawdown during the summer (Figs. I-25 and I-26). An explanation for this may be the difference between reported irrigation use and estimated water use (Fig. I-29). In 1985 the reported use was 4 inches per acre less than the estimated use. The difference is due to greater summer rainfall in the northern Englevale area than at Lisbon. In the middle and southern part of the aquifer study area, the precipitation was close to that observed at Lisbon. The model matches observed water levels much better in this area (Fig. I-27 and I-28). Though there can be large differences between reported and estimated

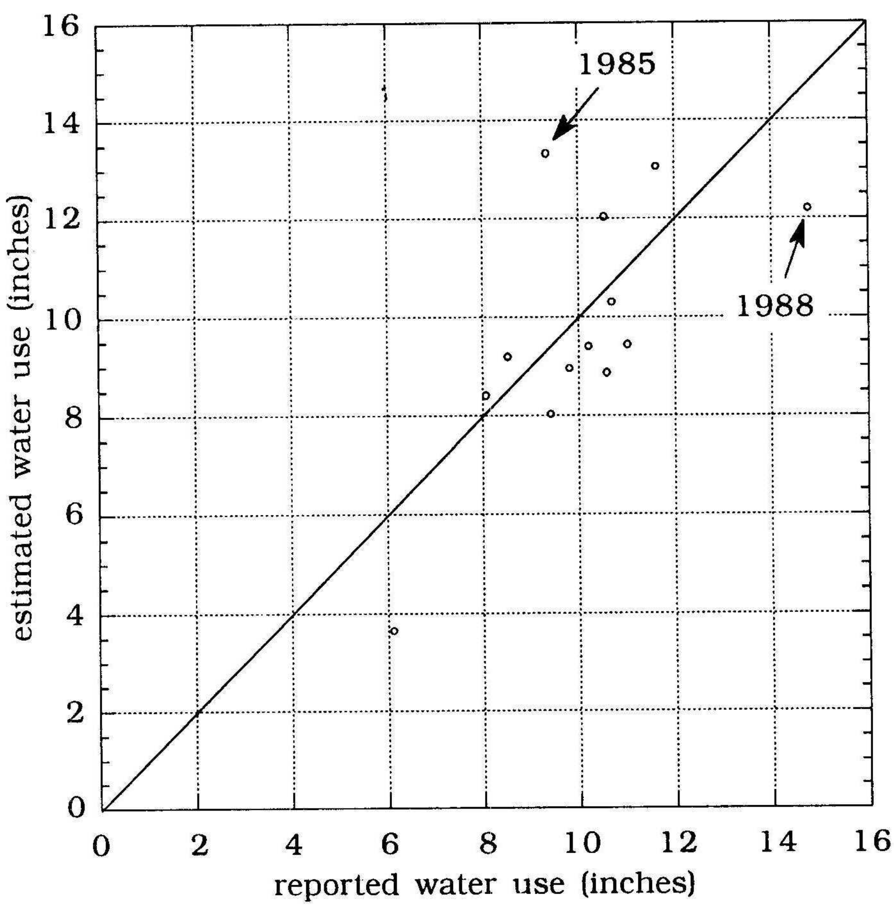

Fig. I-29. Comparison of water use estimated from Lisbon climate data using the soil moisture budget model to reported water use for Englevale aquifer for 1976 to 1989.

use for any given year, the budget model does a good job of reproducing long-term average use and its variability(Fig. I-29). For the period 1976 to 1989 the reported mean use was 10.5 inches per acre versus 10.2 inches per acre for the estimated water use. Because estimates of water use and natural losses to evapotranspiration are based only on temperature, significant differences between observed and estimated annual values are to be expected. Factors affecting evapotranspiration that are not accounted for include wind speed, humidity, solar radiation and surface roughness. Also, many of the large differences between simulated and measured water levels are due to differences in precipitation between Lisbon and the Englevale area. However, the model has the ability to estimate average water use over long periods with a reasonable reproduction of water use variability and the ability to reproduce changes in water levels from 1976 to 1989. Therefore, the model appears to be a valid method to explore how the aquifer would respond to different climatic scenarios.

Scenarios Exploring Effect of Long-Term Climatic Variability on Water Levels

Based on the ability to reasonably reproduce 1976-1989 water levels, the use of the soil moisture budget model to generate input for the Englevale ground water model was considered acceptable to evaluate the long-term viability of the Englevale aquifer at the present level of irrigation development. Climate data were available from 1903 through 1989 for Lisbon, ND. There are some missing data during this long period. Data from Enderlin, Forman, and Verona were used to fill in the missing Lisbon climate data.

Figures I-30 to I-34 show simulated water levels with and without irrigation from 1904 to 1989. The top curve shows how water levels would have responded with no irrigation. Simulated water level fluctuations of 3 to 4 feet occur under natural conditions. This is consistent with observed water levels in aquifers not significantly affected by irrigation. The 1930s, 1950s, and 1980s are periods of low water levels, though the differences from the wet periods are not large. The lower curve shows how water levels would have responded assuming that all irrigation in areas covered by the Englevale model started in 1904. The model covers the northern part of the aquifer terminating approximately one mile south of the Ransom-Sargent County line. The two periods of lowest water levels were the 1930s and the 1980s. The simulated hydrographs do not indicate any long term downward trend. This analysis tends to indicate that the aquifer is not over-appropriated in terms of the climate during this century.

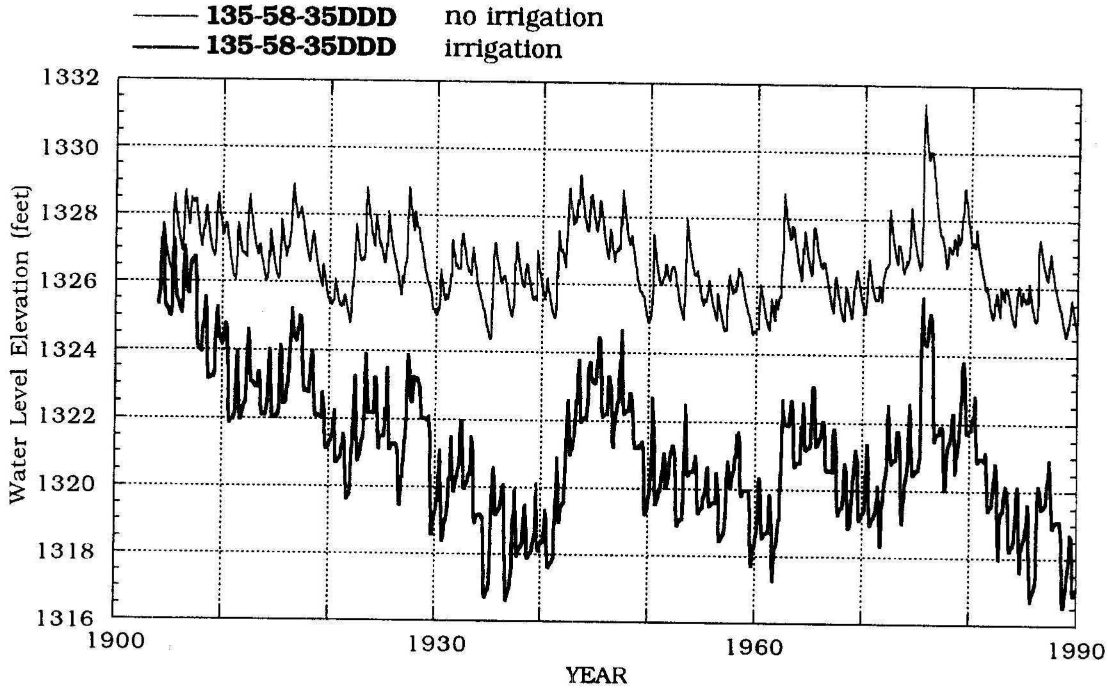

Fig. I-30. Comparison of simulated water levels for observation well 135-058-35DDD assuming no irrigation and all present irrigation development started in 1904.

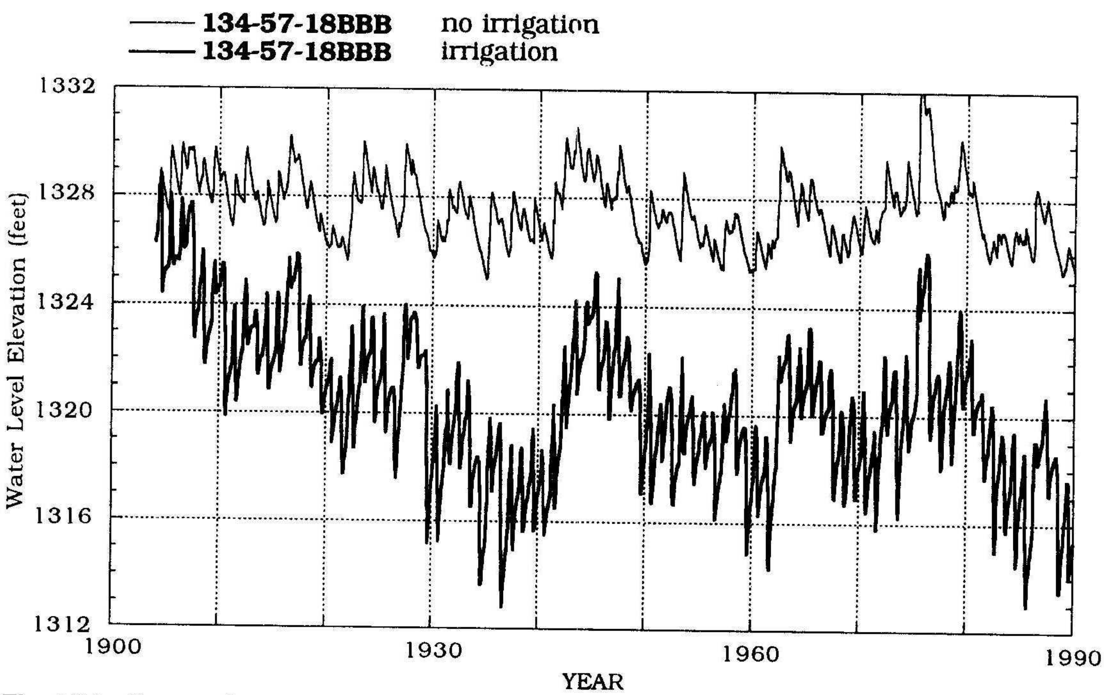

Fig. I-31. Comparison of simulated water levels for observation well 134-057-18BBB assuming no irrigation and all present irrigation development started in 1904.

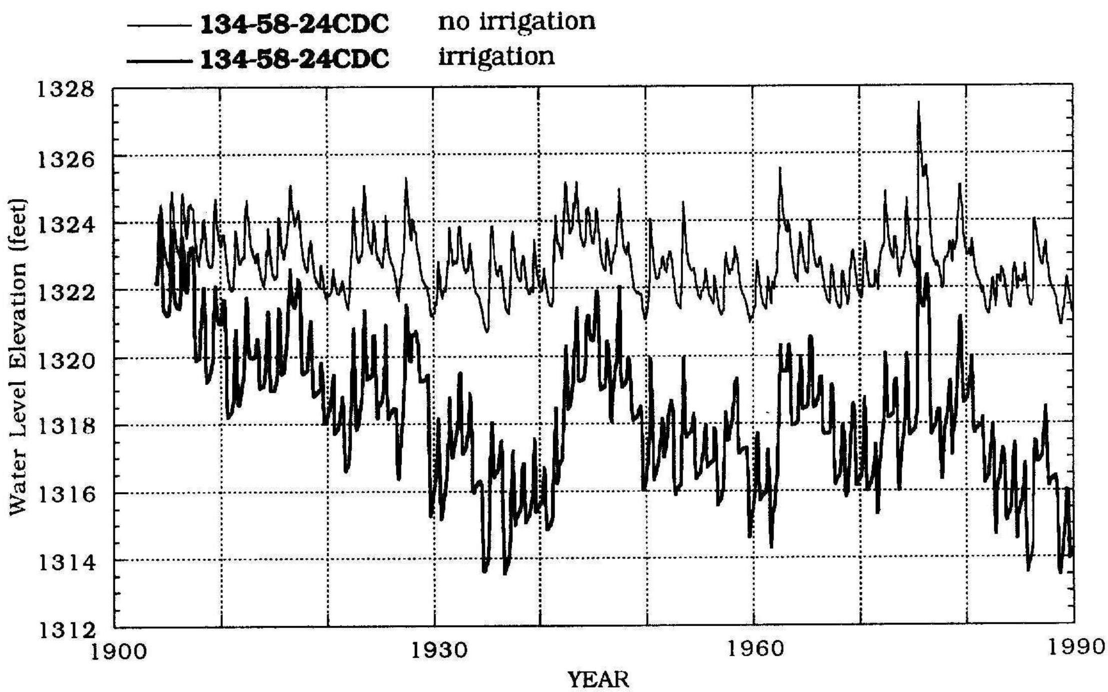

Fig. I-32. Comparison of simulated water levels for observation well 134-058-24CDC2 assuming no irrigation and all present irrigation development started in 1904.

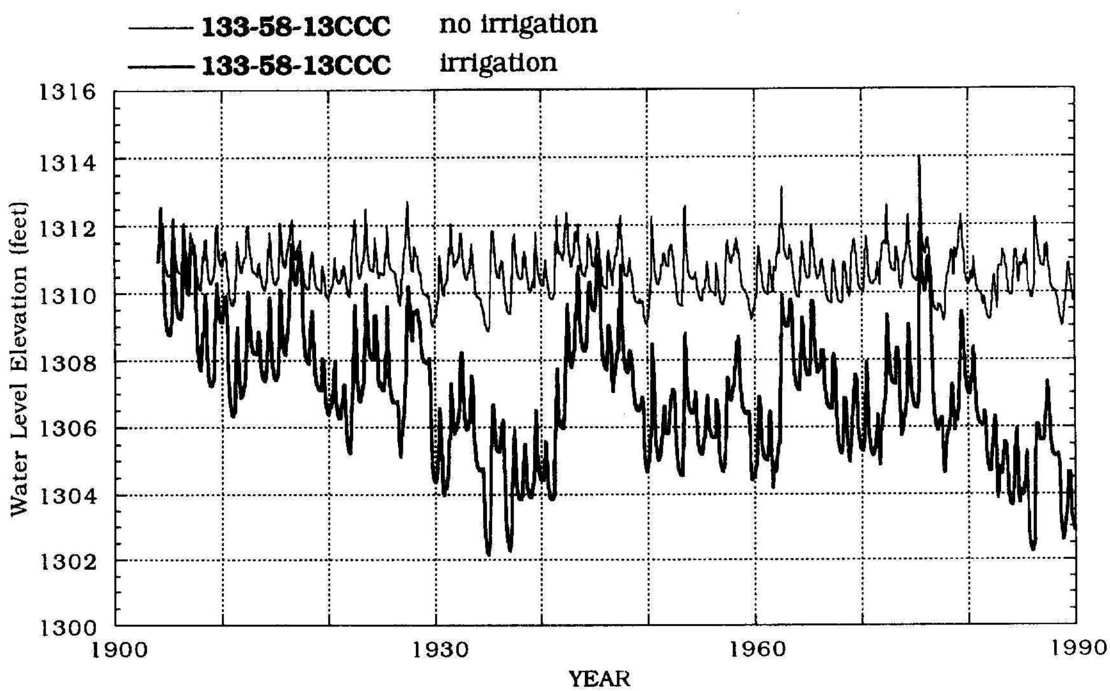

Fig. I-33. Comparison of simulated water levels for observation well 133-058-13CCC assuming no irrigation and all present irrigation development started in 1904.

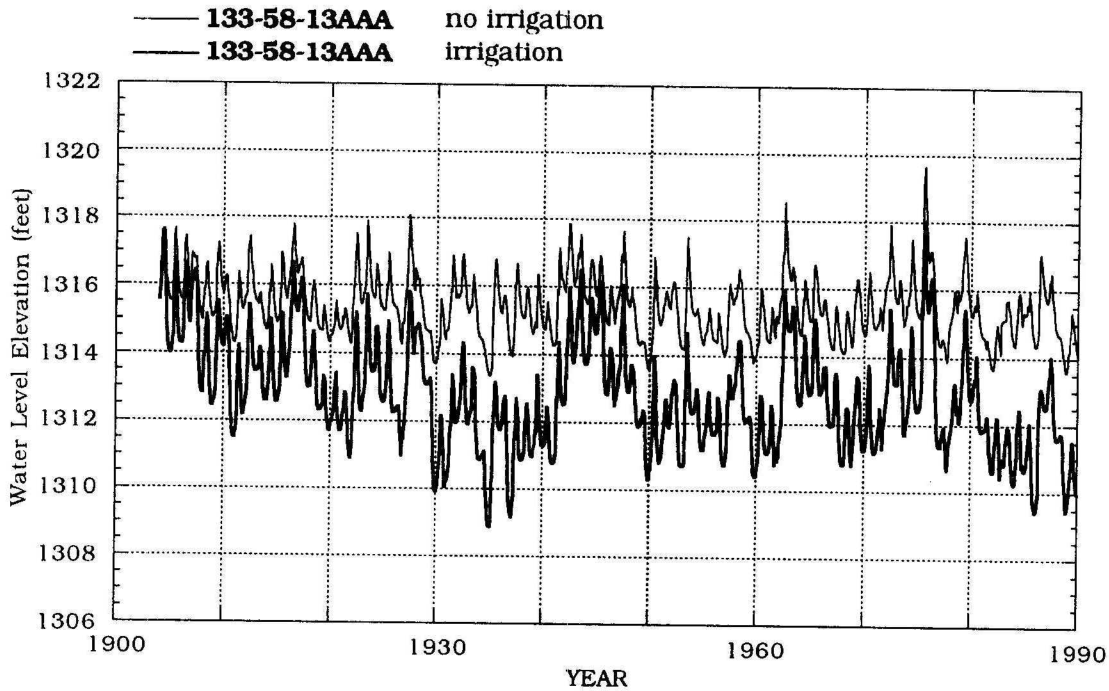

Fig. I-34. Comparison of simulated water levels for observation well 133-058-13AAA1 assuming no irrigation and all present irrigation development started in 1904.

A review of the climate data indicated that the Lisbon climate data for the 1930s were much wetter than other area stations. Simulations were made using McLeod climate data from 1913 to 1989. Precipitous water-level declines occurred during the 1930s in the simulations(Figs. I-35 and I-36). Many irrigation wells in the model went dry at this point in the simulation. Because the model has no provisions for re-saturating wells, these wells remained dry through the remainder of the simulation. A similar response would be expected when using Oakes climate data because it is very similar to the McLeod data. Further simulations need to be performed to examine realistic recovery rates from the simulated decline during the 1930s.

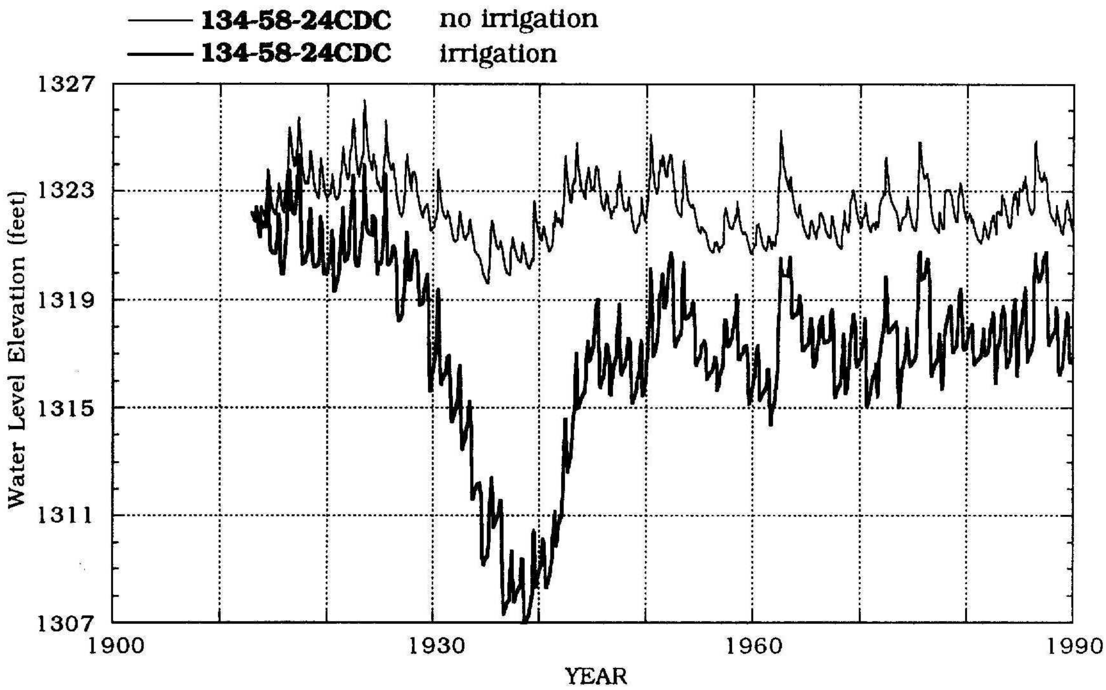

Fig. I-35. Comparison of simulated water levels for observation well 134-058-24CDC2 assuming no irrigation and that all irrigation development started in 1913. Englevale aquifer ground-water model input was generated from McLeod climate data using the soil moisture budget model. Irrigation wells went dry during drought of 1930s and lost from remainder of simulation.

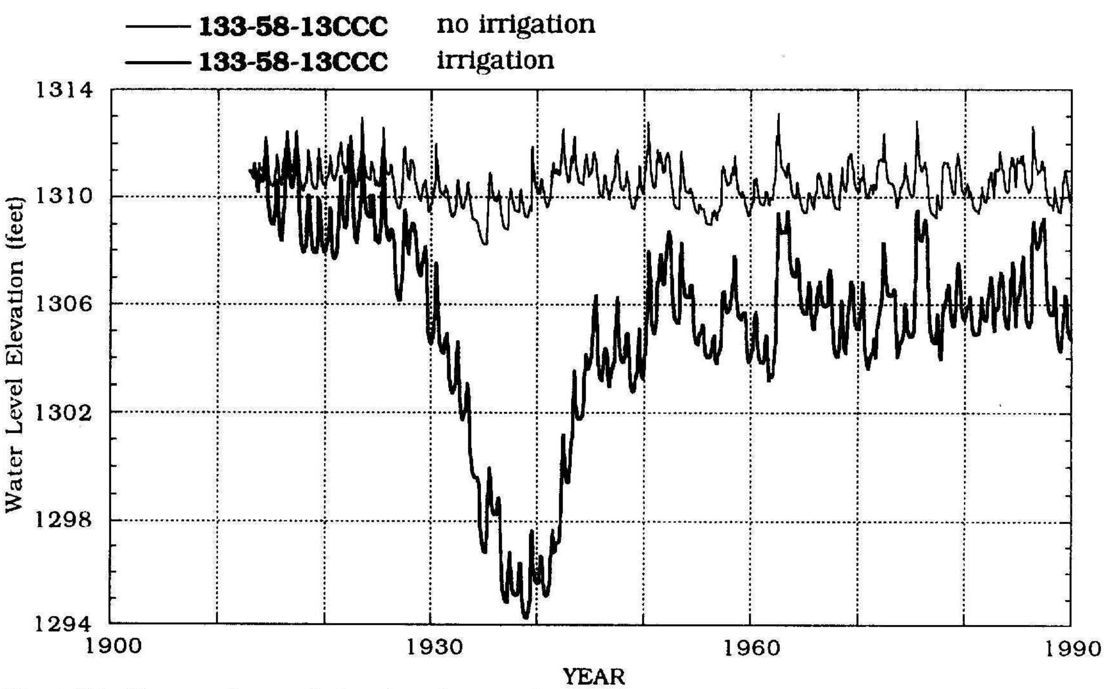

Fig. I-36. Comparison of simulated water levels for observation well 133-058-13CCC assuming no irrigation and that all irrigation development started in 1913. Englevale aquifer ground-water model input was generated from McLeod climate data using the soil moisture budget model. Irrigation wells went dry during drought of 1930s and lost from remainder of simulation.

Long-Term Trends in Aquifer Recharge and Discharge as Related to Climate

Figure I-37 to I-39 shows the simulated long-term trends in ground-water recharge for Lisbon, McLeod, and Oakes using the soil moisture budget model. The simulation indicates that recharge during the 1980s was as low as any time during this century based on Lisbon climate data. However, Fig. I-40 shows that both Oakes and McLeod would have had considerably less recharge than Lisbon during the 1930s. The figure also emphasizes the potential variability in recharge within a small region due to apparently random component of decade length climate variability.

Water use in inches per acre estimated from Lisbon climate data is shown in Fig. I-41. Estimated 5-year average water use ranges from 7 to 11 inches per acre with the period since 1976 using around 10 inches per acre. This is near the high end of the range. Figure I-42 shows a comparison of 5-year water use for Lisbon, McLeod, and Oakes. The 1930s and 1980s are periods of high water use. The drought of the 1950s did not have a severe impact on water levels because of the low water use through this period. The potential evaporation used to calculate annual water use is based solely on temperature.

The trends in water use (Fig. I-42) are very similar to the regional summer temperature trends (Fig. I-43). The estimated water use and regional temperature data show an increasing trend from the beginning of the century up to the present. The 1930s appear as a period of anomalously high temperatures. The greatest temperature change occurs in the spring with a significant warming trend beginning around 1970 (Fig. I-44). This could greatly affect available topsoil moisture at planting and the amount of ground-water recharge. While the springs were exceptionally warm during the 1980s, the falls have been cooler than normal (Fig. I-45). Annual temperature trends are shown in Fig. I-46.

The long-term temperature trend raises some concern that water use in the future will be higher than the simulated mean for 1904 to 1989. There is no discernible trend in annual precipitation for the region (Fig. I-47) though the falls have been wetter than normal during the 1980s (Fig. I-48). The Lisbon area generally is wetter than the southeast region average since it is at the east end of an area extending from Richland to Logan and Mcintosh counties. However, Lisbon was exceptionally dry during the 1980's compared to the rest of the region(Fig. I-49). There is significant variability in climate both spatially and temporally that have significant impacts on ground-water recharge and water use from the aquifer. In the short run, climate variability will probably have larger impacts than any long-term trend in the climate data.

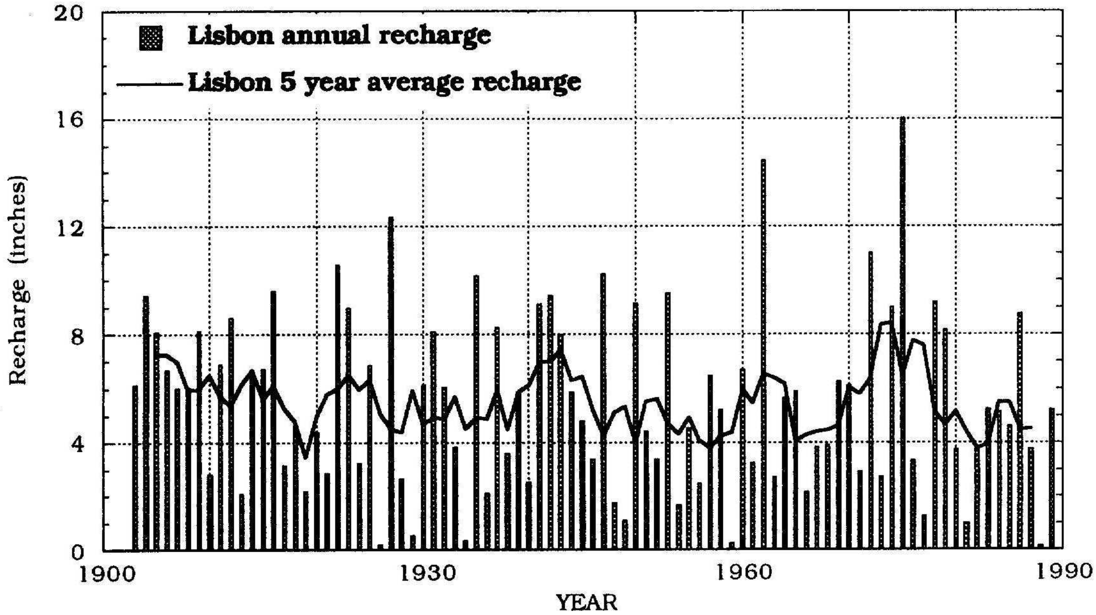

Fig. I-37. Annual recharge calculated from Lisbon composite climate data using the soil moisture budget model assuming a 4.2" soil water holding capacity in the root zone.

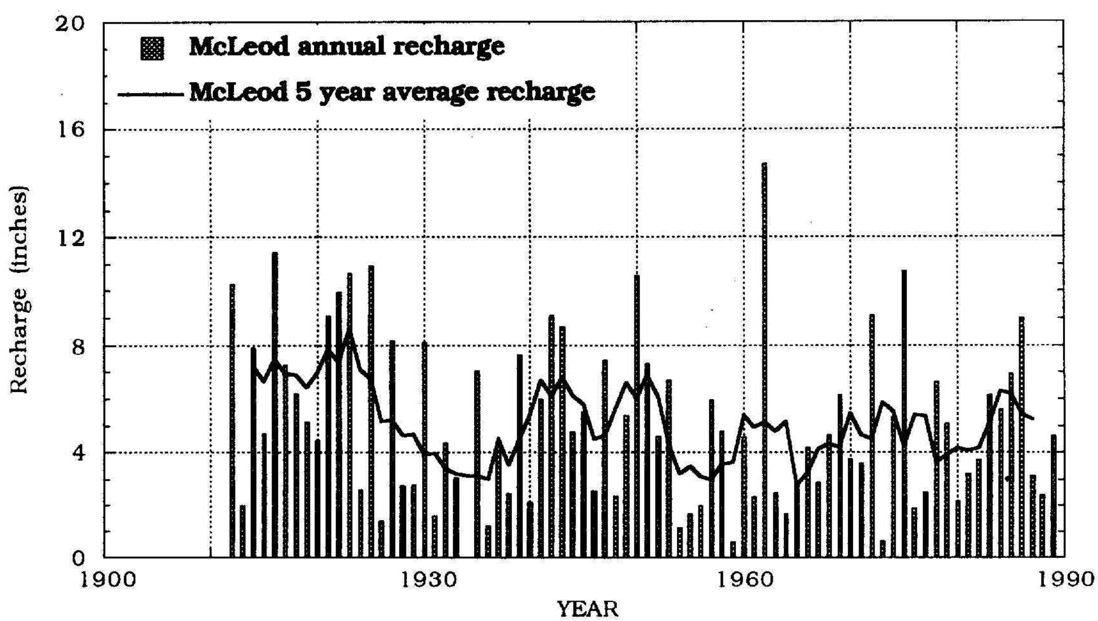

Fig. I-38. Annual recharge calculated from McLeod composite climate data using the soil moisture budget model assuming a 4.2" soil water holding capacity in the root zone.

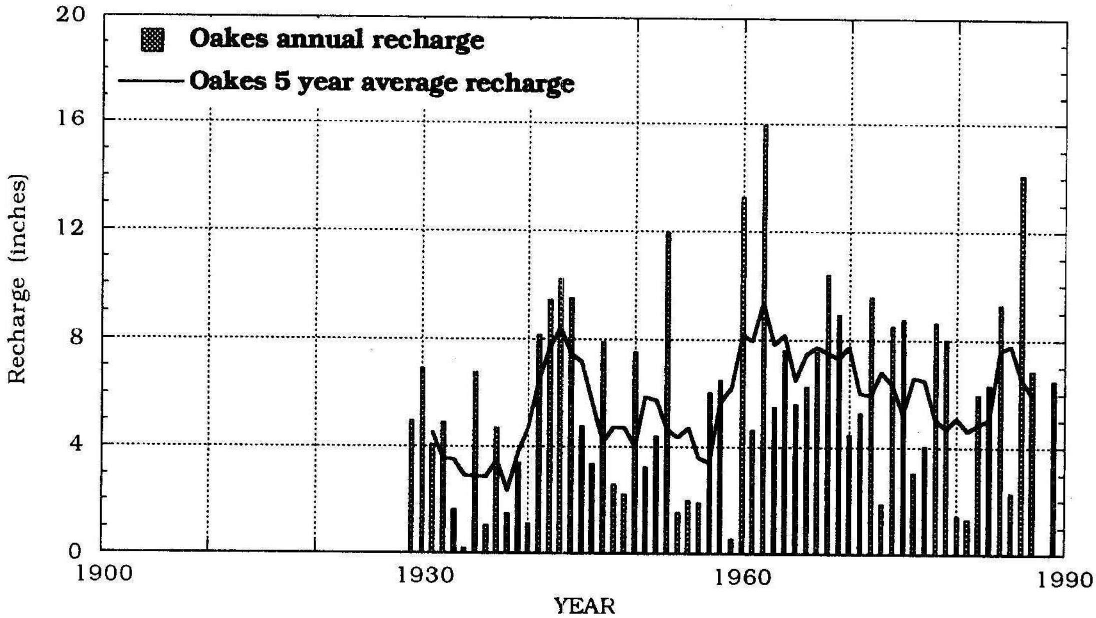

Fig. I-39. Annual recharge calculated from Oakes composite climate data using the soil moisture budget model assuming a 4.2" soil water holding capacity in the root zone.

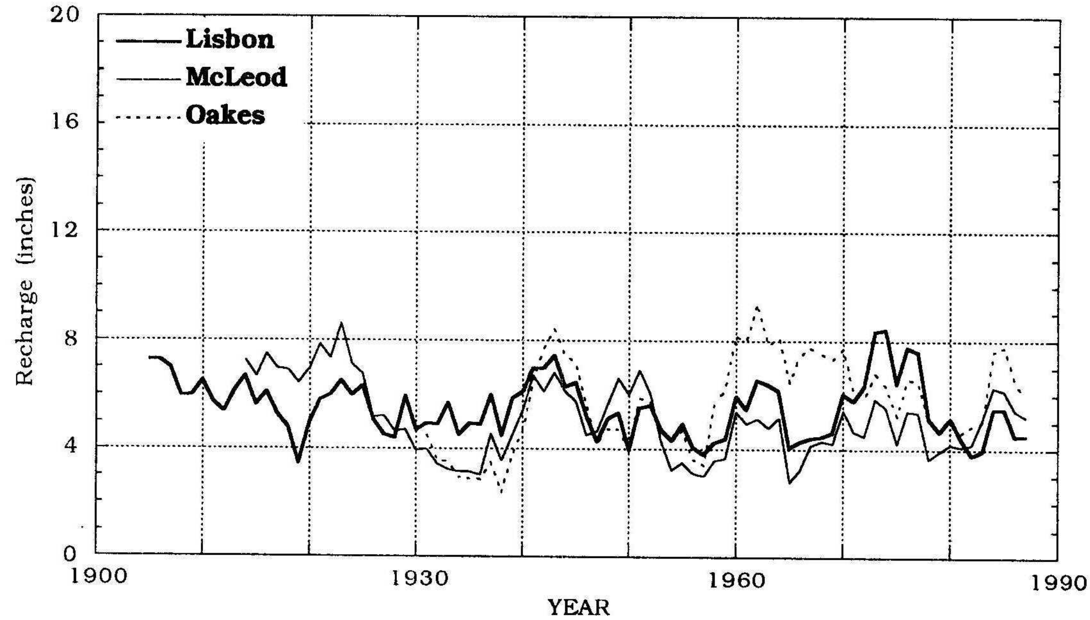

Fig. I-40. Comparison of 5-year moving averages of annual recharge calculated from composite climate data for Lisbon, McLeod, and Oakes using the soil moisture budget model assuming 4.2" soil water holding capacity.

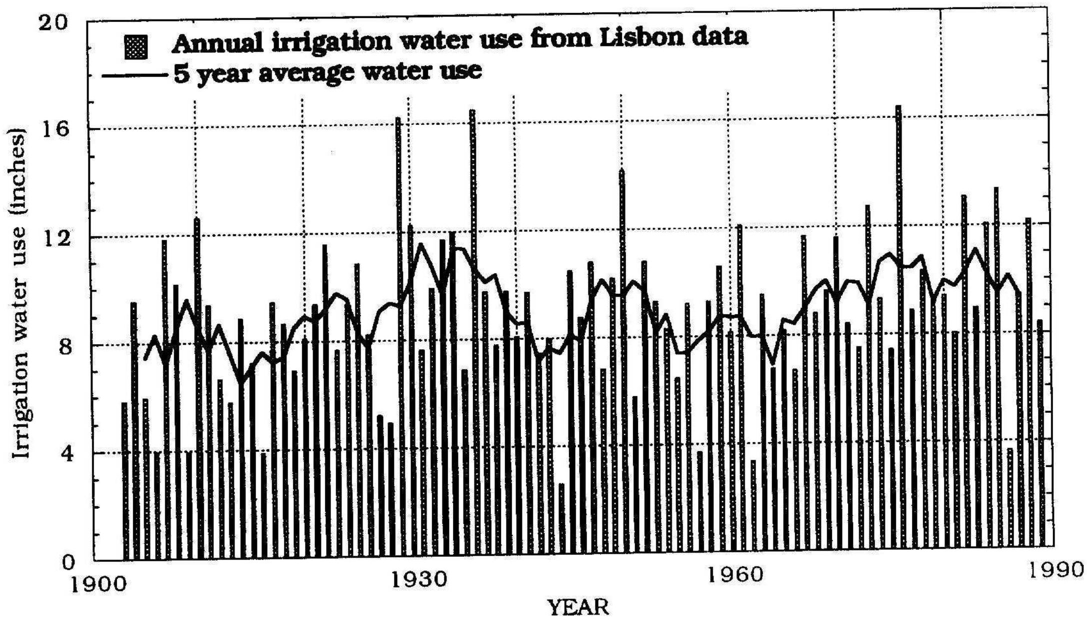

Fig. I-41. Irrigation water use calculated from Lisbon climate data using the soil moisture budget model assuming a 4.2" soil water holding capacity.

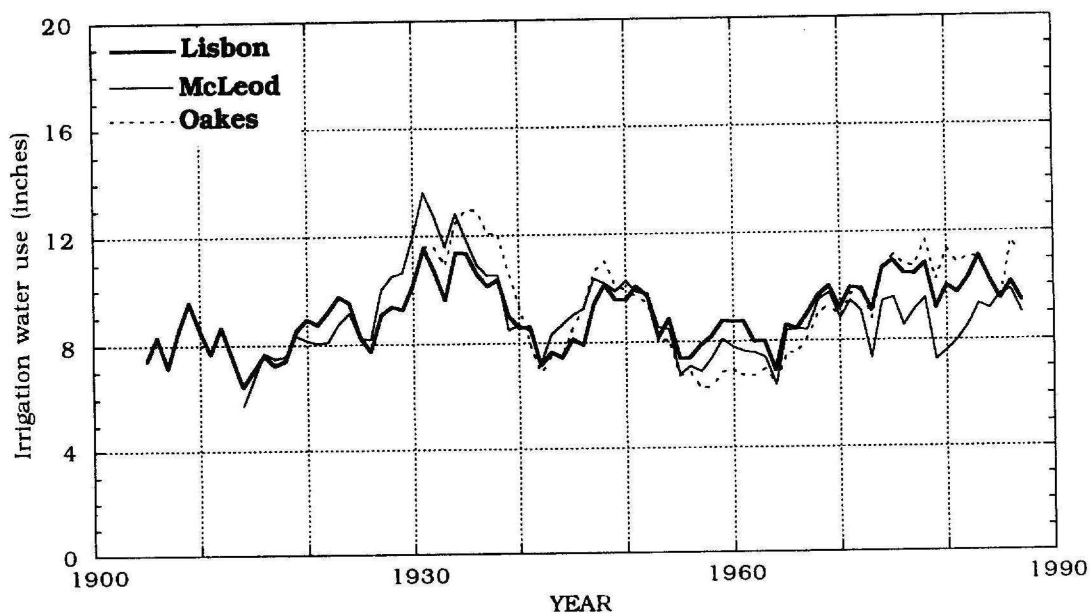

Fig. I-42. Comparison of 5-year moving average of irrigation water use calculated from composite climate data for Lisbon, McLeod, and Oakes using the soil moisture budget model assuming a 4.2" soil water holding capacity.

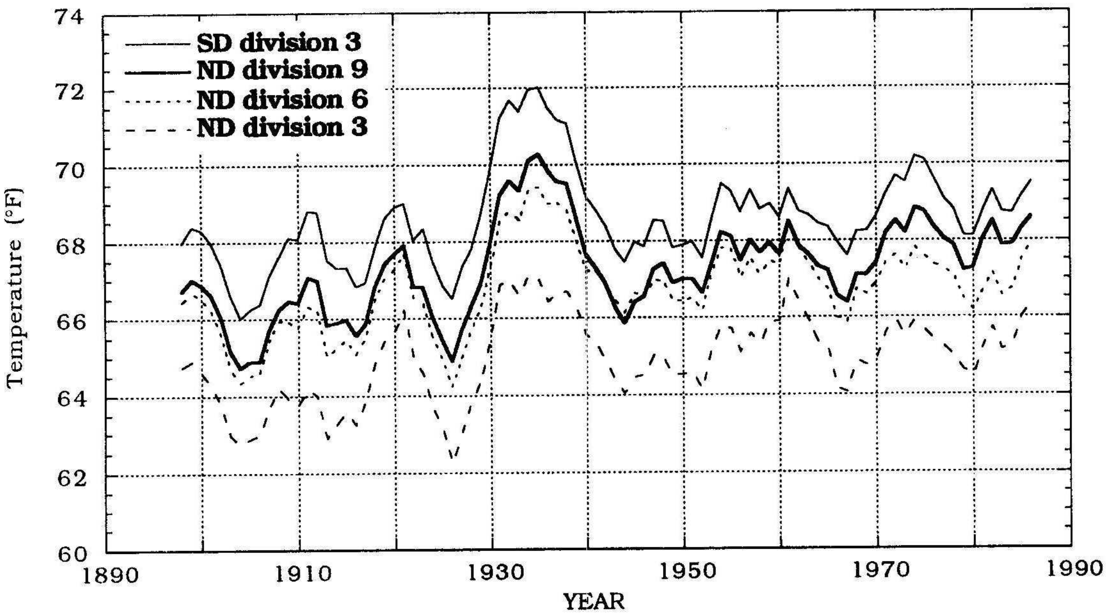

Fig. I-43. Comparison of 5-year moving average of June to August temperatures for NE South Dakota (division 3), SE North Dakota (division 9), east central North Dakota (division 6), and NE North Dakota (division 3).

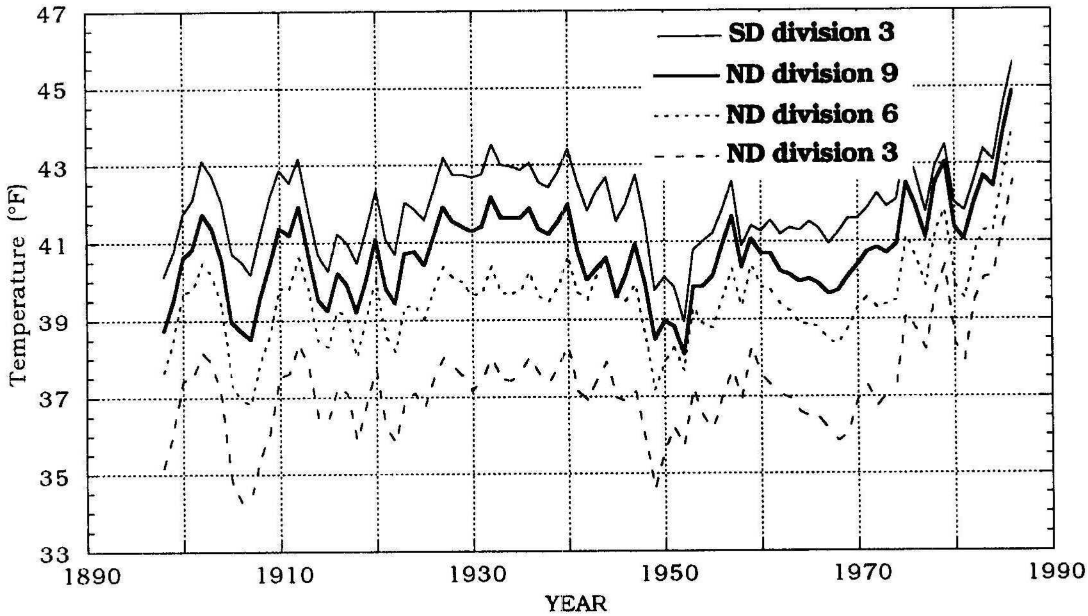

Fig. I-44. Comparison of 5-year moving average of March to May temperatures for NE South Dakota (division 3), SE North Dakota (division 9), east central North Dakota (division 6), and NE North Dakota (division 3).

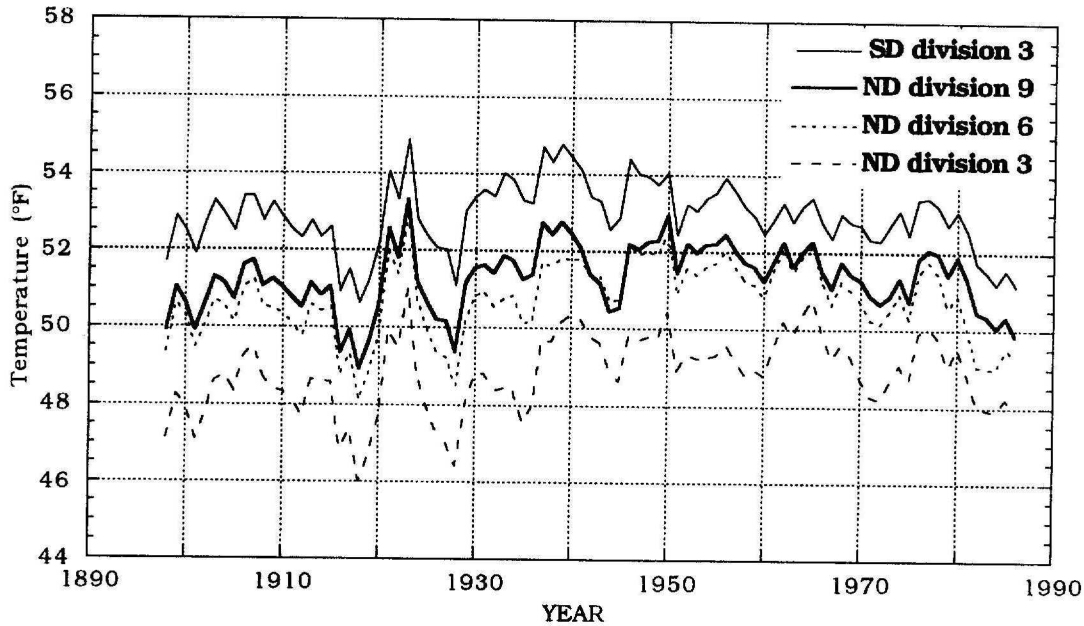

Fig. I-45. Comparison of 5 year moving average of September to October temperatures for NE South Dakota (division 3), SE North Dakota (division 9), east central North Dakota (division 6), and NE North Dakota (division 3).

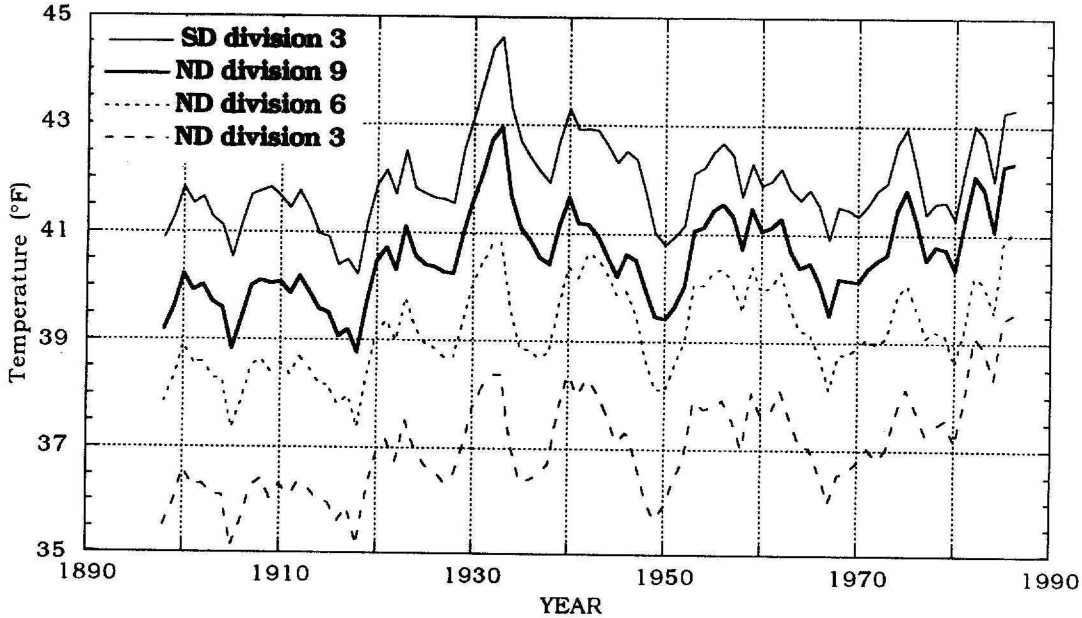

Fig. I-46. Comparison of 5-year moving average of annual water year temperatures for NE South Dakota (division 3), SE North Dakota (division 9), east central North Dakota (division 6), and NE North Dakota (division 3).

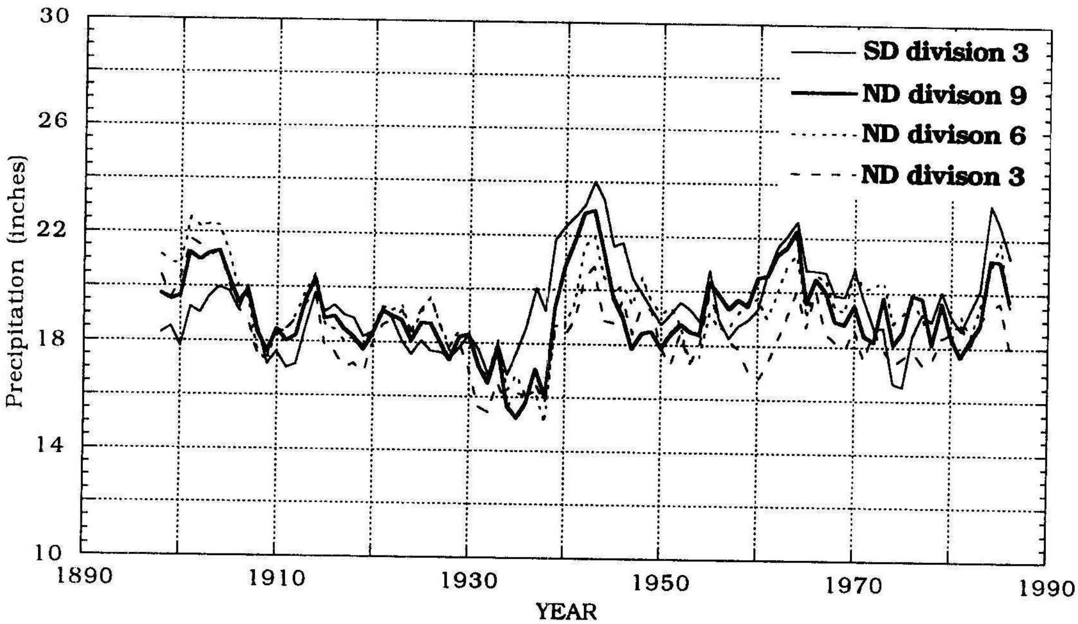

Fig. I-47. Comparison of 5-year moving average of annual water year precipitation for NE South Dakota (division 3), SE North Dakota (division 9), east central North Dakota (division 6), and NE North Dakota (division 3).

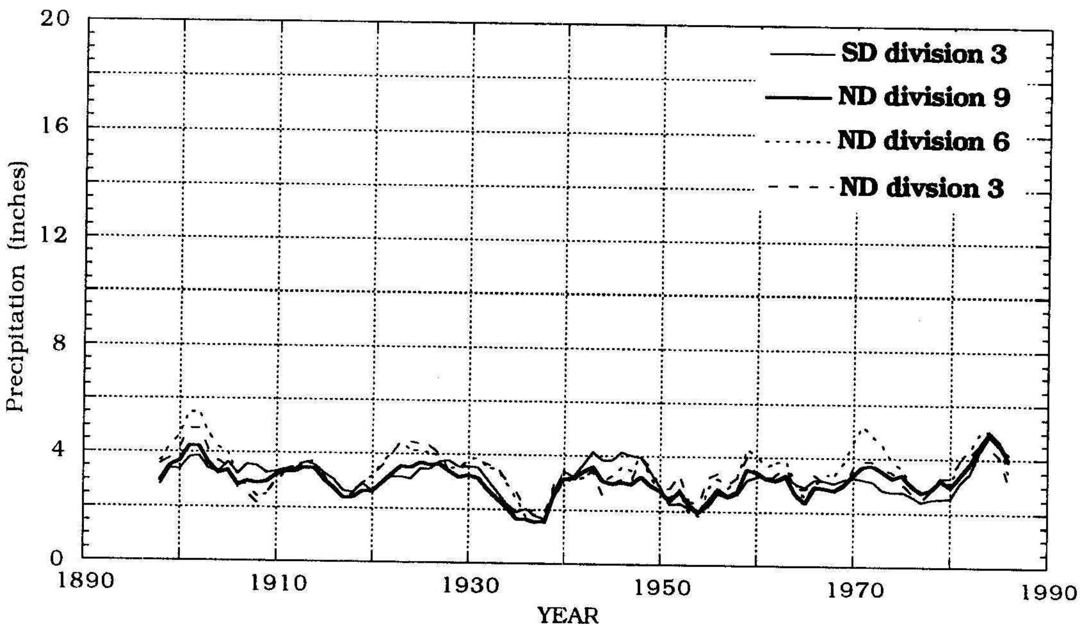

Fig. I-48. Comparison of 5-year moving average of September to October precipitation for NE South Dakota (division 3), SE North Dakota (division 9), east central North Dakota (division 6), and NE North Dakota (division 3).

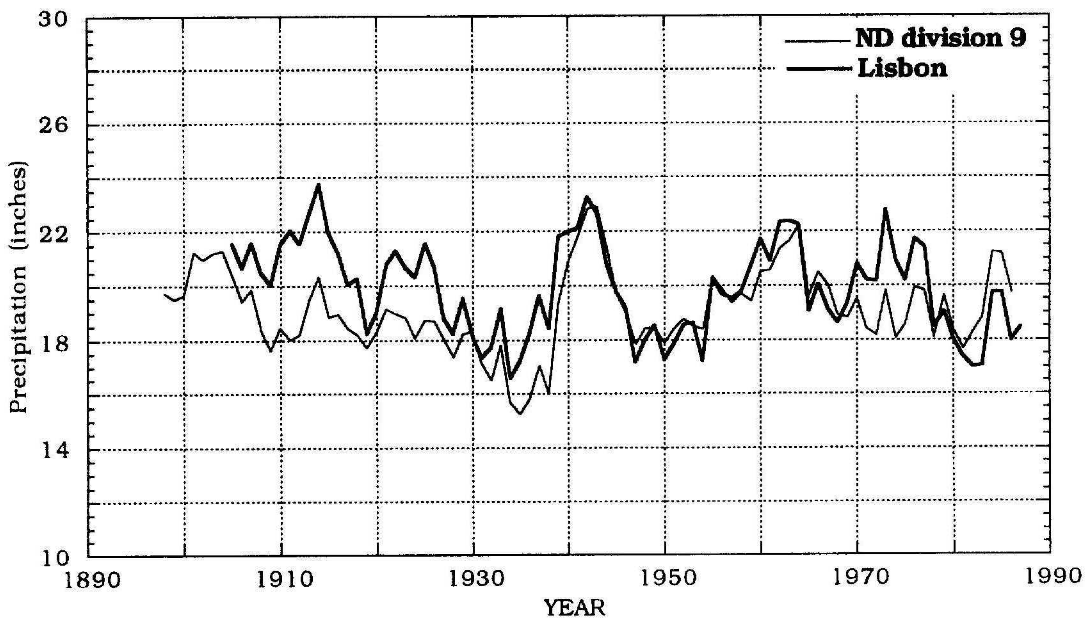

Fig. I-49. Comparison of 5-year moving average of annual water year precipitation at Lisbon to SE North Dakota (division 9) average.

Table I-1 provides a summary comparing recharge, water use, and potential ground-water evapotranspiration estimated from Lisbon, Oakes and McLeod data for different periods. There is considerable temporal and spatial variability in the estimates. The 1904 to 1989 estimated values for Lisbon result in sustainable long-term irrigation in the simulations. The 1982 to 1989 Lisbon estimates result in slow long-term mining of the aquifer. The study indicates that the aquifer is not over-appropriated in the long term based on either Lisbon or Oakes data, but is over-appropriated based on McLeod data. However, the McLeod station has the lowest 1951 to 1980 reported normal precipitation in its vicinity and therefore is probably not representative of the area. Also, the data indicates that the aquifer is over-appropriated based on Lisbon climate data for the period starting in 1976 when much of the irrigation development occurred. The projected over-appropriation is not large, ranging from a 0.3 to 1 foot per decade decline in water levels. If present climatic patterns persist, in the short run these small rates of water-level decline will be masked by the much larger annual fluctuations due to climate variability.

TABLE I-1. **Comparison of recharge, water use and evapotranspiration from ground water estimated with the soil moisture budget model.**

YEARS	STATION	Average Recharge (inches)	Average Irrigation Water Use (inches)	Average ET_{gw} (inches)
1904-1989	Lisbon	5.21	9.02	25.82
1913-1989	McLeod	5.05	8.89	25.90
1930-1939	Lisbon	5.45	10.40	27.11
	McLeod	3.98	11.63	30.72
	Oakes	3.52	11.64	30.48
1930-1989	Lisbon	5.39	9.32	26.28
	McLeod	4.70	9.01	26.15
	Oakes	5.70	9.55	26.29
1948-1989	Lisbon	5.13	9.28	26.42
	McLeod	4.66	8.61	25.35
	Oakes	6.09	9.21	25.61
1960-1974	Lisbon	5.77	8.85	25.94
	McLeod	4.58	8.35	24.14
	Oakes	7.69	8.25	23.28
1976-1989	Lisbon	4.51	10.24	27.50
	McLeod	4.51	9.18	26.20
	Oakes	5.56	11.30	29.10
1982-1989	Lisbon	4.57	10.14	26.34
	McLeod	5.21	9.51	25.71
	Oakes	6.42	11.23	27.97

Sensitivity of Water Levels to Recharge and Water Use

Though the analysis indicates that the Englevale aquifer is not over-appropriated in the long term based upon the 1904 to 1989 Lisbon climate record, the model does show that wells in areas of small saturated thickness will experience problems during periods of long-term drought. The climate of North Dakota shows a high degree of variability on a decade length time scale. It is not possible to predict what the climate of the next few decades will be. However, the past record indicates that it is likely that future decades will be wetter than the 1980s. To allow for the large uncertainties within the modeling procedure and uncertainty of future climate, additional model runs were made varying water use and ground-water recharge. Figures I-50 and I-51 show the effects of 10 percent and 20 percent reductions in recharge from that estimated by the soil moisture budget model. The 20 percent reduction in recharge results in a four-foot decline in long-term water levels, but the model indicates the aquifer will still support the present level of development. However, for this simulation some wells had to be relocated to nearby thicker sections of the aquifer.

The amounts of water pumped for irrigation were increased by 10 percent and 20 percent in two other simulations. Figure I-52 shows a comparison of simulated water levels at 100 percent and 120 percent of estimated water use. Results are similar to the reductions in recharge. Again, a new equilibrium is established in the aquifer indicating the present level of development is sustainable.

Further model studies were undertaken to evaluate the aquifer behavior given the climate of the 1980s and how improved irrigation efficiency might affect long-term water levels. The first simulation was made by repeating the years 1980 to 1989 in the model three times. The model indicates that the aquifer is not sustainable with the climate that existed from 1980 to 1989. With three drought years of 1980, 1981, and 1988 and only 1986 with appreciably above average recharge, water levels continued to decline across the simulated period resulting in many wells going dry. A less extreme climatic scenario was tried next by repeating the years 1982 to 1989 and extending the simulation to 2013. The results of the simulations are shown by the bottom line in Fig. I-53, I-54, and I-55. Simulated water levels continued to decline to the end of the simulation in 2014 until they were about two feet lower than in 1989. Average recharge for the period 1982 to 2014 is 4.6 inches. This is comparable to the amount of recharge occurring during drought periods for the 1904 to 1989 simulation (Table I-1). The simulation indicates that the northern and southern parts of the aquifer show similar response to equivalent climatic stress (compare 100% case Figs. I-54 and I-55). This suggests that the difference in water-level response to irrigation between the area north of Englevale and that near the Ransom-Sargent county line is due to climatic differences and not to differences in irrigation pumping or ability to salvage water from evapotranspiration.

The effect of increasing irrigation efficiency was also explored in the above simulation from 1976 to 2013. Simulations were made assuming a reduction of 10 and 20 percent in water pumped for irrigation. The results are shown by the middle and top line respectively in Fig. I-53, I-54, and I-55. The simulation indicates that in the case of a slight over-appropriation, a 10 percent reduction in pumping can have a significant impact on water levels with the reduction resulting in water levels over two feet higher. A similar simulation was run using the 1904 to 1989 data with 80 percent, 90 percent, and 100 percent of estimated water use. It can be seen in Fig. I-56 that the greatest effect of reduced pumping occurs during the extended dry periods when water levels are lowest. When water levels are high as during the 1940s, a large fraction of the conserved water is lost to evapotranspiration.

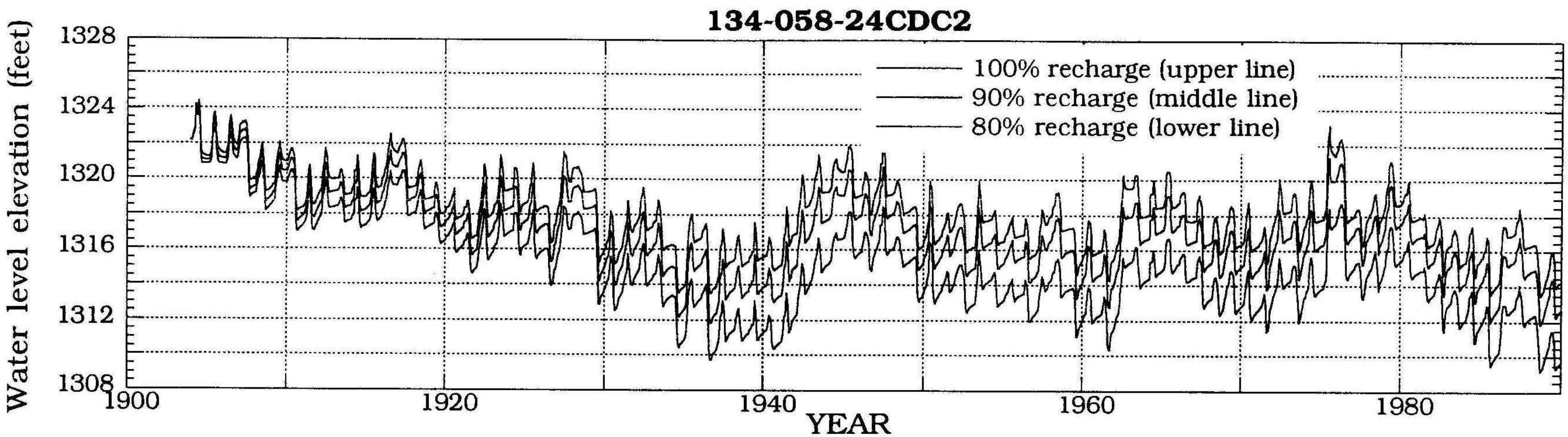

Fig. I-50. Comparison of the effect of reduced recharge rates on simulations using 100%, 90%, and 80% of recharge generated by soil moisture budget model as input to Englevale groundwater model at observation well 134-058-24CDC2. Simulations assume all present irrigation started in 1904.

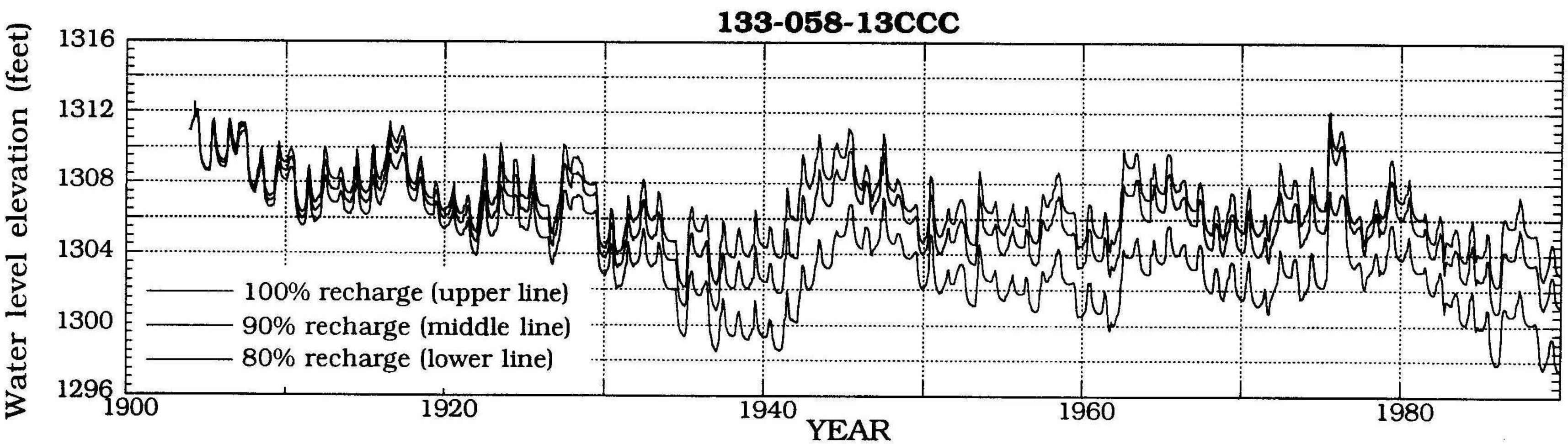

Fig. I-51. Comparison of the effect of reduced recharge rates on simulations using 100%, 90%, and 80% of recharge generated by soil moisture budget model as input to Englevale groundwater model at observation well 133-058-13CCC. Simulations assume all present irrigation started in 1904.

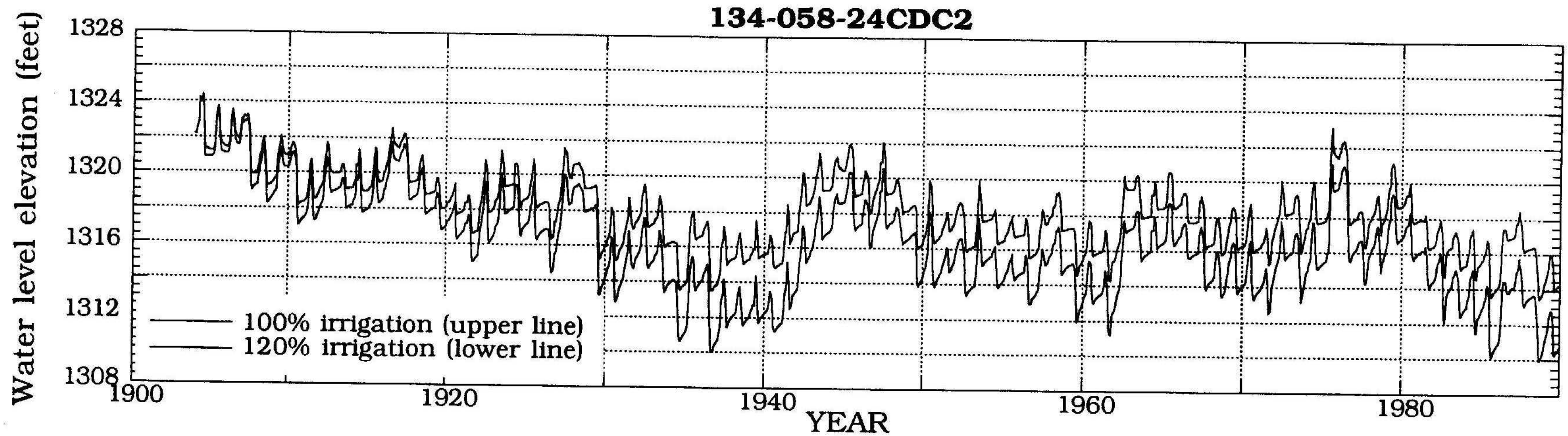

Fig. I-52. Comparison of the effect of increased irrigation water use on simulated water levels using 100% and 120% of irrigation water use calculated by soil moisture budget model as input to the Englevale groundwater model at observation well 134-058-24CDC2. Simulation assumes all irrigation started in 1904.

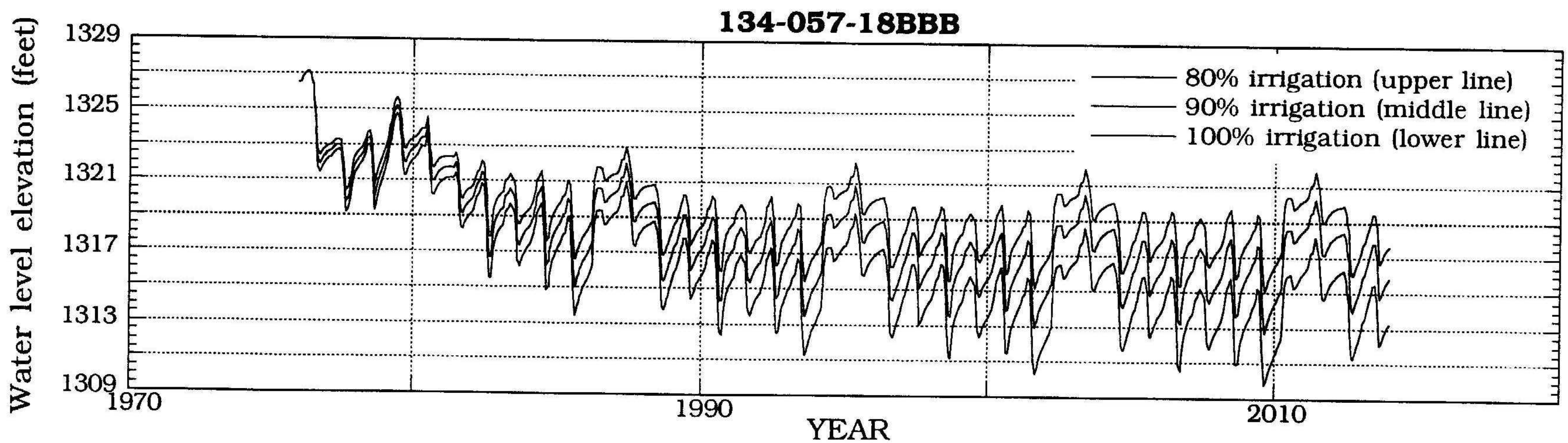

Fig. I-53. Comparison of the effect of decreased irrigation water use on simulated water levels using 80%, 90% and 100% of irrigation water use calculated by soil moisture budget model as input to the Englevale groundwater model at observation well 134-057-18BBB. Model repeats 1982 to 1989 period three times to extend 1976 to 1989 simulation.

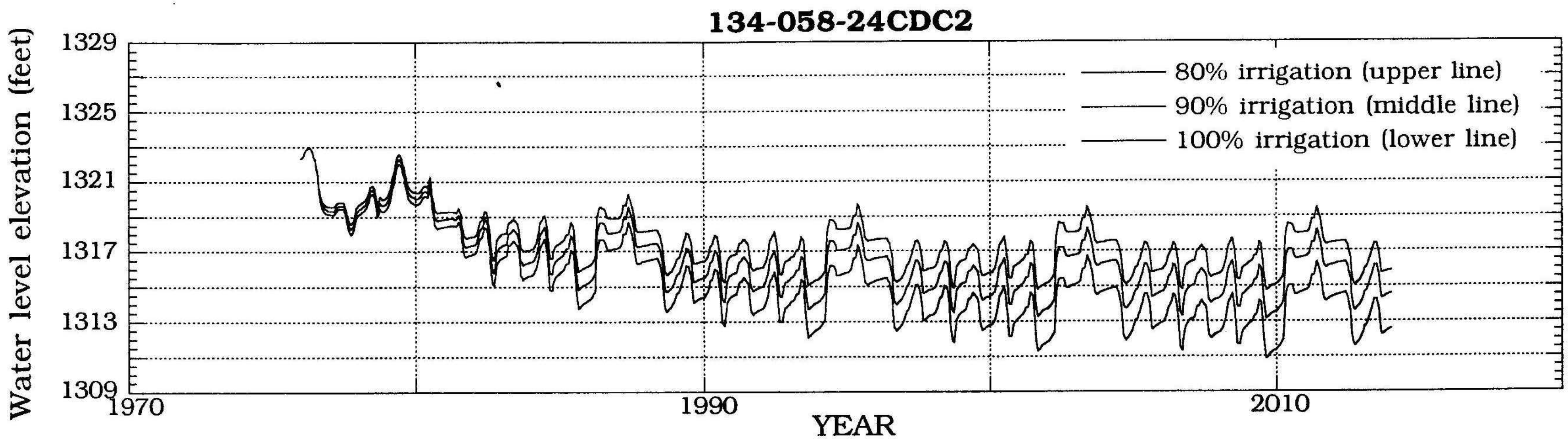

Fig. I-54. Comparison of the effect of decreased irrigation water use on simulated water levels using 80%, 90% and 100% of irrigation water use calculated by soil moisture budget model as input to the Englevale groundwater model at observation well 134-058-24CDC2. Model repeats 1982 to 1989 period three times to extend 1976 to 1989 simulation.

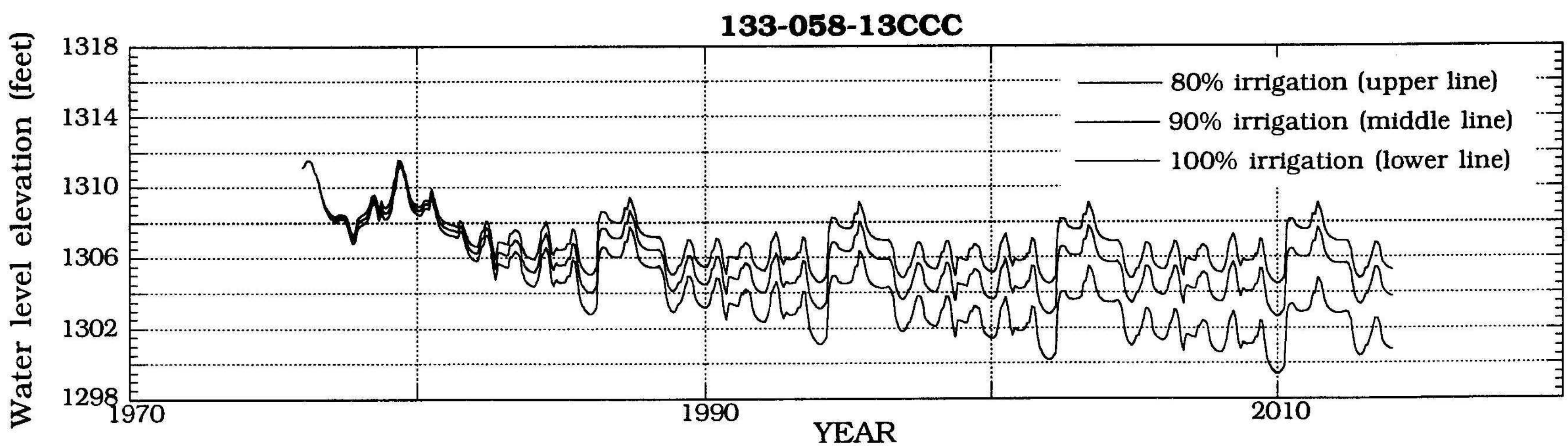

Fig. I-55. Comparison of the effect of decreased irrigation water use on simulated water levels using 80%, 90% and 100% of irrigation water use calculated by soil moisture budget model as input to the Englevale groundwater model at observation well 133-058-13CCC. Model repeats 1982 to 1989 period three times to extend 1976 to 1989 simulation.

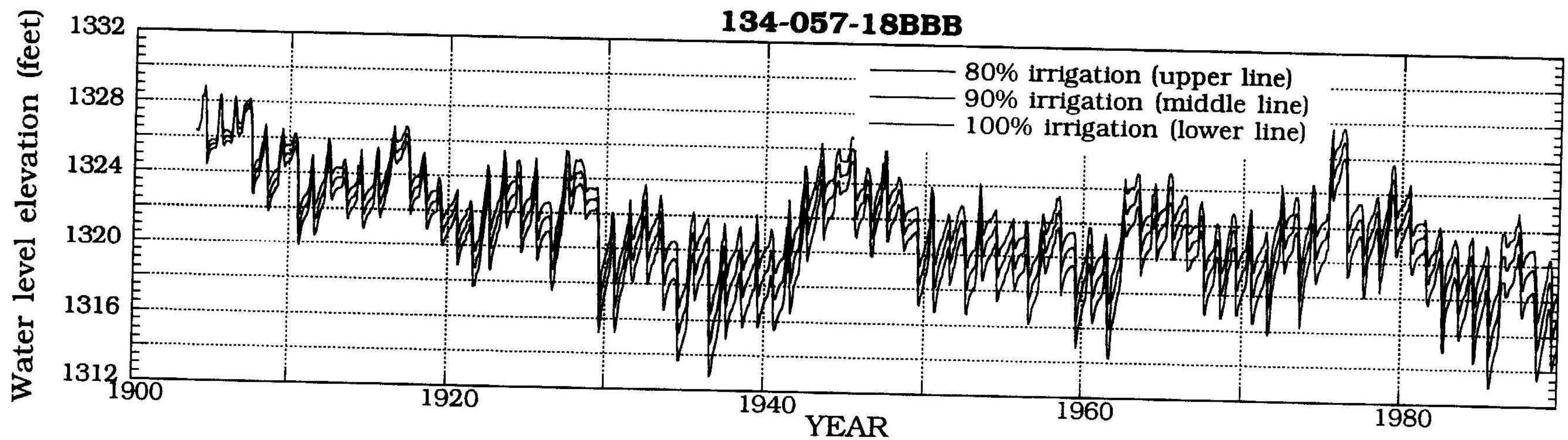

Fig. I-56. Comparison of the effect of decreased irrigation water use on simulated water levels using 80%, 90% and 100% of irrigation water use calculated by soil moisture budget model as input to the Englevale groundwater model at observation well 134-057-18BBB. Simulation assumes all irrigation started in 1904.

Figure I-57 shows the 5-year moving average for recharge and discharge from the model. Prior to irrigation development, evapotranspiration (ET) should equal recharge. The model indicates that the present level of irrigation development, given this dry simulation, results in ET being reduced by 75 percent. This leaves very little water to be salvaged from ET by further water-level declines. This factor along with the slow water level decline observed in Fig. I-53, I-54, and I-55 indicate that the present level of development is very close to the edge of sustainability given this climatic scenario. Any reduction in ground-water recharge or increase in water use would probably result in long-term mining of ground water.

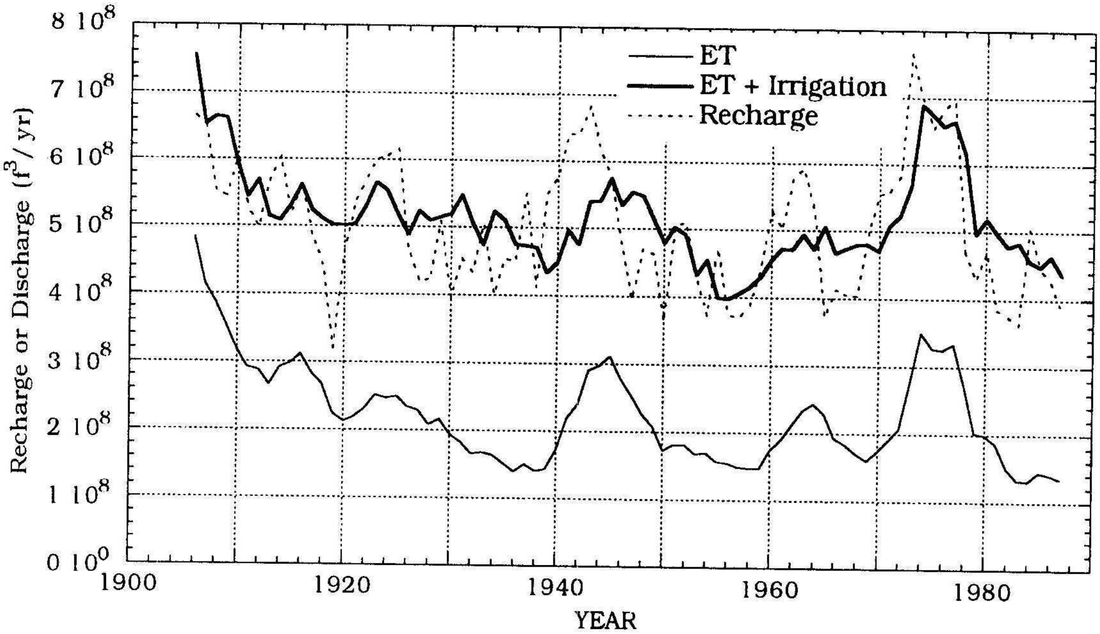

Fig. I-57. Five-year moving average of annual recharge, evapotranspiration (ET), and total discharge (ET + Irrigation use) for Englevale aquifer ground-water model. Model input generated from Lisbon climate data using soil moisture budget model. Assumed all irrigation started in 1904.

Summary of Simulation Results

In summary, the analysis indicates that the Englevale aquifer is not over-appropriated in the long term based on Lisbon climatic records from 1904 to 1989. But, there are many simplifying assumptions in the model and considerable variation in climate within the area of the Englevale aquifer. This means there is considerable uncertainty regarding this conclusion. However, it appears that the aquifer is not significantly over-appropriated with the rate of water-level decline not exceeding 0.1 feet per year over the long term. The dominate influence on water levels over the next twenty years will come from decade-scale climatic variability and not long-term over-appropriation. Periods of extreme drought as exhibited by the Oakes and McLeod climate data for 1930s, and extremely wet periods such as the 1940s will totally overwhelm any long-term trend in water levels in the short run. During periods of long-term drought such as the 1930s and 1980s, wells in areas of limited available drawdown will experience significant declines in well yields. The analysis also shows that if these problem wells can be relocated to areas of greater saturated thickness, the present level of development is probably sustainable most of the time. Improvements in irrigation efficiency resulting in lower losses to evaporation can either reduce the need to move wells, increase the reliability of the system, or provide water for additional irrigation.

Issues of whether the aquifer is over-appropriated based on some long-term climatic average are probably not relevant because of the large impacts of decade length climate variability on Englevale aquifer water levels. The focus of aquifer management must be in terms of the reliability of the supply of irrigation water in terms of decade length patterns of climate change. Because of the limited amount of water available from storage and the large variability in climate, stressing the system to near its long-term average yield capabilities will cause serious supply problems during periods of drought such (as the 1930s as represented by the McLeod data set). The issue of risk in availability of water supply versus number of acres irrigated has not been adequately explored in managing systems such as the Englevale aquifer. This is not to say that management for sustainable long-term aquifer yield is not important. But, it will be very hard to define what the long-term sustainable yield of an aquifer is because of the noise in water level trends resulting from large annual variation in climate, the long period it can take an aquifer to achieve a new equilibrium, and long-term fluctuations in climate.

ARTIFICIAL RECHARGE

An objective of this study was to address the issue of using artificial recharge to solve the problem of declining water-levels that have occurred in the Englevale aquifer. The previous section indicated that the Englevale aquifer is most likely not significantly over-appropriated in the long term. However, the aquifer is sensitive to decade-scale climate variability which can result in a significant reduction in irrigation during periods of prolonged drought. It therefore appears that the benefits of an artificial recharge project would be to smooth out the effects of climate variability. That is, water must be stored in the system during wet periods for use during dry periods. An analysis was made of the ability to store additional ground water in the aquifer in the area north of Englevale. The area north of Englevale along with the southern part of the eastern channel are most vulnerable to a reduction in well yields with a decline in water levels.

The Englevale ground-water model was used to evaluate the ability to store additional water in the Englevale aquifer. The simulations were performed using average annual recharge, water use, and potential evapotranspiration from ground water estimated from 1904 to 1989 Lisbon climate data using the soil-moisture budget model. Three simulations were performed using no artificial recharge, 1,000 acre-feet per year and 2,000 acre-feet per year. The simulations were run for a period of 20 years with artificial recharge starting in the 11th year of the simulation. All simulations assumed that the present level of irrigation development started in the first year of the simulation. The first ten years of the simulations were to allow development declines to occur so that the system was approaching a new equilibrium when artificial recharge was started. The artificial recharge was along the eastern side of Section 12, Township 134 North, Range 58 West (Fig. I-19). Figures I-58 and I-59 show the effect of artificial recharge on water levels at observation wells near the recharge facility. Simulated water levels next to the recharge facility (Fig. I-58) show a rapid increase during the first four years of operation, but show little annual increase by the 10th year. Figure I-60 shows the cumulative change in aquifer storage resulting from an artificial recharge rate of 2,000 ac-ft/yr. During the first three years, almost all of the water added to the aquifer is retained within the aquifer. This corresponds to the rapid water-level rise seen in Fig. I-58 during this period. After that time, progressively more of the water added to the aquifer is lost to evapotranspiration and runoff in the low area along the west side of Sections 12, 13, 24 and 25, Township 134 North, Range 58 West. This area would likely become untillable if an artificial recharge facility was installed due to its becoming even wetter than it was prior to irrigation development. Water is also discharged to other low areas to the south and west as the ground-water mound spreads. This is shown by the slow increase in simulated water levels at observation well 134-058-36CCC (Fig. I-61). Table I-2 summarizes the declining effectiveness of trying to store artificial recharge water in the aquifer north of Englevale.

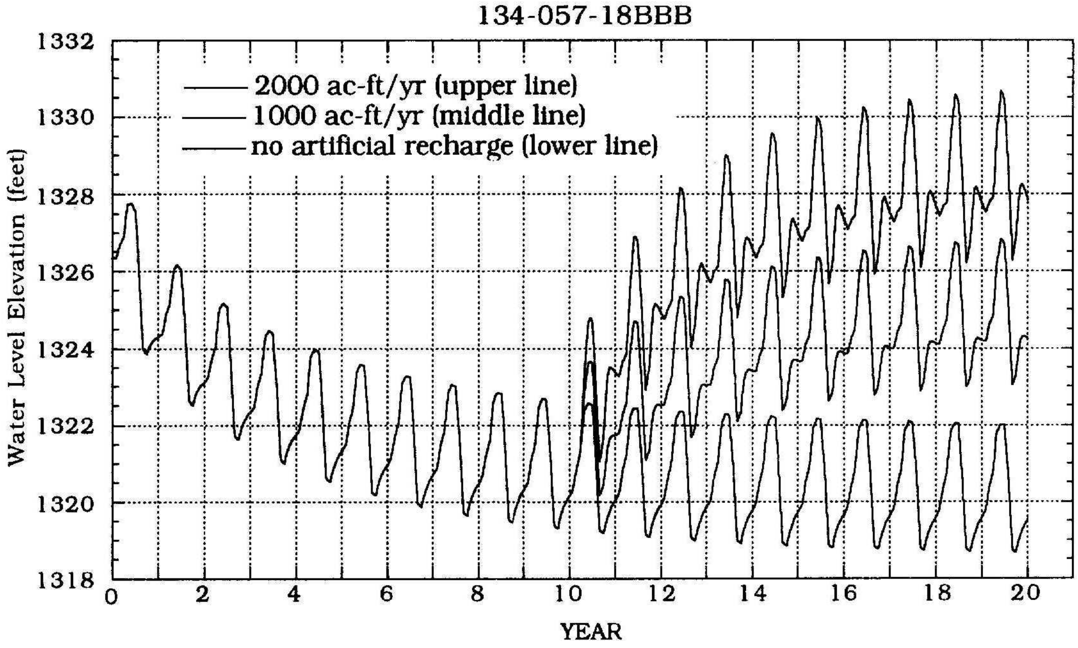

Fig. I-58. Comparison of simulated water levels at observation well 134-057-18BBB with no artificial recharge, and artificial recharge of 1,000 ac-ft/yr and 2,000 ac-ft/yr from basins located east side section 12, T134N, R58W.

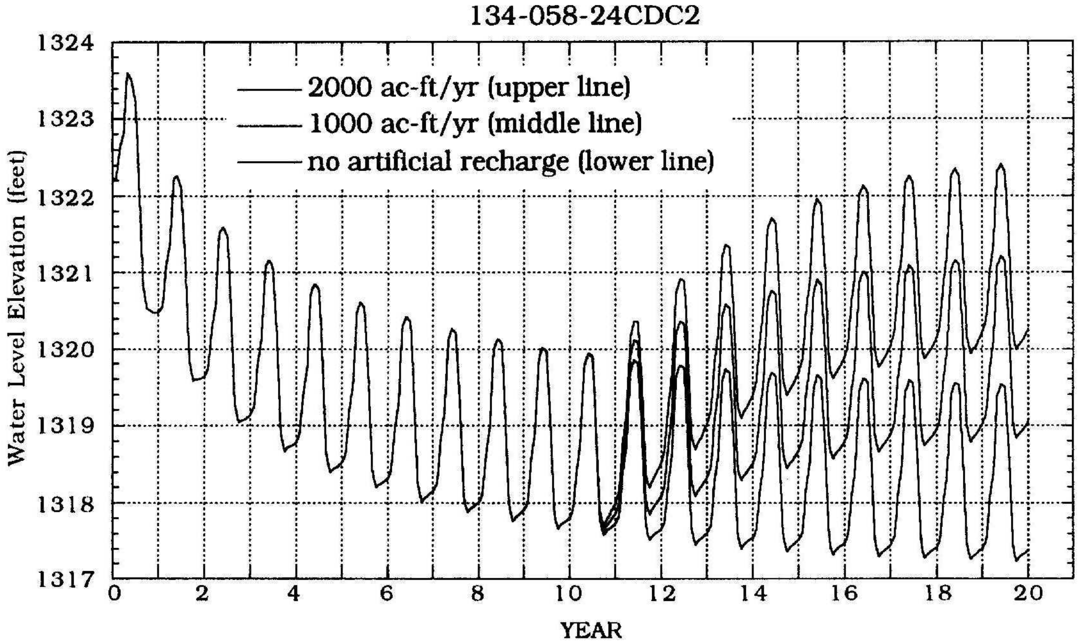

Fig. I-59. Comparison of simulated water levels at observation well 134-058-24CDC2 with no artificial recharge, and artificial recharge of 1,000 ac-ft/yr and 2,000 ac-ft/yr from basins located east side section 12, T134N, R58W.

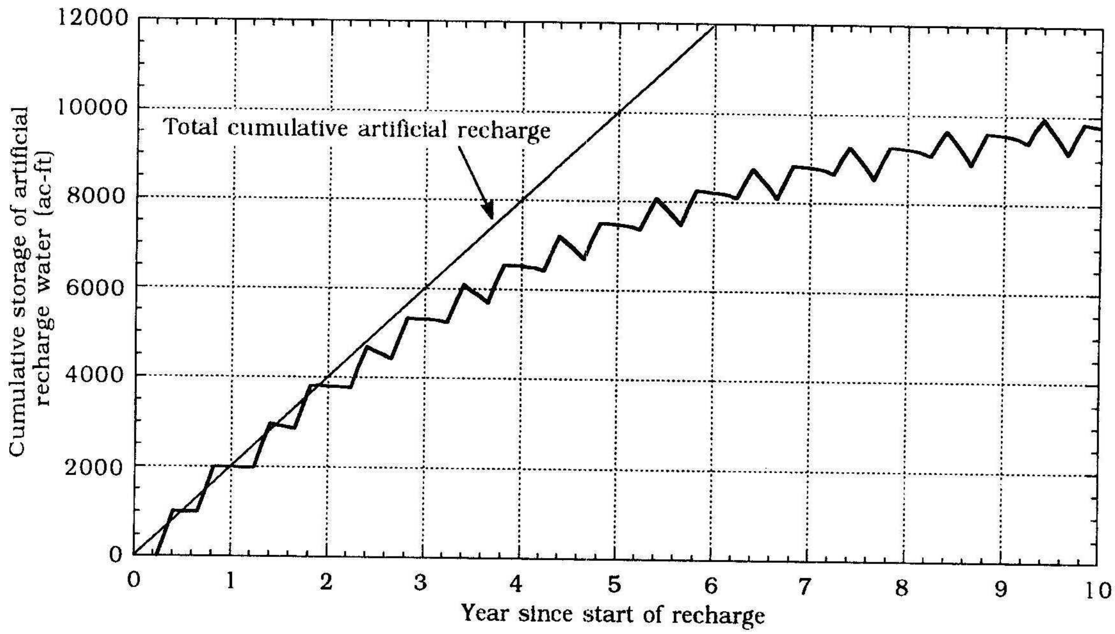

Fig. I-60. Cumulative storage of artificial recharge water in the aquifer with an annual artificial recharge rate of 2,000 ac-ft/yr. Recharge basins are located along east side Section 12, T134N, R58W.

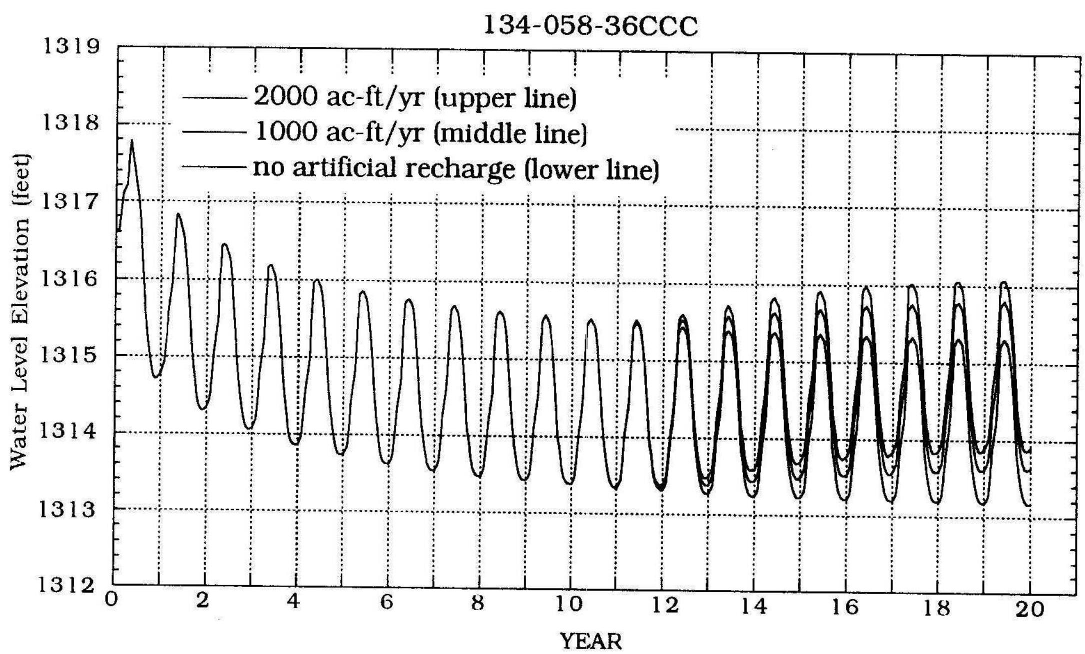

Fig. I-61. Comparison of simulated water levels at observation well 134-058-36CCC with no artificial recharge, and artificial recharge of 1,000 ac-ft/yr and 2,000 ac-ft/yr from basins located east side section 12, T134N, R58W.

By the fifth year of recharge facility operation, approximately 50% of the recharge water is lost. The height of the ground-water mound developed near the artificial recharge facility depends on the recharge rate. Consequently, so is the amount of additional water that can be stored in the aquifer. As can be seen in Fig. I-58 and I-59, the rate of increase in water levels for both the 1,000 ac-ft/yr and 2,000 ac-ft/yr case achieve equilibrium at about the same time though the height of the mound for the 2,000 ac-ft/yr case is significantly higher. Therefore, the amount of water that can be stored in the aquifer is dependent on the recharge rate and not the amount of time that recharge has occurred.

Table I-2. Table of percent of annual recharge water lost to evapotranspiration and runoff for simulated recharge basin located east side of Section 12, Township 134 North, Range 58 West.

years of artificial recharge	Percent of annual recharge lost to evapotranspiration 1,000 acre-feet/year	2,000 acre-feet/year
1	0	0.4
2	8	11
3	22	26
4	35	41
5	47	54
6	57	64
7	65	73
8	72	80
9	78	87
10	82	90

The simulation shows that it is not practical to store over three to five years of recharge water in the aquifer. If recharge is ceased, the ground-water mound will dissipate in about the same amount of time as was needed to create the mound. After the initial mound is established in 4 to 5 years, a declining rate of annual recharge would be needed to maintain the ground-water mound to provide additional water during a drought period. This implies that very large quantities of water would need to be pumped in relation to the amount of water that can be stored in the aquifer because of the loss of recharge water to evapotranspiration. Development of artificial recharge to provide additional stability to the availability of water from the aquifer is not practical unless the recharge water can be provided at very low cost.

Artificial recharge could be used to supply irrigation water if the system does prove to be over-appropriated. It could also provide water to irrigate additional acreage. However, because of the inability to store large quantities of water in the aquifer for long time spans, artificial recharge does not appear to be a solution to the water-level problems that have occurred during the 1980s.

Ironically, the effectiveness of artificial recharge can be improved by lowering the water level in the aquifer significantly below what it is now. This would allow more water to be stored in the aquifer before significant quantities of water are lost to evapotranspiration. The relocation of many wells to the thicker parts of the aquifer would be required. In order to maintain the present level of stability in water available from the aquifer, relocation of wells will be required if artificial recharge is to be developed to irrigate additional acreage.

SUMMARY AND CONCLUSIONS

The aquifer does not appear to be over-appropriated based on an analysis using 1904 to 1989 climate records for the Englevale area. However, with the present level of irrigation, problems will occur during periods of drought such as the 1980s. Based on McLeod and Oakes data, a drought as severe as the 1930s would require a significant reduction in irrigation. Artificial recharge is not a viable method of solving water-supply problems resulting from long drought periods such as the 1930s and 1980s. A limited amount of water can be stored in the aquifer without significant quantities being lost to evapotranspiration. Therefore, water cannot be banked during wet period for use during dry periods. Recharge facilities would need to be idle during wet periods since much of the water would be lost to evapotranspiration. Because of the large fixed costs for an artificial recharge facility to serve the Englevale aquifer, artificial recharge is not economically practical to increase the reliability of the water supply.

Assuming that the aquifer is not over-appropriated in the long term, the only effective method to increase stability of the water supply from the aquifer appears to be to increase the available drawdown. This could be achieved by moving wells in the areas of thin saturated thickness to the thicker parts of the aquifer. To evaluate the effectiveness of moving wells will require further analysis of climate variability and establishing the available drawdown in the existing wells to determine how many wells would need to be moved.

Artificial recharge can be used to increase the number of acres irrigated in the Englevale area. The reliability of the supply would depend on the reliability of adequate flows in the Sheyenne River. Since 3 to 5 years of storage are available in the aquifer without large loss to evapotranspiration, large annual variations in the river supply would be smoothed out. The design, operation, and economics of an artificial recharge facility to expand irrigation are discussed in Part II of this report.

Part II. FEASIBILITY OF EXPANSION OF IRRIGATED LAND IN THE ENGLEVALE AREA, USING ARTIFICIAL RECHARGE OF THE ENGLEVALE AQUIFER TO STORE WATER FROM THE SHEYENNE RIVER

One objective of the Sheyenne River artificial recharge study was to examine the feasibility of importing water at periods of high flow from the Sheyenne River, and storing the water in the Englevale aquifer for use in irrigation. Part I of this report concluded that the problem of stabilization of current use of the Englevale aquifer can be solved through changes in well location and water conservation. For this reason, the feasibility of artificial recharge is examined solely as an expansion option for land and water use in the Englevale area. Considerations in feasibility assessment will be availability and quality of water from the Sheyenne River and its suitability for irrigation; the safety of Sheyenne River water from the standpoint of aquifer contamination; the suitability of land in the Englevale area for both irrigation expansion and artificial recharge; logistics and costs of a pumping and conveyance system from the Sheyenne River to a recharge facility; land and management requirements and costs for an artificial recharge facility; and well and water redistribution system costs for retrieval and use of waters recharged to the Englevale aquifer. Cost analyses and options will consider the potential benefits of conservation measures, such as deficit irrigation scheduling and use of low energy and pressure systems (LEPA).

Types of Artificial Recharge Facilities

There are many types of artificial recharge facilities including injection well, surface spreading, and basin facilities. Because of long-term renovation problems caused by the clogging of water-bearing formations surrounding the well screen, use of injection wells is usually limited to water with low turbidity. Most applications are for injection of treated municipal waters.

The use of surface spreading is most practical in areas where land is relatively inexpensive, and where low permeability layers are not located near the soil surface. However, even minor soil development can cause considerable decreases in permeability resulting in extensive land requirements. For example, it was found at Oakes that infiltration rates were approximately 12 feet per day for the surface soil, while subsoils frequently had infiltration rates between 40 and 75 feet per day (Shaver and Schuh, 1988). It is therefore frequently advantageous to excavate below the zone of soil formation to cleaner sands which can be more simply and intensively managed. **For this reason, this report will deal exclusively with the basin option.**

Area Soil and Vadose Zone Suitability For Artificial Recharge

Efficient operation of an artificial recharge facility requires minimal impedance of water movement to the aquifer. Normally, water delivered to the aquifer begins to mound at the water table beneath the center of the recharge facility and redistributes outward. If the rate of redistribution is small compared with water infiltration through the surface and the thickness of the unsaturated zone is small, a ground-water mound may intersect the basin surface, greatly decreasing the infiltration capacity of the basin.

A relatively impermeable layer between the basin floor and the water table will cause a perched water table to form and a mound will form above the impermeable layer. If the layer is only slightly less permeable than overlying soil materials, then water will simply spread laterally over the impeding layer until the area of it covered by the mound and the head of the mound forming over it are sufficient for the impeding layer to conduct the quantity of water infiltrating from above. In many cases, such a "leaky mound" will be small and will not intersect the basin floor.

A layer of very low permeability can cause several problems. The first potential problem would be the formation of a large mound that would intersect the basin floor, thereby greatly decreasing basin infiltration. The second problem would be the prevention of infiltration waters from reaching the aquifer in time for effective aquifer augmentation. The third problem would be potential water damage to nearby crops or structures caused by water logging as the ground-water mound approaches land surface. For these reasons, suitability of an area for artificial recharge is partially dependent upon the lithology of soil and vadose materials overlying the aquifer.

During the summer of 1991, eight test holes were augered near Englevale in the area of the proposed recharge facility. All test holes were made with a flight auger to enable detailed examination of lithology. Test hole locations are shown on Fig. II-1, and lithologies are described in Appendix A. With the exception of one test hole in Section 13 (134-58-13DAD1) where a layer high in silt was located between 4.0 and 5.0 feet, all lithologies consisted of sands or gravelly sands at two feet below land surface.

The ideal choice of basin floor material would be a medium sand. While gravels are more permeable, they tend to allow finer suspended sediments to penetrate deeper below the basin floor which can eventually cause irrecoverable clogging. However, mixtures of sand and gravel or layers of sand and gravel should work well in the long term provided they are properly managed.

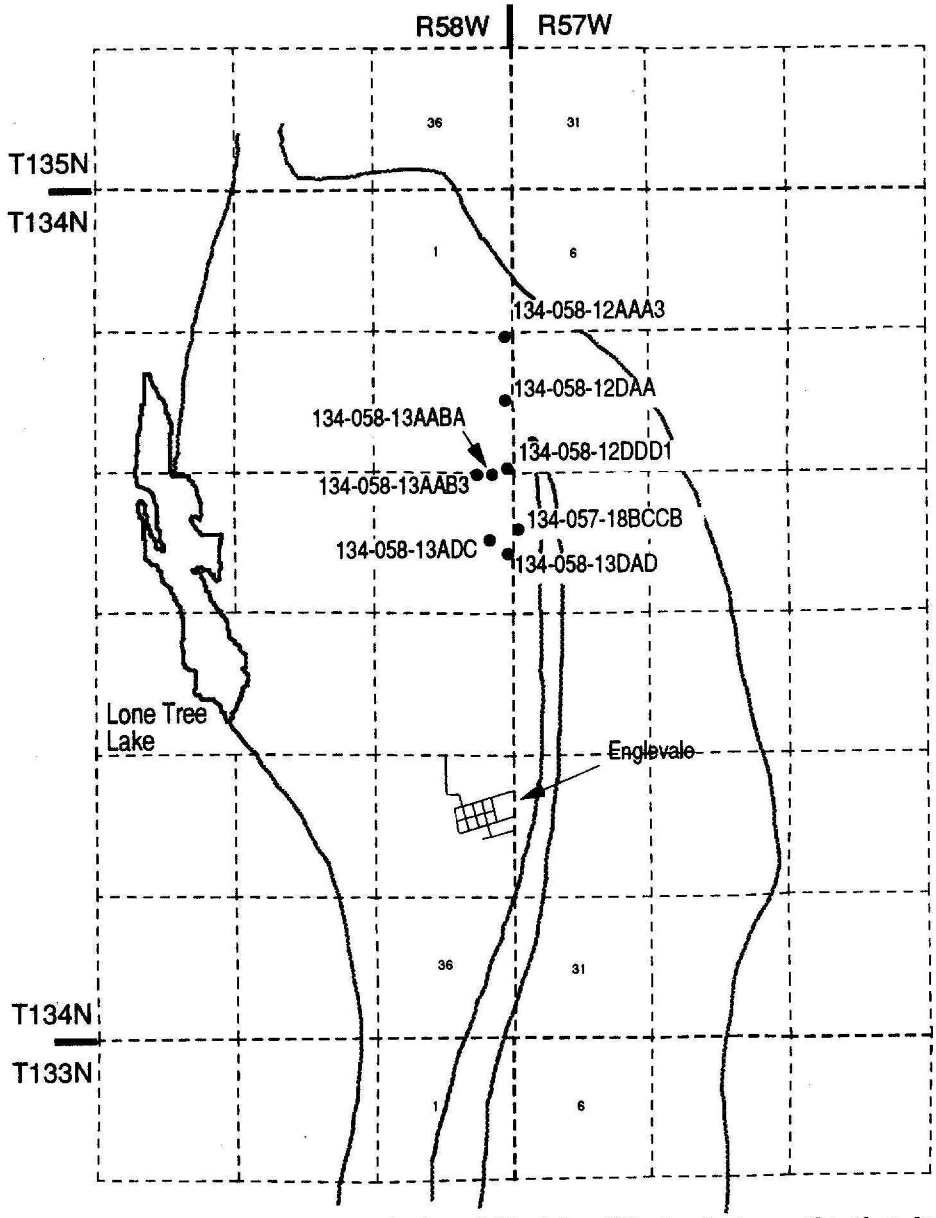

Fig. II-1. Location of test holes drilled for lithologic investigation in the Englevale area during 1991.

No impermeable materials were located between land surface and the water table. A few layers with some silt were found, but these constitute a minor impediment to flow. No impervious clay layers were found above the water table. **Vadose zone materials tested near Englevale appear to present no serious impediment to the development of an artificial recharge basin facility.**

SHEYENNE RIVER WATER SUPPLY

The first potential limiting factor for artificial recharge near Englevale is the supply of water from the Sheyenne River. Because there are competing demands, and the flow in the Sheyenne River is highly variable, evaluation of water-supply reliability is essential.

The flow of water in the Sheyenne River near Englevale is regulated by releases from Lake Ashtabula which is formed by Baldhill Dam. Baldhill Dam, located just north of Valley City (Fig. I-1), is operated by the U.S. Army Corps of Engineers (COE) for flood control and water supply. The flow of water in the Sheyenne River below the dam depends on the amount of water entering the reservoir during the spring runoff. In many years, runoff into the reservoir is sufficient to cause the COE to evacuate water from the flood control pool throughout the summer season. During low flow years when no water accumulates in the flood pool, the COE maintains a minimum release which has frequently been inadequate to meet existing water demands, especially during the critical months of July and August. However, even during years with minimal releases there is often some runoff which originates below Baldhill Dam that would be available for appropriation during the spring months.

Other Demands

Most competing demands for Sheyenne River water are by irrigators who use water predominantly from June 1 to September 30. Flows during other months are available for appropriation. The total demand for water below the USGS Gaging Station at Lisbon (Gaging Station #05058700; T. 134 N. R. 56 W. Sections 1 and 2) is 5,216 acre-feet with a total combined pumping rate of 53.3 cubic feet per second (cfs). A total of 3,556 acre-feet are permitted for irrigation of 2,700 acres and a little over 1600 acre-feet are permitted for municipal use by West Fargo. Although the combined pumping rate of all the permitted users is 53.3 cfs, most of these permits involve irrigation and consequently, not all the users are pumping water at any one time. Experience has shown that a flow of about 25 cfs at Lisbon during the irrigation season will satisfy downstream demands.

Even during the months when no irrigation is occurring, there would have to be some live flow left in the stream to allow for livestock and other downstream uses. Maintenance of a 5 cfs minimum flow should be adequate for this purpose.

Lisbon Data

Reliability of the water supply from the Sheyenne River is evaluated using stream flow data from the Lisbon Gaging Station (#05058700) which has a period of record dating back to 1956. Examination of duration data is one method of assessing the reliability of the water supply.

Table II-1 lists the percents of time that different flow rates have been equaled or exceeded for each month from March through October.

Table II-1. Percent of time flow rates match or exceed designated flow rates, during designated months at Lisbon, North Dakota.

Flow (cfs)	% Time Flow Exceeded							
	March	April	May	June	July	August	Sept	Oct
0.5300	100	100	100	100	100	99.8	100	99.5
0.7900	100	100	100	100	99.9	99.7	99.8	99.3
1.200	100	100	100	100	99.5	99.5	99.4	98.4
1.700	100	100	100	100	99.2	99.2	99.0	98.1
2.600	100	100	100	100	98.9	98.6	98.5	98.0
3.800	100	100	100	100	96.5	97.3	97.9	98.0
5.700	100	100	100	99.7	94.8	94.3	96.3	97.5
8.500	100	99.9	99.9	99.5	91.4	86.4	91.3	95.7
13.00	99.6	99.2	99.2	95.9	85.8	75.5	80.3	89.8
19.00	94.6	96.5	91.9	90.5	73.6	55.5	63.4	72.5
28.00	88.5	89.1	79.2	81.2	57.5	36.9	42.5	50.8
42.00	82.9	80.9	70.1	71.3	47.1	27.0	25.1	33.5
62.00	72.3	73.9	61.0	59.4	36.0	19.2	15.9	23.9
92.00	55.9	65.4	57.4	43.5	27.8	13.6	7.25	16.4
140.0	44.2	57.8	49.2	31.4	20.2	6.64	4.71	7.50
200.0	36.0	54.3	38.9	22.6	13.1	4.27	3.82	1.99
300.0	24.2	49.0	26.6	13.3	8.44	2.18	1.57	0.380
450.0	14.0	40.5	17.3	6.67	5.31	1.04	0.690	0.280
670.0	9.31	31.3	8.63	2.65	1.61	0.190	0.00	0.00
990.0	7.37	21.6	3.51	1.67	1.04	0.00	0.00	0.00
1500	3.87	12.9	2.09	0.200	0.660	0.00	0.00	0.00
2200	0.830	5.62	1.14	0.200	0.280	0.00	0.00	0.00
3300	0.180	1.90	0.850	0.100	0.190	0.00	0.00	0.00
4900	0.00	0.00	0.00	0.00	0.0900	0.00	0.00	0.00

Frequency data, developed using 30-day low flows, show the recurrence intervals for various mean monthly flows. For example, in Table II-2 an April having a mean discharge of 42.2

cfs has a recurrence interval of 10 years. Table II-2 was developed using a Log Pearson III frequency analysis based on 30-day low flows from the data collected at the Lisbon Gage.

Table II-2. Low flow recurrence intervals for designated mean monthly flows on the Sheyenne River at Lisbon, North Dakota.

Rec In Years	Mean Monthly Flow (cfs)							
	March	April	May	June	July	August	Sep	Oct
100	18.72	6.207	6.577	9.623	3.796	5.908	7.186	6.347
50.0	23.88	10.48	9.372	12.97	5.065	6.573	7.954	7.569
20.0	34.58	22.28	15.91	20.13	7.878	7.922	9.472	9.954
10.0	48.31	42.23	25.42	29.50	11.77	9.628	11.33	12.82
5.00	72.93	88.13	44.72	46.35	19.38	12.69	14.55	17.65
2.00	163.8	321.1	130.8	106.5	52.13	24.57	26.14	33.83
1.25	378.3	1012	379.2	234.9	147.2	57.63	54.80	68.42
1.11	593.0	1744	658.8	349.5	258.4	97.68	86.16	101.0
1.04	964.9	2995	1184	527.4	477.8	183.0	147.0	155.7
1.02	1328	4155	1727	683.5	716.7	284.8	213.9	207.9
1.01	1776	5501	2422	859.2	1038	434.9	305.6	271.3

Another method for analyzing the reliability of the water supply is to look at the historical daily flow record and assuming different pumping strategies, estimate the volume of water which would have been available for use each year. For development of Tables II-3A, II-3B, II-3C, it was assumed that 25 cfs would have to be passed downstream for senior appropriators between June 1 and September 15 each year and that 5 cfs would have to be passed downstream the rest of the time. It was also assumed that water would not be used for recharge at Englevale before April 15 each year, and the latest date would be October 31. Tables were developed for delivery rates of 10 cfs (Table II-3A), 15 cfs (Table II-3B), and 20 cfs (Table II-3C). The quantities are broken down into spring, summer and fall. The assumed spring season runs from April 15 to May 31, the summer season from June 1 to August 31, and the fall season from September 1 to October 31. The mean volume of water which would have been available under the 10 cfs option is 2,640 acre-feet per year and 3,554 acre-feet and 4,171 acre-feet under the 15 cfs and 20 cfs options, respectively.

Table II-3A. Unappropriated water available for use at Lisbon, North Dakota, based on 10 cfs pumping capacity.

10 cfs Option				
Year	Quantity of Water Available (ac-ft)			
	Spring	Summer	Fall	Total
1957	920	880	1220	3020
1958	920	0	620	1540
1959	180	0	420	600
1960	920	900	920	2740
1961	920	0	920	1840
1962	920	1840	1220	3980
1963	920	0	300	1220
1964	920	700	920	2540
1965	920	1840	920	3980
1966	920	1840	920	3980
1967	920	900	920	2740
1968	920	1000	920	2840
1969	920	1220	920	3060
1970	920	1000	300	2220
1971	920	1520	920	3360
1972	920	700	920	2540
1973	920	0	820	1740
1974	920	600	500	2020
1975	920	1720	920	3560
1976	920	0	920	1840
1977	920	0	920	1840
1978	920	900	1020	2840
1979	920	1840	920	3980
1980	920	0	1220	2140
1981	920	1220	820	2960
1982	920	1520	920	3360
1983	920	1220	1220	3360
1984	920	900	920	2740
1985	920	600	920	2440
1986	920	1840	920	3980
1987	920	1240	1220	3380
1988	920	200	220	1340
1989	920	300	920	2140
1990	920	700	300	1920

Table II-3B. Unappropriated water available for use at Lisbon, North Dakota, based on 15 cfs pumping capacity.

	15 cfs Option			
Year	Quantity of Water Available (ac-ft)			
	Spring	Summer	Fall	Total
1957	1380	450	1830	3660
1958	1380	0	750	2130
1959	0	0	630	630
1960	1380	1200	0	2580
1961	1380	0	450	1830
1962	1380	2760	1830	5970
1963	1380	0	0	1380
1964	1050	1050	1380	3480
1965	1380	2760	1830	5970
1966	1380	2760	1830	5970
1967	1380	900	0	2280
1968	1380	1500	1380	4260
1969	1380	1830	1380	4590
1970	1380	1500	0	2880
1971	1380	2130	1050	4560
1972	1380	1050	1380	3810
1973	1380	0	1230	2610
1974	1380	900	0	2280
1975	1380	2490	1380	5250
1976	1380	0	1380	2760
1977	1380	0	1230	2610
1978	1380	1260	1500	4140
1979	1380	2760	1830	5970
1980	600	0	1830	2430
1981	1380	1740	1110	4230
1982	1380	2280	750	4410
1983	1380	1830	1830	5040
1984	1380	1290	1380	4050
1985	1380	900	1380	3660
1986	1380	2220	1380	4980
1987	1380	1830	1830	5040
1988	1380	300	180	1860
1989	1110	300	1380	2790
1990	300	0	450	750

Table II-3C. Unappropriated water available for use at Lisbon, North Dakota based on 20 cfs pumping capacity.

Year	20 cfs Option			
	Quantity of Water Available (ac-ft)			
	Spring	Summer	Fall	Total
1957	1840	560	2400	4800
1958	1400	0	0	1400
1959	0	0	0	0
1960	1840	1600	0	3440
1961	280	0	0	280
1962	1840	3680	2440	7960
1963	1840	0	0	1840
1964	1320	1400	1320	4040
1965	1840	3680	2120	7640
1966	1840	3680	2440	7960
1967	1840	1200	0	3040
1968	1840	1880	0	3720
1969	1840	2440	0	4280
1970	1840	1960	0	3800
1971	1840	2680	1320	5840
1972	1840	1280	0	3120
1973	1520	0	280	1800
1974	1840	1160	0	3000
1975	1840	3360	1760	6960
1976	1840	0	1840	3680
1977	600	0	1080	1680
1978	1840	1520	2040	5400
1979	1840	3680	1880	7400
1980	480	0	2440	2920
1981	1400	2280	1440	5120
1982	1840	3040	1000	5880
1983	1840	2440	2440	6720
1984	1840	1600	1840	5280
1985	1840	1120	1720	4680
1986	1840	2800	1320	5960
1987	1840	2480	2440	6760
1988	1800	0	240	2040
1989	1480	400	1000	2880
1990	0	0	520	520

Operation of Baldhill Dam

The reliability of the water supply from the Sheyenne River is also partially dependent on the operating policies of the COE for Baldhill Dam. Changes in releases made from the reservoir directly affect the quantity of water available. For instance, if the COE increases the minimum release made from the reservoir during the summer in low runoff years, the reliability of the water supply could be improved. It is doubtful that detrimental changes would be made since any water accumulated in the flood pool would still have to be released for safety reasons. Discussions have taken place between the State Water Commission (SWC) and the COE regarding the reservoir operation. However, for the purpose of this study, it will be assumed that the COE will continue its current operational policies.

Assessment of Reliability of Irrigation Water Supply

Previous analysis has shown that there is a significant quantity of Sheyenne River water available for appropriation by a water user who could use it in the spring and fall. The quantity available varies between 2,640 acre-feet per year and 4,171 acre-feet per year depending on the size of the conveyance facility. However, because of needs for planning, users of artificial recharge water must be able to assess the reliability of recharge water supplies. Moreover, pipeline construction costs for water conveyance from the Sheyenne River to the project area is the single largest project expense. It is therefore essential that the reliability of river water supplies be evaluated, and that preliminary project cost estimates be based on pipe sizes that are appropriate for the available supply.

Other factors that must be considered in evaluating the reliability of water supply for users of the recharged waters include: (1) the storage capacity of the aquifer, and (2) the efficiency of the aquifer in preventing recharged waters from discharging through other nonuse sinks. Storage capacity and use efficiency considerations are influenced by the thickness of the aquifer, depth to water table, and location of the recharge facility in relation to natural discharge areas. As water levels rise, activation of evaporation from slough areas located west of the project area may occur. Preliminary model simulations (Part I) have indicated that recharge waters would begin to reach the slough area after a period of three to four years and that partial losses to evaporation would then occur.

In addition to using unallocated spring and fall waters, another artificial recharge benefit is that of using the aquifer as a storage reservoir to dampen annual variations of supply. Ideally, the ability to pump more water to the aquifer in wet years could be used to offset years when less water is available. The potential storage time in the aquifer is simulated by treating river supplies as moving averages. For example, if the aquifer can be used reliably as a reservoir for 3-year periods without large loss, then a 3-year moving average cycle best reflects the overall supply for aquifer water use. If the aquifer can reliably be used as a reservoir for 10-year periods, then a 10 year moving average of Sheyenne River water supplies would best reflect the available water for storage in the aquifer.

Probability plots of supplies for 10, 15, and 20 cfs design options, and for 1-, 3-, 4-, and 10-year moving averages of river supplies were plotted. Moreover, the options of combined Spring and Fall pumping only (labeled SF) and combined Spring, Summer, and Fall pumping (labeled SSF) were considered. SSF supplies included only unallocated water during the summer months. One example of these plots is given in Fig. II-2, which describes the probability distribution of SF water supplies from the Sheyenne River for 15 cfs conveyance. The aquifer reservoir effect can be seen on Fig. 11-2. With annual storage only, a water user could be assured of only 1,000 acre-feet per year of water 95 percent of the time, and only about 1,350 acre-feet per year 90 percent of the time. However, aquifer storage adequate to average out supply variability in 10-year cycles would provide 2,200 acre-feet per year of additional water in 19 years out of 20, and only slightly more water in 9 years out of 10. The advantages of using the aquifer reservoir to stabilize supply are thus clear.

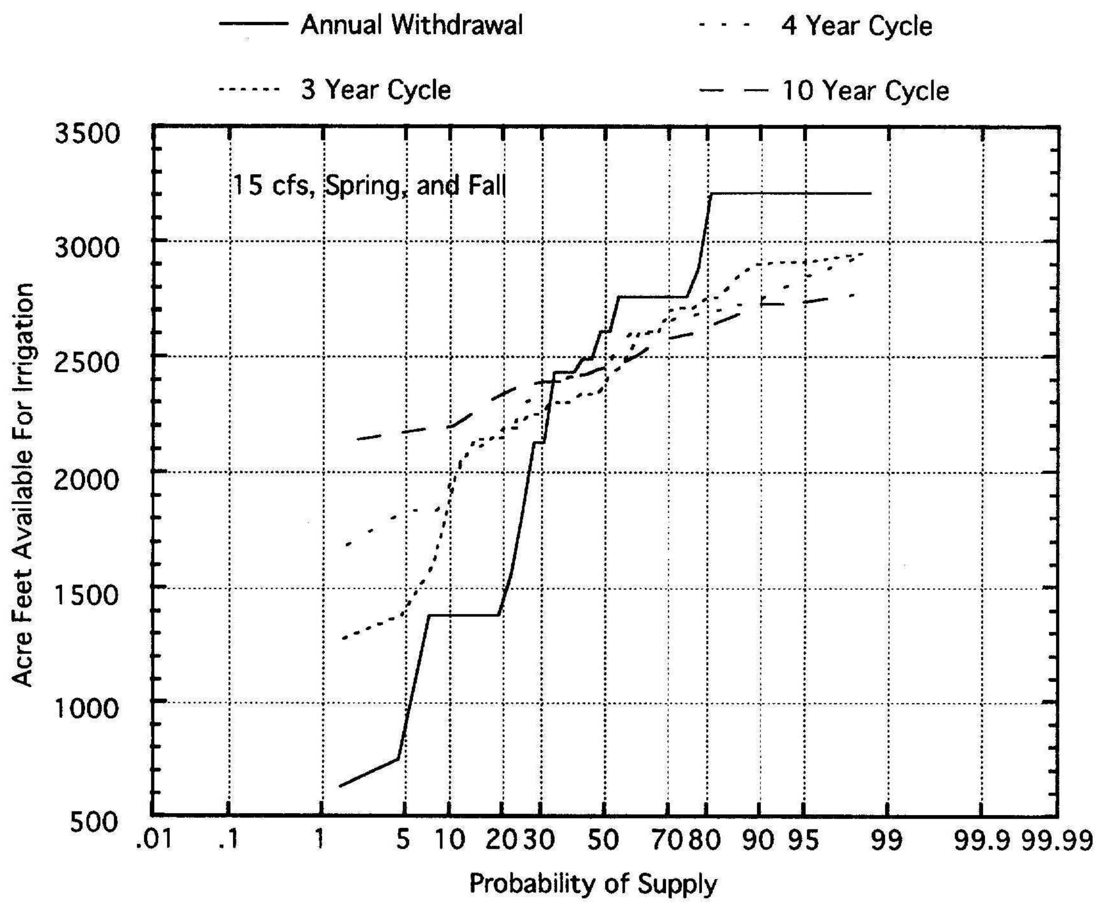

Fig. II-2. Probability plot for the Spring and Fall (SF) water supply from the Sheyenne River at a 15 cfs pumping rate.

From probability plots, irrigation water supplies for each conveyance system at set probability levels were extracted for plots of comparative supply efficiency at different design conveyance rates. Comparative data for SF and SSF river supplies are shown on Figs. II-3 and II-4 respectively.

For both SF and SSF, water supplies either decrease or increase at a decreasing rate with additional increments of pumping rate above 15 cfs. The decreasing efficiency of conveyance is caused by the larger likelihood of having days when a 20 cfs pump cannot be used because of low flows in the Sheyenne River. Because of the large cost of pipeline construction, it is considered unlikely that marginal gains in water delivery could justify the cost of additional pumping capacity above 15 cfs. All additional calculations in this feasibility study will include only the 15 cfs option.

All feasibility options will be based on the storage cycle represented by the three to four-year moving average of supplies from the Sheyenne River, because of the decreased efficiency of aquifer storage over long times caused by discharge from the Englevale slough complex. For the SF option, this represents 1,400 to 1,800 acre-feet per year in 19 years out of 20, about 1,900 acre-feet per year for 9 years out of 10, and 2,200 acre-feet per year for 8 years out of 10. For the SSF option, this represents 1,900 to 2,500 acre-feet per year for 19 years out of 20, 2,000 to 3,000 acre-feet per year for 9 years out of 10, and approximately 3,300 acre-feet per year for 8 years out of 10. **Based on this range of potential irrigation supplies, the supply options for recharge will be 1,800, 2,500, and 3,000 acre-feet per year.**

Average annual water use for irrigation in the Englevale area is about 10 inches per year. Assuming a somewhat conservative average annual water use of 12 inches per year, potential irrigated acreage options would then be 1,800, 2,500, and 3,000 acres. **Assuming 125 irrigated acres per quarter section, this would yield options of 14, 20, and 24 potentially irrigated quarter sections.**

A slightly greater efficiency for the 15 cfs option could be gained by employing a twin-pump design that would allow the use of lower flows. Moreover, greater certainty of supply might be assured by partial appropriation of summer flows when needed for the smaller water supply options based only on SF flows (approximately up to 2,500 acre-feet).

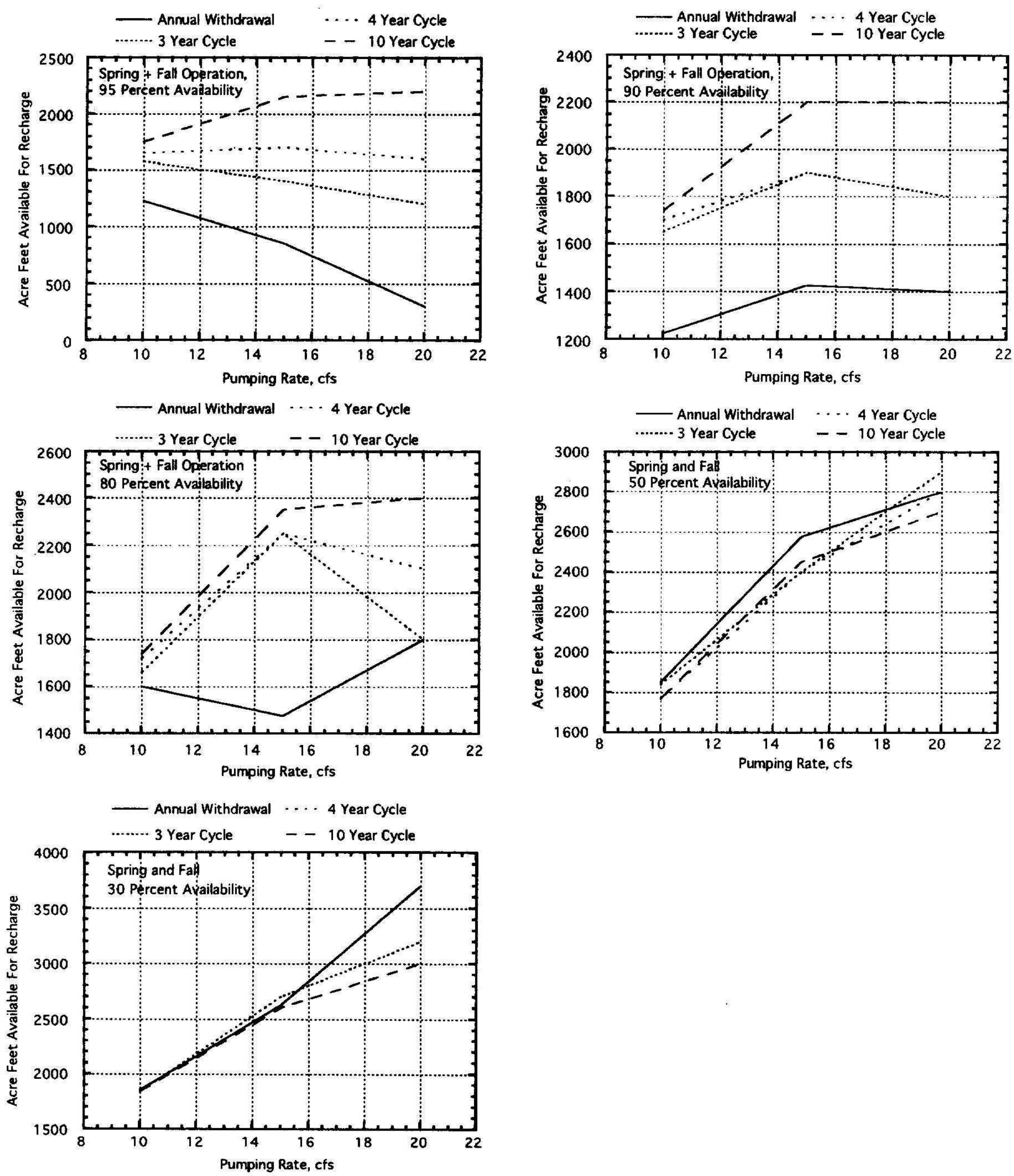

Fig. II-3. Spring and Fall (SF) water availability as a function of pumping rate and supply certainty on the Sheyenne River at Lisbon, ND.

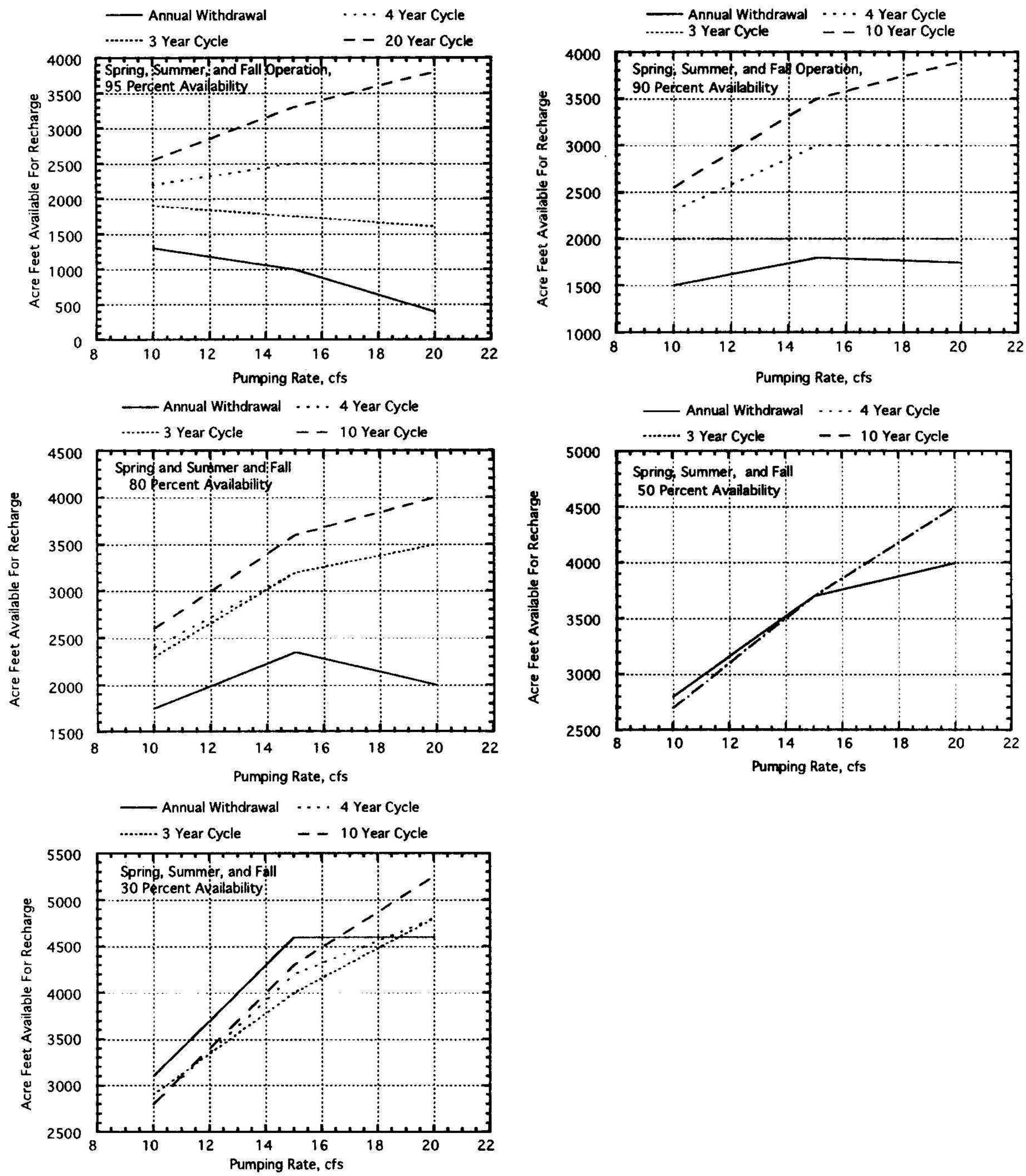

Fig. II-4. Spring, Summer and Fall (SSF) water availability as a function of pumping rate and supply certainty on the Sheyenne River at Lisbon, ND.

SHEYENNE RIVER WATER QUALITY

Three water quality concerns must be considered in implementation of an artificial recharge facility near Englevale. These are: (1) the possibility of aquifer contamination with inorganic or organic solutes that would render the quality of the aquifer waters less desirable for irrigation or drinking, and which might prove harmful to human beings, livestock, or crops; (2) the suitability or unsuitability of the water for infiltration into the soil through a basin facility; and (3) the potential effect of water chemistry on the recharge facility operation.

Inorganic Water Quality

There are several elements and compounds of concern regarding water quality. Nitrates are reduced to nitrites in infants and in excessive amounts, can cause methemoglobinemia (oxygen starvation of the blood). Arsenic, selenium, lead, and mercury are acutely toxic to humans and animals in small dosages. Excessive fluoride can cause mottling of teeth. Excessive sulfates can have a laxative effect, and excessive sodium may be unhealthy for people with high blood pressure. Iron can cause problems of color and staining. Total dissolved solids (TDS), also indicated by electrical conductivity (ECE), affect the taste of water and suitability for irrigation use. Sodium adsorption ratio (SAR) is an indicator of potential effects of sodium on soil cation exchange and structure. Boron, an essential micronutrient for most plants, is of particular concern for irrigation because of the differing sensitivities of crops to excessive amounts.

Data for each inorganic compound are available for selected measurement periods on the Sheyenne River. The USGS has monitored inorganic water quality of the Sheyenne River using samples taken from the Kindred gaging station (Gaging Station #05059000) on a seasonal (and sometimes monthly) basis since 1956. Trace elements including boron, arsenic, lead, zinc, selenium, and mercury were monitored for a ten-year period from 1969 to 1979. Maximum, minimum, median, and midpoint values ([maximum + minimum]/2) are presented for trace elements in Table II-4.

Maximum values for arsenic, lead, and zinc were all well below EPA maximum allowable contaminant levels (MCL). All mercury values were also below MCL, although the maximum mercury level did approach the MCL. However, the midpoint value is less than half of the MCL, and the median value is essentially 0 (negligible). This indicates that the quantity represented by the maximum value is rare. Maximum nitrate levels were less than 25 percent of MCL indicating that human toxicity from high nitrates is not a problem. The maximum boron level is only 20 percent of the U.S. Salinity Laboratory threshold value for possible danger to the most sensitive crop (U.S. Salinity Laboratory Staff 1954). Boron crop toxicity is not a serious threat using water taken from the Sheyenne River.

Additional water samples were taken by the SWC from a bridge located at T. 135 N. R. 57 W. SE quarter of Section 17 from April through September of 1991 to provide water quality information for a Sheyenne River station near the proposed point of diversion. Results (Table II-5) indicated that selenium, arsenic, mercury, and lead were all below MCL. Iron and boron were also below levels of concern. Measured nitrate and fluoride levels were all below levels of health concern. Sodium levels were also low. Sulfate levels were sometimes close to suggested maximum allowable contaminant levels (SMCL), but did not exceed them.

Manganese was somewhat high, but not a threat to human health, livestock, or crops. High manganese can cause problems with precipitation causing clogging of pipes and staining causing black discoloration of plumbing fixtures. Human toxicity of manganese occurs at levels much higher than those found in the Sheyenne River. The EPA suggested maximum contaminant (SMCL) value of 0.05 mg/l for avoiding staining is conservative, and precipitation generally does not occur below 0.150 mg/l.

The total dissolved solids (TDS) threshold for "brackish" water is 1,000 mg/l. All water with TDS less than 1,000 mg/l is classified as fresh. The EPA SMCL is 500 mg/l. The maximum TDS measured by the USGS at Kindred was barely into the brackish category. However, the minimum value was very low. The overall midpoint value for USGS data was 613 mg/l, which is very close to the SMCL. Moreover, this value was likely skewed by a few very large values, and overall average TDS were likely near or below SMCL. SWC 1991 TDS data (Table II-6) for the Sheyenne River had an overall average of 536 mg/l, which is very close to SMCL. Moreover, the highest values were during mid to late summer, when the least pumping from the river would occur, so that the TDS of waters for proposed use are likely below SMCL.

It appears that Sheyenne River water is suitable and even desirable for human, livestock, and crop use. However, some comparison with existing water quality of the Englevale aquifer should also be made. Statistics for water chemistry data on 35 selected wells in T. 135 N. R. 57 W., and 55 wells in R. 58 W. are shown in Table II-7.

The range of nitrate levels in the Englevale aquifer wells (Table II-7) indicate maximum nitrate values much larger than those found in the Sheyenne River, including some above MCL. The addition of Sheyenne River water would improve rather than deteriorate the nitrate status of the Englevale aquifer. The aquifer and river fluoride levels are very similar. The mean sulfate level is somewhat higher in the river. However, the range for river samples was within the range encountered in the aquifer. Bicarbonate levels are similar. Chloride levels are higher in the river than in the aquifer. Calcium and magnesium levels are similar. Sodium is higher in the river, but levels are not excessively high and should not cause a problem. TDS is slightly higher in the river, but the difference in total TDS would likely not be detectable for any use of the aquifer. The

Table II-4. Selected water chemistry parameters for the Sheyenne River measured at the USGS Kindred Gaging Station between 1956 and 1990. Trace elements were measured1956 to 1990. Species measured at each sampling varied. N is number of samples.

Parameter	Boron µg/l	Arsenic µg/l	Lead µg/l	Zinc µg/l	Selenium µg/l	Mercury µg/l	ECE µmhos	lab pH	Nitrate mg/l	TDS mg/l	SAR
MCL	2000	500	500	5000	10	2			44		
min	20	0.0	0.0	0.0	0.0	0.0	289	6.9	0.0	185	0.6
max	400	20.0	40.0	140.0	14.0	1.60	1550	8.7	8.6	1040	4.0
median		5.0	0.5	20.0	1.0	0.0	-	-	-	-	-
midpoint	160	10.0	20.0	70.0	7.0	0.80	920	7.8	4.3	613	2.3
N	35	15	14	11	15	10	35	35	35	35	35

Table II-5. Trace elements for water samples taken by the SWC from the Sheyenne River bridge at Township 135, Range 57, Section 17D in 1991.

Sample Date	Boron µg/l	Iron mg/l	Manganese mg/l	Selenium µg/l	Lead µg/l	Mercury µg/l	Arsenic µg/l	Lithium µg/l	Molyb-denum µg/l	Strontium µg/l
MCL	2000*	0.3**	0.05**	10	50	2	50			
4-16-91	160	0.020	0.12	0.0	0.0	0.0	0.0	0.0	0.0	0.00
4-29-91	160	0.020	0.30	0.0	0.0	0.0	0.0	0.0	0.0	0.000
5-9-91	140	0.030	0.14	1.0	1.0	0.30	1.0	70.0	0.0	480.0
5-20-91	130	0.030	0.63	1.0	1.0	0.0	2.0	80.0	0.0	570.0
6-10-91	130	0.030	0.22	0.0	0.0	0.10	3.0	80.0	0.0	540.0
7-22-91	180	0.020	0.12	0.0	0.0	0.10	6.0	70.0	3.0	630.0
9-4-91	210	0.020	0.07	0.0	0.0	0.10	0.0	80.0	3.0	470.0

* U.S. Salinity Laboratory plant toxity standard in place of MCL.

* * USEPA "Secondary Maximum Contaminant Level" (SMCL) in place of MCL.

Table II-6. Selected water chemistry parameters for samples taken by the SWC from the Sheyenne River bridge at Township 135, Range 57, Section 17D in 1991. SMCL is the EPA suggested maximum contaminant level.

Sample Date	ECE µmhos	field pH	lab pH	field temp °C	silica mg/l	calcium mg/l	magnesium mg/l	potassium mg/l	sodium mg/l	flouride mg/l	bicarbonate mg/l	sulfate mg/l	chloride mg/l	Nitrate mg/l	TDS mg/l	SAR
SMCL										2		250	250	44	500	
4/16/91	860	-	7.80	8.00	10.0	62.0	28.0	7.70	71.0	0 20	249	180	39	0.60	538	1.9
4/29/91	945	8.26	8.02	11.0	9.00	68.0	31.0	8.30	77.0	0 20	260	200	18	0.00	580	1.9
5/9/91	1048	8.36	7.87	16.0	8.70	74.0	35.0	9.80	95.0	0.20	350	230	50	0.60	656	2.3
5/20/91	1170	8.52	7.60	20.	11.0	86.0	40.0	17.0	100.	0.20	296	270	65	1.00	770	2.2
6/10/91	1105	8.46	7.86	22.0	16.0	76.0	38.0	11.0	100.	0 20	301	260	51	0.30	719	2.3
7/22/91	1004	8.17	7.45	24.0	25.0	71.0	35.0	11.0	89.0	0.30	303	220	44	1.20	649	2.2
9/4/91	976	8.30	8.36	20.0	11.0	54.0	37.0	20.0	95.0	0.70	324	180	40	0.10	62	2.4

Table II-7. Selected water chemistry parameters for water samples taken from the Englevale aquifer in Township 134 N; Sections 7, 18, 19, and 30 of Range 57 W; and Sections 1, 10, 11, 12, 13, 14, 23, 24, and 25 of Range 58 W.

Statistical Parameters	Location Township 134 - Range	ECE µmhos	lab pH	field temp °C	calcium mg/l	magnesium mg/l	potassium mg/l	sodium mg/l	flouride mg/l	bicarbonate mg/l	sulfate mg/l	chloride mg/l	Nitrate mg/l	TDS mg/l	SAR
Minimum	57	470.0	7.3	6.0	36.0	9.70	2.0	1.5	0.0	216.0	26.0	1.80	1.0	271.0	0.0
Maximum		1035.0	8.2	16.0	140.0	48.0	49.0	7.0	0.30	314.0	320.	23.0	100.0	711.0	1.9
N		35.0	10.0	35.0	35.0	35.0	35.0	35.0	35.0	35.0	35.0	35.0	35.0	35.0	35.0
Mean		681.4	7.69	8.83	91.0	28.5	10.7	3.3	0.120	271.71	124.2	10.7	16.0	444.4	0.317
Median		640.0	7.66	8.50	86.0	26.0	6.50	3.2	0.10	269.0	100.0	10.0	3.10	412.0	0.20
Std Deviation		147.1	0.318	1.66	26.2	9.48	10.8	1.19	0.071	26.1	68.9	5.97	26.0	114.0	0.42
Std Error		24.8	0.10	0.28	4.43	1.60	1.82	0.20	0.012	4.4227	11.6	1.00	4.40	19.2	0.072
Minimum	58	440.0	7.16	6.0	59.0	4.90	2.0	1.60	0.00	211.0	12.0	1.30	0.30	259.0	0.0
Maximum		870.0	7.9	15.0	130.0	37.0	31.0	5.10	0.20	350.0	210.0	54.0	49.0	583.0	0.8
N		55.0	13.0	55.0	57.0	57.0	57.0	57.0	57.0	57.00	57.0	57.0	57.0	57.0	57.0
Mean		646.9	7.57	9.2	89.1	27.3	7.15	3.00	0.10	276.9	109.5	9.84	3.65	413.6	0.178
Median		650.0	7.50	9.0	89.0	28.0	5.60	2.90	0.10	268.0	110.0	8.50	1.00	412.0	0.10
Std Deviation		120.3	0.21	1.8	19.3	5	5.01	0.86	0.058	30.4	49.5	8.58	7.38	83.2	0.15
Std Error		16.2	0.06	0.24	2.56	0.770	0.663	0.114	0.00	4.03	6.56	1.137	0 978	11.0	0.02

average temperature for the river water in the spring is higher (13.75 degrees C) than the aquifer (8.83 and 9.2 degrees C) but within the range of aquifer temperatures.

In conclusion, **from the standpoint of human or livestock consumption, there appears to be little likelihood of significant degradation of the Englevale aquifer from inorganic solutes in the Sheyenne River. Water stored in the aquifer should be of good quality for human and livestock consumption and for crop use.**

Irrigation Suitability of Sheyenne River Water

The potential problem of boron crop toxicity has been examined and found to be negligible. In addition, salt and sodium suitability for crop use and soil compatibility must be considered. Ranges of sodium adsorption ratio values (SAR) presented in the USGS Kindred data (Table II-4) and the SWC data for 1991 (Table II-6) are low. According to standards set in the North Dakota Irrigation Guide (USDA-SCS 1982), the sodium hazard which includes both potential crop toxicity and soil compatibility, is low. However, ECE values indicate a high salinity hazard. Some potential problems irrigating very salt-sensitive plants might be indicated. However, the river water is compatible with soils having a clay loam or coarser texture, which includes most Englevale area soils.

Water from the Englevale aquifer is of low sodium hazard and a medium salt hazard. However, there is little difference between the Englevale aquifer water and Sheyenne River water since the salt hazard for the aquifer is in the upper portion of the medium hazard range, and the salt hazard for the river is in the lower portion of the high hazard range.

In summary, sodium is low in both the Englevale aquifer and the Sheyenne River with no resulting soil or crop limitations. Some slight salt hazard for very sensitive crops is indicated for the Sheyenne River water. However, there is little actual difference in salinity hazard between the river and the aquifer waters and no significant degradation of the aquifer would be expected.

Organic Contaminant Human and Livestock Health Risk

Human health risk, livestock risk, and crop risk from Sheyenne River water were evaluated using USGS data from Kindred for 1978 through 1979 (Russ Harkness, written communication 1991) and a periodic comprehensive pesticide screen of the Sheyenne River conducted by the SWC during the Spring and Summer of 1991.

Organic contaminants surveyed by the USGS include aldrin, chlordane, DDE, DDT, diazinon, endrin, ethion, heptachlor, lindane, malathion, methyl trithion, parathion, PCB, silvex, toxaphene, trithion, 2,4-D, and 2,4,5-T. Of these DDT and 2,4,5-T are no longer in use. Samples were taken on June 28, 1977, August 23, 1977, May 24, 1978, July 26, 1978, May 2, 1979, June

1, 1979, and August 1, 1979. Some samples were taken from bottom materials and others were taken directly from the river water.

USGS organic contaminant survey results are summarized on Table II-8. DDT, DDE, PCB, and heptachlor were all detected in bottom mud in amounts exceeding existing health protection standards. However, no detections were made in the water itself. Previous studies near Oakes, North Dakota indicated that almost all of the clay settles out in the top 3 inches of an artificial recharge basin with a surface composed predominantly of medium sand (Schuh and Shaver, 1988). Under conditions most conducive to sediment movement, no indications of clay translocation were observed beneath a maximum depth of 20 inches. Similar studies elsewhere have indicated that as much as 90 percent of suspended sediment is removed within the top inch (Goss et al., 1973). Removal of settled bottom mud during pumping should be minimal with a properly constructed intake in the river. Few if any soil-born contaminants would be expected to penetrate beneath the surface of a recharge facility to a significant distance because of natural filtration during the recharge process.

Table II-8. Organic contaminants detected in samples taken by the USGS from the gaging station at Kindred in 1978 and 1979.

Common Name	Detection Date Month Day Year	Type of Sample	Concentration µg/kg
DDD	June 28, 1977	Bottom Material	0.1
	August 23, 1977	Bottom Material	0.3
	May 24, 1978	Bottom Material	0.1
	June 1, 1979	Bottom Material	0.3
	August 1, 1979	Bottom Material	0.1
DDE	August 23, 1977	Bottom Material	0.2
	May 24, 1978	Bottom Material	0.1
	May 24, 1978	Water Sample	0.01
PCB	August 23, 1977	Bottom Material	2
HEPTACHLOR	May 24, 1978	Bottom Material	1.2
2,4-D	August 23, 1977	Bottom Material	1.0
	August 23, 1977	Water Sample	0.1
	May 24, 1978	Water Sample	0.03
	July 26, 1978	Water Sample	0.03
	June 1, 1979	Water Sample	0.07

The herbicide 2,4-D was the only contaminant detected repeatedly (5 detections) in the Sheyenne River water by the USGS survey. All detections were below MCL and also below lifetime health advisory levels (LHA) by a factor of 1,000 or more. No significant human or livestock health risk is indicated by the USGS study. Crop risks are also negligible.

Results of the SWC 1991 pesticide survey are summarized on Table II-9. All detected chemicals were acid extractions. No base extractions or carbamates were detected. Detections followed a pattern of known agricultural usage. For example, 2,4-D, MCPA, and dicamba detections were concentrated in the period from mid-April to mid-May, which corresponds to the predominant spraying period for small grains. In addition, there were two detections of bromoxynil (at very low levels) and one of picloram in mid-May.

No significant risk to human health or livestock was indicated. 2,4-D detections were all at least 100 times smaller than EPA LHA and MCL levels. There was one MCPA detection which was approximately half of the LHA level (there is no MCL for MCPA). Dicamba has an LHA of 200 µg/l but no MCL. The single detection of dicamba is about 10 times lower than the LHA. There are no published LHA or MCL values for bromoxynil. However, current evidence indicates a low health hazard for bromoxynil, and a very general suggested level of concern of 140 µg/l was indicated by an EPA toxicologist (personal communication, Bob Benson, August 10, 1990). Sheyenne River detections were all 7,000 times lower than the suggested level of concern. Picloram has no MCL, but has an LHA of 400 µg/l. The two picloram detections were 4,000 times lower than LHA.

In general, the sporadic detection of surveyed chemicals as well as low detection levels indicate human and livestock health risks from artificial recharge using Sheyenne River water are not significant.

Table II-9. Results of pesticide survey for Sheyenne River, taken from the bridge at Township 135 N, Range 57 N, Section 17D in 1991. C is the concentration in the river water. MDL is the minimum detection limit. BDL is below detectable limit.

Pesticide (common name)	4-17-91 C ppb	4-17-91 MDL ppb	4-30-91 C ppb	4-30-91 MDL ppb	5-21-91 C ppb	5-21-91 MDL ppb	6-11-91 C ppb	6-11-91 MDL ppb	8-8-91 C ppb	8-8-91 MDL ppb	9-4-81 C ppb	9-4-81 MDL ppb
alachlor	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2
cyanazine	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2
dimethoate	BDL	0.6	BDL	0.5	BDL	0.5	BDL	0.5	BDL	0.5	BDL	0.5
metholachlor	BDL	0.6	BDL	0.5	BDL	0.5	BDL	0.5	BDL	0.5	BDL	0.5
pendimethalin	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2
propazine	BDL	2.0	BDL	2.0	BDL	2.0	BDL	2.0	BDL	2.0	BDL	2.0
terbufos	BDL	0.6	BDL	0.5	BDL	0.5	BDL	0.5	BDL	0.5	BDL	0.5
trifluralin	BDL	0.2	BDL	0.1	BDL	0.1	BDL	0.1	BDL	0.1	BDL	0.1
phorate	BDL	0.1	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2
fonofos	BDL	0.1	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2
pramitol	BDL	0.1	BDL	0.1	BDL	1.0	BDL	1.0	BDL	1.0	BDL	1.0
linuron	BDL	0.1	BDL	0.6	BDL	0.5	BDL	0.5	BDL	0.5	BDL	0.5
butylate	BDL	0.1	BDL	0.2	BDL	1.0	BDL	1.0	BDL	1.0	BDL	1.0
eptc	BDL	0.1	BDL	0.2	BDL	1.0	BDL	1.0	BDL	1.0	BDL	1.0
atrazine	BDL	2.0	BDL	3.0	BDL	2.0	BDL	3.0	BDL	3.0	BDL	3.0
chlorpyrifos	BDL	0.2	BDL	0.1	BDL	0.1	BDL	0.1	BDL	0.1	BDL	0.1
diallate	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2	BDL	0.2	0.2	0.2
ethalfluralin	BDL	0.2	BDL	0.1	BDL	0.1	BDL	0.1	BDL	0.1	BDL	0.1
methyl parathion	BDL	0.2	BDL	0.05	BDL	0.05	BDL	0.05	BDL	0.05	BDL	0.05
metribuzin	BDL	0.2	BDL	0.1	BDL	0.1	BDL	0.1	BDL	0.1	BDL	0.1
propachlor	BDL	0.3	BDL	0.4	BDL	0.4	BDL	0.4	BDL	0.5	BDL	0.5
simazine	BDL	2.0	BDL	3.0	BDL	3.0	BDL	3.0	BDL	3.0	BDL	3.0
triallate	BDL	0.2	BDL	0.1	BDL	0.1	BDL	0.1	BDL	0.1	BDL	0.1
2,4-D	**0.6**	0.1	BDL	0.1	**0.6**	0.1	BDL	0.2	BDL	0.5	BDL	0.5
MCPA	BDL	20	BDL	200	**66**	40	BDL	20	BDL	40	BDL	40
bromoxynil	**0.02**	0.01	BDL	0.01	BDL	0.1	BDL	0.01	BDL	0.01	**0.01**	0.01
dicamba	BDL	0.1	**23.2**	0.1	BDL	0.1	BDL	0.1	BDL	0.1	BDL	0.1
picloram	BDL	0.1	BDL	0.1	**0.1**	0.1	BDL	0.1	**0.1**	0.1	0.00	0.1

Organic Contaminant Crop Risk

Only one of the detections poses concern for use of Sheyenne River water in artificial recharge. The 23.2 µg/l detection of dicamba is of concern because of the high sensitivity of dry beans. The worst possible case would be direct irrigation from the river with dicamba concentrations at the 23.2 µg/l detection level. Common applications concentrations for foliar spray (based on active ingredients at 0.067 to 0.28 kg/ha, and spraying 19 to 374 l/ha) give a minimum concentration of 179,000 µg/l and a maximum concentration of 14,740,000 µg/l. Normal foliar concentrations are thus about ten thousand to one million times greater than concentrations detected in the river.

A total of 0.012 kg per hectare of dicamba would be applied if one foot of irrigation water were applied to a field at the concentration detected. This would be about one fifth of the minimum foliar herbicide application of 0.067 kg/ha (0.6 lb./acre). If dicamba occurred at the detected level during only one day of an assumed 100 days of pumping recharge water from the river, the amount of dicamba added to the field through irrigation would be 1/500 of the minimum foliar herbicide application per acre. These levels might still be of concern for highly susceptible legumes.

Three other factors must be considered regarding dicamba. First, the irrigation waters are not directly applied. They are mixed in the recharge basins, sometimes for several days. They are also strained through a sediment layer (and sometimes a calcite cement layer) which would be expected to attenuate chemical movement. Finally, dilution in the aquifer would cause a substantial decrease in concentration.

Placement of any significant quantity of dicamba in the aquifer might be objectionable to some despite low health and crop risks. However, it is stressed that only one detection of dicamba was made. There were no repeated detections to confirm a trend as with 2,4-D. Neither are there any firm indicators that similar quantities of dicamba might be detected in other years or even at other times within the year monitored.

In summary, the single dicamba detection should not be considered a serious impediment to artificial recharge from the Sheyenne River. The single detection simply indicates that the possibility of excessive dicamba levels for the intended use should be considered. It does not indicate that a serious problem exists or that similar detections would be commonplace. Finally if significant levels of contamination did exist on a repeated basis, problems of contamination could be addressed by agencies regulating the use of pesticides.

Water Chemistry Effect on Artificial Recharge

Water chemistry can have a significant effect on the operation of an artificial recharge facility. High sodium can cause slaking of the basin surface decreasing infiltration. Organic acids present in organic filters that can be applied to the recharge basin surface can increase infiltration rate and basin effectiveness (Schuh and Shaver, 1988, Schuh, 1991). Precipitation of salts can cause cementation and clogging of the basin surface (Bouwer and Rice, 1989, Schuh 1990).

One problem of likely concern in the Englevale project area is the precipitation of calcite, dolomite, and iron on the basin floor. Bouwer and Rice (1989) and Schuh (1990) indicated that an increase in pH can cause precipitation and clogging of the basin floor for waters near saturation or oversaturated with respect to calcium or magnesium carbonate. A solution saturated with respect to dolomite or calcite will usually have a pH between 7.8 and 8.2. This is the common pH range for the Sheyenne River (Tables II-4 and II-6). A pH above these levels would likely cause precipitation of calcite and dolomite.

Increasing pH in late spring and summer is usually caused by algae growth in natural bodies of water. Algal photosynthesis removes bicarbonate and increases the dissolved oxygen in basin water. This, in turn, increases pH. Increased pH is more marked in static water or in reservoir waters having large detention times. Two trends in pH are observed for the Sheyenne River: (1) Since 1956 there has been a gradual increase in both annual minimum and maximum pH values measured by the USGS. This may be influenced by increased detention time of Sheyenne River water, caused by the filling of Lake Ashtabula, which would cause increased photosynthesis. Photosynthetic organisms could also have been influenced by increased nutrient loading of Lake Ashtabula. (2) An increase in pH in 1991 from 7.8 in mid April to 8.52 in mid May was observed. This increase may have occurred in Lake Ashtabula or it may have occurred partially in-stream.

Saturation indices for the 1991 SWC data were computed using WATEQF (Plummer et al., 1976). Input data included field measured pH and temperature, calcium, magnesium, sodium, potassium, bicarbonate, silica, sulfate, nitrate, chloride, fluoride, phosphate, strontium, iron, and manganese. Early season electrical potential (Eh) was estimated using the Sato relationship (Plummer et al., 1976). For July and September data, Eh was estimated using dissolved oxygen (DO) measurements taken by inserting the DO meter directly into the Sheyenne River. Results are summarized on Table II-10. Saturation indices greater than 0 indicate that precipitation of the indicated salt or compound would be thermodynamically plausible. Precipitation of calcite, dolomite, and iron hydroxide are all indicated to be plausible for the Sheyenne River. Maximum likely precipitation of calcite and dolomite are indicated for mid-May.

The operation of a basin, with ponding and increased detention time of water would be expected to increase basin pH. Tests at Oakes indicated that basin pH could reach levels as high as 9.0 with two feet of basin ponding (Schuh 1990). Saturation indices for WATEQF simulations using water chemistry for each measured date, but with pH values of 8.5 and 9.0 are summarized on Table II-10. Simulated saturation indices indicate that basin detention could increase the likelihood of dolomite and calcite precipitation. Possible iron hydroxide precipitation is also indicated. Analysis of the Oakes test basin indicated that iron mineral precipitation did occur (Schuh, 1990).

Table II-10. Saturation Indices simulated using WATEQF for Sheyenne River water chemistry data measured in 1991, and for simulated higher pH values that might be caused by algal influence on standing water in infiltration basins (in italics).

M-D-YR	pH	Calcite	Dolomite	Gypsum	Silica gel	Quartz	Fe(OH)3
4-16-91	**7.80**	**0.188**	**0.132**	**-1.409**	**-0.835**	**0.502**	-
4-29-91	**8.26**	**0.727**	**1.273**	**-1.357**	**-0.929**	**0.402**	-
5-09-91	**8.36**	**1.028**	**1.977**	**-1.32**	**-1.02**	**.0302**	
5-20-91	**8.52**	**1.202**	**2.379**	**-1.226**	**-0.981**	**0.333**	
6-10-91	**8.46**	**1.134**	**2.301**	**-1.286**	**-.0.846**	**0.465**	
7-22-91	**8.17**	**0.884**	**1.815**	**-1.363**	**-0.673**	**0.633**	**1.725**
9-04-91	**8.39**	**0.961**	**2.061**	**-1.543**	**-0.977**	**0.337**	**1.531**
4-16-91	*8.50*	*0.861*	*1.482*	*-1.416*	*-0.84*	*0.497*	
4-29-91	*8.50*	*0.951*	*1.724*	*-1.360*	*-0.932*	*0.399*	
5-09-91	*8.50*	*1.155*	*2.233*	*-1.324*	*-1.023*	*0.299*	
5-20-91	*8.50*	*1.184*	*2.343*	*-1.225*	*-0.980*	*0.334*	
6-10-91	*8.50*	*1.169*	*2.372*	*-1.287*	*-0.847*	*0.463*	
7-22-91	*8.50*	*1.183*	*2.416*	*-1.371*	*-0.682*	*0.624*	*1.558*
9-04-91	*8.50*	*1.060*	*2.260*	*-1.546*	*-0.981*	*0.333*	*1.462*
4-16-91	*9.00*	*1.296*	*2.362*	*-1.432*	*-0.855*	*0.482*	
4-29-91	*9.00*	*0.727*	*1.273*	*-1.357*	*-0.929*	*0.402*	
5-09-91	*9.00*	*1.566*	*3.490*	*-1.347*	*-1.046*	*0.275*	
5-20-91	*9.00*	*1.587*	*3.159*	*-1.246*	*-1.009*	*0.305*	
6-10-91	*9.00*	*1.569*	*3.182*	*-1.310*	*-0.879*	*0.432*	
7-22-91	*9.00*	*1.577*	*3.217*	*-1.396*	*-0.717*	*0.590*	*1.154*
9-04-91	*9.00*	*1.465*	*3.085*	*-1.573*	*-1.009*	*0.305*	*1.058*

In summary, chemical constituents in the Sheyenne River water would likely promote the precipitation of calcium and iron salts on the basin floor. This would decrease infiltration rate through the basin floor. The precipitation of dolomite and calcite can be minimized by maintaining shallow water depths in the basin, which decreases the detention time of water in the basin and minimizes algal growth (Bouwer and Rice, 1989). Periodic cleaning of the basin to break the cementation of the surface layer can also renovate the basin.

Some periodic monitoring or special investigations of water quality will be desirable or even necessary. **The operational budget should include monitoring or investigative costs.**

CONVEYANCE SYSTEM COST AND DESIGN

Route Selection

Route selection for water conveyance from the Sheyenne River to the recharge site is the first step in the preliminary design of the delivery system. Once the desired destination point has been selected, the route can also be chosen based on the length, terrain, ease of access, obstructions, and other criteria.

The objective is to recharge the north end of the Englevale aquifer with recharge facilities located near the Englevale road which lies along the township line between Ranges 57 and 58 West. Recharge would most likely occur beginning at or south of Highway #27 with the facilities proceeding in a southerly direction. Therefore, the logical route would be along that township line directly north to the Sheyenne River. This would be the shortest, most direct route between the river and the location where the water is to be recharged. The terrain would be similar for any route selected, with a very steep climb out of the river valley followed by a relatively flat course. The water would have to cross the divide between the Sheyenne River basin and the Wild Rice River basin.

At the proposed point of diversion, the interbasin divide consists of a range of hills having an elevation of just over 1,380 ft. The river elevation is about 1,120 feet and the resulting static lift is about 260 feet. It will be necessary to pipe the water over this divide. The pipeline would extend from the Sheyenne River to one-half mile north of Highway #27 where the water would then be conveyed by open channel.

Selecting a route along a section line has the advantage of site access and should also facilitate easement acquisition. Only the mile nearest the river would not be immediately accessible by motor vehicle. Figure II-5 shows a topographic profile for the selected route. The elevations were taken from USGS 7.5 minute topographic quadrangles.

Potential Access Difficulties

Because of the steep and forested valley sides, it will be necessary to clear cut the trees along the route. It will be necessary to maintain a treeless corridor for the pipeline to ensure that growth of tree roots does not damage the pipeline. An access road or trail will also be needed for the mile nearest the Sheyenne River.

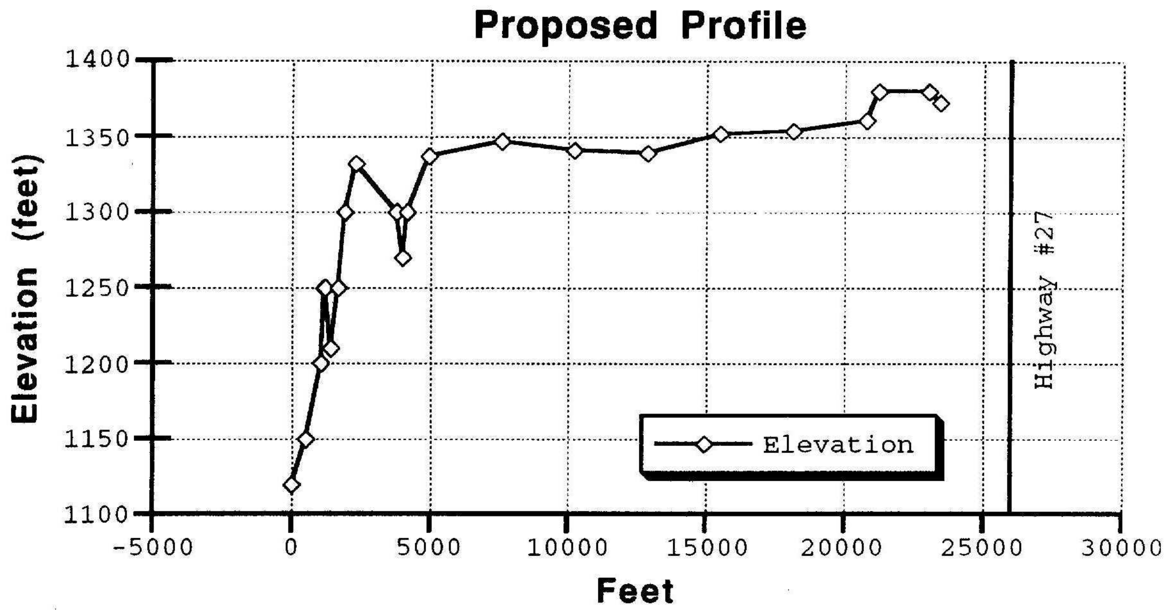

Fig. II-5. Land surface elevation profile for the route of a proposed pipeline from the Sheyenne River to an artificial recharge facility at Englevale, ND.

Pipeline Materials

Various materials can be used for pressure pipelines. In selecting materials, price as well as performance characteristics of materials for the design task must be considered. Some more common materials used for pressurized water-transmission lines include plastic, ductile iron, and reinforced concrete. Advantages of ductile iron or reinforced concrete include ability to withstand bending loads, strength and durability during handling and installation, and ability to withstand negative pressures. Disadvantages include susceptibility to corrosion and greater initial costs.

Use of plastic pipe has increased in recent years. Polyvinyl chloride (PVC) accounts for 90 percent of all plastic pressure water pipe (Mosser, 1990). PVC advantages include corrosion resistance, smooth wall for minimal friction head loss, water inertness, along with ease of transport, handling, and installation due to light weight. Disadvantages include requirement of extra care in bedding and backfilling, and in general handling (Tullis, 1989). Cost estimates for PVC and reinforced concrete are considered in this feasibility study.

Pipeline Design and Cost (PVC Option)

The total length of the pipeline would be approximately 23,420 feet from the river to the quarter line of Section 1, T. 134 N. R. 58 W. The total static head to be overcome is about 260 feet. Based on the analysis of the supply of water available from the Sheyenne River, the system would be designed for a 15 cfs capacity. Using nominal diameters, the velocities resulting from using various pipe sizes are listed in Table II-11.

Table II-11. Water velocity (V), static head (H), and frictional pressure (h_f) as a function of PVC pipe size.

Nominal D (in)	V (fps)	H (ft)	h_f (ft)	($H+h_f$) (ft)	($H+h_f$) (psi)
14"	14.0	260'	729	989	429
16"	10.7	260'	381	641	278
18"	8.5	260'	215	475	206
20"	6.9	260'	128	388	168
24"	4.8	260'	53	313	136
30"	3.1	260'	18	278	120

Reducing the pipe diameter increases the flow velocity. Since the friction loss is dependent on flow velocity, reducing the pipe diameter also increases the friction loss. Frictional loss can be calculated using the Hazen-Williams method (eq. II-1) for plastic pipe where it is assumed C = 150 (Uni-Bell, 1982).

$$f = (0.0984)\, Q^{1.85}/d_i^{4.87} \qquad \text{(II-1)}$$

where: f = head loss in ft/100 ft.

Q = flow in gpm

d_i = inside diameter in inches

Surge pressures must also be considered in pipeline design. Abrupt changes in velocity create transient surges also known as 'water hammer'. Surges result from the opening and closing of valves, or the starting and stopping of pumps. The magnitude of this pressure is a function of the type of pipe and the change in velocity. A rule of thumb for PVC pipe is to design for 16 psi for each one foot per second (fps) in velocity change (Mosser, 1990). Using this rule of thumb, estimating the surge pressure and adding it to the static and frictional pressures results in the specifications shown in Table II-12.

Table II-12. Surge pressure and total pressure adjusted for surge.

Nominal D	Velocity (fps)	($H+h_f$) (psi)	Surge (psi)	Total Pressure psi
18"	8.5	206	136	342
20"	6.9	168	110	278
24"	4.8	136	77	213
30"	3.1	120	50	170

Maximum flow velocities in PVC piping systems are normally limited to 5 feet per second (Uni-Bell 1982). Therefore, a 24-inch pipe would result in velocities just within the specified limit. The total pressure (working pressure plus surge pressure) should not exceed the pressure rating of the pipe (Uni-Bell, 1982). Therefore, a 24-inch pipe with a pressure rating greater than 213 psi should be selected.

A local pipe supplier provided the price estimates for PVC pipe shown in Table II-13. All cost figures are in units of \$/ft. Specifications for standard dimension ratio (SDR) 17 (250 psi) iron pipe size (IPS) outside diameter (O.D.) pipe are also listed in the Uni-Bell PVC Pipe Handbook. However, the local pipe supplier was unable to locate a manufacturer who is currently

Table II-13. Approximate cost of PVC pipe in dollars per foot for both IPS and cast iron schedule at specified diameters and pressures.

IPS Schedule O.D. Pipes

Pressure Rating	Diameters			
	16"	18"	20"	24"
SDR 32.5 (125 psi)	8.90	11.50	14.25	20.65
SDR 26 (160 psi)	11.15	14.23	17.78	25.70
SDR 21 (200 psi)	13.72	17.45	--	--

Cast Iron Schedule O.D. Pipes

Pressure	Diameters				
	16"	18"	20"	24"	30"
SDR 32.5(125 psi)	10.85	14.75	16.90	23.90	--
SDR 25 (165 psi)	13.90	17.60	21.61	31.00	47.60
SDR 21 (200 psi)	--	--	25.30	37.40	--
SDR 18 (235 psi)	19.00	24.00	29.40	40.90	--

manufacturing this pipe series. Based on this data, the 24-inch SDR 18 (235 psi) C.I.O.D. pipe should be selected. The actual velocity and pressures need to be recalculated using the actual inside diameter.

Nominal Diameter = 24"
Inside Diameter = 25.8 - (2)(1.433) = 22.9 inches
Velocity = Q/A = 5.2 fps
hf = 66 feet
SH = 260' + 66' = 326' = 141 psi
Surge = (16)(5.2) = 83.2 psi
Total Pressure = 141 psi + 83 psi = 224 psi
224 less than 235 Therefore pipe specifications are acceptable

Using $10/ft for installation, the **total cost** would be:
($40.9 + $10)(23,420) = **$1,192,078**

It may be possible to use a pipe with a smaller pressure rating once the pipeline is beyond the Sheyenne River valley. Out of the total 260 ft of static head, 227 ft are overcome after the first 7,580 feet of the route. Reducing the total pressure by 227 ft or 98 psi results in a pressure of 126 psi. Therefore, DR 25 (165 psi) pipe could be used for the final 15,840 feet.

Then the **total cost** would be:
($40.9 + $10)(7580) + ($31.00 + $10)(15,840) = **$1,035,262**

Pipeline Design and Cost (Reinforced Concrete Option)

The friction losses for reinforced concrete pipe can also be calculated using the Hazen-Williams method (Uni-Bell, 1982) with C = 130 for concrete (Clark, 1977).

$$f = (.2083)(100/C)^{1.85}(Q^{1.85}/d_i^{4.87}) \qquad \text{(II-2)}$$

Table II-14 presents the flow velocities and resulting friction losses for various sizes of reinforced concrete pipe. For reinforced concrete pipe, a 40% allowance for surge pressure was included in the pipe selection. Given the calculated heads, a local pipe supplier provided the following cost estimates:

20 inch	\$35/ft
24 inch	\$40/ft
30 inch	\$45/ft

Table II-14. Water velocity, static head (H), and frictional pressure (h_f) as a function of reinforced concrete pipe size.

Diameter (inches)	V (fps)	H (ft)	h_f (ft)	$(H+h_f)$ (ft)	$(H+h_f)$ (psi)
14"	14.0	260'	950'	1210'	524
16"	10.7	260'	496'	756'	328
18"	8.5	260'	279'	539'	234
20"	6.9	260'	167'	427'	185
24"	4.8	260'	69'	329'	143
30"	3.1	260'	23'	283'	123

With reinforced concrete pipe, the nominal d is the actual inside diameter. Therefore, no recalculation is necessary. It also appears that changing the pressure rating of the pipe does not significantly change the cost. Therefore the 24-inch pipe should be selected to achieve velocities in the 5 fps range.

Cost = (\$40 + \$10)(23,420 ft) = **\$1,171,000**

Power Requirements

The sum of the static and friction head loss is approximately 330 feet for both options. Using this data and a pumping rate of 15 cfs, the power requirements can be calculated using eq. II-3:

$$P = Q\,g\,H = (15\text{ cfs})(62.4\text{ lb/ft}^3)(330\text{ft}) = 308{,}880\text{ ft-lb/sec} \qquad \text{(II-3)}$$

where: Q = discharge

g = density of water

H = total head

Assuming 80% efficiency:

$$P = 308{,}880 \text{ (ft-lb/sec)}/.80 = 386{,}100 \text{ ft-lb/sec} \quad \text{(II-3a)}$$

$$(386{,}100 \text{ ft-lb/sec})(3.766 \times 10^{-7} \text{ kwh/ft-lb}) = 0.1454 \text{ kwh/sec} \quad \text{(II-3b)}$$

$$(.1454 \text{ kwh/sec})(86{,}400 \text{ sec/d})(30 \text{ d/m}) = 376{,}890 \text{ kwh/m} \quad \text{(II-3c)}$$

where: kwh = kilowatt-hour

ft = foot

h = hour

sec = second

d = day

m = month

Cass County Rural Electric Cooperative provided a cost estimate for this power requirement. One month of continuous operation would cost $9,830, and the cost for a month of non-operation would be $500. **An average year pumping 3,000 acre-feet would cost about $38,700. An agreement may be possible whereby the buried power line located near the proposed canal could be changed to an overhead line in exchange for right of way along the canal.**

Canal

A canal would be used to convey water from the quarter line on the east edge of Section 1, T. 134 N., R. 58 W. to the southeast corner of Section 12. Without a detailed survey of existing topography, accurate estimation of the required excavation is difficult. Using the USGS 7.5 minute topographic quadrangles for the area and an excavation cost of $1.25 per cubic yard, an estimated cost of $31,000 for the mile south of Highway #27 was developed. The terrain for this mile of channel is relatively flat. However, the half mile north of the highway includes a drop of over 20 feet. It can be assumed that the canal will cost about the same per foot as the mile of canal south of the highway, but the cost of drop structures will also need to be included. The cost of these structures was estimated from information provided by the U.S. Soil Conservation Service (SCS).

excavation ($31,000)(1.5)	**$46,500**
4 drop structures @ $10,000	**40,000**
Road Crossing & Head Works	**10,000**
Total	**$96,500**

Channel Dam

A channel dam will be required in the Sheyenne River to provide an adequate depth of water for the intake facility. A sheet pile weir would be used. Rip rap would be placed on the downstream side of the weir to within one foot of the top of the weir. This would provide a gradual descent to the downstream river bed reducing the danger of drowning for canoers and swimmers. A low-flow crossing would also be constructed downstream to provide access to the intake facility. Using channel dimensions from the nearby Kathryn Dam project, the following cost estimate was developed.

Mobilization	**$ 5,000**
PZ22 Sheet Piling 600 linear feet @ $30/ft	**18,000**
Rock Rip Rap 150 cubic yard @ $30	**4,500**
Rock Rip Rap Filter	**1,000**
Low Water Crossing	**10,000**
Site Work	**4,000**
Total	**$42,500**

Intake Facility and Pumphouse

Figures II-6 and II-7 illustrate a potential intake and pumping facility. Two box culverts equipped with trash racks could be used to convey the water from the river into two pumping chambers. Two pumps would be used to lift the water from the chambers into the pipeline. The motor assembly would be kept above the 100-year flood plain and would be enclosed in an insulated metal building. The cost estimate is:

Mobilization	**$ 5,000**
Site Preparation	**5,000**
Piling (300 lf @ 33.33)	**10,000**
Concrete (50 cy @ 250)	**12,500**
Rebar (7500 lb @ 0.666)	**5,000**
Pump House	**12,500**
Precast Box Culvert	**30,000**
Electrical Hookup	**8,000**
Steel Hookup Piping	**6,000**
Rock Rip Rap (30 cy @ 33.33)	**1,000**
Total	**$95,000**

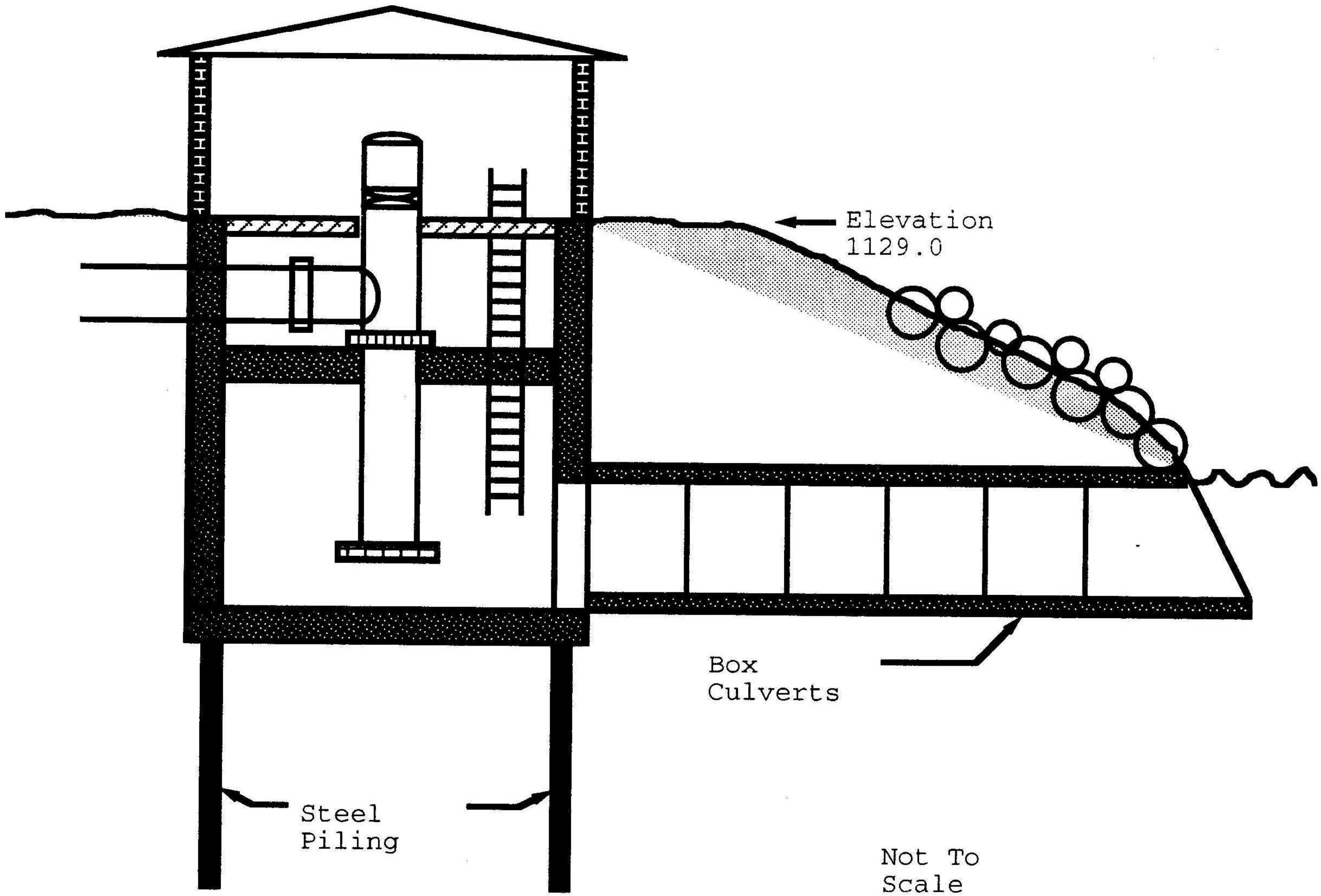

Fig. II-6. Conceptual diagram of pumphouse and intake structure for Sheyenne River artificial recharge.

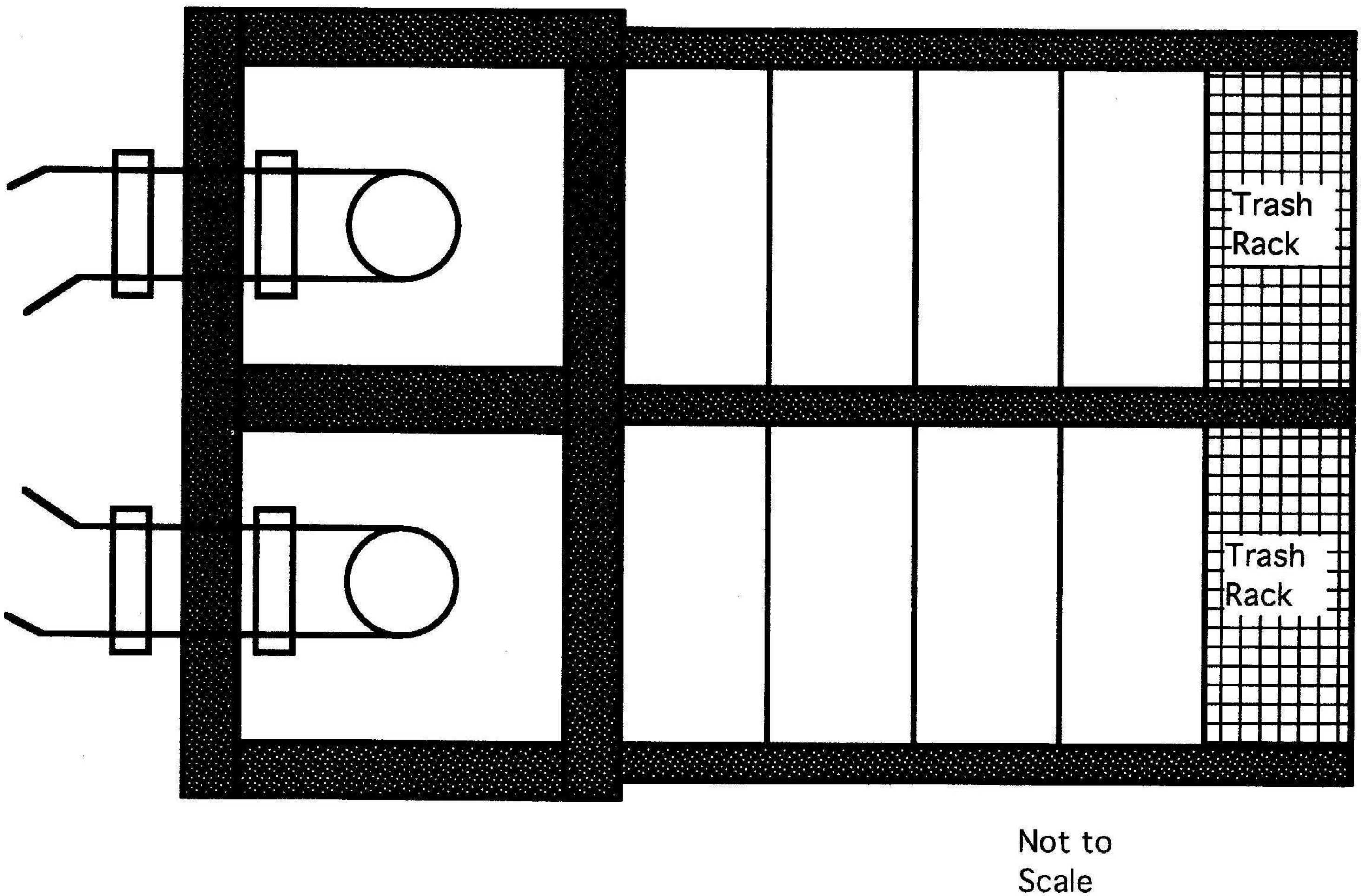

Fig. II-7 Conceptual diagram of pumphouse layout

Pumps

A cost estimate for the required pumps was obtained from a local supplier. Two 400 hp vertical turbine pumps were selected. The cost of each pump and motor assembly was estimated at $35,000, for a **total cost of $70,000.** The pump supplier estimated that the bowls may have to be replaced and the motors rewound after about 15 years at a cost of approximately $25,000 each or a total of $50,000. Some periodic replacement of bearings and shafts may also be necessary.

Right of Way

An estimate for the right of way cost of the conveyance system from the river to Highway #27 is also required. The right of way cost for the conveyance south of this point is included in the recharge facility cost estimate. Right of way costs are based on land value estimates obtained from the Ransom County Treasurer's office. Costs of $300/acre for permanent easements for the pipeline and $360/acre for title for the canal were used. It was assumed that a 100 ft strip would be required for the pipeline easement and that a 50 ft wide strip would be needed for the canal. Therefore, the cost of the pipeline easement would be $16,129, and the cost of the right of way for the canal would be $1,091. Total right of way cost for the conveyance system would be $17,220.

Summary of Costs

Following is a summary of the costs involved with the conveyance system.

Pipeline	**$1,035,000**
+ 10% appurtenances	**103,500**
Pump	**70,000**
Canal	**96,500**
Channel Dam	**42,500**
Intake & Pumphouse	**95,000**
Subtotal	**$1,442,500**
+ 11% legal and administration	**158,500**
+ 11% engineering	**158,500**
+ 11% contingencies	**158,500**
Right of Way	**17,000**
Total Capital Cost	**$1,935,000**

An **annual cost of $38,700 for power and a maintenance cost** will need to be included in the following economic analysis.

CONSTRUCTION AND MAINTENANCE OF AN ARTIFICIAL RECHARGE FACILITY

An infiltration basin facility with a maximum water supply rate of 15 cfs from the Sheyenne River is assumed for feasibility analysis. It is assumed that all basins will be constructed in the corners of irrigated lands and will be interconnected by canals that also serve as infiltration facilities. A diagram of potential recharge sites and their configuration is shown on Fig. II-8. It is assumed that for greatest efficiency, basins first constructed would be those closest to the first inlet point in the SE corner of Section 1, and in the NE corner of Section 12. The 0.5 mile of canal north of Highway #27 is predominantly in glacial till and is assumed to be clay lined. Further examination of this assumption, and the possibility of fracturing of the till should be examined before finalizing design. It is considered to be for conveyance alone, and should have no function as part of the infiltration facility.

All conveyance ditches south of Highway #27 are considered to be a part of the basin facility, with active surface infiltration area. Depths of conveyance ditches south of Highway #27 and basins are set at approximately two feet below land surface and are designed primarily as a portion of the water infiltration system. Conveyance ditches between basins are 12 feet wide at the bottom with 3:1 side slope. Only the 12 foot bottom area is included as infiltration surface. A total width of 50 feet is allocated for purchase. Land cost is $360.00 per acre for both basin and ditch.

General Operational Considerations

Artificial recharge with sediment laden water is highly complex, and infiltration can be impeded by many factors including: (1) sediment clogging, (2) solid and gaseous products of microbial respiration and metabolism, (3) dissolution of subbasin salts, or precipitation of salts from influent water, (4) dispersion of basin floor materials, (5) flocculation, (6) ion-exchange reactions, and (7) swelling of clays due to hydration.

To deal with these clogging problems many remedial methods have been developed including: (1) natural recovery through drying and cracking, (2) sediment removal or cleaning of the basin floor, (3) tillage, (4) flocculation and pretreatment of water, (5) basin depth control, (6) coarse media filtration and grass filtration, and (7) use of organic mat filters.

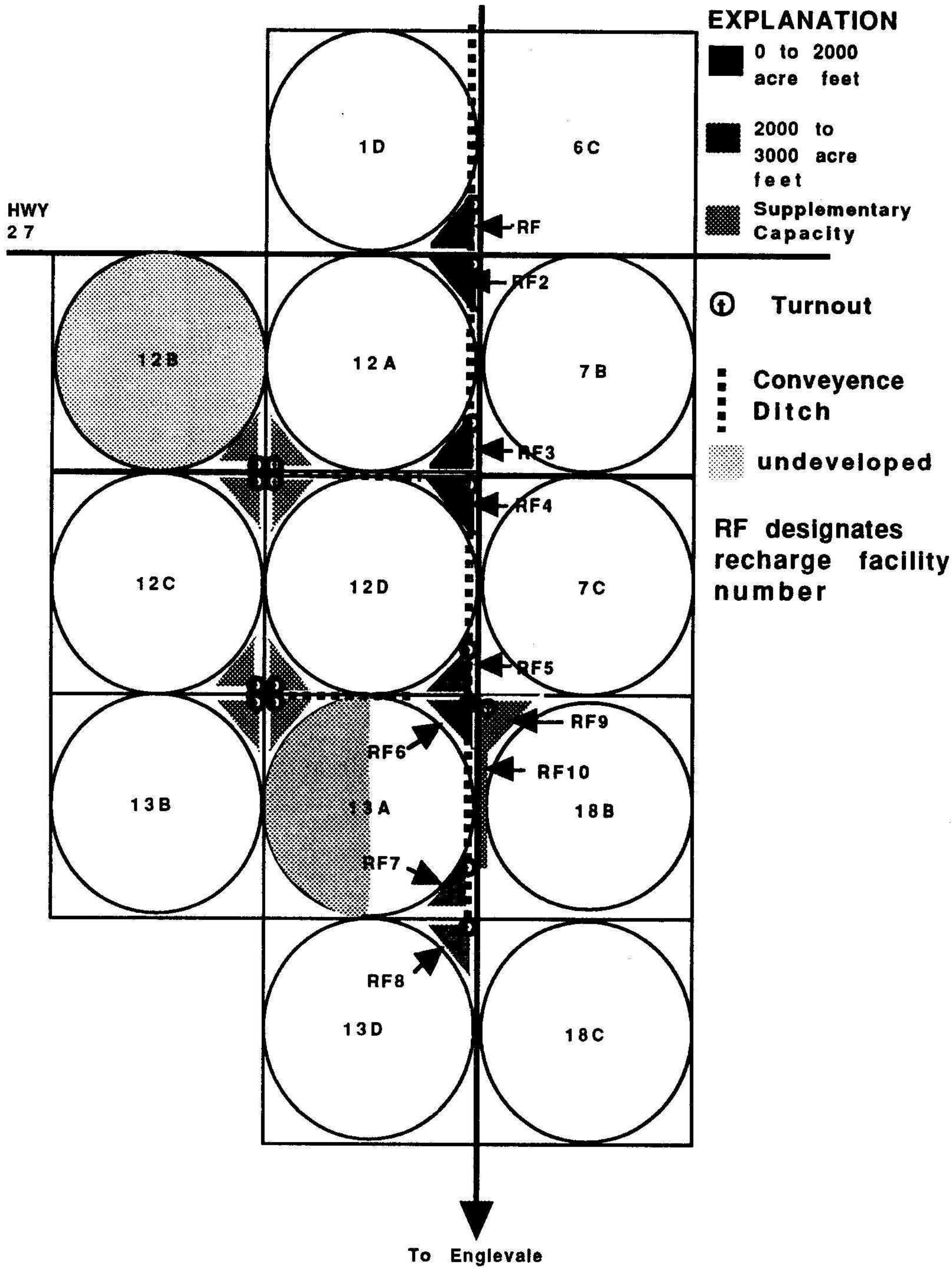

Fig. II-8. Location and layout of proposed artificial recharge facilities.

Clogging and potential remedial measures have been described and summarized by Schuh and Shaver (1988) for a test basin near Oakes, North Dakota and will not be discussed in detail in this study. A detailed operational management plan would include an analysis of specific management strategies and consideration of some remedial measures for basin clogging that might arise as a result of less common operational conditions. For this study, only sediment clogging and calcium carbonate cementation (the two primary clogging agents of the Oakes study) will be considered. The only remedial measure considered will be the standard procedure of cleaning the basin (i.e. removing the shallow layer of deposited sediment from the basin surface).

Basin Size and Operational Requirements

A basin operational plan consists of a series of cycles for each year which consist of alternating periods of infiltration and cleaning. Many factors affect the nature and duration of operational cycles. In turn, these operational cycles affect the number of acres required for operation of a recharge basin, the amount of time and expense involved in the operation of the basin, and the relative and total fixed and variable costs in the overall plan.

The following discussion of the effects of operational cycles will be based on an infiltration and clogging rate measured on the Oakes Test basin in the fall of 1986 (Schuh and Shaver, 1988). For a test basin with bottom soil ranging from fine-loamy sand to medium sand and operated with water from the James River having about 50 mg/l suspended solids, the change in infiltration rate over time could be described by:

$$i = 10.23\, t^{-0.59} \qquad \text{(II-4)}$$

where: i =infiltration rate (feet/day)

t = time (days).

The cumulative recharge over time for the Oakes basin was

$$I = 24.95\, t^{0.41} \qquad \text{(II-5)}$$

where: I =cumulative infiltration (feet).

Cleaning between basin operations consists of allowing the basin floor to dry and then performing a shallow cleaning with a blade to a depth of about one inch. The time needed to clean the basin partially depends on how long it takes for the residual water in the basin to drain (Fig. II-9). The time needed for the ponded water to fully infiltrate into the soil and for drying to begin ranges from only about a half-day for a two day operational period (final i = 6.8 feet per day (ft./d)) to as much as 4.5 days for a 60 day operational period (final i=0.91 ft./d). These calculations are based on an estimated basin head of one foot (the midpoint estimate for the initial head of two feet, and a final head of 0 feet.) Following the cessation of ponding, an additional period of five days is allowed for the soil to dry and for the cleaning operations to be performed. The final estimate for days of maintenance per basin is also shown in Fig. II-9, and varies from a minimum of 6 days to a total of 9.5 days.

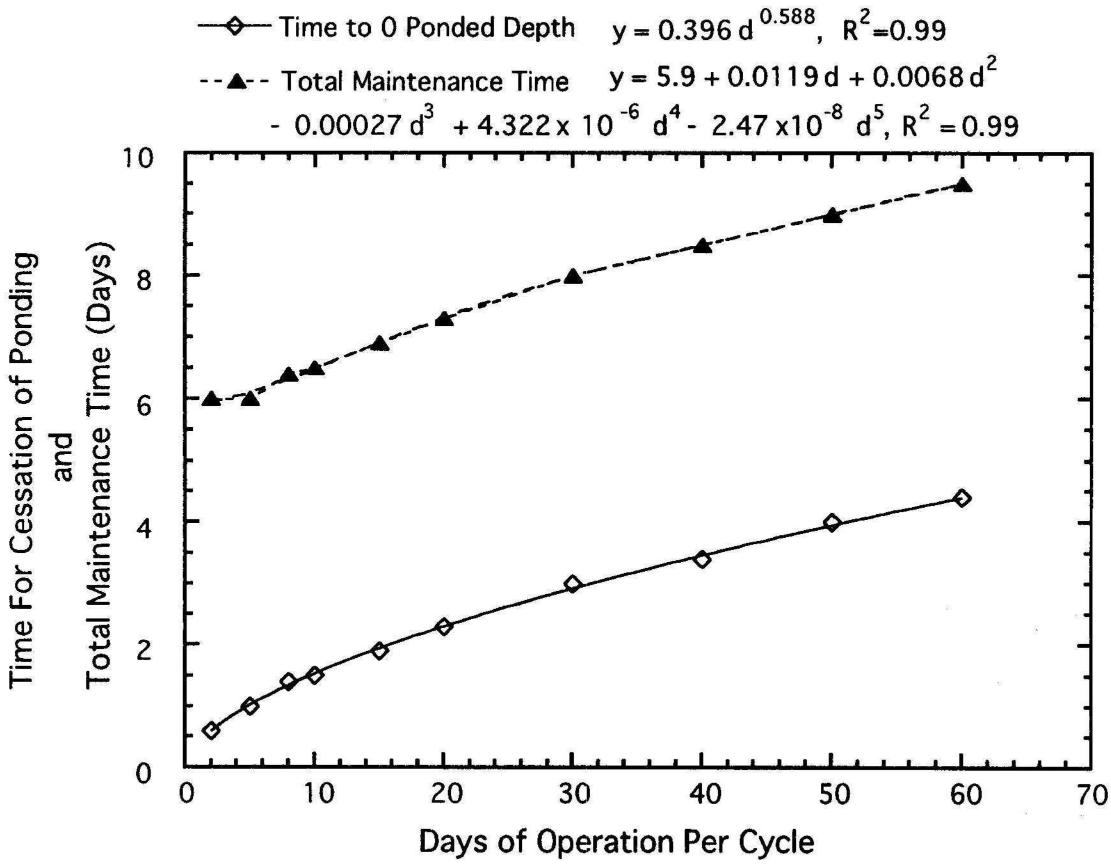

Fig. II-9. Time needed for a basin having a two-foot initial ponded depth to fully drain following cessation of delivery of water, and total maintenance time for each basin cycle including drainage time and cleaning time. y = time for cessation of ponding, and d = number of days per operational cycle.

For the design operational period (105 days, including 60 days in spring and 45 days in fall) a limited number of operational cycles can be performed. Cleaning time is subtracted from actual operational time. However, in general the shorter cycles require more cycles of cleaning and therefore have more down time. This is shown on Fig. II-10. Cleanings vary from as many as 14 per year for two days of operation per cycle to as few as two for 60 days of operation per cycle.

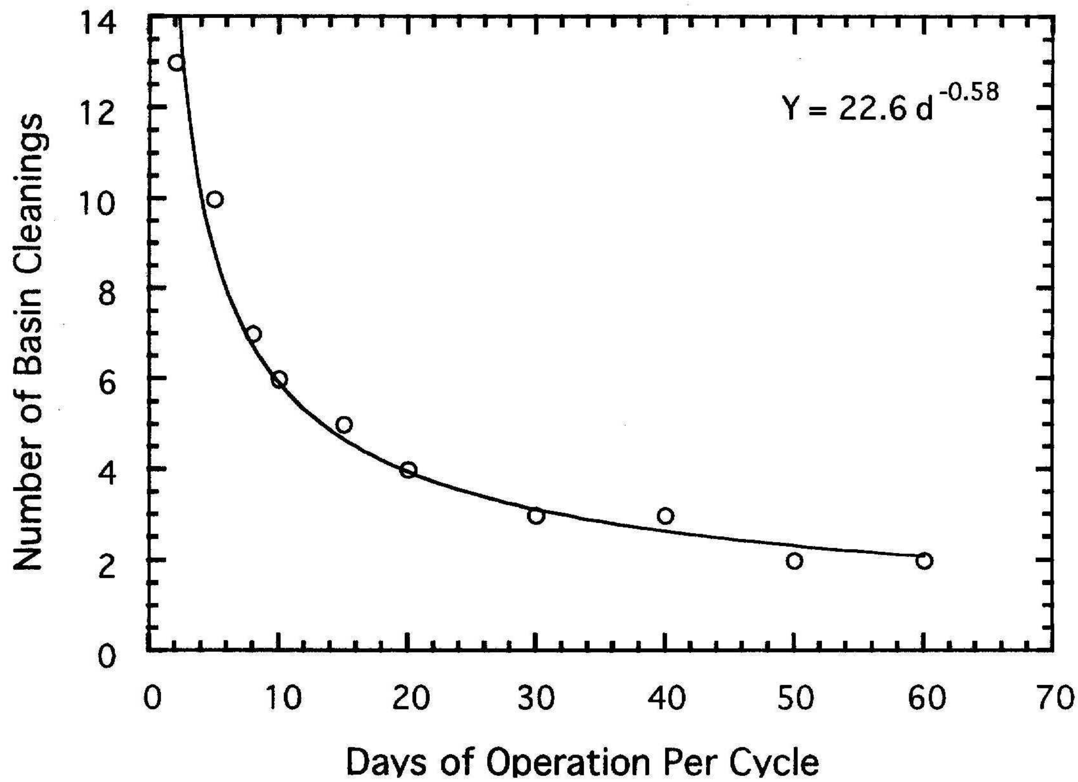

Fig. II-10. Number of basin cleanings per 105 days of operation as dependent on the length of operational cycles. Y = number of basin cleanings; d = days of operation per cycle.

Without accounting for cleaning time, estimates of average infiltration rate are made by calculating I using eq. II-5 for the final day of operation per cycle, and then dividing by the operational days per cycle. Average i for each operational cycle is shown on Fig. II-11. Average i varies from as much as 16 ft./d for 2 operational days per cycle, to as little as 3 ft./d for 60 operational days per cycle. In general, average infiltration for shorter operational periods is considerably larger than for longer periods. However, shorter operational periods require longer total cleaning times. The overall recharge rate adjusted for cleaning time varies much less with cycling time (Fig. II-11). Nonetheless, the shorter time periods remain the most productive in terms of recharge rate.

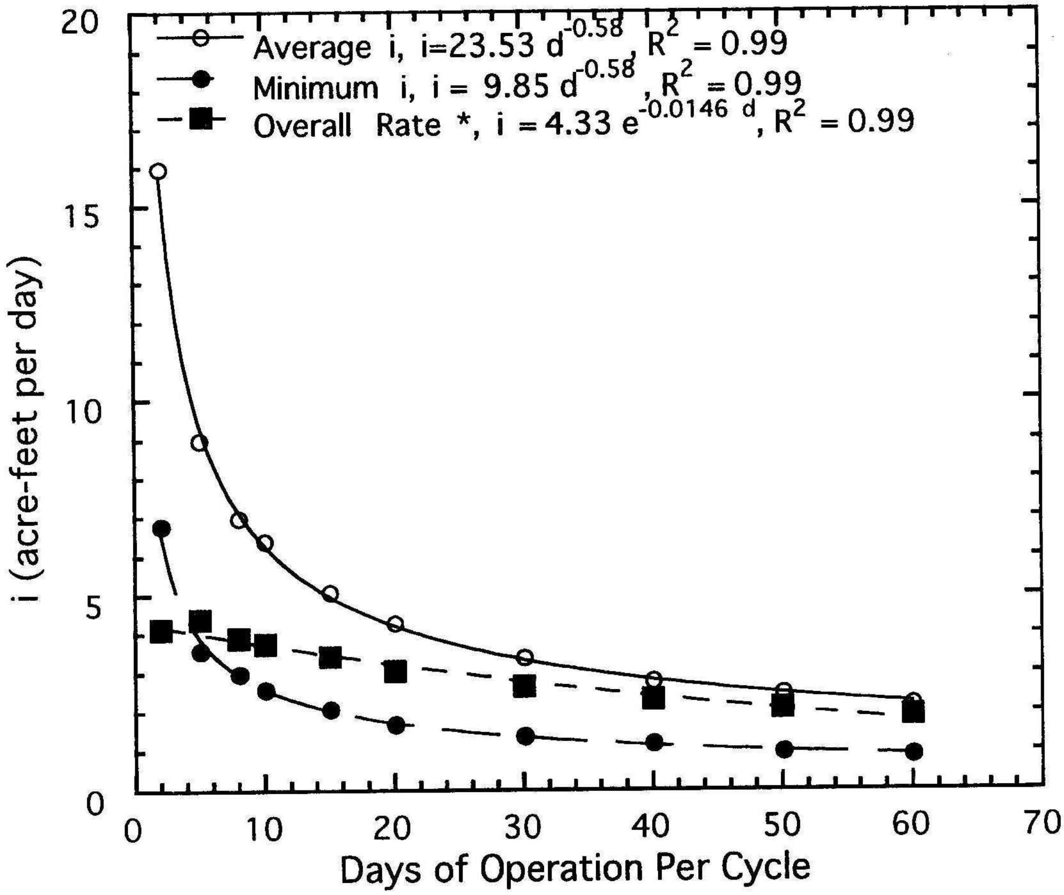

Fig. II-11. Average infiltration rate for an artificial recharge basin for different lengths of operational cycle, based on measurements made in 1986 on a test basin near Oakes, North Dakota using turbid water from the James River. The "Minimum" infiltration rate value is the rate measured at the end (most highly clogged) part of the basin operation. The "Overall rate" is the average rate adjusted to include cleaning and down time during a full 60 day operational period.

Total required basin area under optimal pumping conditions is illustrated on Fig. II-12 for 1,800, 2,500, and 3,000 acre-feet in 105 days of pumping. Assuming that water can be delivered to the recharge basin at the maximum infilitration rate and maximum flexibility of operation were achieved, then a maximum of 14 acres would be required to recharge 3,000 acre-feet under the least intensive operational scheme (60 day operational cycles and two cleanings per year). However, this would require the availability of as much as 75 cfs from the river at certain times and the ability to convey it to the basin. The expense of pipe, pumping facilities, and the limitations of the river water supply make such operational schemes impractical.

The maximum capacity of 15 cfs in the proposed conveyance system results in an increased land requirement for the basin. There will be some days in which less than 15 cfs is available from the river during which pumping will not be possible. Therefore, it is important that the facility always be able to receive waters at maximum rate in order to make up possible deficits. **For this reason, the total infiltration capacity of the basin should always be at least equal to the 15 cfs conveyance capacity of the delivery system.**

The amount of required infiltration area varies considerably with the choice of operational management cycles even under the maximum delivery constraint. In Fig. II-12, required acreage for a basin designed to receive 15 cfs under all conditions varies from as little as 5 acres for the two day operational cycle to 33 acres for the 60 day cycle. In evaluating appropriate basin size, both land costs and operational costs must be considered.

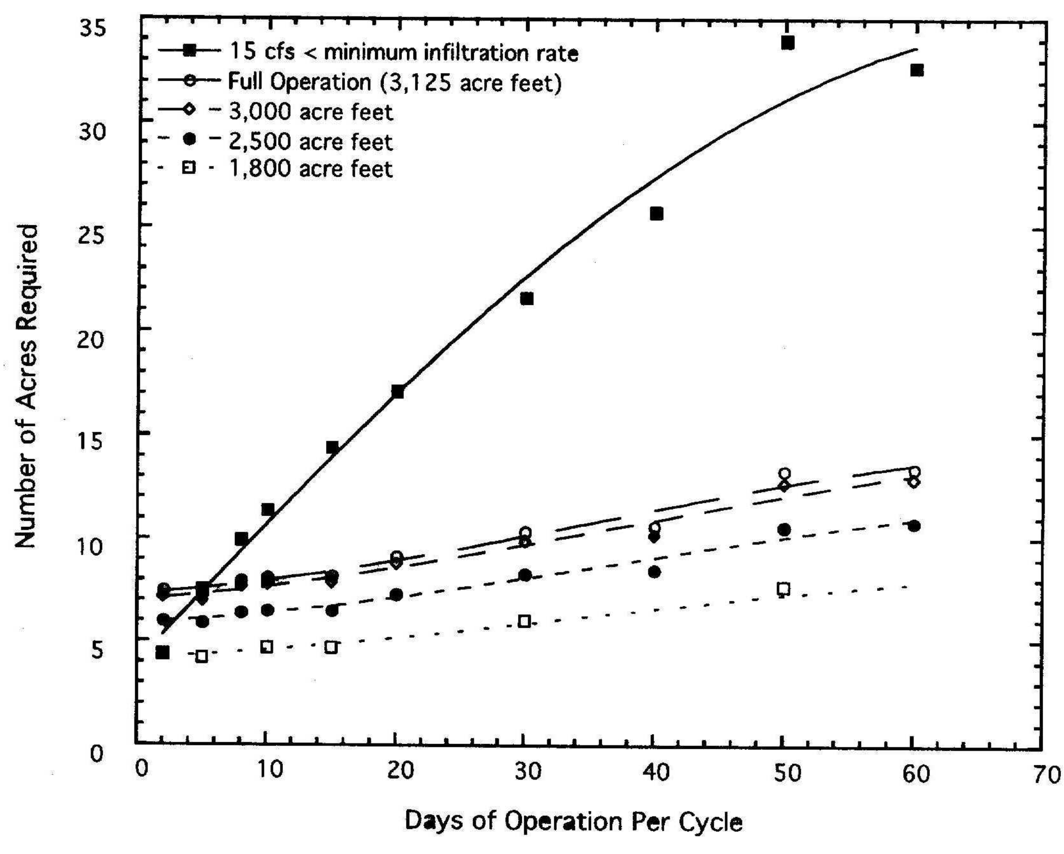

Fig. II-12. Number of acres of infiltration surface required to recharge 1,800, 2,500, and 3,000 acre-feet per year, and the number of acres required to assure that the recharge facility will be able to receive and recharge the full 15 cfs from the river at the most limiting time (with minimal average infiltration rate), for each management operational cycle (adjusted for cleaning time).

Land Cost versus Operational Cost

Both fixed (land and equipment) and variable costs should be considered in an evaluation of basin size allotment. In addition, convenience and risk should be included in an evaluation of appropriate basin size. Basin cleanings (Fig. II-10) vary considerably with operational plans and each cleaning requires the maintenance and operation of suitable equipment. An estimate of cleaning costs was based on the estimated mean contract costs of four different scrapers contracted by the SWC construction division. The estimated cleaning rate for a 1-inch shaving was about 1 acre per hour. Estimated cost is $48 per acre per operation. A similar estimate of about $50 per hour for a land leveler was obtained from NDSU Extension contract rate tables (Haugen and Aakre, 1991).

Required basin area was estimated using the 15 cfs operational curve for each basin operational cycle. The cost per cleaning was then estimated by multiplying basin acres times the $48 per acre. Cost per cleaning was multiplied by the number of required cleanings for each operational cycle to arrive at annual variable cost. Variable (operational) costs were compared

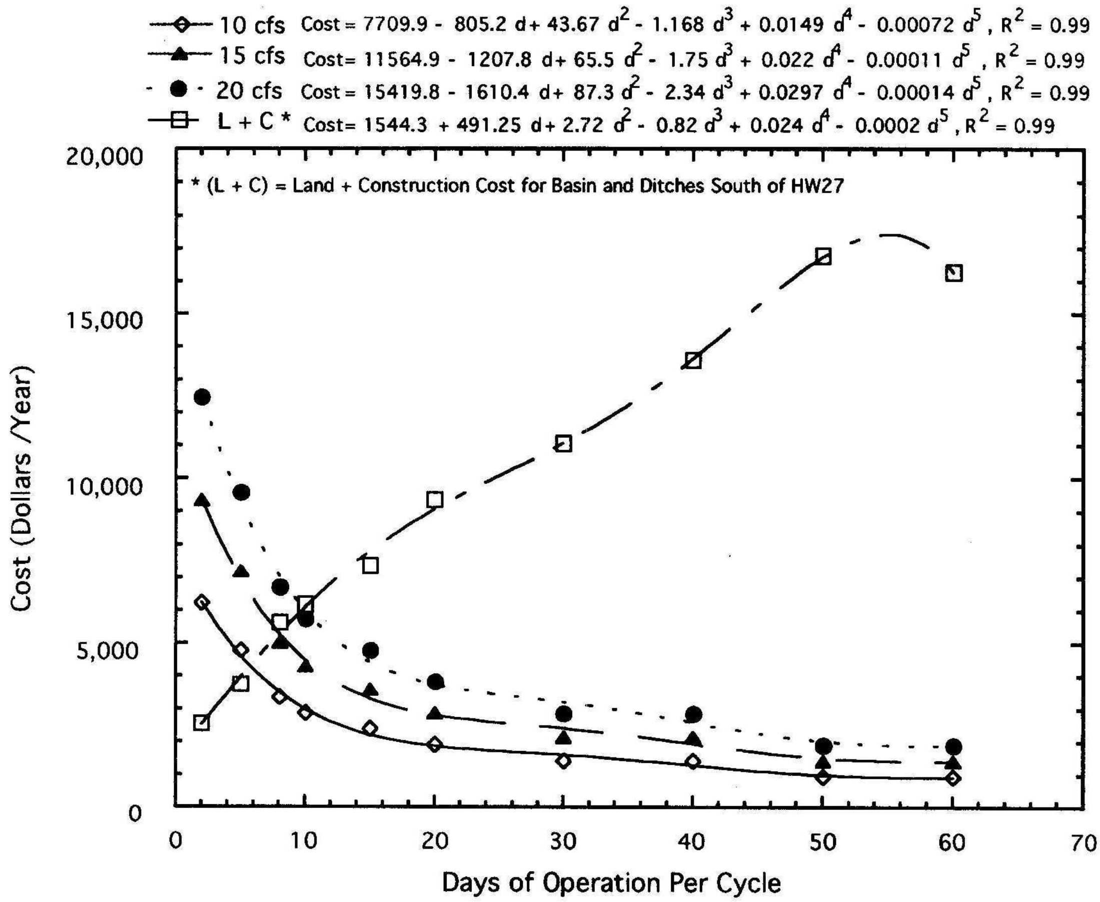

Fig. II-13. Comparison of maintenance cost at 10,15, and 20 cfs pumping rates, and total land + construction costs (L+C) for basin and infiltration ditches, as required for varying operational periods.

with the annual fixed land cost which was calculated using an estimated cost of $360.00 per acre (the mean value for 6 townships in the vicinity of the basin). Land cost was amortized over 30 years at a rate of 8 percent. Annual land cost and annual operational costs for various operational cycles are shown in Fig. II-13. Results indicate that land cost and operational cost were approximately equal for the 10 day operational cycle.

With longer cycling periods, land costs increased faster than operational costs decreased. To find the most cost efficient cycling scheme, annual land and operational costs were summed and plotted versus operational cycle (Fig. II-14). The least cost is incurred with the 10-day operational cycle (which requires approximately 11 acres of basin surface). Little additional cost is incurred using the 20 day cycle which requires approximately 18 acres of basin surface. While it would seem that 10 to 18 acres would be the best choice, other factors including risk, flexibility of operation, and convenience should also be considered in the final selection of the basin size.

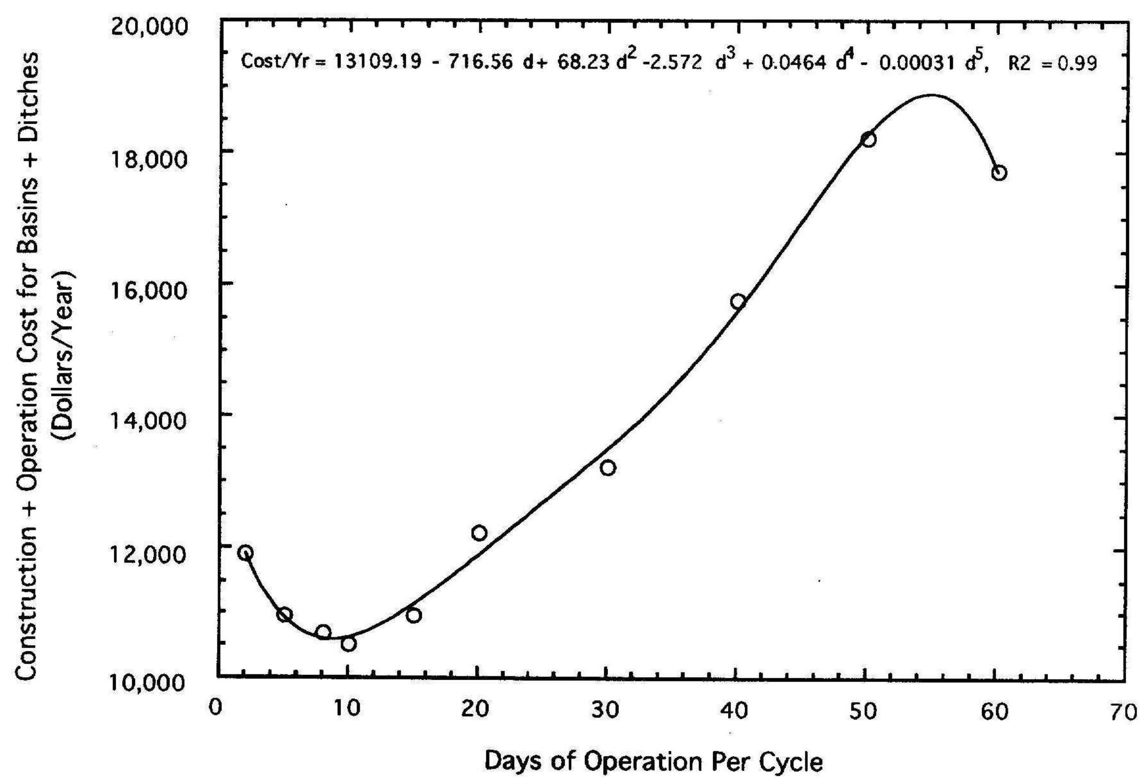

Fig. II-14. Total annual cost as a function of days per operational cycle.

Risk and Flexibility of Operation

Risk and flexibility of operation are important considerations in assessing proper recharge basin sizes. Risk factors include the following uncertainties: (1) errors in estimation of actual clogging rates; (2) weather conditions, and equipment and labor availability at critical times for cleaning; and (3) the possibility of gradual degradation of basin capabilities. Uncertainties caused by possible variations in river water availability have already been considered in the basin design criteria for the 15 cfs conveyance system.

(1) Basin size estimates are based on experiments on similar soil materials and water of similar chemistry which were conducted about 40 miles from the proposed site. Also, curve fitting procedures for estimation of long-term (60 day) infiltration rates will include some error. Effects of these errors could be offset by switching to an operational cycle with a shorter period. However, to have this option requires that longer periods be planned into the initial design.

(2) Uncertain weather conditions are an important consideration. Approximately 5 days have been allotted for drying and cleaning following the cessation of ponding in the basin. Frequent heavy rains could prevent cleaning and seriously inhibit total recharge capability in some years. The more cleaning periods required, the larger the risk incurred. The best strategy to offset this risk is to design a basin for longer operational cycles that requires fewer cleanings.

Uncertainty in the scheduling of contractors, the availability of local labor (if the irrigation district chooses to undertake its own maintenance), and the possibility of equipment breakdowns could impair the performance of cleaning operations at critical times. Impact of malfunctioning equipment or of other timing factors can be minimized by conservative design requiring fewer cleanings.

(3) Despite cleaning of the basin surface, it is possible some degree of deterioration of infiltration rate could result from gradual accumulations of clay particles that move deeply into the soil profile. Cleaning and removal could be expensive. Effects of deterioration could be partially offset by moving to shorter operational cycles. To insure this flexibility, a longer operational period would be required in the initial design.

Convenience is another factor worth considering. Short operation cycles require considerable attention and intensive management. This might be inconvenient during spring planting operations or fall harvest operations. Moreover, intensive management might require the hiring of a manager for part of the year which would further increase operational costs. The dollar value of convenience is difficult to assess and can only be evaluated by the operators.

Approximate Construction and Maintenance Cost

Table 11-15 includes the simulated total basin fixed (land + construction) and maintenance cost for different basin sizes based on operational schemes previously discussed. Estimated construction cost is based on a design following the proposed layout shown in Fig. II-8, beginning at Highway #27 and expanding southward as required. Infiltration area includes both canal and basin surface. For simplicity, it is assumed that all canals and basins will have about a two foot depth below land surface with canal gradients following land surface slopes. To have an adequate canal gradient to maintain flow in the canals and prevent over-topping of the canal given some undulation in the topography, the canals and basins may need to be deeper in some places. With an approximate width of 12 feet on the canal bottom and 3 to 1 side slopes, the canal width will be 24 feet. A total width of 50 feet including spoil and access is assumed. Land purchase or easement cost is set at $360.00 per acre. Construction cost includes land purchase and excavation costs at $1.25 per cubic yard. Construction and total cost also include 33 percent for legal and administrative cost, engineering cost, and contingency. For construction and land purchase, 8 percent interest amortized over a period of 30 years is assumed.

Three cost options will be considered in the total cost analysis. The first option will be for the 20-day operational cycle (Fig. II-12) which assumes 17.1 acres of infiltration area. The second option is the 30-day operational cycle (21.6 acres); and the third option is the 60-day (maximum capacity) cycle (32.7 acres). Only canals south of Highway # 27 are included as infiltration surface. Fixed basin cost is

FIXED BASIN COST

Option 1 (17.1 acres) Construction + 33% = $141,000
Option 2 (21.6 acres) Construction + 33% = $166,000
Option 3 (32.7 acres) Construction + 33% = $244,000

Maintenance cost for the three options is considered on an annual basis.

ANNUAL BASIN MAINTENANCE COST

Option 1 (20 days per cycle) Maintenance Cost = $3,200 per year
Option 2 (30 days per cycle) Maintenance Cost = $2,400 per year
Option 3 (60 days per cycle) Maintenance Cost = $1,600 per year

The difference in total cost per year for basin and conveyance between Option 1 and Option 3 is $7,626.00 per year for the entire facility(Table II-15). It would seem likely that option 3 would be the most beneficial option to pursue because it provides the most flexibility and requires the least attention.

Table II-15. Estimated cost for construction and operation of an artificial recharge basin based on operational and maintenance cycles required to allow 15 cfs delivery to the basin at all times. Fixed cost includes both land and construction cost. Fixed cost is over 30 years at 8% interest.

Basin infiltration area (acres)	Canal infiltration area (acres)	Total infiltration area (acres)	Fixed Cost + 33% (dollars)	Maintenance cost per year (dollars)	Construction cost per year (dollars)	Total cost per year dollars
4.37	0.00	4.37	38,389	10,390	3,409	13,799
7.53	0.00	7.53	56,487	7,992	5,017	13,009
9.92	0.75	10.67	84,794	5,594	7,532	13,126
11.36	0.75	12.05	93,037	4,795	8,264	13,059
14.40	0.75	15.15	110,435	3,996	9,810	13,805
17.10	1.50	18.60	140,505	3,196	12,480	15,676
21.60	1.50	23.10	166,252	2,397	14,767	17,164
25.70	2.25	27.95	204,333	2,397	18,150	20,547
34.00	2.25	36.25	251,792	1,598	22,366	23,964
32.70	2.25	34.95	244,347	1,598	21,704	23,302

ENGLEVALE AREA SOIL SUITABILITY FOR IRRIGATION

Much of the Englevale area land is suitable or conditionally suitable for irrigation. Land in the outwash plain is mostly of the Renshaw and Sioux series. While the Sioux series is classified as nonirrigable because of coarseness and large relief, the Renshaw is irrigable, and has few limitations (USDA-SCS, 1982). For the Renshaw soil a maximum ECE of 3,000 and a maximum SAR of 9 are recommended. This is well within the range of both aquifer and Sheyenne River water properties. Soils formed in glacial till in the Englevale area are primarily of the Barnes and Svea series. Both are classified as conditionally irrigable (USDA-SCS, 1982). The required conditions (ECE < 1800, SAR < 6, and 1/2 inch leaching in fall to prevent salt buildup in the soil profile) are easily met within the water quality distributions of the Englevale aquifer and the Sheyenne River. Fig. II-15 provides an approximate map of irrigable and conditionally irrigable land near Englevale.

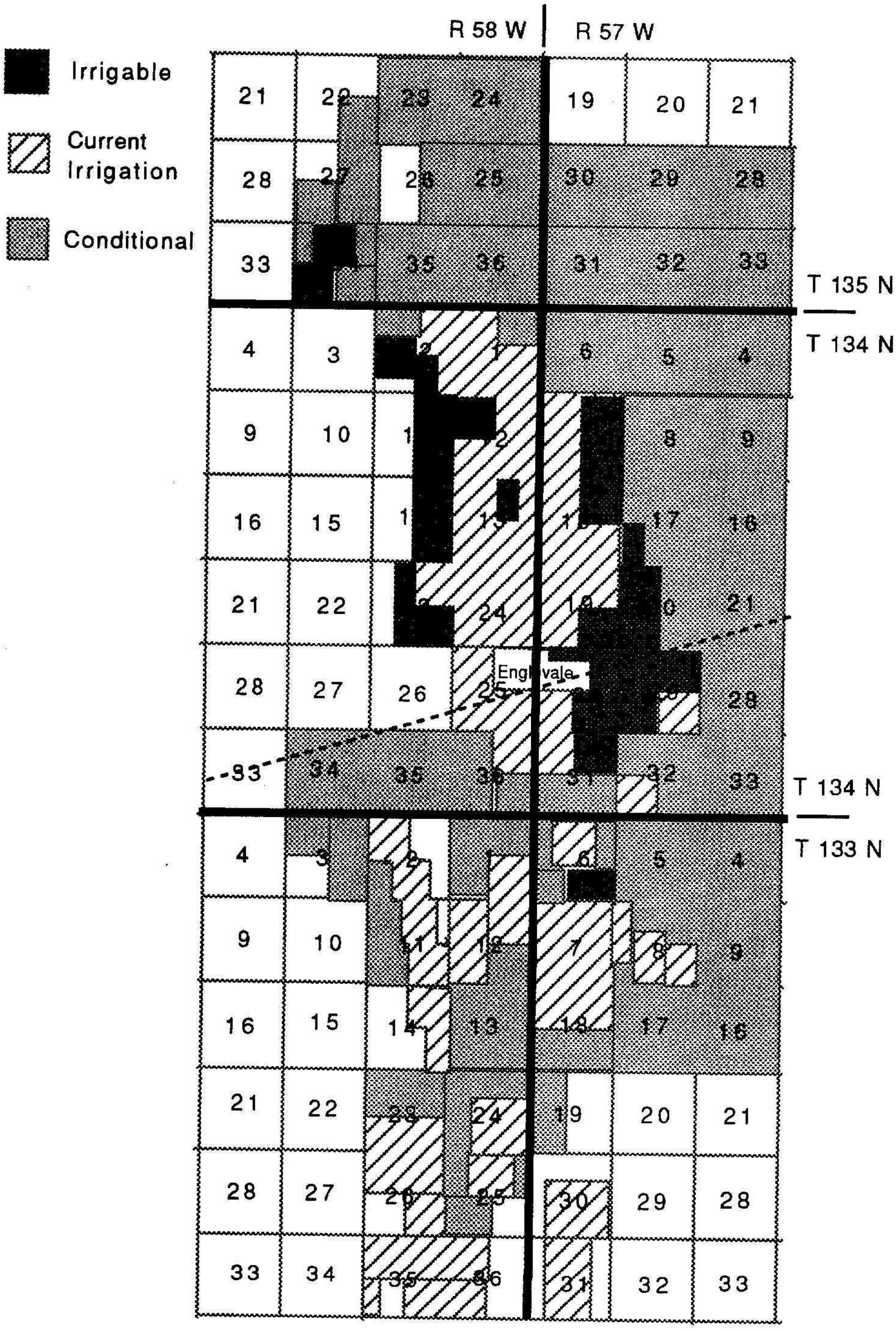

Fig. II-15. Location of potential irrigable land near Englevale.

WATER DISTRIBUTION SYSTEM

Recharge waters must be pumped from the Englevale aquifer to appropriate fields. This requires a well field and a distribution system. Because of existing well locations and problems with shallow well placement discussed in Part I, well field placement must be highly selective and limited to the deeper portions of the aquifer. It will seldom be possible to place a well directly beneath the field of use.

The Englevale aquifer model discussed in Part 1 was used to test the feasibility of adding 22 wells to the northern part of the Englevale aquifer, each assumed to supply one quarter section, to simulate the effects of pumping recharge waters from the Englevale Aquifer. The study indicates that the development of additional wells to pump recharged water is possible without threatening existing water rights, though further studies of the hydrogeology would be required before actual design of the well field. The proposed well field is located in an approximate north to south linear pattern from Section 1 to Section 25 of T. 134 N. R. 58 W. The proposed distribution of the well field is shown in Fig. II-16.

For feasibility analysis, the same configuration of one well per quarter section is proposed. Average water use of 12 inches per year is assumed. Acreage options evaluated are 1,800, 2,500, and 3,000 acres. Using 130 acres per quarter section, this converts to 14 quarter sections (and wells), 19 quarter sections (and wells), and 23 quarter sections (and wells), respectively. The wells are not placed on the irrigated quarter sections.

Although there is considerable irrigable land in the Englevale area, it is assumed that irrigated lands will be efficiently located with respect to the project facility. Two groups of irrigable land were selected based on proximity to the proposed well field from the potentially irrigable land in Englevale area(Fig. II-15). The selection criteria were: (1) distance from the well field to the center of the irrigable quarter section, and (2) the well spacing required to minimize interference effects between wells. Group 1 consists of fields located within one mile of the well field. Group 2 consists of fields located within 2.5 miles of the well field. Fields selected are shown on Fig. II-16.

However, it is stressed that the specific irrigation tracts are selected for purposes of feasibility analysis alone. Other options and configurations could be arranged to accommodate local land use practices and requirements. It will be assumed that all Group 1 lands will be irrigated before Group 2 lands are irrigated.

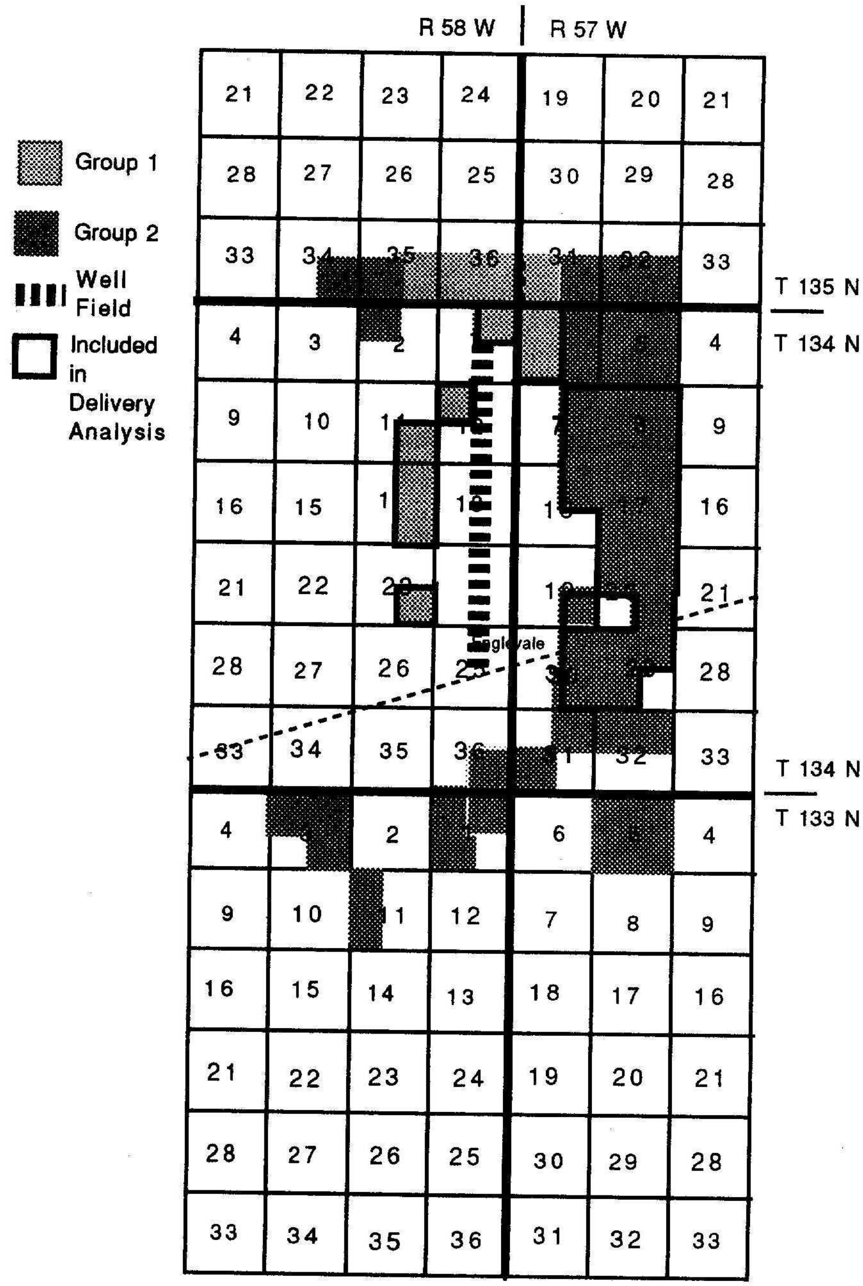

Fig. II-16. Approximate proposed well-field locations, and illustration of land groups used for distribution feasibility analysis.

Well Field

Each well is assumed to be constructed for an average aquifer saturated thickness of 35 ft. and a water-table depth of 30 ft. below land surface. Each well will consist of 11.8 feet of 12-inch diameter #80 slot stainless steel screen ($1050 per screen) and 53.5 feet of 12-inch steel casing ($21 per foot). A 75 hp pump with a lift of 60 feet and an output of 800 gpm is assumed ($11,000 per pump). Drilling cost is assumed to be about $30/ft. Development cost is based on one hour of development per foot of well screen at $75 per hour, and $500 is allocated for well testing.

WELL FIELD COST

Pump cost/well	$11,000
Well screen cost/well	1,000
Casing cost/well	1,100
Drilling cost /well	2,000
Development cost/well	900
Well testing	500
Total cost/well	$16,500
+ 11% legal and administration	1,800
+ 11% engineering	1,800
+ 11% contingencies	1,800
Total + contingency per well	$21,900

Total cost 14 wells	(1,800 acre ft)	$307,000
Total cost 19 wells	(2,500 acre ft)	416,000
Total cost 23 wells	(3,000 acre ft)	504,000

Distribution System

The well field will be located along an approximate north-south transect near the centers of Sections 1, 12, 13, 24, and 25 all of T. 134 N., R. 58 W. A preliminary assessment of each quarter section's potential for irrigation development was conducted. Based on this assessment, it was estimated that the first 8 quarters to be developed would be located 0.5 mile from the well field, and that the next 20 quarters developed would average 1.75 miles from the well field.

Assuming 900 gpm per system and using a 10-inch diameter pipe results in a velocity of 3.7 fps (using the nominal diameter). The cost of 10-inch, 100 psi, PVC irrigation pipe was quoted by a local supplier at $2.50 per foot. Another local irrigation supplier provided the same cost estimate and also estimated the cost of installation as $1.00 per foot. Therefore, the cost for the first 8 pivots would be $3.50 per foot for 3,960 feet (0.5 mile plus 0.25 mile from the edge of the quarter to the center) or $13,860 per pivot.

The cost for the next 20 pivots would be $3.50 per foot for 10,560 feet (1.75 miles plus 0.25 mile from the edge of the quarter to the center) or $36,960 per pivot.

Some money might be saved by manifolding wells and systems together to share a common transmission line from the well field. For the first 19 pivots located 0.5 mile from the well field, if two systems were manifolded together there would be 10.5 mile of 14" pipe ($4.50 per foot) which would convey the water to a corner common to the two quarters being irrigated. Each quarter would then have 1,867 feet of 10-inch pipe from the corner to the center of the quarters. Assuming the cost of installation for the 14-inch pipe at $1.50 per foot the cost can be calculated as follows:

($4.50 + $1.50)(2640ft) + 2($2.50 + $1.00)(1867ft) = $28,909 for two pivots
or $14,454 per quarter.

Therefore little money would be saved by manifolding two systems together for these closer systems.

If two systems were manifolded together for the systems located farther from the well field, there would be 1.75 miles of 14-inch pipe and two segments of 10-inch pipe each 1,867 ft long. The cost would be:

($4.50 + $1.50)(9240ft) + (2)($2.50 + $1.00)(1867ft) = $68,509
or $34,255 per system.

Potential for cost savings by manifolding delivery systems is most promising for the systems located farther from the well field. If four systems were manifolded together, an 18-inch main transmission line would result in a velocity of 4.5 fps using the nominal diameter. The cost of this pipe is \$9.21 per foot. There would be 1.75 miles of 18-inch pipe, with an assumed installation cost of \$2.00 per foot. There would also be four segments of 10-inch pipe, each 1,867 feet long. The cost would be:

(\$9.21 + \$2.00)(9240ft) + (4)(\$2.50 + \$1.00)(1867ft) = \$129,718 or \$32,430 per system.

Based on these estimates, it appears that minimal savings would be achieved by manifolding the delivery systems together. The costs per quarter are approximately \$14,000 per quarter for the first 8 quarters located 0.5 mile from the well field and approximately \$35,000 per quarter for the 20 quarters averaging 1.75 miles from the well field.

TOTAL DELIVERY COST

14 quarter sections	**\$322,000**
19 quarter sections	**\$497,000**
23 quarter sections	**\$637,000**

Additional consideration of 33% for legal and administrative cost, engineering cost, and contingency cost results in

ADJUSTED TOTAL DELIVERY COST

14 quarter sections	**\$428,000**
19 quarter sections	**\$661,000**
23 quarter sections	**\$847,000**

TOTAL PROJECT COST

A total project construction and operation cost is projected based on the cost factors presented in this feasibility study. Annual cost is based on interest (i) at 8%, and payments amortized over n years, where n=30 according to the equation

$$\text{cost/y} = \text{construction cost}\ [\ i(1+i)^n/((1+i)^n - 1)] + \text{annual operational cost} \quad \text{(II-6)}$$

TOTAL CONSTRUCTION COST

	14 quarter sections	19 quarter sections	23 quarter sections
WELL FIELD CONSTRUCTION	$307,000	$416,000	$504,000
WATER DISTRIBUTION SYSTEM	$428,000	$661,000	$847,000
CONSTRUCTION OF CONVEYANCE FACILITY	$1,935,000	$1,935,000	$1,935,000
CONSTRUCTION OF INFILTRATION FACILITY (32.7 ACRE OPTION)	$244,000	$244,000	$244,000
TOTAL COST AT FIELD	$2,914,000	$3,256,000	$3,530,000

CONSTRUCTION COST PER YEAR

Amortized Cost Per Year	14 quarter sections	19 quarter sections	23 quarter sections
8% interest, 30 year amortization	$259,000 ($142/acre)	$289,000 ($117/acre)	$314,000 ($105/acre)
8% interest, 40 year amortization	$244,000 ($134/acre)	$273,000 ($111/acre)	$296,000 ($99/acre)

OPERATIONAL COST PER YEAR

Pumping Cost/year	$38,700
Basin maintenance cost/year	1,600
Pumping station maintenance cost/year	1,800
Sampling + monitoring	1,500
Total Operational Cost / Year	$43,600

TOTAL COST PER YEAR

Total Construction + Operation Cost Per Year	14 quarter sections	19 quarter sections	23 quarter sections
8% interest, 30 year amortization	$302,000 ($166 /acre)	$333,000 ($135 /acre)	$357,000 ($119 /acre)
8% interest, 40 year amortization	$288,000 ($158 /acre)	$317,000 ($128 /acre)	$340,000 ($114 /acre)

The total cost includes the cost of water recharged to the aquifer and of the facilities to convey water from the aquifer to the field. However, the cost to pump water from the aquifer to the field is not included nor is the cost of the center pivot irrigation system. Also not considered are the maintenance costs for the well field which would include pump column and bowl replacement, pump repair, and well rehabilitation. Assuming maintenance cost of $4000 per well every 10 years, annual maintenance costs per well would be $240 and $260 respectively for 30 and 40 year amortization periods. Annual well maintenance costs would be less than $2 per acre. These cost must be considered as a part of each fields operational cost.

The recharge facility cost figures are conservative. In some cases, engineering and design cost, and legal and administrative cost might be avoidable. For example, basin excavation and delivery system preparation might be avoidable to a large degree. If costs for basin and distribution construction are adjusted to include only 11% (contingency) safety margin, compared with the 33% margin, the adjusted cost savings per year would approximately $ 5.00 per acre. This is shown in the figures included below.

Construction Cost Per Year Using 11% Design, Administrative, and Contingency Margin For Basin and Distribution System Design and Construction Instead of 33% Margin

TOTAL COST AT FIELD

(14 quarter sections)	**$2,803,000**
(19 quarter sections)	**$3,107,000**
(23 quarter sections)	**$3,350,000**

Amortized Cost Per Year	14 quarter sections	19 quarter sections	23 quarter sections
8% interest, 30 year amortization	$249,000 ($137 /acre)	$276,000 ($112 /acre)	$298,000 ($100 /acre)
8% interest, 40 year amortization	$235,000 ($129 /acre)	$261,000 ($106 /acre)	$281,000 ($94 /acre)

Operational Cost per Year

Pumping Cost/year	$38,700
Basin maintenance cost/year	1,600
Puming station maintenance cost/year	1,800
Sampling + monitoring	1,500
Total Operational Cost / Year	$43,600

Total Cost per Year

Total Construction + Operation Cost Per Year	14 quarter sections	19 quarter sections	23 quarter sections
8% interest, 30 year amortization	$293,000 ($161 /acre)	$320,000 ($129 /acre))	$341,000 ($114 /acre)
8% interest, 40 year amortization	$279,000 ($153 /acre)	$304,000 ($123/acre)	$325,000 ($109 /acre)

WATER CONSERVATION EFFECTS ON TOTAL COST

In recent years, significant advances in efficiency of irrigation power and water use have been made. New approaches include: (1) deficit irrigation, (2) low pressure drop nozzle irrigation, and (3) low energy precision application (LEPA) systems. Adoption of these practices could considerably alter the cost effectiveness of an artificial recharge project.

Deficit irrigation involves the early filling of the soil profile to field capacity, then irrigating at a reduced rate to allow a gradual depletion of the water in the soil profile during the growing season so that minimal leaching water loss occurs. Deficit irrigation has been used beneficially on many crops. Special attention must be given to avoid stressing crops at critical times. Because of low soil moisture storage, extremely coarse sands and gravelly soils might prove to be somewhat more risky and less forgiving of errors on crops that are extremely sensitive to moisture stress. Potatoes, for example, might be somewhat difficult to deficit irrigate on very coarse soils. However, deficit irrigation has been successfully used on beans in the Oakes area (Dr. Earl Stegman, personal communication, February 14, 1992). For some crops and situations, deficit irrigation practices would be worth considering as a conservation practice.

Low Pressure Drop Nozzle systems have been used to avoid wind and evaporative loss by forming larger water droplets and placing the water closer to the crop canopy (and soil). LEPA systems attempt to further avoid evaporation through direct placement of water in a small area directly at the soil and below the crop canopy (bubble nozzles). In the Southern Great Plains, low irrigation efficiencies of 65% have been claimed for high pressure irrigation systems. More recent research at the Bushland Texas Agricultural Research Service Station has indicated that these estimates are excessively low (Dr. Howell, personal communication, February 14, 1992). LEPA systems have claimed up to 95 to 99 percent efficiency, and low pressure drop nozzles have claimed up to 90 percent efficiencies.

LEPA systems have not been researched in North Dakota. However, it is likely the lower evapotranspiration rates in the Northern Great Plains would result in greater efficiency from using high pressure irrigation practices, and that gains from adopting LEPA systems would not be as great. Also, the benefits of using LEPA systems would be questionable on extremely coarse sand and gravel soils. Preponderance of large pores with minimal soil storage might result in poor soil redistribution of water below the surface in the root zone. Attempts to fill the soil profile using bubble nozzles might cause losses below the root zone from direct percolation due to uneven distribution of water on the soil surface. It would seem that a low pressure drop nozzle with overlapping spray might prove to be as beneficial as a LEPA system under such conditions.

As a rough approximation, 10 to 20 percent increases in water use efficiency would not be unreasonable to expect from adoption of combined deficit irrigation and low pressure drop systems. Benefits, however, are highly dependent on management. For simplicity, economic consideration of conservation measures will be viewed simply as an increase in irrigated acreage. For example, if 10 percent less water per acre is used, 10 percent more acres can be irrigated. Under conservation, the size of the pumping and conveyance system from the Sheyenne River would not be altered. The infiltration basin would still be the same size. Both the distribution system for moving water from the well field to the irrigated fields and the well field would be affected. The well field is assumed to increase in cost by a percentage equal to the increase in irrigated acreage. To calculate the additional distribution system cost, it is assummed that additional acreage is located at an average distance of 1.75 miles from the well field. Fractional quarters are used for comparative purposes. It is assumed that actual development would occur in 130 acre steps. A cost summary for 10 and 20 percent conservation savings is given on Table II-16. If conservation measures could reduce the pumping rate per quarter below 700 gpm, additional savings of about $4 per acre could be realized by using 8 inch diameter pipe for the distribution system.

ECONOMIC BENEFITS OF EXPANDED IRRIGATION

The method used to estimate direct benefits of irrigation to landowners in the Englevale area simply compares the returns from an irrigated scenario with a dryland situation. Published statistics and conversations with local producers were used to estimate crop rotations and budgets for irrigated and dryland situations. The increased per acre returns of irrigated over dryland crop production are the direct irrigation benefits to producers.

Converting from dryland to irrigated agriculture also causes changes in the regional economy. Regional economic benefits are estimated using the North Dakota Input-Output Model (Coon et al., 1989). Irrigated agriculture's more intensive farming practices result in increased purchases of inputs as well as sales of outputs, and present additional opportunities for value-added activities in the state. These changes in business activity result in additional profits to the agriculture sector, other sectors serving agriculture, as well as support additional jobs across the regional and statewide economy.

Crop Rotations and Composite Acres

To represent the mix of crops grown across the area, a composite acre was developed for irrigated and dryland crop production for the Englevale area (Table II-17). A composite acre will not likely be what any one producer grows in one year. Instead, it will represent the mix of crops most producers would grow in the region over time. Crop budgets were estimated for dryland and

Table II-16. Estimated cost summary for artificial recharge project, assuming 10 or 20% water savings from using water conservation practices.

Water Efficiency Gain %	Quarter Sections Irrigated	Well + Distribution System Cost Dollars	Total Cost Dollars	Fixed Cost/Year (30 year) Dollars	Fixed Cost/Year (40 year) Dollars	Total Cost/Year (30 year) Dollars	Total Cost/Year (40 year) Dollars	Cost per acre-yr (30 year) Dollars	Cost per acre-yr (40 year) Dollars
10	15.4	838,000	3,010,000	267,000	252,000	311,000	296,000	155	148
	20.9	1,208,000	3,388,000	301,000	284,000	345,000	328,000	127	121
	25.3	1,509,000	3,689,000	328,000	309,000	371,000	353,000	113	107
20	16.8	927,000	3,107,000	276,000	261,000	320,000	304,000	146	139
	22.8	1,338,000	3,518,000	312,000	295,000	356,000	338,000	120	114
	27.6	1,666,000	3,846,000	342,000	323,000	385,000	366,000	107	102

Table II-17. Distribution of crops for dryland and irrigated composite acres, Englevale area, 1992

Crop	Dryland	Irrigated
	---- % of cropland ----	
Corn	10	66
Dry Edible Beans	2	34
Wheat	50	--
Barley	20	--
Sunflower	8	--
Fallow	8	--
Soybeans	2	--
TOTAL	100	100

irrigated composite acres using crop production data from North Dakota State University (Haugen and Aakre, 1991) and a report estimating irrigation benefits of the Garrison Diversion Project (Leitch et al., 1991).

Effects of Adopting Improved Irrigation Technology and Water Conservation Practices

As stated earlier, improvements in irrigation technology such as the LEPA system permits operators to improve water application efficiency, apply less water per application, and generate net returns which are higher than low and high pressure alternatives (Hornbaker and Mapp, 1988). Reduced costs associated with the well, pump, and motor (reduced pumping costs) provide most of the improved net returns. In the analysis, irrigation electricity costs will be reduced by 20 percent to conservatively estimate impacts of adopting LEPA technology. Overall water use of an aquifer may be reduced by as much as 40 percent through adoption of irrigation technologies and farming practices which conserve water such as deficit irrigation, checkbook scheduling method, furrow diking, or limited tillage (Ellis et. al., 1988).

Expanded Acres of Irrigation

Benefits to producers and the region are examined for three potential irrigated acreage expansions. It was assumed 130 acres per quarter section of land would be irrigated using current irrigation and tillage practices. The three expansion scenarios are:

Quarter Sections	Total Acres
14	1,820
19	2,470
23	2,990

Benefits from a 10 percent and a 20 percent gain in water efficiency are also estimated. The reduced water use is translated into 10 and 20 percent increases in irrigated acreage.

	Quarter Sections	Total Acres
10% Water Efficiency Improvement	15.4	2,002
	20.9	2,717
	25.3	3,289
20% Water Efficiency Improvement	16.8	2,184
	22.8	2,964
	27.6	3,588

Results

Two types of economic effects occur when cropland is converted from dryland to irrigated. Per acre net returns change, affecting the well-being of the producers. Secondly, on-farm production activity increases as a result of intensified cropping and from farmers switching to higher value crops. These changes generate additional off-farm business activity and jobs.

Benefits to Producers

Switching to irrigated agriculture from dryland would increase net returns to producers in the Englevale area by $88 per acre annually (Appendix B). In determining the $88 per acre increase in net return, a $50.30 per acre cost of irrigation ownership was used. This includes cost of irrigation well, pump, and center pivot. The cost analysis for the artificial recharge project also includes the cost of the well and irrigation pump. The well costs estimated for the artificial recharge project are $15 and $14 per acre for the 30 and 40 year amortization periods respectively. **The well costs must be subtracted from the total artificial recharge project costs to obtain the cost of artificial recharge water above the cost of normal center pivot irrigation.**

Net returns of $92 per acre per year are used for LEPA affected acres to account for a $5 per acre net energy savings. Total net returns to area producers range from $160,160 per year for 14 additional irrigated quarter sections to $330,096 per year for 27.6 quarters (Table II-18)

Table II-18. Annual net returns to irrigation over dryland, Englevale area, 1992.

Scenario/ Quarter Sections (130 acres/quarter)	Total Net Returns
Current Practices:	
14	$160,160
19	$217,360
23	$263,120
10% Water Savings and 20% Energy Savings:	
15.4	$184,184
20.9	$249,964
25.3	$302,588
20% Water Savings and 20% Energy Savings:	
16.8	$200,928
22.8	$272,688
27.6	$330,096

Regional Benefits

Regional benefits accrue from the increased input purchases, additional sales, and improved value-added opportunities associated with the more intensive farming practices of irrigated agriculture. Irrigation expenditures and returns which are over and above those from dryland farming are inserted into appropriate sectors in the North Dakota Input-Output Model (I-O

Model). The I-O Model estimates increases in total business activity, retail trade, personal income, business and personal services, the finance, insurance, and real estate sector, as well as secondary employment. The I-O Model also estimates increases in value-added industries associated with irrigated crop production such as livestock production or agricultural manufacturing and processing.

Estimated increases in total business activity in the region from expanded irrigation ranged from $1.2 million to nearly $2.5 million (Table II-19). Regional increases in livestock production ranged from $38,000 to $74,000 with increases in agricultural manufacturing and processing ranging from $21,000 to $42,000.

Table II-19. Increases in retail trade, personal income, business and personal services, finance, insurance, and real estate (FIRE), total business activity, and employment due to irrigation expansion, Englevale, 1992.

Scenario/ Irrigated quarters (130 a/qtr)	Retail Trade	Personal Income	Business & Personal Services	FIRE	Total Business Activity	Secondary Employment
	Thousands of Dollars Per Year					Persons
Current Practices						
14	493	421	60	74	1,256	13
19	669	572	82	100	1,707	18
23	810	693	99	121	2,066	23
Savings[a] 10% Water 20% Energy						
15.4	537	476	66	82	1,394	14
20.9	729	645	90	111	1,891	21
25.3	873	773	109	134	2,288	26
Savings[a] 20% Water 20% Energy						
16.8	586	519	73	89	1,521	16
22.8	796	704	99	122	2,063	23
27.6	963	851	119	147	2,497	30

[a] Assumed water and energy savings from the adoption of improved irrigation technologies and soil and water conservation practices.

BENEFIT VERSUS COST FOR USE OF ARTIFICIAL RECHARGE

The cost analysis for the artificial recharge facility presented previously has determined the total cost for the construction and operation of the river pumping station, pipeline, canals, and recharge basins; and the construction costs for well field and pipeline to deliver water to the center of the field to be irrigated. It does not include well field maintenance costs or power costs for pumping water from the well field.

Benefits to the producer are considered to be returns to the producer above those accrued from dryland agricultural (Appendix B). To compare the costs of the proposed artificial recharge project to the benefits, the cost of the wells and pumps must be subtracted from the project cost since these costs are included in the irrigation ownership costs used in determining the benefits of irrigation. The distribution system is a cost above normal irrigation costs and must be included in the artificial recharge facility costs. Table II-20 presents the cost of the artificial recharge facility excluding the cost of the irrigation wells and pumps that are used in the following analysis.

Comparison of direct economic benefits and costs to users of prospective artificial recharge waters (delivered to the field) indicate that profitable operation is unlikely under current prices and conditions. Only with best management practices (20 percent water and 20 percent electricity savings through water conservation practices), and the largest feasible irrigated acreage (27.6 quarter sections) is the project profitable. Benefit to cost ratios are illustrated for 8 percent interest using 30 and 40 year amortization periods with current water use practices, with practices resulting in 10 percent water conservation and 20 percent electrical use conservation, and with 20 percent water conservation and 20 percent electrical use conservation (Fig. II-17). A benefit-cost ratio of one means that the return on irrigation with Sheyenne River water would be equal to the return on dryland agriculture.

The results indicate that under current crop rotations and prices, the use of artificial recharge from the Sheyenne River would involve relatively large risk and low likelihood of reasonable returns to the grower. However, a change in rotations, such as the introduction of other high value crops into the rotation in place of corn (for example sweet corn, onions, carrots, potatoes), or a substantial upward shift in crop prices could increase the profitability of artificial recharge very quickly.

The benefit-cost analysis indicates that the profitability of the project is strongly dependent on the number of acres that can be irrigated. The number of acres that can be irrigated are dependent upon the efficiency with which the water can be used and the risk that the irrigators are willing to accept of not having sufficient water during periods of low flows in the

Table II-20. Estimated cost summary for artificial recharge project with well and pump costs deleted. This is cost of the project above the cost of traditional center pivot irrigation from ground water.

Scenario/ Irrigated quarters (130 a/qtr)	Total Cost/Year (30 year) Dollars	Total Cost/Year (40 year) Dollars	Cost per acre per year (30 year) Dollars	Cost per acre per year (30 year) Dollars
Current Practices				
14	275000	262000	151	144
19	296000	282000	120	114
23	312000	298000	104	100
Savings[a] 10% Water 20% Energy				
15.4	281000	268000	140	134
20.9	304000	290000	112	107
25.3	322000	307000	98	93
Savings[a] 20% Water 20% Energy				
16.8	287000	273000	132	125
22.8	312000	296000	105	100
27.6	331000	315000	92	88

Sheyenne River. Any further consideration of the project should explore the effect on the benefit cost ratios of increasing risk of limited availability of water during dry periods as acres irrigated increases.

These results provide general guidelines to the profitability of using Sheyenne River water to recharge the Englevale aquifer to irrigate additional acres. The irrigators will need to evaluate the costs associated with the project upon their own individual farming operations.

Another consideration is the overall economic benefits to the Englevale region resulting from expanded high value crop production which is based on use of artificial recharge water. Regional increases of total business activity ranging from $1.2 to $2.5 million per year and secondary employment of between 13 to 30 persons would be expected to result from increased economic activity rooted in artificial recharge development (Table II-19). While such additional regional benefits should be considered by governmental entities considering financing or otherwise assisting in the construction of an artificial recharge facility on the Sheyenne River, they would have little effect on the direct profitability of using artificial recharge water for farm operators in the Englevale area.

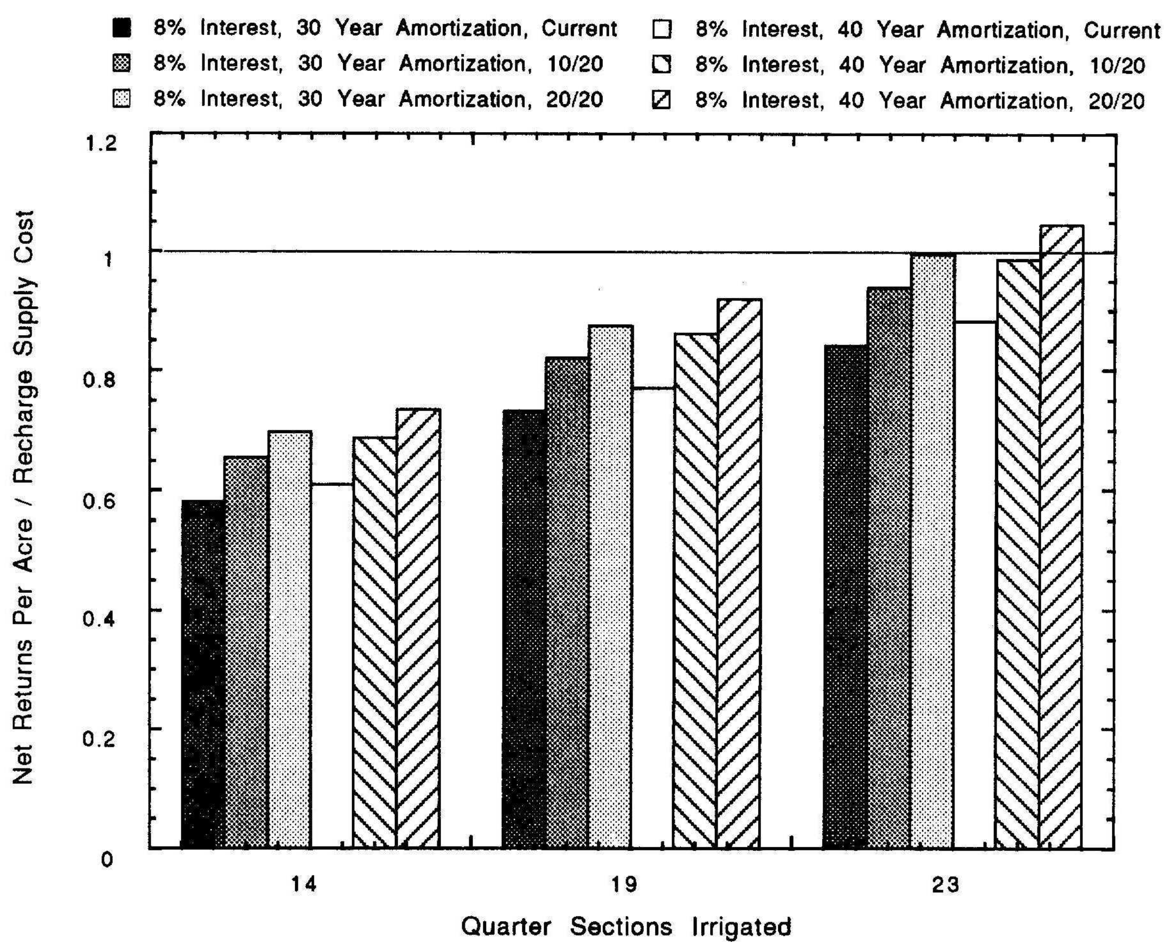

Fig. II-17. Ratio of total net returns from irrigation per acre to cost per acre for recharge water delivery at the quarter section for (1) Current Irrigation Practices, (2) 10 percent water savings and 20 percent electrical cost savings using water conservation systems and practices; and (3) 20 percent water savings and 20 percent electrical cost savings using water conservation systems and practices using 33 percent design, administrative, and contingency margin.

CONCLUSION

Sheyenne River water supplies available for aquifer storage are optimal for a pumping capacity of about 15 cfs from the river. Based on the historical flows, the Sheyenne River could supply about 1,400 to 1,800 acre-feet per year in 19 years out of 20; 1,900 acre-feet per year in 9 years out of 10; and 2,200 acre-feet per year in 8 years out of 10. These figures assume using a 3 year moving average supply cycle and only spring and fall flows. If available summer flows are included, the Sheyenne River could supply up to 3,000 acre-feet in 9 years out of 10, and about 3,300 acre-feet in 8 years out of 10. If aquifer storage is sufficient to allow for longer periods of storage, larger amounts of water would be available. This means that if a deficit develops, the ability to use larger periodic flows could be used to make up the deficit. Also, the use of two pumps instead of one would allow for use of lower flow rates than 15 cfs and would increase overall water availability. Based on this range of potential supplies with proper water management, between 1,800 and 3,000 acre feet could be appropriated and stored in the Englevale aquifer.

Water quality of the Sheyenne River is suitable for artificial recharge of the Englevale aquifer. Trace elements and nitrates are not present in concentrations likely to cause serious problems. Generally, the basic inorganic water quality of the Sheyenne River is comparable to that of the Englevale aquifer and pose no serious threat to the quality of the aquifer. Some pesticides have been detected in the Sheyenne River, but their concentrations are well below EPA health advisory standards. One detection of dicamba was made in concentrations that might be of concern for direct irrigation on beans. However, there was no repeat of the detection, and it seems very likely that the detection was due to unusual circumstances. No serious problem with pesticide contamination was noted. However, if appropriated waters are to be used for bean crops, some periodic testing of the water would be advisable.

There are ample soils suitable for irrigation in the Englevale area. Sheyenne River water quality is also suitable for irrigation on local soils. The area soil and vadose materials were sufficiently coarse to allow for minimally impeded movement of water from approximately two feet below land surface to the Englevale aquifer. The artificial recharge facility consists of basins placed in the corners of center pivot irrigated land and connected by distribution ditches. About 32 acres of land would be needed to provide ample infiltration area with maximum flexibility and least difficulty in operation and maintenance scheduling.

Total fixed cost for an artificial recharge facility including: (1) a structure for pumping and conveyance of water from the Sheyenne River to the area of use, (2) a basin recharge system, (3) a well field for pumping water from the aquifer, and (4) a distribution system for moving water from wells to fields would be approximately $2,914,000 for 14 additional irrigated quarter sections and $3,530,000 for 23 additional irrigated quarter sections. Operational cost, including pumping cost from the river, basin cleaning and management, and water monitoring cost (not counting pumping cost from the aquifer to the fields or well and irrigation pump maintenance costs) would be approximately $43,600 per year. Annual cost at 8 percent interest and amortized over 30 years would be approximately $357,000 per year ($119 per acre) for 23 additional irrigated quarter sections, and $302,000 per year ($166 per acre) for 14 additional irrigated quarter sections. Approximately $7 less per acre per year would be achieved by a 40 year amortization schedule. Substantial additional savings could be made through efficient implementation of the recharge plan. Savings of $14 to $20 per acre could be achieved by using water conservation practices thereby spreading the costs over more acres.

It is unlikely that a 20 percent water savings through conservation could be achieved at Englevale. However, the assumed water use for this analysis was 12 inches which is 20 percent higher than the actual average annual water use of 10 inches. Therefore, it is realistic to expect that with modest conservation gains and the lower water requirements of some soils in the expansion area, that cost per acre as low or lower than the 20 percent conservation option could be achieved.

The ratio of total net returns per year to total cost per year indicates that profitable operation under current commodity prices and current crop rotations is unlikely. However, current benefits and costs are nearly balanced and changes in commodity prices or changes in crop rotations to include higher value crops would greatly enhance the profitability of using artificial recharge water from the Sheyenne River. Current profitability is marginal and risky. Enhanced returns could greatly increase profitability and greatly reduce risk.

Regional indirect benefits between $1.25 and $2.5 million in increased economic activity and as many as 30 new jobs in region might result from the use of artificial recharge water from the Sheyenne River for irrigation of high value crops in the Englevale area.

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APPENDIX A. LITHOLOGIC LOGS FOR EXPLORATORY HOLES AUGERED NEAR ENGLEVALE. LITHOLOGY WAS EXAMINED USING SPLIT SPOON SAMPLES.

TEST HOLE A-1

LOCATION:	134-058-12AAA3	**DATE DRILLED:**	8/6/91
ELEVATION: (FT,MSL)	1347	**DEPTH: (FT)**	37.5

DEPTH (FT)		**DESCRIPTION OF DEPOSITS**
0.0	0.5	Loam, silty; dark brown
0.5	1.0	Sand, slightly gravelly; predominately medium yellow brown
1.0	1.5	Sand, slightly gravelly;very fine sand to very fine pebble; predominately medium sand; moderately well sorted; some shale in gravel fraction; yellow brown
1.5	5.0	no sample recovered
5.0	6.5	Gravel, sandy; fine sand to coarse pebble; predominately coarse to very coarse sand
6.5	7.5	Gravel; predominately very fine pebble; poorly sorted; many shale pebbles in gravel
7.5	10.0	Sand, gravelly; fine sand to medium pebble; predominately coarse to very coarse sand; poorly sorted; 20% shale in gravel fraction
10.0	11.0	Gravel, sandy, silty
11.0	12.5	Gravel, sandy; very fine sand to coarse pebble; predominately very coarse sand; poorly sorted; top part shalely
12.5	13.0	Sand, gravelly; predominately very coarse sand; fine sand to fine pebble; a little silt; moderately sorted
13.0	15.9	Gravel, sandy; predominately coarse to very coarse sand
15.9	16.5	Gravel, silty; gravel up to 2.5" gravel; gray
16.5	17.5	Gravel, sandy; fine sand to medium pebble; predominately coarse sand; yellow brown
17.5	18.0	Gravel, silty; up to 2" gravel
18.0	20.0	Gravel, sandy; fine sand to coarse pebble; predominately very coarse sand; poorly sorted

TEST HOLE A-1 Continued

20.0	20.5	Gravel, silty; predominately coarse sand; reddish brown
20.5	22.5	Gravel, sandy; fine sand to medium pebble; predominately coarse to very coarse sand; yellow brown
22.5	22.8	Gravel, silty; gravel +2"
22.8	25.0	Gravel, sandy; fine sand to coarse pebble; predominately coarse sand; moderately sorted; dark yellow brown
25.0	25.5	Gravel, sandy, slightly silty; brown
25.5	25.9	Gravel, silty, sandy; gray
25.9	28.5	Gravel, sandy; fine sand to coarse pebble; predominately coarse sand; moderately sorted
28.5	30.0	Gravel, sandy; predominately coarse sand; more shale; wet, top of capillary fringe
30.0	34.0	Gravel, sandy; medium sand to coarse pebble; predominately very coarse sand; poorly sorted; 0.5" silty clay parting near 32'
34.0	35.0	Gravel; coarse sand to very coarse pebble; predominately very fine pebble; shalely
35.0	36.5	Gravel; very coarse sand to very coarse pebble; predominately very fine pebble
36.5	36.8	Clay, silty, sandy, gravelly; gray
36.8	37.5	Gravel; very coarse sand to very coarse pebble; predominately very fine pebble

TEST HOLE A-2

LOCATION:	134-058-12DDD1	**DATE DRILLED:**	8/6/91
ELEVATION: (FT,MSL)		**DEPTH: (FT)**	33.0

DEPTH (FT)		DESCRIPTION OF DEPOSITS
0.0	0.5	Loam; black
0.5	0.9	Loam, sandy, slightly gravelly; dark gray
0.9	2.5	Gravel, sandy; predominately medium sand to medium pebble; bimodal distribution; predominately carbonates; dark yellow brown
2.5	2.9	Gravel, sandy; predominately coarse sand; yellow brown
2.9	5.0	Gravel, sandy; fine sand to coarse pebble; predominately coarse sand; moderately sorted; appears bimodal
5.0	7.5	Gravel, sandy; fine sand to coarse pebble; predominately coarse sand; very slightly silty; somewhat bimodal; 30% gravel; pale yellow brown
7.5	11.0	Sand, gravelly; fine sand to coarse pebble; predominately coarse sand; 20% gravel; moderately sorted; gravel mostly shale
11.0	12.0	Sand, gravelly; medium sand to very coarse pebble; predominately very coarse sand; 15% gravel; 60% shale; dark brown
12.0	12.5	Gravel, sandy; fine sand to medium pebble; predominately coarse sand; somewhat bimodal; 30% gravel
12.5	15.0	Sand, slightly gravelly; very fine sand to medium pebble; predominately coarse sand; 10% gravel
15.0	17.5	Sand, gravelly; very fine sand to medium pebble; predominately coarse sand; 15% gravel; 30% shale
17.5	21.0	Sand, gravelly; fine sand to coarse pebble; predominately coarse sand; 25% gravel; 20 to 30% shale; bimodal
21.0	21.5	Sand, gravelly; fine sand to medium pebble; predominately very coarse sand; 20% gravel; moderately sorted; 35% shale
21.5	23.0	Sand, slightly gravelly; fine sand to medium pebble; predominately medium to coarse sand; moderately well sorted

TEST HOLE A-2 Continued

23.0	25.5	Sand; very fine to very coarse sand; predominately medium sand; 10 to 20% shale
25.5	27.5	Gravel, sandy; medium sand to coarse pebble; predominately very coarse sand; 40% gravel; 25 to 40% shale; water level ≈27'
27.5	28.5	Sand, gravelly; fine sand to medium pebble; predominately coarse sand; 25% gravel; 30 to 40% shale
28.5	31.5	Gravel, sandy; medium sand to coarse pebble; predominately very coarse sand; 40 to 50% gravel; moderately sorted; +50% shale
31.5	32.5	Sand; fine to coarse sand; predominately medium sand; well sorted; predominately quartz
32.5	33.0?	Sand, slightly gravelly; fine sand to fine pebble; predominately coarse sand; 30 to 40% shale

TEST HOLE A-3

LOCATION:	134-058-13AABA	**DATE DRILLED:**	8/6/91
ELEVATION: (FT,MSL)		**DEPTH: (FT)**	36.5

DEPTH (FT)		**DESCRIPTION OF DEPOSITS**
0.0	1.0	Loam, silty; black
1.0	2.5	lost core
2.5	3.5	Sand, slightly gravelly; very fine sand to fine pebble; predominately medium sand; 5% gravel; moderately well sorted; yellow brown
3.5	5.0	Gravel, sandy; fine sand to coarse pebble; predominately very coarse sand; bimodal; 15% shale increasing to 30% with depth; yellow brown
5.0	6.5	Sand, gravelly; very fine sand to medium pebble; predominately coarse sand; 15% gravel; shalely
6.5	7.5	Sand, gravelly; fine sand to coarse pebble; predominately coarse sand; bimodal
7.5	8.5	Sand, slightly gravelly; very fine sand to fine pebble; predominately medium sand; moderately sorted; brownish gray
8.5	10.0	Sand, gravelly; very fine sand to medium pebble; predominately coarse sand; less than 10% shale
10.0	13.0	Sand, gravelly; fine sand to coarse pebble; predominately coarse sand; 25% gravel; bimodal, gravel predominately fine; 20 to 30% shale
13.0	14.5	Gravel, sandy; fine sand to very coarse pebble; predominately coarse sand; 30% gravel; bimodal, gravel predominately medium; 20 to 30% shale; yellow brown
14.5	15.3	Gravel, sandy, silty; silt to medium pebble; predominately coarse sand; brownish gray
15.3	17.5	Sand, gravelly; fine sand to fine pebble; predominately coarse sand; 20% gravel; moderately sorted; 20 to 30% shale; dark yellow brown
17.5	20.0	Sand, gravelly; as above except a little more silt

TEST HOLE A-3 Continued

20.0	23.4	Sand, gravelly; very fine sand to fine pebble; predominately coarse sand; 10% gravel; moderately sorted; 15% shale; moderate yellow brown
23.4	24.0	Sand, gravelly; fine sand to medium pebble; predominately coarse to very coarse sand; 10% gravel; 60% shale; dark brown
24.0	25.9	Sand, slightly gravelly; very fine sand to fine pebble; 5% gravel; 20 to 30% shale
25.9	26.5	Sand, gravelly; very fine sand to fine pebble; predominately coarse sand; 20% gravel; bimodal; 60 to 70% shale; shale predominately coarse to very coarse sand
26.5	27.1	Sand, gravelly; very fine sand to medium pebble; predominately coarse sand; 20% gravel; 15% shale; moderate yellow brown
27.1	27.5	Gravel, sandy; fine sand to medium pebble; predominately coarse sand; bimodal; gravel is mostly shale, predominately fine gravel
27.5	28.0	Clay, silty, sandy, gravelly; dark brownish gray
28.0	30.0	???? split spoon plugged by large stone
30.0	31.5	Gravel; coarse sand to coarse pebble; predominately very fine gravel
31.5	32.0	Sand, gravelly; predominately coarse sand; 15% gravel; 40% shale
32.0	34.5	Gravel, sandy; very fine sand to fine pebble; predominately coarse sand; 50% gravel; 60 to 70% shale; black to salt and pepper
34.5	35.0	Gravel, sandy; predominately very fine pebble; 60% gravel; 90% shale; black
35.0	36.5	Gravel, sandy; predominately very coarse sand; 50% gravel; thin till lens form 36.0' to 36.3' consisting of clayey zone with 2.5" stone

TEST HOLE A-4

LOCATION:	134-058-13DAD	**DATE DRILLED:**	8/6/91
ELEVATION: (FT,MSL)		**DEPTH: (FT)**	35.0

DEPTH (FT)		DESCRIPTION OF DEPOSITS
0.0	1.0	Loam; dark gray
1.0	1.5	Loam;
1.5	2.5	Sand; very fine to very coarse sand; predominately medium sand; moderately well sorted; dark yellow brown
2.5	3.9	Sand; very fine to fine sand; predominately fine sand; well sorted; moderate yellow brown
3.9	5.0	Silt; moderate yellow brown
5.0	6.0	Silt, sandy; silt to very fine sand; moderate yellow brown
6.0	8.0	Sand, gravelly; very fine sand to medium pebble; predominately coarse sand;; 5 to 10% gravel; moderately well sorted; 10% shale; yellow brown
8.0	11.0	Sand, gravelly; very fine sand to coarse pebble; predominately coarse sand; 25% gravel; bimodal; 5 to 15% shale
11.0	12.9	Sand, gravelly; fine sand to medium pebble; predominately coarse sand; 10% gravel; 20% shale; dark yellow brown
12.9	13.5	Sand, gravelly; fine sand to medium pebble; predominately coarse to very coarse sand; 20% gravel; 25-35% shale; dark yellow brown
13.5	15.5	Sand, gravelly; very fine sand to medium pebble; predominately medium to coarse sand; 15% gravel; 15 to 20% shale
15.5	20.5	Sand, gravelly; fine sand to fine pebble, some coarse pebble 17.0 to 17.2; predominately coarse to very coarse sand; 20% gravel; moderately well sorted; 10% shale; three zones of poorly sorted sandy gravel containing 60% shale at 18.5 to 18.7, 19.0 to 19.2, and 19.5 to 19.7
20.5	22.0	Sand, gravelly; fine sand to medium pebble; predominately coarse sand; 25% gravel; moderately sorted; gravel mostly shale

TEST HOLE A-4 Continued

22.0	22.7	Sand, slightly gravelly; fine sand to very fine pebble; predominately coarse sand; 5% gravel; moderately sorted; 30 to 40% shale; coarse fraction predominately shale
22.7	23.0	Gravel, sandy; fine sand to fine pebble; predominately coarse to very coarse sand; 35% gravel; moderately sorted; 15% shale
23.0	23.5	Sand, gravelly; fine sand to medium pebble; predominately coarse sand; 10% gravel; moderately sorted
23.5	25.0	Sand, gravelly; medium sand to coarse pebble; predominately coarse sand; 15% gravel; moderately sorted; contains 0.2' zones of shale gravel
25.0	26.5	Sand; fine sand to coarse sand; predominately coarse sand; well sorted; predominately quartz, some shale
26.5	27.5	Sand, slightly gravelly; fine sand to very fine pebble; predominately coarse sand; 5% gravel; coarse fraction predominately shale
27.5	32.0	Sand, gravelly; fine sand to fine pebble; predominately coarse to very coarse sand; 10 to 20% gravel; moderately sorted; 20% shale; some 0.2' shalely zones (more gravelly) with up to 80% shale, water level $\approx$28'
32.0	32.7	Gravel, sandy; coarse sand to medium pebble; predominately very coarse sand to fine pebble; 55% gravel; poorly sorted; 10% shale, mostly carbonates
32.7	34.8	Gravel, sandy; medium sand to medium pebble; predominately coarse sand; lower part has silt and clay
34.8	35.0	Clay, very silty, sandy, very gravelly; clay appears to fill interstitial space in gravel matrix

TEST HOLE A-5

LOCATION:	134-058-13ADC	**DATE DRILLED:**	8/7/91
ELEVATION: (FT,MSL)		**DEPTH: (FT)**	36.0

DEPTH (FT)		**DESCRIPTION OF DEPOSITS**
0.0	0.7	Loam, fine sandy; dark gray
0.7	1.5	Gravel, sandy; very fine sand to medium pebble; predominately medium to coarse sand; 30% gravel
1.5	2.5	Sand, gravelly; very fine sand to fine pebble; predominately medium sand
2.5	4.0	Sand, gravelly; very fine sand to coarse pebble; predominately medium to coarse sand; 20% gravel; 15% shale
4.0	5.0	Sand; very fine to medium sand; predominately fine to medium; well sorted; yellow brown
5.0	7.0	Sand, slightly gravelly; very fine sand to very fine pebble; predominately; medium sand; 5% gravel; moderately well sorted; yellow brown
7.0	7.5	Sand, gravelly; very fine sand to fine pebble; predominately medium sand; 15% gravel; 20 to 30% shale; gravel predominately shale
7.5	9.0	Sand, gravelly; fine sand to coarse pebble; predominately coarse sand; 25 to 30% gravel; 10% shale
9.0	10.0	Sand, gravelly; fine sand to fine pebble; predominately coarse sand; 10% gravel
10.0	12.5	Sand, gravelly; fine sand to medium pebble; predominately coarse sand; 25% gravel; 35% shale
12.5	13.5	Gravel, sandy; fine sand to medium pebble; predominately coarse sand; 35% gravel; bimodal; +40% shale, gravel mostly shale
13.5	16.5	Sand, gravelly; fine sand to fine pebble; predominately medium to coarse sand; 10% gravel; 30 to 40% shale, coarse fraction mostly shale
16.5	17.5	Gravel, sandy; fine sand to coarse pebble; predominately coarse to very coarse sand; 30% gravel; moderately sorted; 50% shale

TEST HOLE A-5 Continued

17.5	18.5	Sand, gravelly; fine sand to fine pebble; predominately medium to coarse sand; 5% gravel; 15% shale
18.5	20.0	Sand, gravelly; fine sand to medium pebble; predominately medium to coarse sand; 15% gravel; moderately sorted; 20 to 30% shale
20.0	20.5	Sand, very silty; predominately very fine sand; calcareous white mottling
20.5	23.5	Sand, gravelly; very fine sand to fine pebble; predominately medium to coarse sand; 15% gravel; moderately sorted; 20% shale
23.5	24.0	Sand, gravelly; fine sand to coarse pebble; predominately coarse sand; 15% gravel; 50% shale
24.0	26.0	Sand, slightly gravelly; very fine sand to fine pebble; predominately medium to coarse sand; 5% gravel; 15% shale
26.0	27.5	Sand, gravelly; very fine sand to very coarse pebble (1.5"); predominately medium to coarse sand; 20% gravel; 30% shale; coarse pebble at bottom
27.5	29.7	Gravel, sandy; medium sand to medium pebble; predominately very coarse sand; 40% gravel; moderately sorted; 30% shale
29.7	30.0	Clay, sandy, silty, gravelly; gravel to 1.5"
30.0	32.5	Gravel, clayey, sandy; predominately coarse pebble, gravel to 2"; uncertain of permeability
32.5	35.5	Gravel; coarse sand to very coarse pebble; poorly sorted; some clay at 34.8 to 35.0
35.5	36.0	Sand, gravelly; predominately coarse sand; 25% gravel; poorly sorted; 70% shale

TEST HOLE A-6

LOCATION:	134-057-18BCCB	**DATE DRILLED:**	8/7/91
ELEVATION: (FT,MSL)		**DEPTH: (FT)**	37.0

DEPTH (FT)		DESCRIPTION OF DEPOSITS
0.0	1.0	Loam, fine sandy; dark gray
1.0	5.0	Sand, gravelly; very fine sand to medium pebble; predominately fine sand; 10% gravel; brownish gray
5.0	6.0	Sand, gravelly; very fine sand to coarse pebble; predominately medium to coarse sand; 25% gravel; bimodal; 5 to 10% shale
6.0	8.5	Sand; very fine to coarse sand; predominately fine to medium sand
8.5	10.0	Sand; very fine to very coarse sand; predominately fine to medium sand; 15 to 20% shale, very coarse sand predominately shale
10.0	11.0	Sand; very fine to coarse sand; predominately fine to medium sand; 5% shale
11.0	12.5	Sand, gravelly; very fine sand to coarse pebble; predominately medium sand; 10% gravel; moderately sorted; 20 to 30% shale
12.5	14.5	Sand; very fine to coarse sand; predominately fine to medium sand; 15 to 30% shale
14.5	15.0	Sand, gravelly; very fine sand to coarse pebble; predominately medium sand; 7% gravel; moderately sorted
15.0	15.5	Sand, gravelly; very fine sand to medium pebble; predominately fine to medium sand; 25% gravel
15.5	20.0	Sand, gravelly; fine sand to coarse pebble; predominately medium sand; 20% gravel; 15% shale
20.0	23.5	Sand, gravelly; fine sand to coarse pebble; predominately medium sand; 20% gravel, more coarse pebble than above; bimodal; 15% shale
23.5	27.4	Sand, gravelly; fine sand to medium pebble; predominately medium to coarse sand; 20% gravel; bimodal; poorly sorted; 25% shale to 40% in lower 2'
27.4	30.0	Gravel, sandy; fine sand to medium pebble; predominately coarse sand; 30% gravel; moderately sorted; 30 to 40% shale

TEST HOLE A-6 Continued

30.0	32.5	Sand, gravelly; fine sand to medium pebble; predominately coarse sand; 20% gravel; moderately sorted; 40 to 50% shale
32.5	34.0	Gravel, sandy; coarse sand to coarse pebble; predominately very coarse sand; 50% gravel; poorly sorted; 15% shale
34.0	35.0	Gravel; very coarse sand to coarse pebble; predominately fine pebble; 30% shale
35.0	37.0	Sand; fine sand to very coarse sand; predominately medium to coarse sand; well sorted; 60 to 70% shale

TEST HOLE A-7

LOCATION:	134-058-12DAA	**DATE DRILLED:**	8/7/91
ELEVATION: (FT,MSL)		**DEPTH: (FT)**	37.5

DEPTH (FT)		**DESCRIPTION OF DEPOSITS**
0.0	0.8	Loam, fine sandy; dark gray
0.8	1.5	Gravel, sandy; fine sand to coarse pebble; predominately fine to medium sand; 30% gravel; bimodal; poorly sorted; gravel predominately coarse pebble; gray brown
1.5	2.5	Sand, gravelly; very fine sand to medium pebble; predominately medium sand; +10% gravel; moderately sorted; 20% shale
2.5	4.2	Sand; very fine to medium sand; predominately fine to medium sand; moderately well sorted; predominately quartz
4.2	5.0	Sand, gravelly; very fine sand to fine pebble; predominately medium sand; 10% gravel; moderately sorted; 10% shale; dark yellow brown
5.0	5.5	Sand, silty; predominately fine to medium sand
5.5	7.5	Sand, gravelly; fine sand to fine pebble; predominately medium to coarse sand; 10% gravel; moderately sorted; 15% shale
7.5	9.0	Sand; very fine to medium sand; predominately fine sand; well sorted; predominately quartz; pale yellow brown
9.0	12.5	Sand, gravelly; fine sand to very fine pebble; predominately medium to coarse sand; 5 to 10% gravel; moderately sorted; 30 to 40% shale
12.5	14.0	Sand; slightly gravelly; fine sand to very fine pebble; predominately medium to coarse sand; 5% gravel; moderately sorted; 15% shale
14.0	15.5	Sand; gravelly; fine sand to very fine pebble; predominately coarse sand; 5 to 10% gravel; 30 to 40% shale; shale predominately coarse to very coarse sand
15.5	17.5	Sand; predominately fine; well sorted; predominately quartz; pale yellow brown
17.5	18.0	Sand; very fine to very coarse sand; predominately fine sand; moderately well sorted; 20% shale

TEST HOLE A-7 Continued

18.0	21.0	Sand, slightly gravelly; fine sand to very fine pebble; predominately coarse sand; moderately well sorted; a little medium to coarse pebble 19.6 to 20.0
21.0	22.5	Sand, gravelly; fine sand to medium pebble; predominately medium to coarse sand; 15% gravel; moderately sorted
22.5	25.0	Gravel, sandy; fine sand to coarse pebble; predominately coarse sand; 35 to 40% gravel; bimodal; poorly sorted; 20% shale
25.0	28.5	Gravel, sandy; coarse sand to coarse pebble; predominately very coarse sand; 45% gravel; poorly sorted; 30% shale; 25' to 26', some clay filling pores
28.5	30.0	Sand, gravelly; fine sand to medium pebble; predominately coarse sand; 25% gravel; poorly sorted; 30% shale, water level ≈29'
30.0	31.9	Gravel, sandy; predominately very coarse sand; 50% gravel; poorly sorted; 25% shale
31.9	36.5	Sand; predominately fine to medium; moderately well sorted; 20% shale
36.5	37.5	Sand; very fine to very coarse sand; predominately medium sand; moderately sorted; 20% shale

TEST HOLE A-8

LOCATION:	134-058-13AAB3	**DATE DRILLED:**	8/8/91
ELEVATION: (FT,MSL)		**DEPTH: (FT)**	37.5

DEPTH (FT)		DESCRIPTION OF DEPOSITS
0.0	0.8	Loam, fine sandy; black
0.8	1.9	Loam, fine sandy; dark gray
1.9	2.5	Sand, silty; predominately very fine sand; yellow brown
2.5	3.5	Sand; very fine to fine sand; predominately fine sand; well sorted; predominately quartz
3.5	5.2	Sand, silty; predominately very fine sand; well sorted; yellow brown
5.2	6.5	Sand, gravelly; very fine sand to medium pebble; predominately medium sand; 10% gravel; moderately sorted; 15% shale; dark yellow brown
6.5	7.5	Sand, gravelly; very fine sand to very coarse pebble; predominately medium to coarse sand; 20% gravel; moderately sorted; 15% shale; dark yellow brown
7.5	9.0	Sand, gravelly; fine sand to medium pebble, a little coarse pebble; predominately coarse sand; 25% gravel; bimodal; 30% shale
9.0	10.0	Sand, gravelly; fine sand to coarse pebble; predominately coarse sand; 20% gravel; poorly sorted; 15% shale
10.0	16.0	Sand, gravelly; fine sand to fine pebble; predominately coarse sand; 5 to 10% gravel; moderately well sorted; 15% shale; pale yellow brown
16.0	18.0	Sand, gravelly; fine sand to coarse pebble; predominately coarse sand; 20% gravel; moderately sorted; 20 to 30% shale; moderate yellow brown
18.0	18.6	Gravel, sandy; fine sand to coarse pebble; predominately very coarse sand; 50% gravel; poorly sorted; 60 to 80% shale; dark gray
18.6	21.5	Sand, gravelly; fine sand to coarse pebble; predominately coarse sand; 10 to 15% gravel; moderately sorted; bimodal; moderate yellow brown

TEST HOLE A-8 CONTINUED

21.5	23.1	Sand, gravelly; fine sand to fine pebble; predominately coarse sand; 20% gravel; moderately sorted; +60% shale; dark gray
23.1	24.5	Sand, gravelly; fine sand to fine pebble; predominately coarse sand; 10% gravel; moderately sorted; 30 to 40% shale; moderate yellow brown
24.5	25.5	Sand, silty, gravelly; fine sand to medium pebble; predominately coarse sand; 40% shale; slightly cemented by silt and clay filling pore space, crumbles easily; gray brown
25.5	29.0	Sand, gravelly; medium sand to fine pebble; predominately coarse to very coarse sand; +25% gravel; moderately sorted; dusky yellow brown
29.0	30.5	Gravel, sandy; medium sand to medium pebble; predominately very coarse sand; 50% gravel; poorly sorted; 60% shale
30.5	32.5	Gravel, sandy; coarse sand to medium pebble; predominately very coarse sand; 40% gravel; poorly sorted; 30% shale
32.5	37.5	Sand, gravelly; medium sand to fine pebble; predominately coarse sand; 20% gravel; moderately sorted; bimodal; 25% shale; a little coarser lower 2'; 1.5" till pebble at 36.5'

APPENDIX B. CROP BUDGETS FOR AN ACRE OF IRRIGATED, COMPOSITE, IRRIGATED, AND COMPOSITE DRYLAND CROP PRODUCTION, ENGLEVALE AREA, 1992.

	Irrigated Corn Grain w/base	Irrigated Dry Edible Bean	Irrigated Composite (66% corn, 34% bean)	Composite[a] Dryland	Added Dollar Flows[b]	Input-Output Sector
Average Yield	160 bu/acre	2,800 lbs./acre				
10 yr. Ave. Price	$2.25/bu	$0.15/lb.				
Gov't Payment	$24.61	n/a				
Gross Income	$386.21	$420.00	$397.70	$101.59	$296.11	
Expenses			Dollars			
Seed	28.00	27.00	27.66	7.23	20.43	Retail
Fert. & Chem.	88.71	53.67	76.80	11.48	65.32	Retail
Misc.	29.05	6.80	20.83	5.28	15.55	B&P Serv
Ins. & Int.	18.72	18.32	18.58	6.09	12.29	FIRE
Fuel & Lub.	7.11	9.72	8.00	5.97	2.03	Retail
Repairs	14.06	13.39	13.83	6.75	7.08	B&P Serv
Hired Labor	3.65	3.03	3.44	0.00	3.44	Household
Mach. Ownership	25.35	23.36	24.67	24.67	0.00	Retail
Irr. Ownership	50.30	50.30	50.30	0.00	50.30	Retail
Land Ownership	23.34	23.34	23.34	23.34	0.00	FIRE
Irr. O. & M.	33.00	28.50	31.47	0.00	31.47	Retail
TOTAL	320.29	257.43	298.92	90.81	208.11	
Returns to Unpaid Management and Labor	65.92	162.57	98.78	10.78	88.00	Household

Crop budgets were derived from:
Estimated 1991 Crop Budgets: Southcentral North Dakota (Haugen and Aakre 1991)
and A Reevaluation of GDU Irrigation (Leitch et al. 1991).

[a] A composite acre of dryland consists of 50% wheat, 20% barley, 10% corn, 8% sunflowers, 8% fallow, 2% soybeans, and 2% dry edible beans.

[b] Added dollar flows equal irrigated composite acre values less dryland composite acre values.