Illinois State Water Survey

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WATER SUPPLY ALTERNATIVES FOR THE CITY OP DANVILLE

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PREPARED BY
ILLINOIS STATE WATER SURVEY

The Inter-State Water Company serves the City of Danville and 4 neighboring communities. Pumpage in million gallons per day (mgd) has been relatively constant during the past 8 years as follows:

Year	*Pumpage, mgd*
1968	9.1
1976	8.7

Water is obtained from Lake Vermilion, built in 1925, on the North Fork of the Vermilion River. The dependable water supply yield of the lake has been calculated for the 40-year drought condition, following the common practice of assuming that the lake will be only half emptied during the drought. The remaining half of the lake water would sustain fish life and recreation. The yield of Lake Vermilion is decreasing as a result of the deposition of sediment, and is estimated to have the following future yields:

Year	*Yield, mgd*
1980	7.7
1990	7.3
2000	6.8
2010	6.3

For the 5 townships served, the projected population is 76,173 in 1980, and 90,787 in the year 2010. These projections are within the allowable 5% of the Illinois Bureau of Budget projections. Based upon these population figures and appropriate economic factors, future demands are estimated as:

Year	*Water Demand, mgd*
1980	10
1990	11
2000	12
2010	13

Thus, demand may exceed present yield at any future time under 40-year drought conditions. About 6.7 mgd will be needed from some other source by 2010. Fortunately there are several feasible alternatives available for increasing the supply.

Under one option Lake Vermilion can be enlarged by raising the dam and spillway by as much as 5 feet. The 1980 dependable yield would be increased from 7.7 mgd to 13.1 mgd. This yield would decrease over future years because of sediment deposition in the lake:

Year	*Yield, mgd*
1990	12.0
2000	11.0
2010	10.1

Raising the lake level would cost about $230,000. However, it would cost an added $200,000 to provide a culvert under the Duncan Road for access by boats from the upper part to the lower part of the lake and vice versa. Amortized over 50 years the entire project would cost about $35,100 per year.

A second option is to dredge accumulated sediment from Lake Vermilion. Dredged material would be contained in disposal basins created by building levees near the lake. Two small dredges costing $140,000 each, might remove a total of 110 acre-feet of sediment from the lake each year, which would be more than adequate to maintain the lake capacity at the 1980 level. The annual cost of dredging would be $179,000.

A combination of raising the lake level by 5 feet and one dredge would cost $124,600 per year. Raising the lake level by 3 feet and employing two dredges would cost $212,500 per year. Either of these combinations would meet demands over the study period of 1980-2010.

Under a further option, water can be obtained from the Vermilion River at Danville and pumped 2 miles north to the water plant to augment the present supply. Such an auxiliary supply of 6.7 mgd would cost $89,700 per year.

Under another approach, wells might be drilled near the Wabash River northwest of Covington, Indiana. A supply as large as 10 mgd is physically possible, or water could be diverted directly from the Wabash River. However, developing a water supply in Indiana for use in Danville, Illinois, is probably not feasible because of legal and institutional constraints. A permit from the State of Indiana would be required for an intake tower in the Wabash River, or for wells near the river. Indiana officials seem negative toward the idea of exporting water to Illinois.

It seems possible that a maximum water supply of 1.75 mgd might be obtained from a 4-well system within the City of Danville at a cost of $28,800 per year, but this would fall short of the projected deficit.

A major water supply can be developed from well fields 9 to 19 miles north of Danville where major underground water-bearing formations are present in the buried Mahomet Valley. This water can be transported by pipeline to Lake Vermilion or pumped into the North Fork Vermilion River and permitted to flow downstream into Lake Vermilion. The City would need to purchase flowage rights for this water from riparian landowners along the river to insure that the water did reach Lake Vermilion if the river option is selected.

Sites for four well fields have been evaluated. Depending upon the site and means of conveyance selected, a supply of 6.7 mgd would cost per year from $70,400 exclusive of the cost of flowage easements, to $140,600 by pipeline.

Water from any of these new sources would have a quality generally comparable to that from Lake Vermilion, and although slightly higher in mineral content, would be well within the limits for drinking water standards, and readily treatable by conventional methods.

In summary, several alternative sources of water supply for the Danville area are available and have been evaluated in the body of the report in single, dual, or multiple-source combinations.

WATER SUPPLY ALTERNATIVES FOR THE CITY OF DANVILLE

by Krishan P. Singh

1. INTRODUCTION

The study reported here is a response of state government to assist the people of the Danville area in locating sources of water supply to meet the expected future water demands of the area. The project has been carried out by the Illinois State Water Survey at the University of Illinois for the Division of Water Resources under an Agreement For Cooperative Investigation. The work began July 1, 1977, and the report was completed in March, 1978. The purpose has been to evaluate various alternatives that reasonably might pe available to provide future public water supply needs for the Danville region of Illinois.

Acknowledgments. The initiative in establishing this project was carried out by Frank Beal, Director, Illinois Institute for Environmental Quality, in order that the state provide information on water supply to the people of the Danville region. The project was outlined and instituted by Donald R. Vonnahme who was Acting Director of the Division of Water Resources, Illinois Department of Transportation, at the beginning of the project. Paul F. Kramer and later John B. Carlisle maintained liaison with all persons Involved with the project.

This project has been carried out at the State Water Survey under the general supervision of Dr. William C. Ackermann, Chief, and John B. Stall, Head of the Hydrology Section. Adrian P. Visocky and Richard J. Schicht carried out groundwater evaluations in northern Vermilion County and in Indiana, and Ellis W. Sanderson did the groundwater evaluation in and around Danville. Wyndham J. Roberts supplied information on dredging machinery and costs, and Robert H. Harmeson provided information on the quality of water from various surface water and groundwater sources. J. Rodger Adams helped In getting asbestos-cement pipe costs. Anil K. Singhal and Takashi Takenaka, graduate research assistants, helped in processing the data, developing computer programs, and other analyses.

The section on legal and institutional aspects of bringing water from Indiana was compiled largely by John B. Stall and William C. Ackermann. It was based on the information provided by an interview with William J. Andrews, Deputy Director,

Indiana Department of Natural Resources. The section on legal and institutional aspects of using the North Pork Vermilion River to transmit water from wells in northern Vermilion County downstream to Lake Vermilion was based largely on information provided by Cheryl Sylvester of the Illinois Department of Transportation.

Kenneth A. Bidle, Manager, and Amarjit S. Ambiee, Engineer, of the Inter-State Water Company, a privately owned water utility serving Danville, provided full cooperation and helpful information throughout this project.

As a part of this project, water samples were obtained weekly from the Vermilion River near Danville for about 4 months. Field' sampling was done by Eugene Hart, and cooperation in this project was coordinated by Robert Jones, both of the Central Foundry Division of the General Motors Corporation plant at Tilton. Laboratory analyses were carried out by the Illinois Environmental Protection Agency through the cooperation of Ira M. Markwood, Division of Public Water Supplies; Roger Selburg, Permits Section; Dorothy Bennett, Water Quality and Modeling; and Roy Frazier,. Director, Regional Laboratory at Champaign.

Some data on the water quality of the Wabash River were provided by C. Lee Bridges of the Indiana Department of Public Health.

At the State Water Survey the illustrations for this report have been prepared by John W. Brother, Jr., and William Motherway, Jr. The report has been given editorial review by J. Loreena Ivens.

2. FUTURE WATER SUPPLY DEMANDS

The Inter-State Water Company serves the incorporated communities of Danville, Tilton, Catlin, Belgium, and Westville and the Hooten Water District. The service area lies in Blount, Catlin, Danville, Georgetown, and Newell townships of Vermilion County, as shown in Figure 1 along with township numbering and well numbering systems. The amount of water pumped by the company during the most recent 9 years is given below.

Year	*Million gallons per day*
1968.	9.10
1969	8.94
1970	8.83
1971	9.37
1972	8.71
1973	8.87 .
1974	8.52
1975	8.54'
1976	8.65

The figures show a rather small decline even though the population served has increased with years. The decline is attributed to the economic environment of the area, reuse of water by industry, changes in customer bills, and limited service area development.

Forecasting of future water demands is a risky business. Many of the demand projections made for a number of cities 10 to 15 years ago have proved to be considerably higher than the present use because of the all-pervasive effect of pollution control regulations and requirements, and awareness of dwindling resources of good water. Because of the unavailability of a crystal ball that enables one to look into the future, the same function can be performed scientifically by channeling the past experience through a modifier based on the present conditions and some future expectations to develop estimates of water requirements or demands in future years.

Population Projections. The directive of the Bureau of Budget states:

> "These population projections are to be used as guidelines by all State agencies, boards, and commissions in the preparation of required plans, programs and budget documents. To this end, these projections are an integral part of the budget

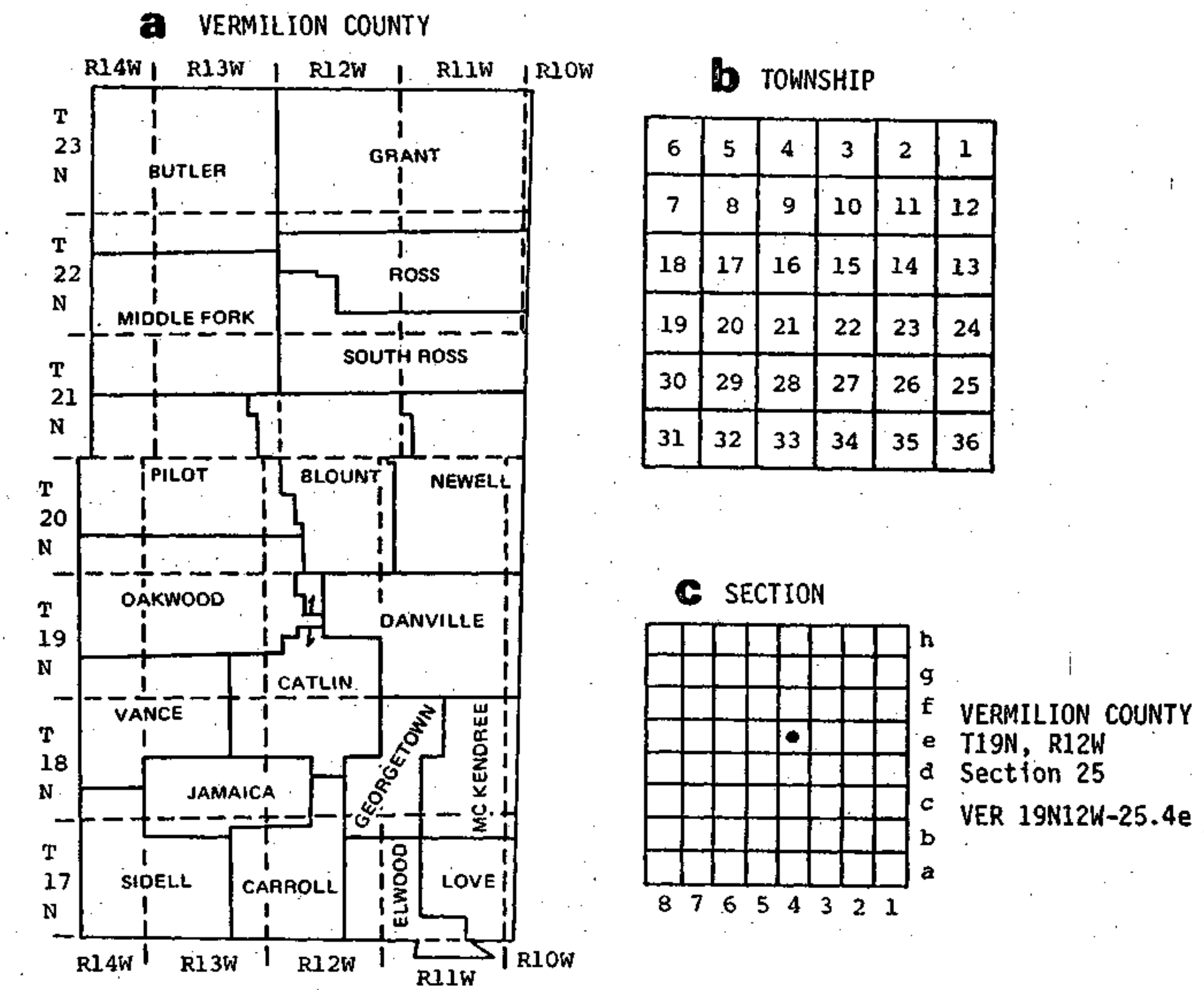


Figure 1. Vermilion County map, sections in a township, and well numbering system

process and where applicable, all agency requests will be reviewed in light of these projections.

"These projections are not intended to represent the optimal or most desirable growth patterns throughout the State, but rather they represent the most probable projections based upon current information.

"In recognition of the inherent uncertainty in any projection technique, State agencies, upon consideration of additional information, may utilize population values within 5% of the data for the year 1995. Variations other than this must be submitted to this Office with detailed supporting information and approved prior to use."

Illinois Bureau of the Budget (IBOB) population projections for the five townships to be served by water supply from Danville are:

Twp	*1980*	*1990*	*2000*	*2010*	*.2020*
Blount	2,560	2,972	3,38l	3,778	4,315
Catlin	3,838	4,380	4,927	5,481	6,220
Danville	44,566	45,021	44,497	44,291	45,871
Georgetown	'9,613	10,229	10,731	11,316	12,389
Newell	13,887	16,526	18,818	20,965	24,105
Total	74,464	79,128	82,354	85,831	92,900
% of county population	74.8	75.6	76.4	77.0	77.4

For the years 1940 through 1970, the relevant data for the five-township area are:

Year	*1940*	*1950*	*1960*	*1970*
Population	61,089	62,723	71,594	72,695
% of county population	70.4	72.0	74.4	74.9

If we assume that the trend of increase in percent of county population will continue to the year 2020, the values of these percentages and populations are calculated to be:

Year	*1980*	*1990*	*2000*	*2010*	*2020*
County population	99,573	104,613	107,805	111,532	119,955
% of county population	76.5	78.1	79.7	81.4	83.0
5-twp population	76,173	81,703	85,921	90,787	99,563

The 1995 population is not more than 5 percent different from the IBOB estimates. Hence, these population estimates, though somewhat higher than the IBOB, can be considered in this study.

Henry, Meisenheimer & Gende, Inc., in their report of water resources and community development of the greater Danville region (HM&G, 1974) developed some population estimates. Their estimates based on U.S. Census data are lower than those given in the preceding paragraph. However, their estimates based on average state growth of 1.0 percent per year yield higher populations in future years as shown below.

Year	*1970*	*1980*	*1990*	*2000*	*2010*	*2020*
Population	72,695	80,300	88,702	97,982	108,233	119,557

The Vermilion County population over the years 1940 to 1970 has increased from 86,791 to 97,047, about 0.37 percent per year. A 0.37 percent increase rate would raise the 1970 population of 72,695 for the five townships to only 87,438 in the year 2020. Allowing for movement of some population to the five-township area, the 2020 population estimate may be in the range of 90,000 to 9'2,000.

Prom consideration of the estimates developed and explained above, the following estimates of future populations for the five-township service area will be used in this study.

Year	*Population*
1980	76,173
1990	81,703
2000	85,921
2010	90,787
2020	99,563

From the values of average water use in million gallons per day (mgd) and in gallons per capita per day (gpcd), the population estimates from Henry, Meisenheimer & Gende (1974) for the years 1980, 1990, 2000, 2010, and 2020 are 77,538, 83,219, 89,005, 98,446, and 106,711, respectively. These estimates exceed those derived above by about 2 percent for the year 1980 to about 7 percent for 2020.

Average Water Use Projections. Many techniques have been developed over the years for transforming future population estimates into water use projections. Studies relevant to the service area under consideration are discussed below.

1. Roberts, Csallany, and Towery (1970) considered nearly 1200 water supplies of incorporated Illinois communities

and developed a demand-population trend curve for Illinois in terms of gallons per capita per day (gpcd):

$$\text{gpcd} = 45 \log (P/40) - 17$$

in which P is the population. It was recommended that the trend curve for a town should be drawn parallel to the Illinois curve. In other words, the value of the intercept or constant, -17 in the above equation, may vary from one town to the other.

In 1970, the Inter-State Water Company pumped an average of 8.83 mgd for a 72,695 population served, giving an average use of 121.5 gpcd. With gpcd and P equal to 121.5 and 72,695, the following relation is obtained

$$\text{gpcd} = 45 \log (P/40) - 25$$

2. Singh, Visocky, and Lonnquist (1972) developed a general relation for average water use for towns in the Kaskaskia River basin. In the relation

$$Q = a\, P^{1.252}$$

Q is in gallons per day (gpd). Value of the coefficient a will vary from one town to the other. For 8,830,000 gpd and 72,695 population served, the coefficient is calculated as 7.23 for the Danville area.

3. Schicht, Adams, and Stall (1976) investigated the effect of manufacturing employment on the per capita water consumption, applicable to towns in northeastern Illinois. The total manufacturing employment in the five-township Danville service area was 13,461 as obtained from the Illinois Manufacturers Directory 1971. For the total 1970 population of 72,695 in the service area, the manufacturing employment as a percent of population comes to 18.52. The gpcd from the graph in the reference is 146.

4. Singh and Adams (1977) considered manufacturing employment, I, and population, P, as the two most significant variables affecting water use. From a statistical analyses of I, P, and Q for 208 towns in six counties (Cook, Du Page, Kane, Lake, McHenry, and Will) a model equation (Q is in gpd)

$$Q = a\, P^{\alpha + \beta (I/P)}$$

was developed in which a, α, and β were evaluated for each county. Conditions for the Danville service area are similar to large towns in Lake County. The sample size in Lake County was 26 and the regression analysis yielded a multiple correlation coefficient of 0.995. The values of a, a, and 3 are 41.29, 1.0721, and 0.1682, respectively.

5. Henry, Meisenheimer, & Gende (1974) employed various

projections, based mainly on different rates of increase in residential, industrial, and commercial use. They presented an average of all projected values in Table 5, p. 17 of their report. Roughly, a 100 percent increase in gpcd is indicated over the period 1980 to 2020.

Future Trends in per Capita Water Use. It may be worthwhile to investigate the present thinking in regard to future trend in per capita water use.

Residential: Levying of sewer taxes in proportion to water use, gradual growth of two-family and multi-family housing, and increasing use of water-saving devices under the impact of conservation measures and increase in sewage treatment costs are expected to hold residential water use to the present level or even reduce it. Any increase, if it occurs, would be rather small. Pumpage by the Inter-State Water Company over the last 9 years shows a rather small decline even though the population served has increased with years. It is interesting to note that the Champaign County Regional Planning Commission (CCRPC, 1973), adopted a gpcd of 107.3 in 1980 and 107.0 in 1990 for Champaign-Urbana.

Industrial: Industries have been cutting down on their water use by changes in process technology, reuse, and installation of water-saving devices. The following information (U.S. Department of Commerce, 1971 and 1975) indicates a considerable decrease in industrial water intake in Illinois.

Year	*Number of manufacturers*	*Value added (million $)*	*Number of employees*	*Water intake- (billion gallons)*
1968	626	9,430	536,400	761.3
1973	709	12,947	527,300	504.6

The total water intake in 1973 was considerably less than in 1968 though the overall manufacturing activity increased. Reduced industrial use is attributed to more stringent pretreatment requirements for discharging industrial wastewaters to municipal sewer systems, higher wastewater treatment costs, and public concern over the amount of industrial wastewaters entering the streams.

Commercial: The considerations applying to residential and industrial use apply equally well to commercial use.

SWS Demand Projections. The projections were made through consideration of the five sources of information already discussed. The manufacturing employment as a percent of population in the service area was maintained at 18.52. Thus, the increase in manufacturing employment will keep pace with the Increase in population served.

Year	*1980*	*1990*	*2000*	*2010*	*2020*
Population	76,173	81,703	85,921	90,787	99,563
Method		*Use in mgd and (gpcd)*			
1.	9.34	10.13	10.73	11.44	12.72
	(122.6)	(124.0)	(124.9)	(126.0)	(127.8)
2.	9.36	10.22	10.88	11.66	13-08
	(122.8)	(125.0)	(126.6)	(128.4)	(131.4)
3.	11.12	11.93	12.54	13.25	14.54
	(146.0)	(146.0)	(146.0)	(146.0)	(146.0)
4.	10.04	10.85	11.47	12.18	13.49
	(131.8)	(132.8)	(133.4)	(134.2)	(135.8)
5.	9.7	12.1	15.3	19.9	25.6
	(127.3)	(148.1)	(178.1)	(219.2)	(257.1)

The 1970 water use for the service area population of 72,695 from the first four methods comes to 8.55, 8.83, 10.6l, and 9.54 mgd, respectively. The actual use was 8.83 mgd. The water use obtained with method 3, 10.61 mgd, is higher than the observed use of 8.83 mgd. The average of the projections from the first four methods for the years 1980, 1990, 2000, and 2010 are 9.96, 10.78, 11.41, and 12.13 mgd, respectively. With an allowance for some increase in water use with years because of the city's efforts to attract more industry, the following demand projections were made:

Year	*1980*	*1990*	*2000*	*2010*
mgd	10	11	12	13

These will be used to define the demand curve, designated the SWS demand curve, which is shown in Figure 2.

Other Demand Projections. The HM&G (1974) projections based on an average of various ad hoc rates of increase in residential, industrial, and commercial use are:

Year	*1980*	*1990*	*2000*	*2010*
mgd	9.7	12.1	15.3	19.9

The 1980 water use conforms to that of the SWS demand curve, but the similarity ends there. Assumption of rapid industrial development with time in future years has led to the geometric increase in projected water demands for the years 1990, 2000, and 2010. The HM&G demand curve is also shown in Figure 2.

A meeting was held on October 24, 1977, to consider the differences in SWS and HM&G demand curves. The meeting was attended by John Weaver from the city of Danville, Harold Meisenheimer from the HM&G, Steve Yen from Harza Engineering, Paul Kramer from the Division of Water Resources, and Richard

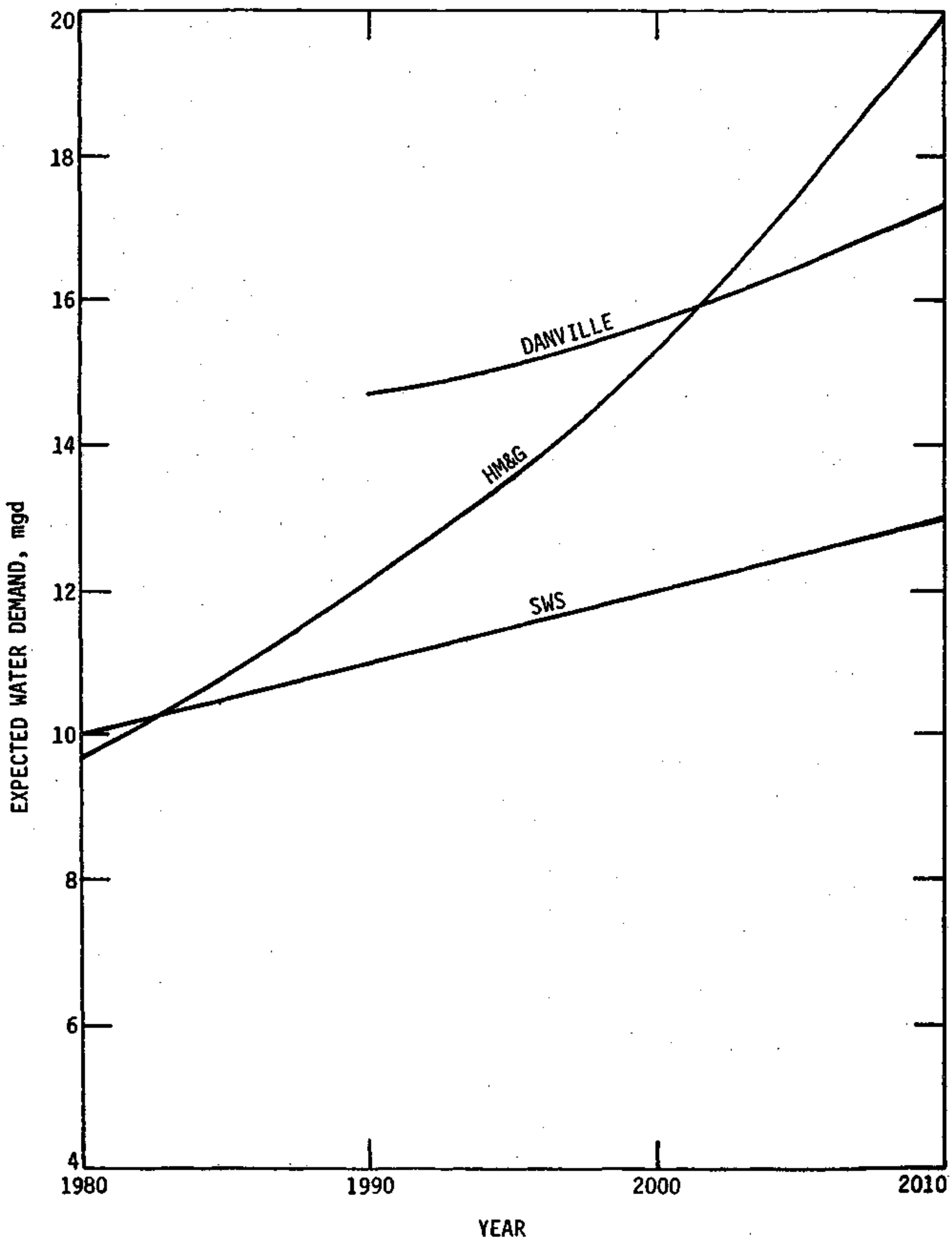


Figure 2. Various demand curves for the period 1980 through 2010 for the Danville service area

Schicht and Krishan Singh from the State Water Survey. The SWS demand curve gives 0.5 to 1.0 mgd more water demand than if things were to proceed as they have in the past and continue that normal growth into the future. The HM&G projections were predicated upon accelerated economic and industrial growth of the city. Danville is one of the few cities that regularly issue industrial development bonds. It was felt, however, that the HM&G projections were rather high. The city and HM&G agreed to derive a new set of values based upon their best prognosis of their hopes and expectations of rapid industrial growth in future years.

The new demand projections are defined by the Danville demand curve in Figure 2 for the years 1990 through 2010. These were obtained by multiplying the HM&G population projections for those years by 176.2 gpcd, a figure based on the design capacity of the sewage treatment works in Danville after making adjustments for infiltration into the sewer system, losses from the sewer system, and industrial cooling water entering the system.

Year	*1980*	*1990*	*2000*	*2010*
Population (HM&G)	77,538	83,219	89,005	98,466
mgd	13.7*	14.7	15.7	17.3
gpcd	176.2	176.2	176.2	176.2

* Calculated for 1980 population as estimated by HM&G.

It is evident that the Danville demand curve yields a demand of 13..7 mgd in 1980 instead of 9.7 mgd from the HM&G demand curve. The Danville curve is practically parallel to the SWS curve though about 4 mgd higher for the range of years 1990 through 2010. This is probably for having an extra water supply capability available to attract industry.

Various alternatives for meeting all three demand curves were explored in order to provide a wide range of options for change if so warranted in future years.

3. LAKE VERMILION: CAPACITY AND YIELD

Lake Vermilion, the present supply source for the Inter-State Water Company, was built in 1925. The drainage area of the North Fork Vermilion River above the dam is 298 square miles. The Illinois State Water Survey carried out sedimentation surveys on the lake in August 1963 and December 1976. The pertinent lake data and that from the two surveys are given below. The surface area and storage capacity are computed for a normal pool level of 577.2 feet above mean sea level.

Year	*Storage capacity (ac-ft)*	*Surface area (acres)*
1925	8514	775
1963	5318	696
1976	4641	608

One acre-foot (ac-ft) of storage corresponds to 0.326 million gallons. Only about l/7th of the original capacity is now left above the Old Dam No. 2 (north of Sections 29 and 30, T20N, R11W).

Storage Capacity in Future Years. The storage capacity expected in future years was estimated by the following procedure:

1. The sediment trap efficiency values for various capacity-inflow ratios were read from Brune's (1953) curve and stored in the computer.

Capacity-inflow ratio	0.06	0.05	0.04	0.03	0.02	0.01	0.005
Trap efficiency,%	79.5	76.6	73.0	67.9	59.1	42.8	25.2

2. From a study of flows at the six nearby gaging stations an average yearly runoff of 0.82 foot was chosen for the North Fork Vermilion River basin. For 298 square miles, the average yearly inflow is 156,390 ac-ft.

3. A preliminary analysis indicated a sediment inflow rate of about 110 ac-ft per year (sediment inflow corresponding to sediment deposited in lower half of the lake). It was decided to run 26 inflow rates, varying from 100 to 125, over the period 1925-2025 to determine which gave the best fit with the data from the 1963 and 1976 surveys.

4. Allowances were made for decrease in volume of sediment trapped in the low-depth upper part of the lake at discrete time intervals. The decrease in volume occurs because of alternate wetting and drying with lake level fluctuations.

5. The computer program was written to calculate capacity at the end of each year. Capacity at the end of a year equals capacity at the beginning of that year minus the storage occupied by the sediment trapped. The capacity inflow ratio at the beginning of the year equals the capacity at that time divided by 156,390. Allowances for decrease in sediment volume, where exposed to alternate wetting and drying, were included in the program.

6. The computer runs indicated that a sediment inflow rate of 116 ac-ft (as laid down in the lower and hence deeper part of the lake) fitted the data well. The derived storage versus time curve is shown in Figure 3. Some numbers of interest are:

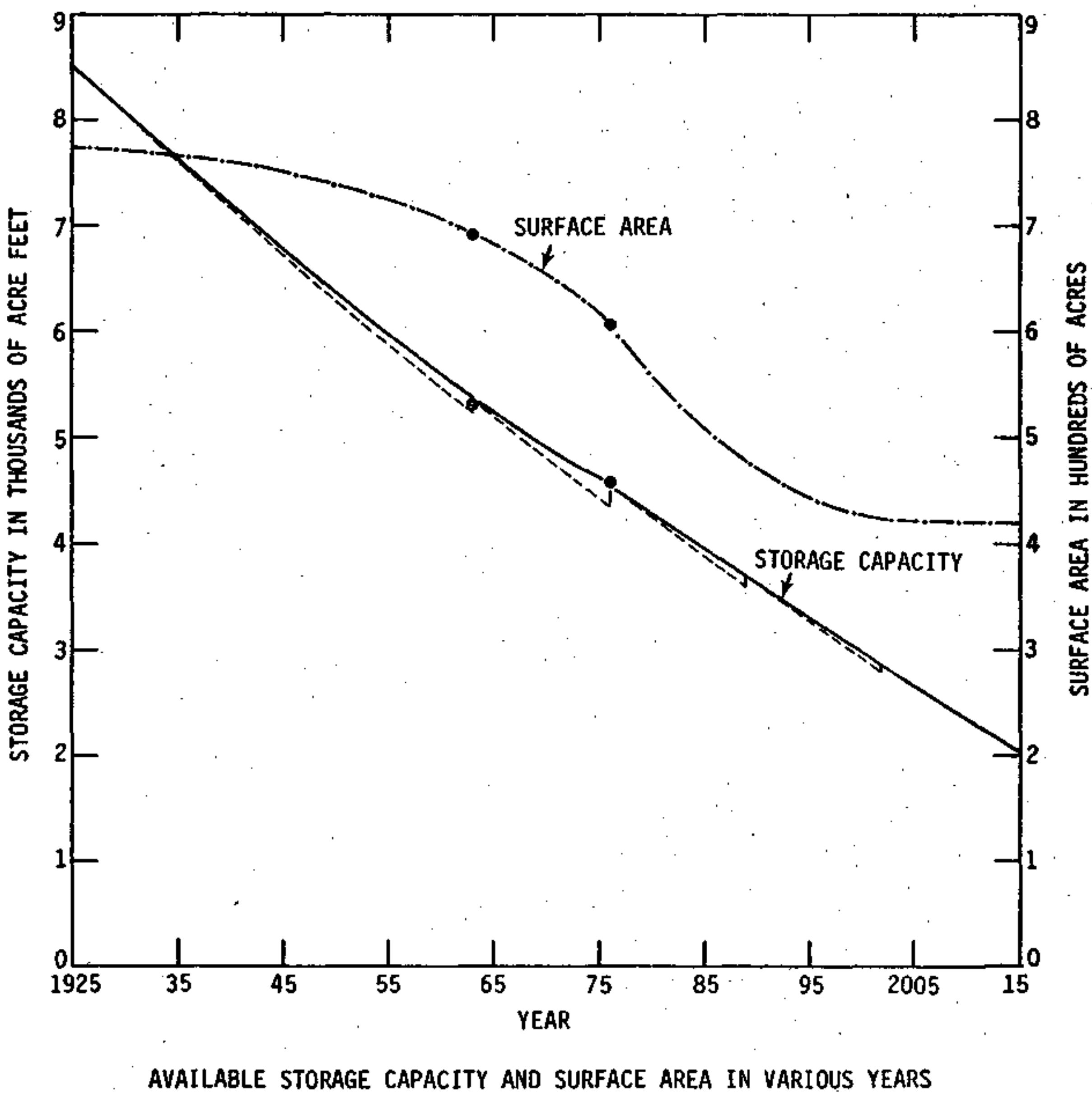
Year	*Storage capacity (ac-ft)*	*Surface area (acres)*
1980	4270	560
1990	3630	473
2000	2980	425
2010	2350	420
2020	1780	415

The surface area in acres was estimated by extrapolating the sediment trapped in different parts of the lake. Such data are available from the 1963 and 1976 surveys.

Future Yields from Lake Vermilion. The Lake Vermilion yield was computed for various storage volumes and associated lake surface areas with 40-, 25-, and 10-year droughts and for both full and half capacity use, by the methodology developed by Stall (1964). The Vermilion River near Danville was used as an index station and the data for net evaporation from a lake surface used were those given for Springfield (Roberts and Stall, 1967). The yield-storage curves for different recurrence intervals and full or half capacity use are shown in Figure 4.

For the expected storage volume in the years 1980, 1990, 2000, and 2010, allowing for the continuing sediment accumulation, the yields from the Vermilion are tabulated below.

Year	*1980*	*1990*	*2000*	*2010*
Storage, ac-ft	4270	3630	2980	2350
Full capacity use, yield in mgd				
40-year drought	10.53	9.8l	9.17	8.28
25-year drought	11.35	10.46	9.71	8.82
10-year drought	13.18	12.24	11.28	10.12
Half capacity use, yield in mgd				
40-year drought	7.67	7.25	6.76	6.30
25-year drought	8.02	7.61	7.12	6.51
10-year drought	9.22	8.72	8.12	7 . 39



3. Storage capacity and surface area of Lake Vermilion, 1925 - 2015

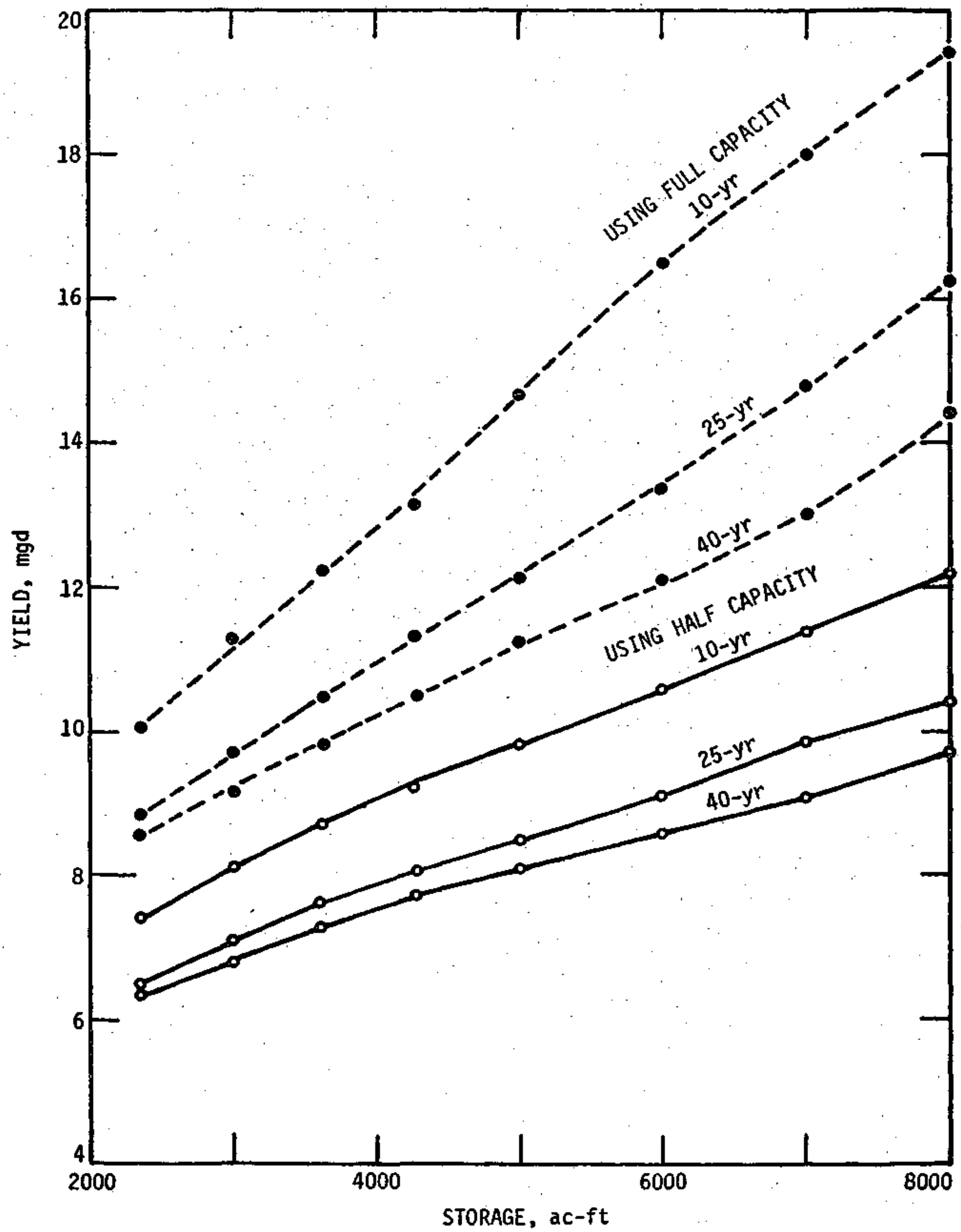


Figure 4. Lake Vermilion storage yield curves for 10-, 25- and 40-year droughts with half and full capacity use

The critical drawdown period in months with half capacity use and 40-year drought varies from about 6.5 to 8.5 months for a storage capacity of 2000 to 8000 ac-ft. The corresponding period with full use is 7.6 to 9.5 months. The yields from the lake used in meeting water demands will be those with half-capacity use and 40-year drought. The remaining half-capacity will serve recreation and other purposes. Any extra net storage created by raising the lake level and/or dredging the lake will-be available for full utilization for water supply.

Low Flow to Lake Vermilion. Monthly mean flows observed in the Vermilion River near Danville, USGS gaging station No. 3-3390, for the years 1933 through 1977 were adjusted to natural flows by subtracting estimated effluent flows from Champaign-Urbana, Rantoul, and Danville under low flow conditions. Values of total effluent discharge from the three towns to the Vermilion River system are estimated as 9.93, 11.81, 14.96, 20.28, and 26.10 cubic feet per second (cfs) for the years 1930, 1940, 1950, 1960, and 1970, respectively (see section 6, Water from the Vermilion River). A value of 26.10 cfs is used for the years 1971 through 1977. The drainage area above the Danville gaging station is 1290 square miles and that for the North Fork Vermilion River above the dam is 298. square miles. The monthly inflow, in mgd, to Lake Vermilion is estimated from

$$(Q - y)(298/1290)/1.548 \text{ or } 0.1492\ (Q - y)$$

in which Q is the observed monthly flow at the Danville gage, and y denotes the total effluent discharge from the three towns to the Vermilion River system. Division by 1.548 converts the cfs into mgd. The calculated monthly inflows to the lake are given in Table 1. The inflows greater than 0.1492 (150-y) are not shown because they exceed 20.9 mgd in 1930 or 18.5 mgd in 1970.,

The period of low flow, its duration, and average inflow over the period are given below for durations of 4 months or more.

No.	*Period*	*Months*	*Average inflow (mgd)*
1	8/40 - 11/40	4	4.69
2	8/44 - 1/45	6	5.09
3	9/53 - 2/54	6	4.23
4	7/54 - 12/54	6	4.02
5	8/64 - 12/64	.5	4.06
6	9/76 - 1/77	5	4,16

It is evident that an average inflow as low as 4 mgd over a period of 6 months can occur once in 40 years. The 6-month period occurred July to December, August to January, or September to February. With the exception of the year 1934, the lake level would be at the normal pool level in the month of June.

TABLE 1. Monthly Mean Inflow to Lake Vermilion for the Years 1933-1977

Water year	Monthly mean inflow to Lake Vermilion in mgd											
	Oct.	*Nov.*	*Dec.*	*Jan.*	*Feb.*	*Mar.*	*Apr.*	*May*	*June*	*July*	*Aug.*	*Sept.*
1933	4.26	17.43								9.34	6.11	14.12
1934		10.76	10.76		13.64			10.90	4.88	2.39	5.74	
1935												6.29
1936	4.02									3.10	2.89	18.22
1937											12.53	13.03
1938												
1939	11.01	12.53										5.64
1940	6.17	6.77	7.44	5.50						13.46	5.41	2.48
1941	3.27	7.59	13.76	15.81							10.76	7.07
1942											12.05	16.07
1943	8.76											11.23
1944	7.13	16.04	9.92	9.22						18.38	5.15	5.62
1945	4.58	6.00	4.79	4.45								
1946											19.34	6.47
1947	10.10										15.10	18.08
1948	13.31											8.86
1949	8.18	17.29										7.86
1950												
1951												
1952											13.79	8.12
1953	5.50	8.37	17.07								8.95	2.92
1954	2.34	3.28	3.64	6.08	7.12	15.89				4.85	6.56	1.65
1955	4.31	2.58	4.19								9.33	4.93
1956												7.80
1957	3.91	5.22	13.04									14.44
1958												
1959	12.86									8.43	4.02	5.97
1960											16.99	10.54
1961	5.19	12.67.	9.15	12.28							14.82	9.93
1962	13.77											
1963		14.58	9.72	6.37	10.37						6.79	3.30
1964	3.60	4.46	2.16	12.92	9.76						6.18	4.82
1965	2.91	11.88	9.73								10.67	
1966										10.72	7.32	10.81
1967	7.33									12.20	8.27	5.55
1968	9.87	15.93										11.84
1969	7.29										10.64	14.71
1970												
1971											18.04	
1972	11.62	7.00										
1973												
1974												
1975	13.56											
1976											11.47	4.87
1977	7.04	5.15	3.43	0.29	12.52							

Note: Blanks under monthly mean inflow denote inflow greater than 0.1492 (150-y) or (22.38-0.1492y) mgd; y varies from 9.93 in 1930 to 26.10 in 1970 and thereafter.

The statistics of the inflow duration curve were analyzed to define the average percent time the water would have to be pumped from other sources to augment the supply from the lake to meet the water demand. The percent time equals months divided

Inflow (mgd)	*Months*	*% time*
< 5	32	5.93
≤ 10	85	15.74
≤ 15	125	23.15
≤ 18	135	25.00

by 540 or the months in 45 years. The yield of the lake is 6.3 mgd for half capacity use and a 40-year drought. The inflow is about 4 mgd under these conditions. This gives about 2 mgd from utilization of half storage in 2010. The estimated pumping for various demand's are given below.

Demand (mgd)	*Range of pumping (mgd)*	*% time*	*% time pumping at full capacity*
10	0 - 4	11	6
15	0 - 9	20	10
20	0 - 14	24	12

An average 20 percent time at full pumping capacity will be assumed for alternative sources augmenting the supply from the lake. This provision allows ample margin for extra pumping during low flow periods because of the uncertainty about future inflows. Annual electrical charges for water transported from 6 to 15 miles are a small fraction of the total annual cost of optimally designed pipelines. Full-time pumping instead of 20 percent time increases the total cost by 1 to 5 percent depending upon the magnitude of static and friction head.

ALTERNATIVES

4. INCREASING YIELD BY RAISING THE LAKE LEVEL

The dam creating Lake Vermilion was constructed in 1925. It was designed by Donald W. Mead and Charles V. Seastone, Consulting Engineers, Madison, Wisconsin. The main section consists of 10 tainter gates, 14 feet wide and 14 1/2 feet high, with 3-foot thick piers. Figure 5 shows a section through the center of a bay. Some levels of interest are the spillway crest at 563.2, normal pool at 577.2, and roadway top at 587.2 feet elevation above mean sea level. To the west of the main section, there are two 4-foot by 4-foot sluice gate outlets with the upstream and downstream inverts at 563.2 and 541.7 feet, and a 12-foot wide trash gate bay with crest at 574.2 feet elevation. At the extreme west there was a 50-foot long free spillway with crest at the normal pool of 577.2 feet. The spillway was later blocked and it is now inoperative.

The structure is founded well into the shale for foundation stability. Upstream and downstream toes are provided for additional resistance against sliding and a system of tile drains to reduce uplift pressures. The consultants estimated the spillway capacity at 28,000 cfs with normal pool and free flow conditions. The tainter gates are operated to keep the lake level at the normal pool though the historical record shows quite a few episodes of a 3-foot or more rise in level, followed by a similar decline because of manual operation of the gates.

Main Spillway Capacity. The capacity was computed for various pool levels, gate openings, and tailwater levels following the methodology outlined in the *Design of Small Darns* (Bureau of Reclamation, 1973) and *Design Criteria* (Waterways Experiment Station, 1952). The computed discharge capacities for various gate openings and upstream lake levels, but with the tailwater below the crest of the dam, are shown in Figure 6. The spillway capacities for the condition when the gates are raised high enough for the nappe to clear the bottom of the gate are shown in Figure 7 under varying degrees of partial submergence. The explanation of symbols in the discharge (Q) formulas In Figures 6 and 7 can be seen in the references cited above. Under free flow conditions and no submergence, the spillway capacity at various lake levels is as given below.

Lake level (feet)	*Spillway capacity (free flow) (cfs)*
577.2	26,600
579.2	32,100
581.2	37,600
583.2	43,800

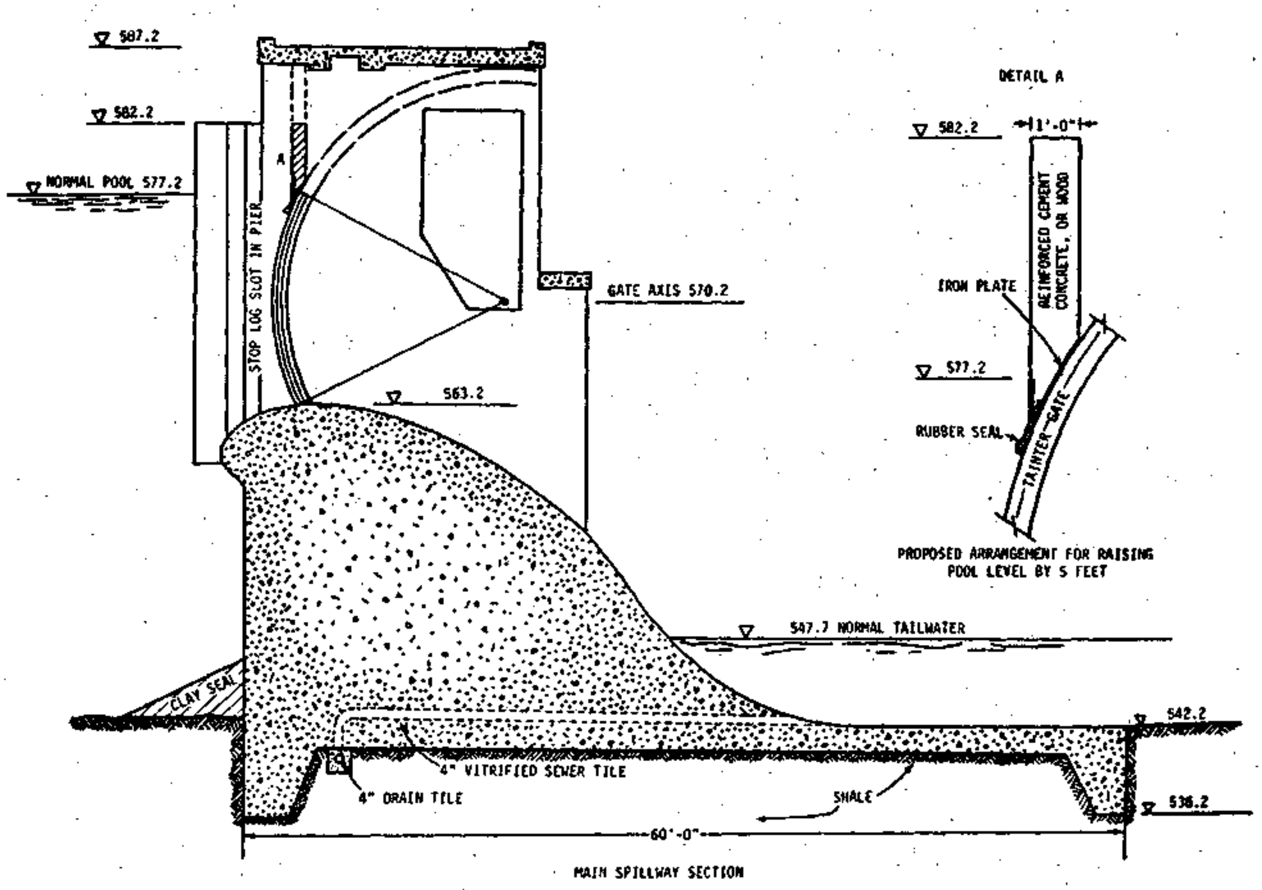


Figure 5. Lake Vermilion Dam cross section and arrangement for raising lake level

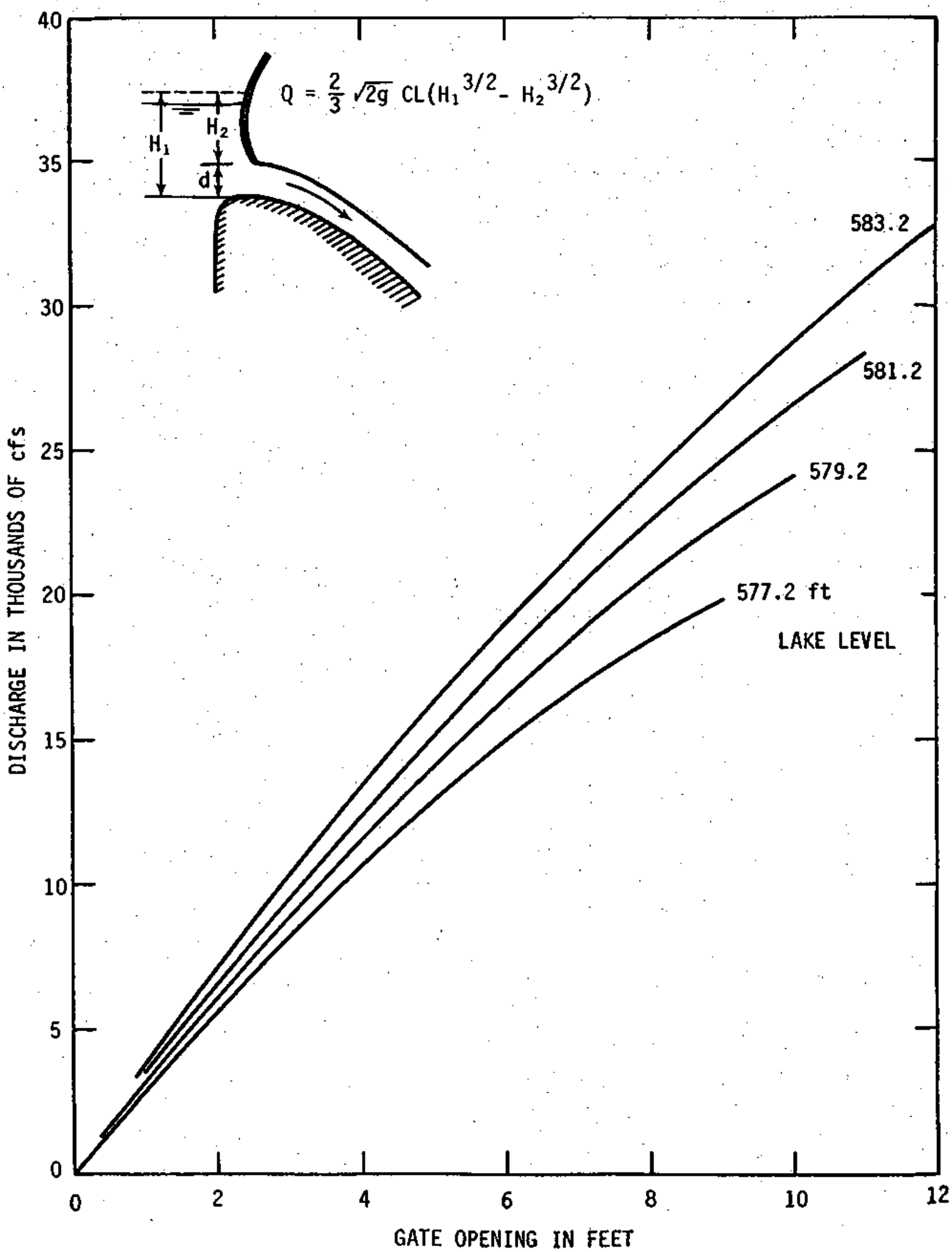


Figure 6. Main spillway capacity for different pool levels with partial gate opening and no submergence

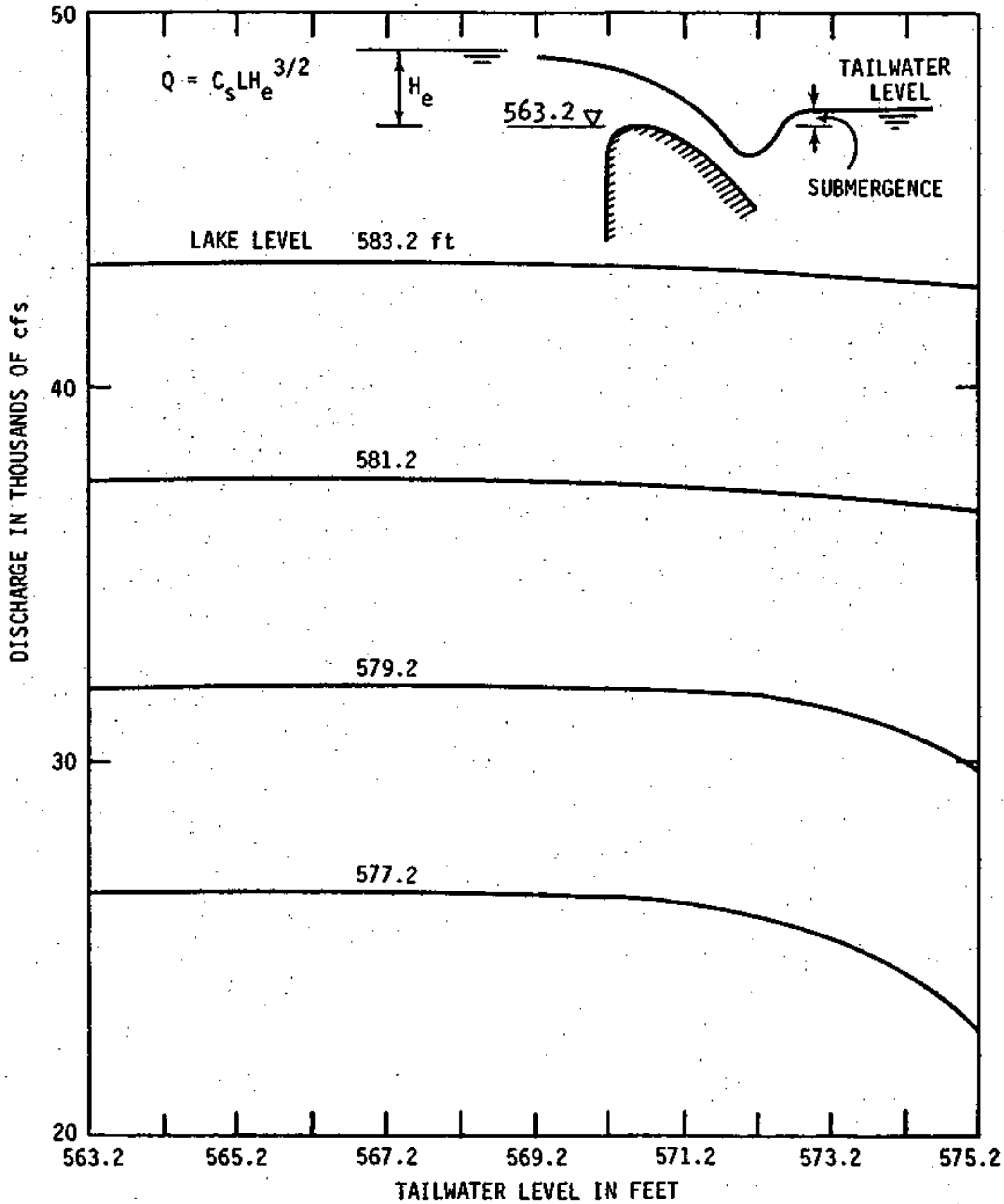


Figure 7. Main spillway capacity under free flow condition and partial submergence

Even a submergence (tailwater level minus crest elevation) of 8 feet reduces the free flow discharge capacity by only about 1 percent.

The two sluices and the trash bay pass 1050, 1330, 1640, and 1890 cfs at lake levels of 577.2, 579.2, 581.2, and 583.2 feet, respectively. Thus, the total discharge capacity at the four lake levels can be taken as 28,000, 33,000, 39,000, and 45,000 cfs.

Tailwater for 100-year Flood. The 100-year flood in the Vermilion River near Danville, 3.2 miles downstream of the confluence with the North Fork Vermilion River, is estimated as 41,400 cfs with the methodology recommended by the Water Resources Council (1976). With the method outlined for ungaged areas by Carns (1973), the 100-year flood at the Lake Vermilion damsite is estimated as 16,700 cfs. Sixteen cross sections were developed in the 4.2-mile reach of the North Pork Vermilion River from the damsite to the confluence with the Vermilion River and 10 cross sections in the 3-2-mile reach of the main river from the confluence with the North Fork to the USGS gaging station near Danville. The flood elevation for the 100-year flood at the gaging station was used to compute (The Hydrologic Engineering Center, 1976) the backwater profiles for various roughness coefficients, starting from the gaging station to the confluence along the main river and from there to the damsite along the North Fork. A summary of pertinent water surface elevations is given in Table 2. The water surface elevation at the gaging station on the Vermilion River near Danville is 530.98 feet **for** the 100-year flood of 41,400 cfs.

The crest of the dam is at 563.2 feet elevation, much higher than the tailwater level obtained even with high values of n. No submergence may be expected for a 500-year flood which is estimated to be 51,000 cfs at the gaging station on the Vermilion River near Danville.

Other Flood Magnitudes. The drainage area of the Vermilion River above the gaging station near Danville is 1290 square miles. Continuous daily flow and flood data are available from

TABLE 2. Water Surface Elevations for the 100-Year Flood in the North Fork and Vermilion Rivers

Mannings roughness n	*Water level at confluence (feet)*	*Mannings roughness n*	*Water level below dam (feet)*
0.03	537.36	0.03 (0.055)	556.71
		0.04 (0.065)	558.32
0.04	539.29	0.04 (0.065)	558.32
		0.05 (0.075)	559.69

Note: Figures in parentheses denote the values of n for the overbank flow.

June 1928 to September 1976. The floods computed for recurrence intervals varying from 2 to 500 years are given in Table 3. The drainage area of the North Fork above the Lake Vermilion dam is 298 square miles. The floods for recurrence intervals up to 100 years were computed with the state standard method (Cams, 1973). These floods are listed in Table 3. The 200- and 500-year floods are estimated from the relationship between the floods on the North Fork and the Vermilion River.

The total existing spillway capacity for the lake at 577.2 feet is about 28,000 cfs, higher than a 1000-year flood. The capacity increases to 33,000, 39,000 and 45,000 cfs with an increase in lake level of 2, 4, and 6 feet, respectively. Therefore, the existing spillway capacity is more than adequate for passing high floods used in the design of such structures.

Raising the Lake Level. The normal lake level at present is 577.2 feet, near the top of the tainter gates. An increase in lake level can be achieved by a simple alteration as shown in Figure 5 for an increase of 5 feet in lake level. The 1-foot thick breast wall, spanning between the piers, can be constructed in reinforced cement concrete or wood. The ten 15-foot long walls, allowing 6 inches on either side for fitting into the grooves cut into the piers and abutment, would have their top level at the desired lake level and would have a flexible rubber seal fitted on the curved iron plate. Even if the breast wall top is kept at 582.2 feet or 5 feet above the present lake level, the lake level can be maintained anywhere in the range from 577.2 to 582.2 feet. This structural modification seems to be the cheapest means of achieving higher lake levels. The height of the trash bay gate can be increased to conform to the new lake level. The construction of breast walls will not alter the free flow discharge capacity of the spillway even with the lake level at 582.2 feet, or 19 feet above the crest, because the nappe height over the crest will be about 12.7 feet, which is less than the opening between the breast wall and the dam crest.

TABLE 3. Flood Magnitudes for the North Fork and Vermilion Rivers

Recurrence interval (years)	*Flood magnitude (cfs)*	
	North Fork	*Vermilion River*
2	4,900	13,130
5	7,840	20,970
10	9,850	26,140
25	12,250	32,480
50	14,660	37,030
100	16,700	41,400
200	18,700	45,630
500	21,250	51,000

The clearance between the present normal lake level and the low steel of the Denmark Road Bridge is 6.2 feet. It is desirable to have a clearance of 5 feet or more for access by boats from the upper part to the lower part of the lake and vice versa. This clearance will not be available if the lake level is raised by more than a foot. A culvert with a 20 feet wide and 15 feet deep opening will be provided under the road to. maintain the access. The top and bottom reinforced cement concrete (RCC) slabs of the culvert under the raodway will be supported by the RCC beams connected by suitably spaced piers along the center line of the culvert.

The available historical record shows quite a few rises in lake level of 3 feet or more. With the high discharge capacity of the spillway, this rise can be limited to a few inches necessary to actuate an automatic gate operation. The automatic operation may let two middle gates open up to 4 feet to keep the water level within 2 inches of the desired level, then allow two adjoining gates to open up to 4 feet, and if the level rises beyond 3 inches of the desired level, allow the other 6 gates to open up to 4 feet. If the lake level still keeps on rising, all the gates can be opened gradually to the extent required to maintain the level as close to the desired level as possible. The automatic controls would actuate the motors to raise the gates. The manual arrangement would also be maintained. The cost of automatic controls would Include new motors, standby power, and any modifications to the existing hoisting mechanism.

The Inter-State Water Company owns or has flowage easements for most of the land to be submerged by raising the lake level by 5 feet. Because any land to be acquired is now subject to frequent flooding, the acquisition costs will be minimal. Therefore no extra costs are allocated for it. The raising of lake level would increase the recreation potential as well as the yield of the lake.

Dam Stability. An analysis of the available structural drawings of the dam indicates that the existing piers can easily bear any water pressure transmitted by the breast walls because of raising the lake level. The safety of the dam against overturning and sliding was tested for lake levels of 577.2 and 582.2 feet. The safety factors against overturning were estimated as 2.4 and 2.0, and against sliding 9.4 and 7.8 with a unit shear resistance of concrete at the foundation shale-concrete contact of 60 pounds per square inch (psi) as determined by the methodology described in the *Design of Small Dams Bureau of Reclamation* (1973)• It is assumed that the shear resistance of mass concrete is less than one-half that of the shale foundation. Values of shear resistance in limestone and sandstone vary from 1200 to 3000 and from 300 to 3000 psi (Eshbach, 1968). Values of shear resistance for shale are not given, but are believed to be 120 to 1000 psi. In these computations, the foundation was considered effective up to the point where the tile drain exits to the tailwater, or a base width of about 40 feet..

An effort was made to gather data on the seismic coefficient applicable to a dam on the shale foundation near Danville. The seismic coefficient recommended for building design (Department of Army, 1973) in the general area is 0.25. However, for a structure on shale rock, the seismic coefficient will be much lower. A check on the factor of safety with the seismic coefficient of 0.1 indicated a decrease of about 15 to 20 percent in the safety factor.

Increase in Storage Capacity. A computer program was developed to calculate the storage capacity of Lake Vermilion for the years 1980 through 2010 with various storage capacity increments in 1980 or in three equal installments for the years 1980, 1990, and 2000. The storage capacity increments are achieved by raising the lake level from the existing 577.2 feet to an elevation commensurate with the desired increase in storage capacity. The results of the computer runs are given in Table 4. Runs were also made for increments in two steps during the years 1980 and 1990.

TABLE 4. Storage Capacity of Lake Vermilion with Varying Storage Increments

Total increment (ac-ft)	*Storage capacity in ac-ft in the year*			
	1980	*1990*	*2000*	*2010*
A. Total increment in the year 1980				
0	4285	3630	2980	2350
600	4885	4203	3465	2834
1200	5485	4776	4004	3335
1800	6085	5354	4554	3851
2400	6685	5934	5111	4380
3000	7285	6517	5674	4920
3600	7885	7101	6242	5467
4200	8485	7688	6812	6019
4800	9085	8274	7384	6576
B. Total increment in 3 equal installments in 1980, 1990, and 2000				
0	4285	3630	2980	2350
600	4485	3826 (4026)	3298 (3498)	2865
1200	4685	4014 (4414)	3662 (4062)	3390
1800	4885	4203 (4803)	4029 (4629)	3922
2400	5085	4393 (5193)	4400 (5200)	4465
3000	5285	4584 (5584)	4775 (5775)	5017
3600	5485	4776 (5976)	5182 (6382)	5573
4200	5685	4968 (6368)	5531 (6931)	6134
4800	5885	5161 (6761)	5911 (7511)	6699

Note: Figures in parentheses give the storage capacity after **adding** the installment in 1990 or 2000.

It is evident from Table 4 that with a single storage increment in the year 1980, the storage capacity and hence the yield for water supply will decrease from a maximum in 1980 to a minimum in 2010. With increments in three installments, the yield may decrease or increase with years depending on the magnitude of the total storage increment. The. storage values in Table 4 were converted to the corresponding yield values to analyze the most desirable method of raising the lake level in the best interest of meeting the water supply demand.

Available Yields. For the existing lake with its normal level of 577.2 feet, the dependable yield is that determined for a 40-year drought and half-capacity use; the other half is reserved for meeting water requirements for recreation, fish, and other purposes. However, any increase in capacity because of raising the lake level above 577.2 feet could be fully utilized for water supply after allowing for extra sediment volume trapped hecause of the added storage capacity. The estimated yields based on this premise are shown in Figure 8. The storage increment achieved by raising the lake level can be estimated from the following information.

Rise in lake level (feet)	*Storage increment (ac-ft)*
1	790
2	1600
3	2440
4	3310
5	4200

Raising the lake level in 1980, or in equal installments in 1980, 1990, and 2000, or in any other fashion depends on the alternatives to meet the deficits in water supply. If the supply from another source is used to augment the yield from the lake and if this supply is available from 1980 onward, the gradual raising of the lake level sufficient to provide a yield to match the difference between the demand and the supply from the other source would be the desired method. There is a possibility that the city of Danville may attract a sizeable industry after some years and thus significantly increase the demand for water. The choice of an adequate source to meet the future increase in demand can be made later from an array of sources with different supply potentials. For the SWS demand curve, the demands can be met up to the year 1995 by raising the lake level by 3 feet in 1980 and gradually raising it by another 2 feet by the year 1995, or by raising the lake level by 5 feet in 1980. In the period 1995 through 2010, the deficit would increase almost linearly from zero to about 2.9 mgd.

Associated Costs. The costs are determined by the total increase in lake level desired. The component costs have been arrived at after discussions with the Division of Water Resources and the Illinois Department of Transportation. The costs

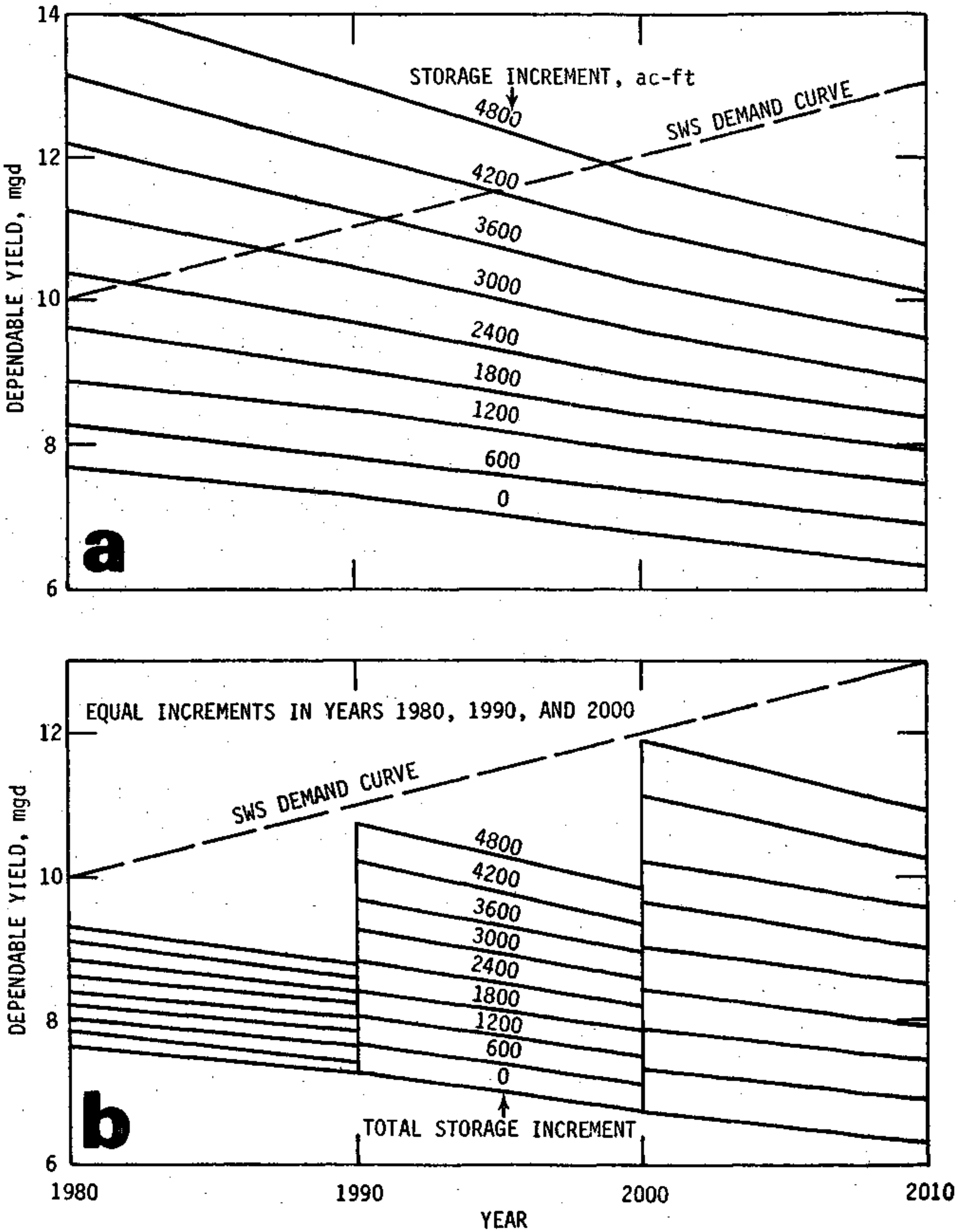


Figure 8. Dependable yield with storage increase from raising lake level

associated with raising the lake level by 1, 2, 3, 4, and 5 feet are given in Table 5. The cost of providing the culvert for maintaining access by boats has been estimated in consultation with the Division of Water Resources. The annual costs for raising the lake level from 1 to 5 feet are shown in Figure 9.

The ice pressures on the gates and piers during severe winter conditions are mitigated at present by a bubbler system. The gates get stuck at the piers because of frozen water and moisture between the gates and seals. Steam is blown from downstream to loosen the gates. A heating mechanism that can be energized during such conditions to keep the gates free and to melt any ice formed on the gate and/or breast wall will not only ensure automatic operation but also increase the safety of the gates and the dam by eliminating ice pressures. The gates have been in operation for more than 50 years. Their condition needs to be checked for sustaining the imposed loads. Cost of a new existing-size gate and hoisting mechanism is $45,000, or a total of $450,000 for the ten gates. Cost of a 19-ft high gate including the hoisting mechanism is $58,000, or a total of $580,000. Cost of new gates is not included because such a need is not dependent on the raising of the lake level. The automatic operation will reduce the present operation, maintenance, and repair (OM&R) cost but will increase the energy cost for the heating mechanism. It is assumed that the total OM&R cost with automatic operation and raising the lake level will remain the same as for the present conditions.

TABLE 5. Cost of Raising Lake Levels to Increase the Yields

Item	*Cost in thousands of dollars for rise in feet of* 1	2	3	4	5
Breast walls and miscellaneous	14	18	22	26	30
Automatic operation, standby power, and modifications to hoisting equipment	138	138	138	138	138
Heating equipment for gates and breast walls	24	40	50	57	62
Culvert with 20 feet wide and 15 feet deep opening	—	200	200	200	200
TOTAL	176	396	410	421	430
Annual cost at 8% and 50 years	14.4	32.4	33.5	34.4	35.1

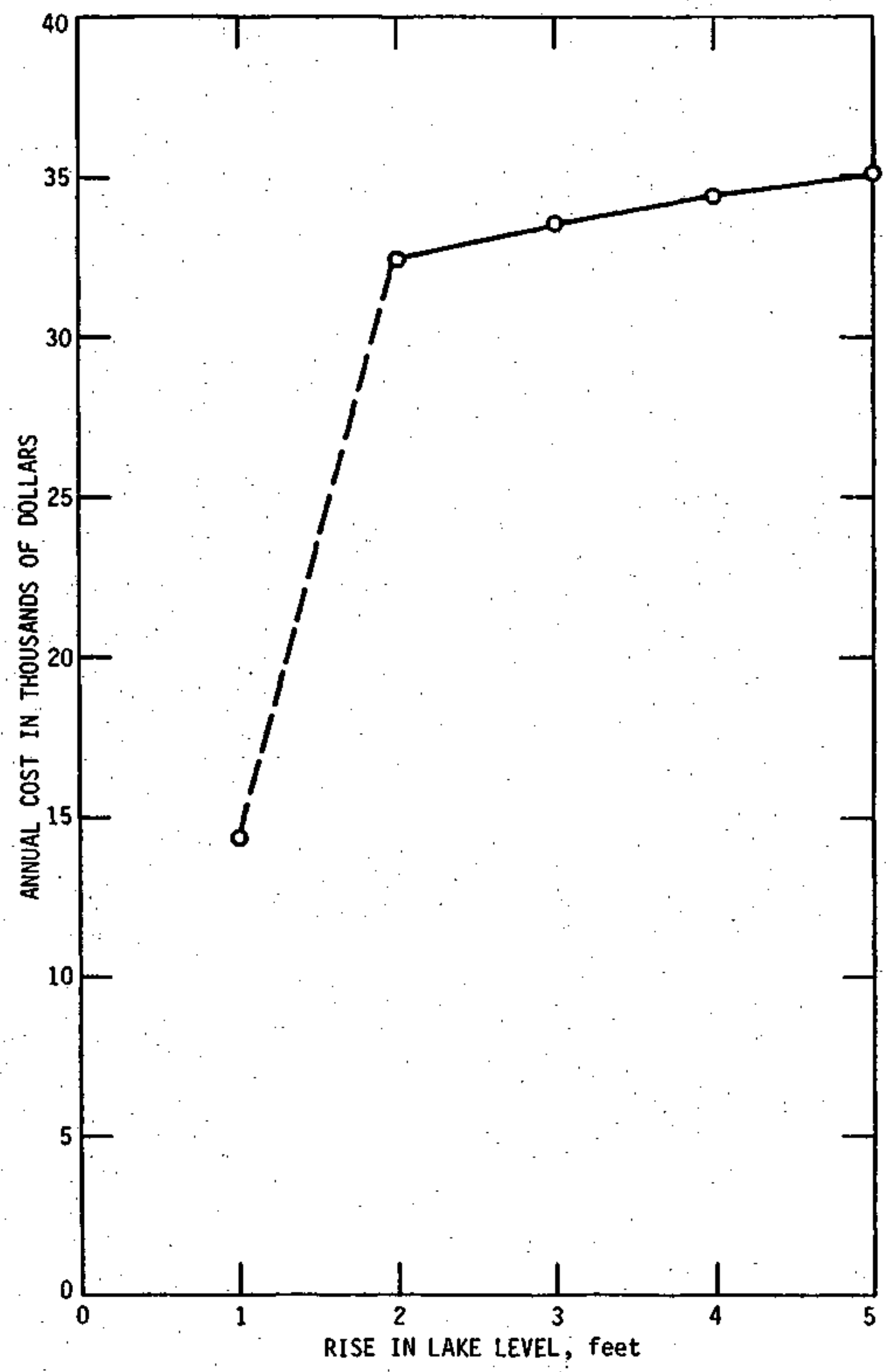


Figure 9. Annual cost of raising lake level

5. DREDGING OP LAKE VERMILION

The water storage capacity of a man-made lake decreases at a slightly decreasing rate with time because of the sediment deposited in the rather quiescent waters of the lake. New lakes are very difficult to plan and build because of the ecological and environmental concerns. It may be worthwhile to keep an existing lake in good condition for serving water supply, recreation, and other purposes, rather than to let it silt up and become mud flats causing significant environmental damage. An economic assessment of penalties for deterioration of a lake is beyond the scope of the present study, but an effort is made to consider the problem from a humanistic viewpoint.

Storage Capacity with Dredging. The computer program developed for estimating the expected storage capacity of Lake Vermilion in future years was modified to allow for annual dredging rates of 30 to 150 ac-ft starting from the year 1980. The resulting storage capacities for these rates and with no dredging are shown in Figure 10a for the present normal lake level of 577.2 feet. The dependable yields were calculated on the same basis as for raising the lake level, that is, the net Increase in capacity (obtained by subtracting the increased sediment volume trapped from the increase in capacity due to dredging) would be fully utilized for water supply but the lake capacity without any dredging or raising of the lake level would be utilized only to the extent of 50 percent. The resulting yields for a drought of 40 years are drawn in Figure 10b.

Raising the Lake Level and Dredging. Computer runs were made with assumptions of 1800, 3000, and 4200 ac-ft increases in storage achieved by raising the lake level in 1980 as well as in three equal installments in 1980, 1990, and 2000, and with four dredging rates of 50, 70, 90, and 110 ac-ft per year. The results for the total storage increment in 1980 and four dredging rates in terms of dependable yield in mgd are shown in Figure 11. The SWS, Danville, and HM&G demand curves are also shown in this figure. The SWS demand curve can be matched by various combinations of storage increment and dredging. Two alternatives, based on a maximum rise in lake level of 5 feet and a maximum annual dredging rate of 110 ac-ft are described below.

A. Minimum rise in lake level with maximum dredging: For an increase of 2400 ac-ft in capacity achieved by raising the lake level by 3 feet and an annual dredging rate of 110 ac-ft, the dependable yield for the years 1980 through 2010 is shown by curve A in Figure 12.

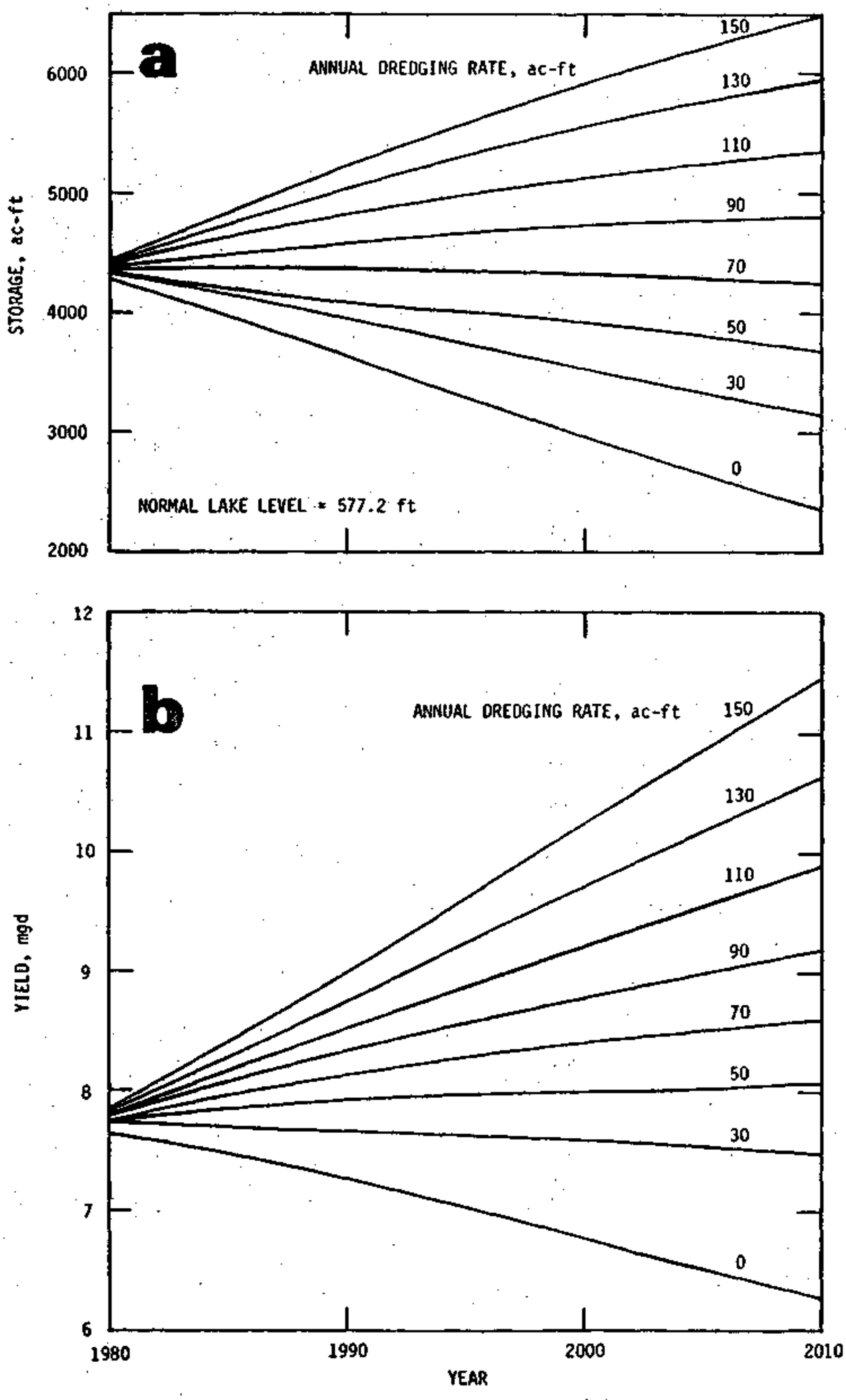


Figure 10. Storage capacity and dependable yield of Lake Vermilion with dredging at different rates

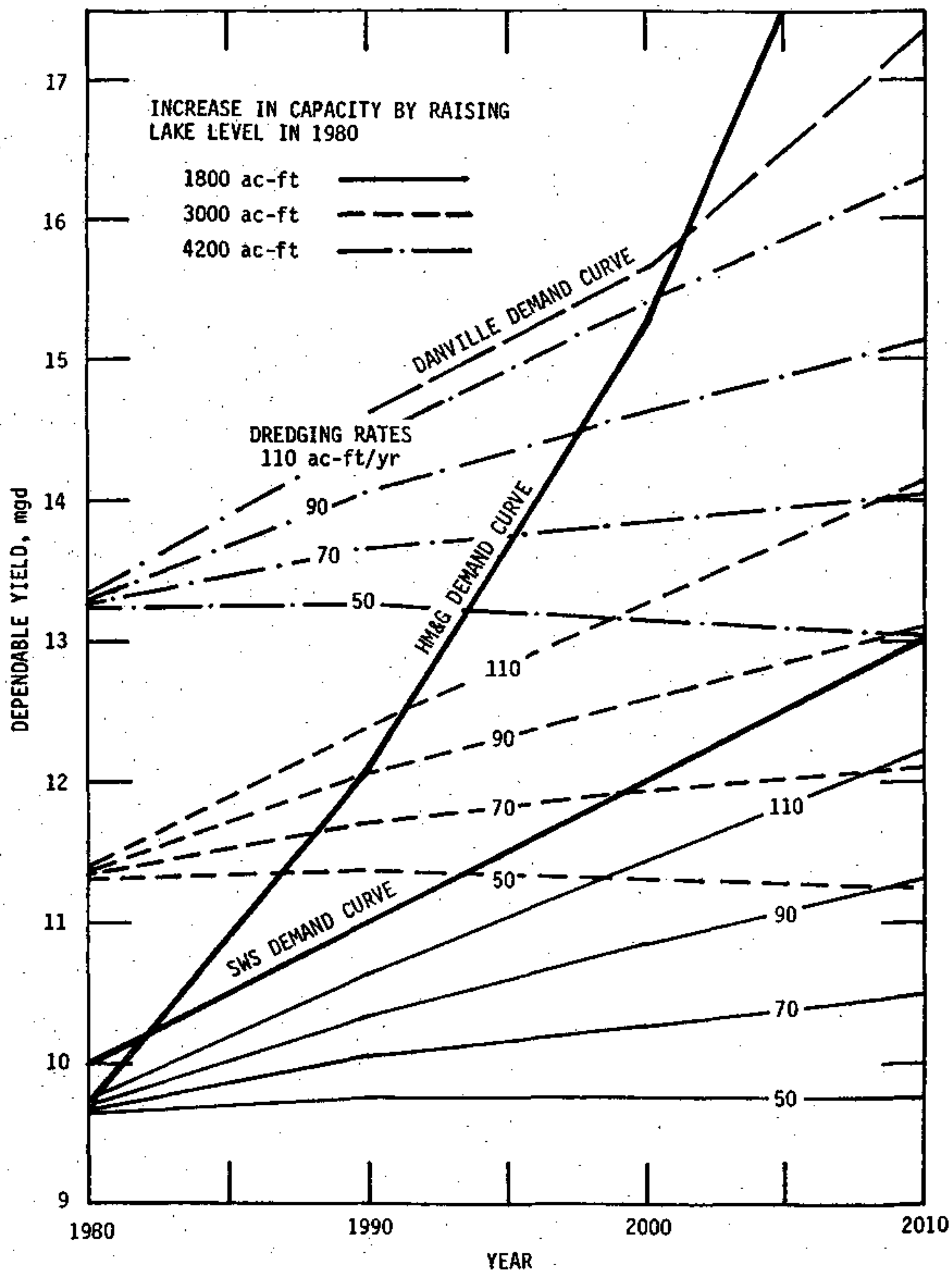


Figure 11. Dependable yield of Lake Vermilion with raising of the lake level and dredging

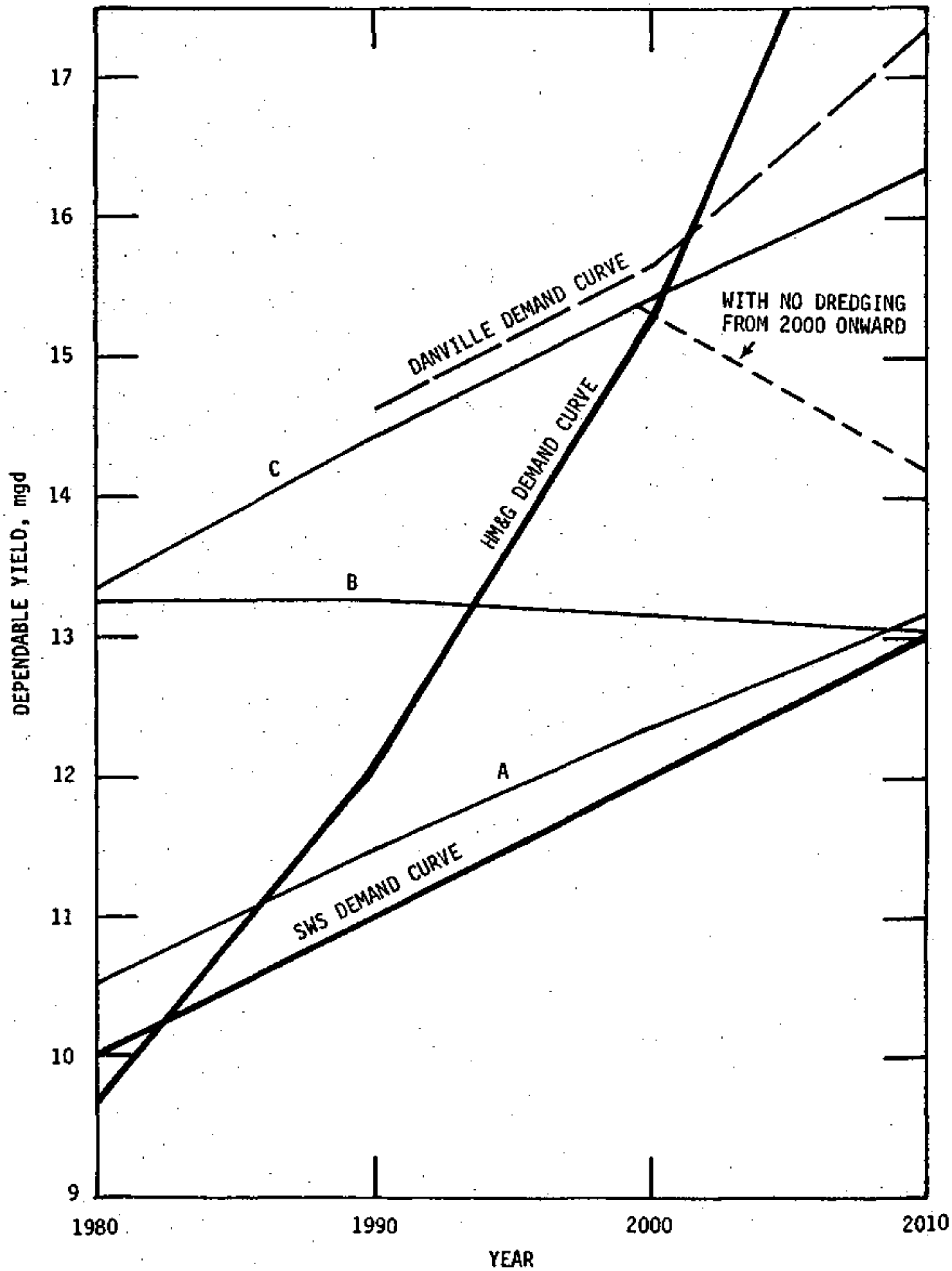


Figure 12. Combinations of raising the lake level and dredging for meeting water supply demands

B. Maximum rise in lake level with minimum dredging: For an increase of 4200 ac-ft in capacity achieved by raising the lake level 5 feet and an annual dredging rate of 50 ac-ft, the dependable yield for the years 1980 through 2010 is shown by curve B in Figure 12.

A storage increment of 4200 ac-ft or a rise in lake level of 5 feet combined with an annual dredging rate of 110 ac-ft can meet the HM&G demand curve until the year 2000 as shown by curve C in Figure 12. The maximum deficit in 2010 would be 19.9 minus 16.3, or 3.6 mgd. This can be supplemented from other sources, such as directly from the Vermilion River upstream of its confluence with the North Fork Vermilion River. Curve C is about 0.5 mgd below the Danville demand curve up to the year 2000.

Cost of Dredging. Small portable dredges are available and these can be readily assembled for use in small and medium size lakes. An 8-inch rotating cutter-head dredge (Roberts, 1976) has an average production rate of 150 cubic yards per hour and can discharge sediment up to a distance of 3000 feet, depending on the lift and other factors. Another type of dredge removes sediment by an auger mounted on the end of an hydraulically operated boom. The sediment slurry can be pumped to distances of 2500 feet and to a maximum elevation of 150 feet above the dredge.

An adequate spoil area is necessary for disposal of the dredged material. The spoil area should be divided into interconnected subareas, separated by well-compacted levees of sufficient height to hold the dredged material up to the desired level and with openings arranged so as to increase the path of flow and thus facilitate maximum sedimentation. Though the Federal Water Pollution Control Act of 1972 provides for controls on sewage sludge disposal, there is little information available on any controls on dredged sediment disposal. There are no major chemical and other industries in the drainage area above Lake Vermilion, and the sediment in the lake does not have concentrations of undesirable chemicals associated with such industries.

The dredging can be carried out from March 15 to November 15, an average of 8 months a year. This gives 172 working days, Monday through Friday, allowing for 3 holidays. At the rate of 8 hours a day, the working hours amount to 1376 a year. Allowing for 12 to 13 percent down hours for maintenance and repairs, a net of 1200 working hours per year can be adopted for computing dredging costs. The annual cost of operating a dredge having an average capacity of 130 cubic yards (cu yd) per hour is estimated,

Operator at $l6,000/year for 9 months	$12,000
Helper, semi-skilled at $12,000/year, for 9 months	9,000
Supervisor at $20,000/year, 1/4 time for 9 months	3,750
Fuel and repairs at $8/hour for 1200 hours	9,600
Total OM&R (operating, maintenance and repair) cost per year	$34,350

Annual cost of $140,000 dredge at 8 percent and 10 years life	$20,850	
Total annual cost	$55,200	
Volume of dredged material including water, in 1200 hours at 130 cu yd per hour	156,000	cu yd
Volume of material dredged from lake equals volume discharged by dredge x 0.6	93,600	cu yd
	or 58.0	ac-ft
Volume of dredged material after drying and compaction under its own weight equals volume dredged from lake x 0.6	34.4	ac-ft
Spoil area built-up each year assuming average 7-foot depth	5	acres
Annual cost of spoil area preparation and restoration @ $2,000 per acre	$10,000	
Annual cost of levees in spoil area, with graded filter so that water drained has minimum of sediment, @ 26¢/cu yd of material dredged from the lake	$24,300	
Total annual cost (dredging, levees, and spoil area preparation, etc.)	$89,500	
Cost per ac-ft of storage created by dredging equals $89,500/58	$ 1,543	
Cost per cu yd of storage created	96¢	
Cost per cu yd of material in spoil area equals 96/0.6 or 160¢	$1.60	

If the material laid on the spoil area is compacted mechanically to achieve a 95 percent or higher Proctor density, the cost per cubic yard and per ac-ft of storage created would be $1.25 and $2,020, respectively, and the spoil area requirement with an average 7-foot depth would be about 4 acres. In other words, the total annual cost with compaction in about 4 acres of spoil area would be $117,200 for 58-ac-ft of storage capacity created in the lake. The estimated annual costs of alternatives A and B for meeting the SWS demand curve are derived below.

A. Raising lake level by 3 feet in 1980 and dredging at 110 ac-ft per year

	Annual Cost
Raising lake level by 3 feet	$ 33,500
Dredging 110 ac-ft/year, using two dredges	
without compaction in spoil area	179,000
with compaction in spoil area	234,400
Total annual cost	
1. Including compaction	267,900
2. Excluding compaction	212,500

B. Raising lake level by 5 feet in 1980 and dredging at 50 ac-ft per year

	Annual Cost
Raising lake level by 5 feet	$ 35,100
Dredging 50 ac-ft/year, using one dredge	
without compaction in spoil area	89,500
with compaction in spoil area	117,200
Total annual cost	
1. Including compaction	152,300
2. Excluding compaction	124,600

The total annual cost for raising the lake level by 5 feet and dredging at the rate of 110 ac-ft per year will be $269,500 and $214,100 for the two cost alternatives, respectively.

Credits for Reclaimed Area. An average of .5 acres of good farmland will be available every year when one dredge is operated and the deposits in the spoil area are not compacted mechanically. An average appreciation of $2000 per acre can be assumed. Thus, there may be an annual credit of $10,000 with the use of one dredge and $20,000 with the use of two dredges. If the deposits are compacted to 95 percent or higher Proctor density, the spoil area could be transformed into prime urban land bordering a lake. This could bring $20,000 or more an acre by planning reclaimed areas suitable for subdivision development. The annual credit in this case could be much higher than $80,000 with the use of one dredge and $160,000 with the use of two dredges.

The Inter-State Water Company owns about 400 acres of low-lying land between the bluffs (bordering the lake on the east and west) and the lake. The area should be sufficient for meeting the spoil area acreage requirements from 1980 through 2010. Annual costs with and without credits for each of the two cost alternatives for raising the lake level and dredging are given in Table 6. It is evident that if credits are given for the reclaimed area, the compaction of spoil areas for turning them into prime urban land would be desirable.

TABLE 6. Annual Cost of Raising the Lake Level and Dredging

Rise in level (feet)	*Dredging rate (ac-ft/yr)*	*Compaction of dredged materials?*	*Annual cost in dollars* without credit	*Annual cost in dollars* with credit
3	110-116	No	212,500	192,500
		Yes	267,900	107,900
5	50-58	No	124,600	114,600
		Yes	152,300	72,300
5	110-116	No	214,100	194,100
		Yes	269,500	109,500

6. WATER PROM THE VERMILION RIVER

Two gaging stations on the Vermilion River, upstream and downstream of the confluence with the North Fork Vermilion River, are the Vermilion River near Catlin, USGS No. 3-3385, and the Vermilion River near Danville, USGS No. 3-3390. The drainage area above these gaging stations is 959 and 1290 square miles, respectively. Continuous daily flow data are available for 19 years, 1940-1958, at Catlin and 55 years, 1922-1976, at the Danville gage. The 7-day 10-year low flows (Singh and Stall, 1973) in the Vermilion River near Catlin and Danville are 19.0 and 33.0 cfs, respectively, and in the North Fork Vermilion River near the confluence it is 0.40 cfs for a drainage area of 307 square miles. The 7-day 10-year low flow map for the general area under consideration is shown In Figure 13.

Low Flow Adjustment. In order to use the low-flow statistics at the Danville gage for determining the availability of water for augmenting the Danville water supply, it was necessary to devise a method for adjusting the low flows to the 1970 condition of effluents entering the Vermilion River. The effluent volume from Champaign-Urbana, Rantoul, and Danville forms a major portion of the river flow at the Danville gage during low-flow conditions. The increase in water use at these three towns from 1910 to 1970 is shown in Figure 14. Ratios of effluent discharge as shown In Figure 13 to corresponding water use in 1970 as shown in Figure 14 were used to convert the water use in different years to effluent discharge under low-flow conditions. These ratios are 0.60, O.65, and 0.74 for Champaign-Urbana, Rantoul, and Danville, respectively. The ratios for Champaign-Urbana and Rantoul are somewhat lower because the total effluent is not being discharged to the streams in the Vermilion River drainage basin. The estimated effluents discharged from the three towns to the Vermilion River system during low-flow conditions are given in Table 7.

TABLE 7. Estimated Effluents Entering the Vermilion River System Upstream of the Danville Gage

	Estimated effluents in cfs				
Year	*Champaign -Urbana*	*Rantoul*	*Danville*	*Total*	*Low flow adjustment*
1930	2.28	2.54	5.11	9.93	16.17
1940	2.94	2.73	6.14	11.81	14.29
1950	4.56	2.93	7.47	14.96	11.14
1960	7.86	3.32	9.10	20.28	5.82
1970	11.20	4.00	10.90	26.10	0.00

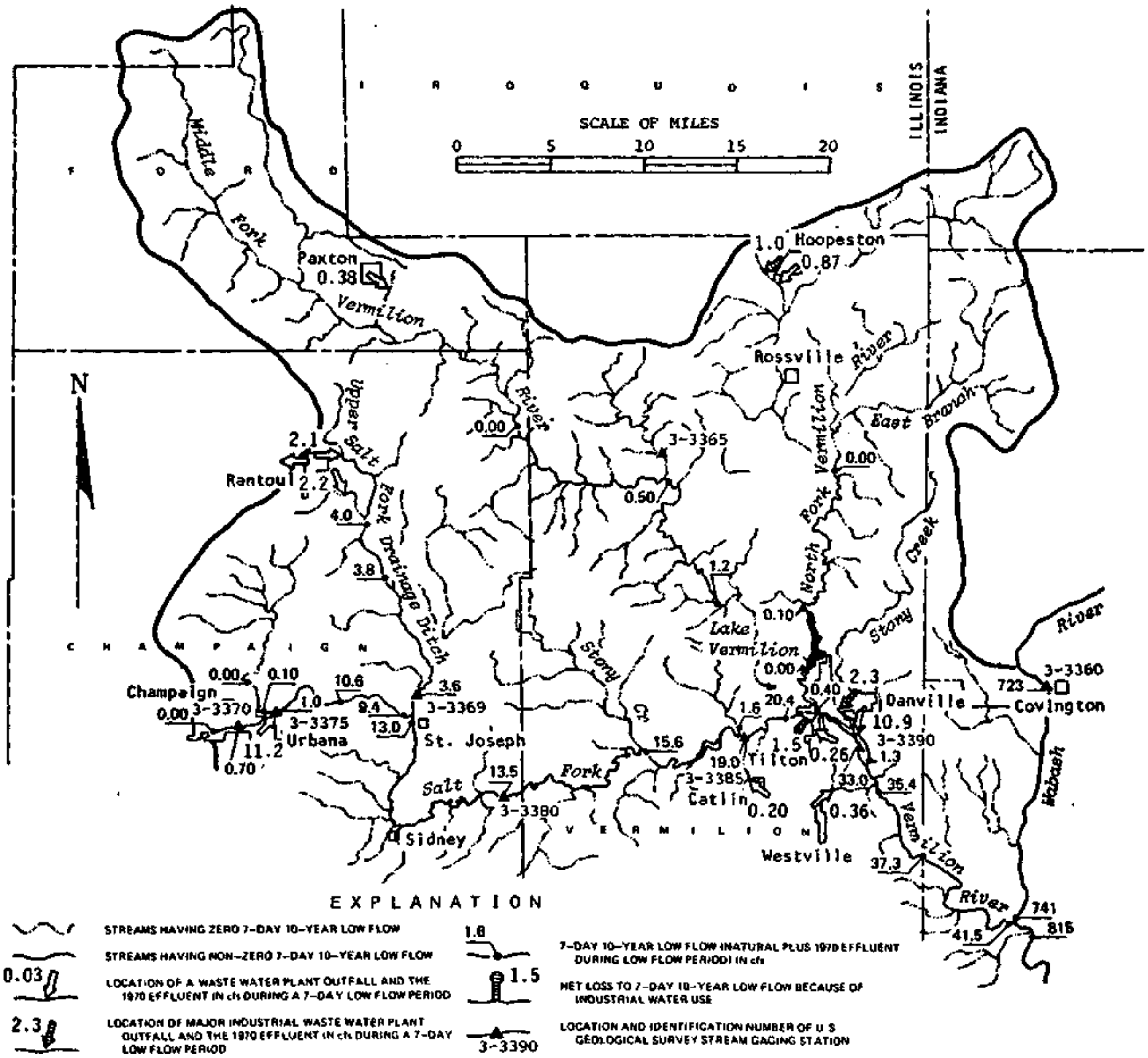


Figure 13. The 7-day 10-year low flow map of the Vermilion River basin

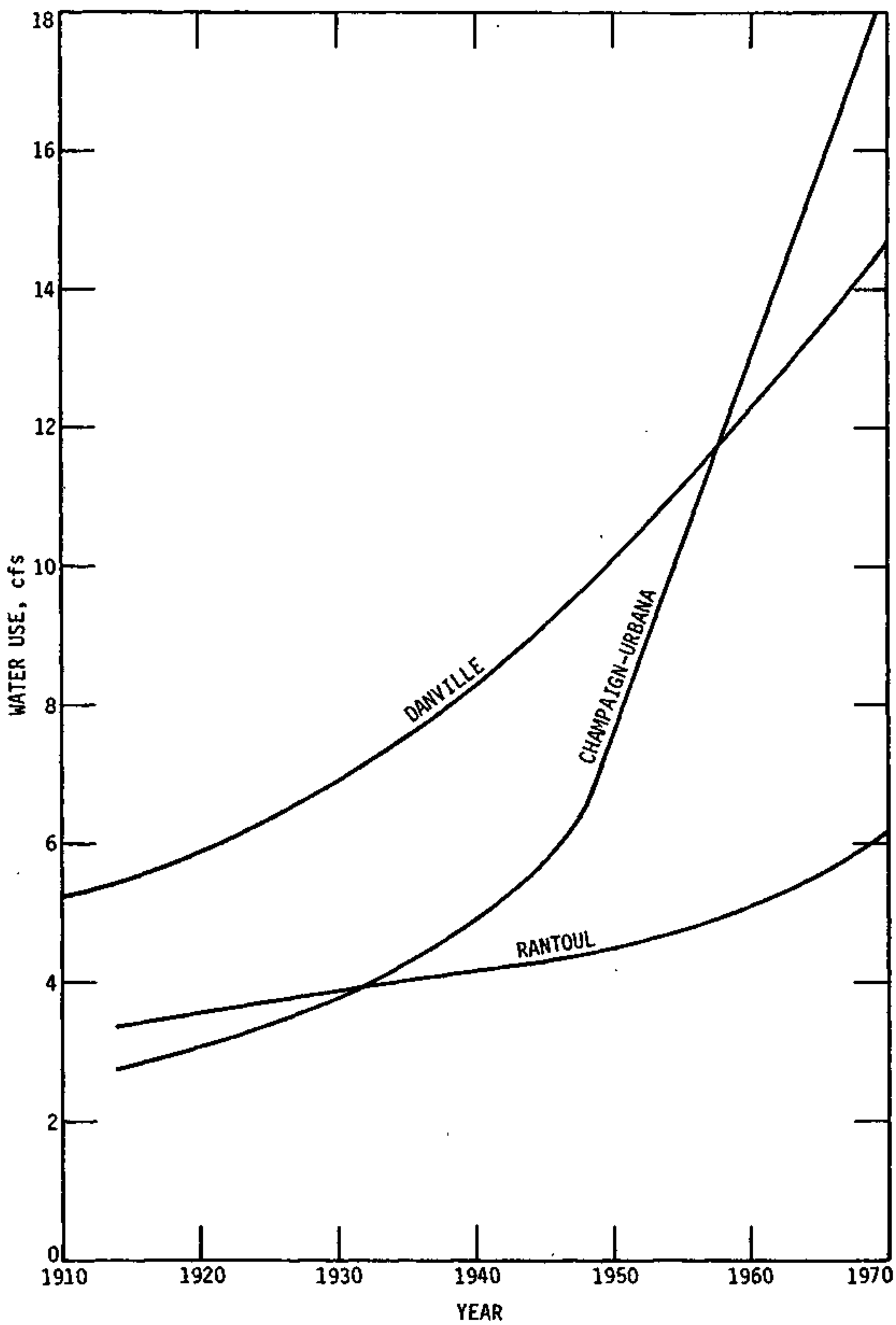


Figure 14. Water use in Champaign-Urbana, Rantoul, and Danville

Also included is the low flow adjustment, or the total effluent flow in 1970 minus that for the year under consideration, to be added to low flows in the years 1930 through 1970 to obtain low flows under the 1970 effluent flow conditions. The methodology provides a rational method for adjusting the observed flows over a number of years to a common base.

Low Flow Statistics and Water Availability. The 31-day low flows for the months January through December were computed with the daily flow data for the calendar years 1933 to 1972 at the Danville gage. The 31-day low flow period in any month could have from zero up to 15 days in the preceding or succeeding month. The month, the date of the middle of the 31-day period, and the magnitude of flow over the period were calculated and printed via a computer program. These low flows were adjusted for the effluent flow condition in 1970. With the 1970-condition 7-day 10-year low flow of 33.0 cfs as the protected flow and any future increase in effluents entering the system not available for water supply, the available flow was obtained by subtracting 33.0 cfs from the adjusted low flows. Typical information developed is shown in Table 8 for the deficit years during 1963 through 1972.

Non-availability of sufficient water from the Vermilion River to meet 5, 10, 15, 20, and 30 cfs water demands was analyzed in terms of deficient months and associated recurrence intervals. The results are presented in Table 9. Available flow in cfs during a 40-year drought and low-flow period of July through December is shown in Figure 15. In a deficient month, the available flow may be somewhere between the flow interval; for example, 15 cfs is not available for 3 months and 20 cfs for 4 months once in 40 years. The extra month of deficient flow may be assigned a value of (15 + 20)/2 or 17.5 cfs. Because of the flow variation within a month, a value of 15 cfs is assumed instead of 17.5 cfs.

TABLE 8. Non-Availability of Water in the Years 1963-1972

		Water available in cfs is less than				
Year	*M or D*	*5*	*10*	*15*	*20*	*30*
1963	M		10,12	9,10,12	9,10,11,12	9,10,11,12
	D		3,19	30,3,19	30,3,7,19	30,3,7,19
1964	M			1,10	1,10	1,10
	D			1,22	1,22	1,22
1969	M					8,9
	D					31,1

Notes: M denotes the month: January = 1, February = 2, and so on.
D denotes the date of the middle of the 31-day period, corresponding to the month in the preceding line.

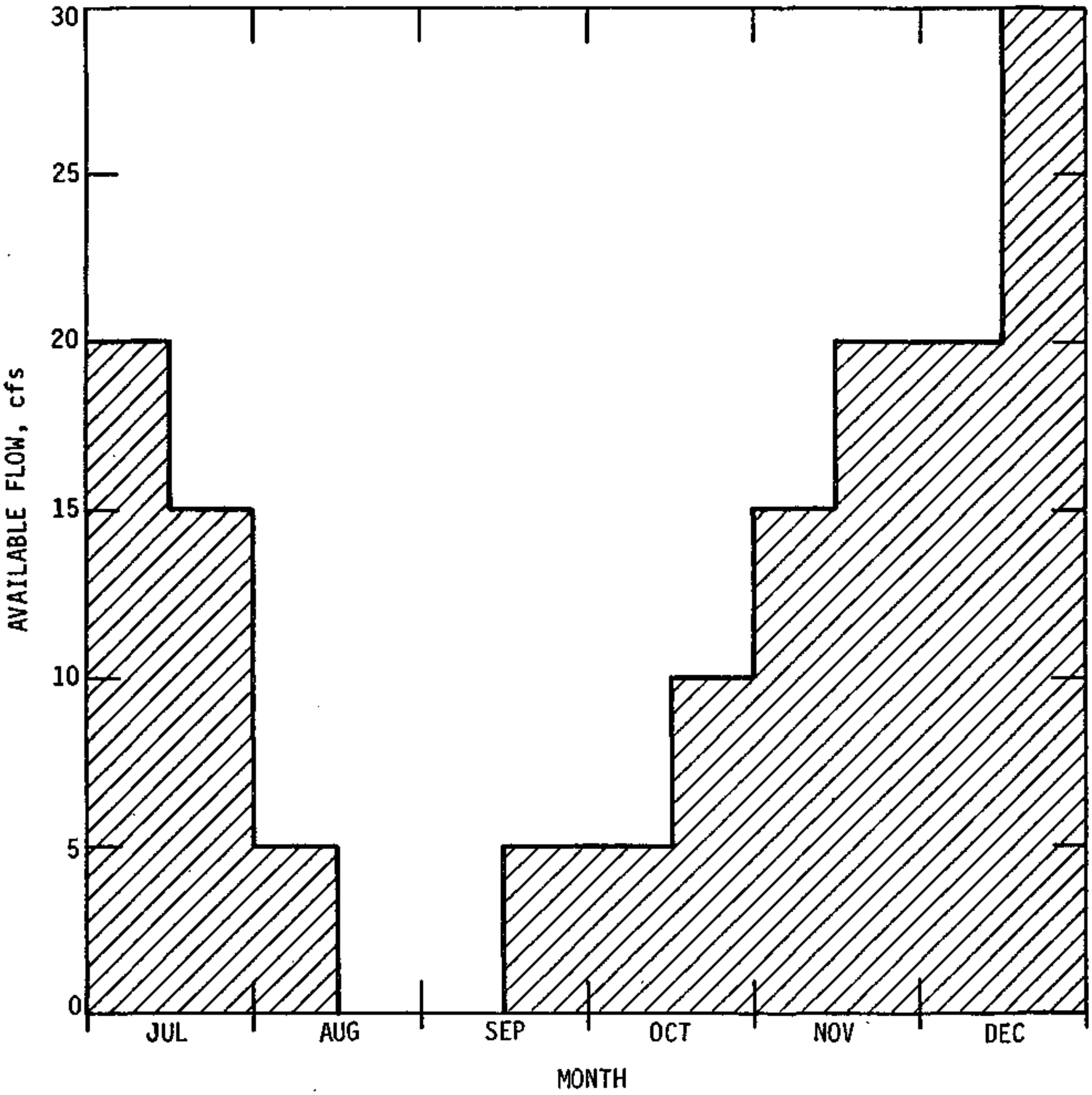


Figure 15. Availability of water from the Vermilion River during the low-flow period of July through December

TABLE 9. Critical Periods when Sufficient Supplies not Available

Recurrence Interval (years)	*Supply rate (cfs)*	*Range of months with deficient periods*	*Average deficient period (months)*
10	5		
	10	9	1.0
	15	8-12	1.5
	20	7-1	1.5
	30	7-1	4.0
20	5	8-9	1.0
	10	7-10	1.5
	15	7-11	2.5
	20	7-1	3.5
	30	7-1	5.0
40	5	8-9	1.0
	10	7-12	2.5
	15	7-1	3.0
	20	7-1	4.0
	30	6-1	5.5

The average yield available in the 6-month low-flow period during a 40-year drought was computed for various pumping capacities. These are plotted in Figure 16. At a supply rate or pumping capacity of 5 cfs, the water can be pumped only 5 months out of 6, giving an average yield of 4.17 cfs. The capacity-yield curve shows that the yield increases at a steadily decreasing rate as the pumping capacity increases and that the potential of this source of water is limited to about 11 to 12. cfs.

Annual Costs. The next step was to compute the costs associated with the intake tower in the Vermilion River, pipeline to carry water from the intake tower to 0.1 mile upstream of the low dam near the treatment plant, facilities for pumping water, and electric charges with pumping at full capacity for an average of 20 percent of the time when Lake Vermilion may not be adequate to meet the water demands. The intent would be to keep the lake level within a few feet of the normal pool.

The intake tower and the proposed 2-mile long pipeline are shown in Figure 17. The maximum static head along this route is 80 feet but a head of 100 feet was assumed to allow for extra losses at the intake. The annual cost of transporting water was computed as explained in Appendix II. Cast iron, ductile iron, or steel pipe with cement lining is considered because the route passes through an urbanized area. The intake tower is estimated to cost $80,000 and an extra cost of $25,000 is allowed for laying the pipeline under the bed of the North Fork

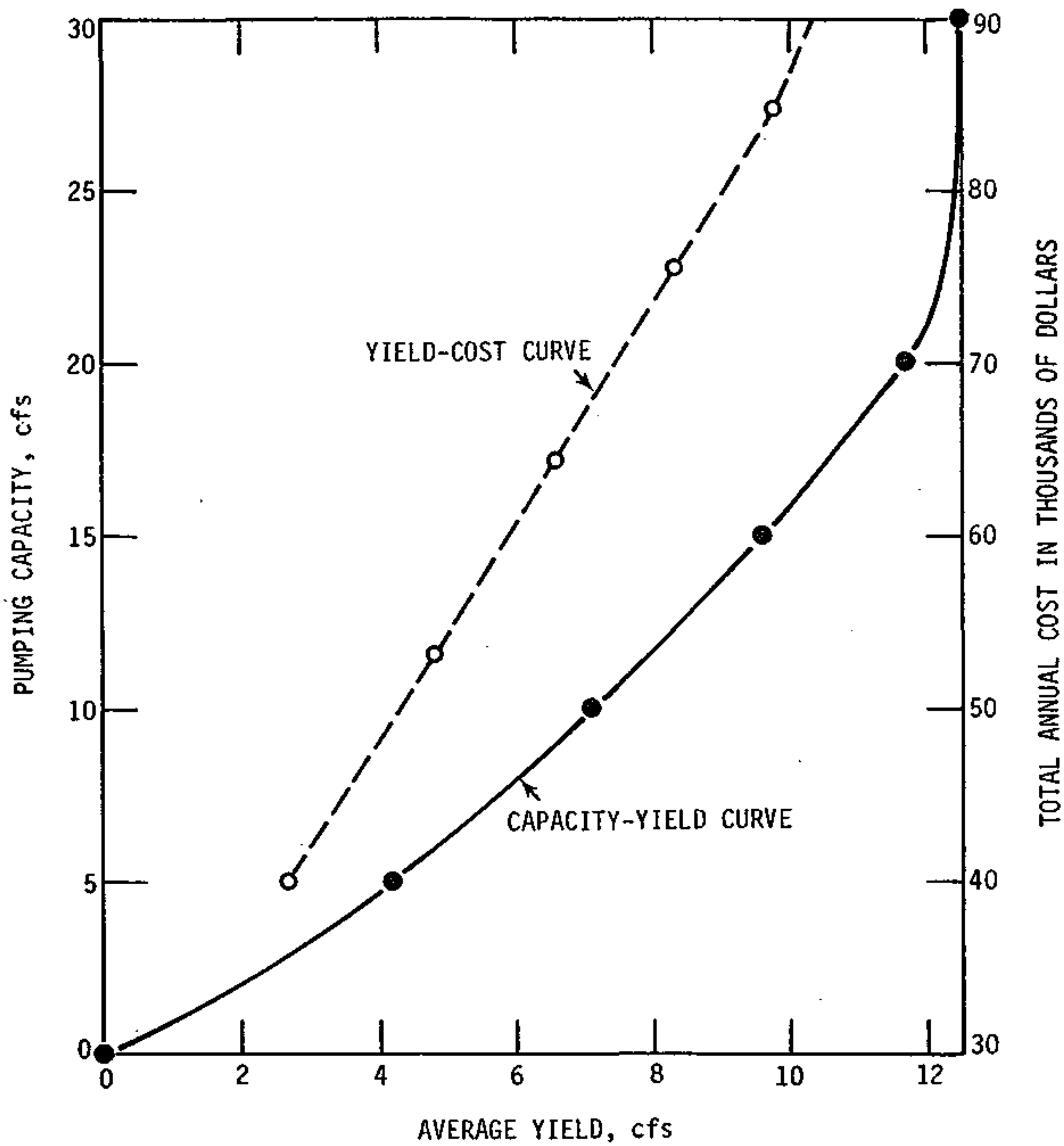


Figure 16. Capacity-yield and yield-cost curves for water from the Vermilion River

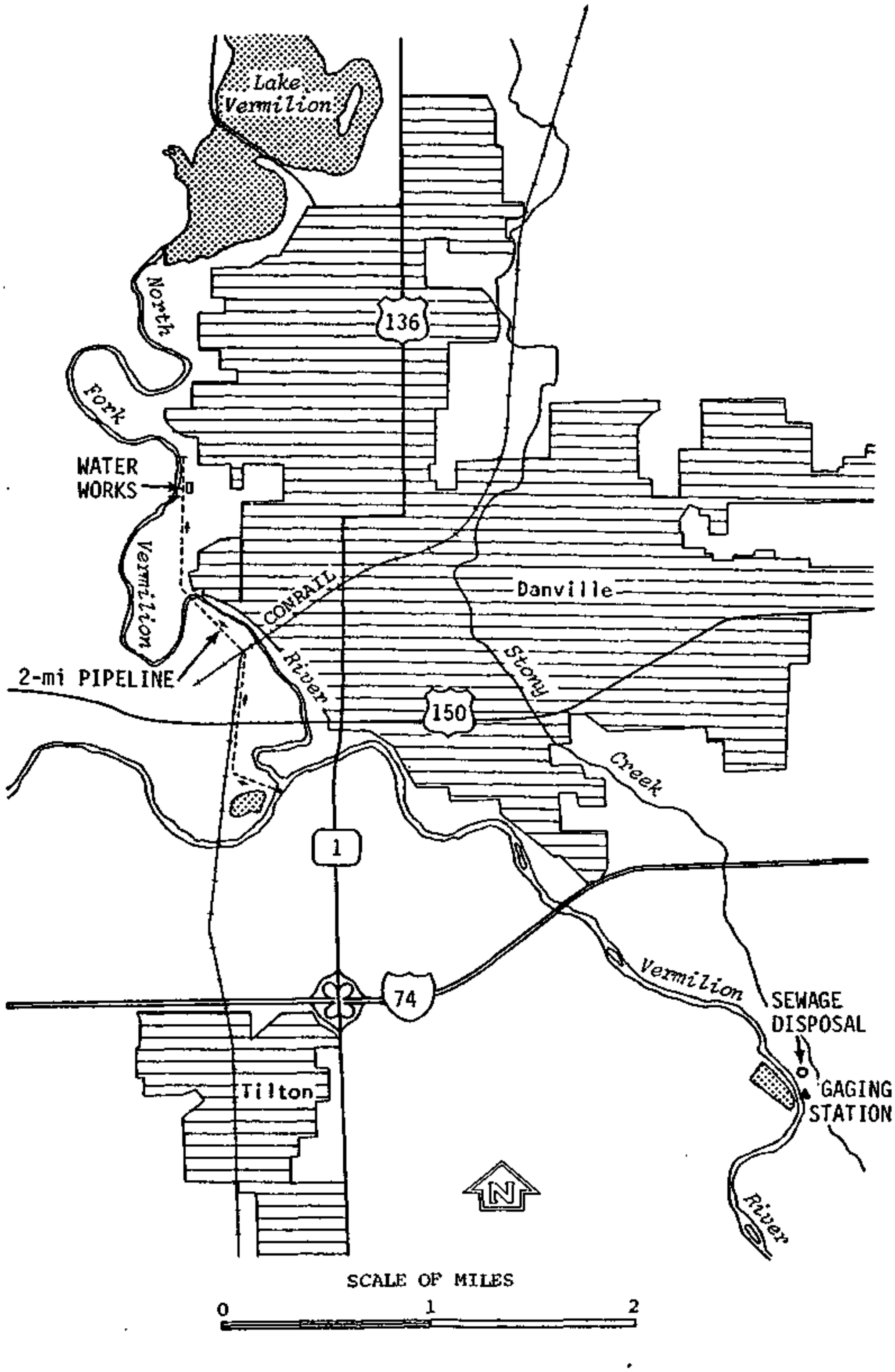


Figure 17. Pipeline for transporting water from the Vermilion River

TABLE 10. Annual Cost of Vermilion River Water

Capacity (mgd)	*Yield (mgd)*	*Yield (cfs)*	*Annual Cost (dollars)*	*Pipe diameter (inches)*
2.0	1.82	2.83	40,500	12
4.0	3.19	4.95	53,400	16
6.0	4.32	6.70	64,200	20
8.0	5.37	8.32	75,400	24
10.0	6.32	9.80	84,600	24
12.0	7.17	11.12	96,000	24

Vermilion River. An amount of $20,000 each is allowed for taking the pipeline under U.S. 150 and Conrail. The annual cost of intake tower and laying pipeline under the highway, railroad, and river he'd amounts to $11,900 on the basis of 50 years life and 8 percent interest. The cost information is given in Table 10.

The yield-cost curve is shown in Figure 16. It is evident that there is no economy of scale In the range of a 2 to 7 mgd supply from the river. This is due to basing the annual cost on the pipe carrying and pumping capacity. The yield, however, increases at a steadily decreasing rate with increase in capacity.

7. WATER FROM THE WABASH RIVER IN INDIANA

The USGS gaging station on the Wabash River that is nearest to Danville is station 3-3360 at Covington, Indiana. The drainage area above this station is 8208 square miles. The daily flow records from January 1940 through December 1972 were analyzed to derive the 31-day low-flow statistics for each of the 12 months.

Availability of Water. The protected flow is assumed as 723 cfs, the 7-day 10-year low flow (Singh and Stall, 1973). This flow was subtracted from the 31-day low flow and the remaining flow was tabulated under four categories equaling or exceeding 10, 25, 50, and 100 cfs. The deficit months in each year for the four categories are given in Table 11 together with the middle date of the 31-day low-flow period.

Non-availability of sufficient water from the Wabash River to meet 10, 25, 50, and 100 cfs water demands in terms of deficient months and associated recurrence intervals was analyzed. The results are presented in Table 12. In deficient months, available flow will be near the middle of the flow range of the interval; for example, 25 cfs is not available for 1.5 months and 10 cfs for 1.4 months once in 40 years. The extra 0.1 month of deficient flow may be assigned a value of (25 + 10)/2 or 17.5 cfs. Because of the flow variation within the month, a value of 10 cfs is assumed available during the 0.1 month.

TABLE 11. Availability of Water from the Wabash River for Danville (Based on 31-day low-flow information)

		Water availability in cfs is less than			
Year	*M or D*	*10*	*25*	*50*	*100*
1940	M				8
	D				23
1941	M	8,9	8,9	8,9	8,9
	D	27,18	27,18	27,18	27,18
1956	M				10
	D				10
1963	M			12	9,10,12
	D			30	30,16,30
1964	M	10	10,11	10,11	1,10,11
	D	29	29,1	29,1	1,29,1

Notes: M denotes the month; January = 1, February = 2, and so on.
D denotes the date at the middle of the 31-day low-flow period corresponding to the month in the preceding line.

TABLE 12. Critical Periods when Sufficient Supplies not Available (during the period July 1 to December 31)

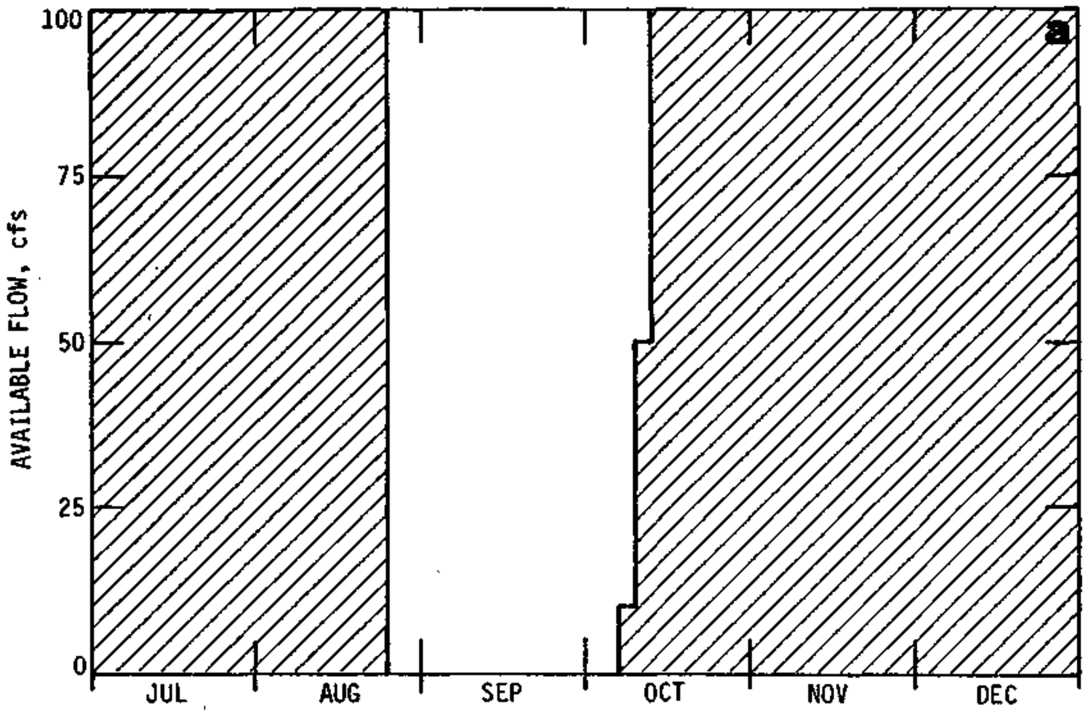
Recurrence interval (years)	*Supply rate (cfs)*	*Range of months with deficient periods*	*Average critical period (months)*
10	10	-	
	25		
	50	8-12	1.0
	100	8-12	1.5
20	10	8-10	1.3
	25	8-11	1.4
	50	8-12	1.5
	100	8-12	1.6
40	10	8-10	1.4
	25	8-11	1.5
	50	8-12	1.5
	100	8-12	1.6

The available flow in cfs during a 40-year drought and the low-flow period of July through December is shown in Figure 18a.

The average yield available during the 6-month low-flow period and a 40-year drought was computed for the four pumping capacities of 10, 25, 50, and 100 cfs as shown below. The pumping capacity—yield curve is drawn in Figure 18b.

Pumping capacity (cfs)	*Average yield (cfs)*		*Average yield (mgd)*
10	$10 \times 4.6/6$	$= 7.67$	4.95
25	$(25 \times 4.5 + 10 \times 0.1)/6$	$= 18.92$	12.22
50	$(50 \times 4.5 + 10 \times 0.1)/6$	$= 37.67$	24.33
100	$\left\{\begin{array}{l}100 \times 4.4 + 50 \times 0.1 \\ \quad + 10 \times 0.1\end{array}\right\}/6$	$= 74.33$	48.02

Cost of Water Supply. The proposed intake tower and the route of the transmission pipeline to Danville are shown in Figure 19. The water is pumped from the Wabash River. The transmission line lies in the right-of-way of U.S. 136 and the railroads except for a distance of one or two miles. The water can be pumped either to the lake above the main dam, or to the river above the water treatment plant. The total length of the pipeline is 13.8 miles. Though the low water level in the Wabash River at the intake tower would be at an elevation of about 476 feet above mean sea level, and at the two discharge points the level would be about 577 and 540 feet, the transmission line crosses a 650-foot contour near Danville. Therefore, a static head of 200 feet was assumed allowing 26 feet extra head. The annual costs of water transmission



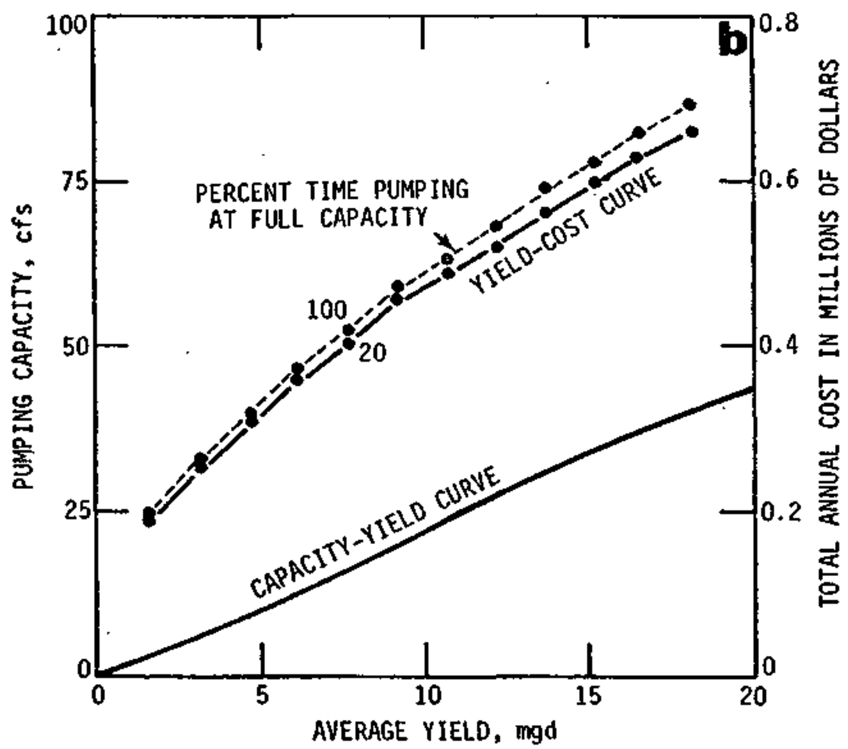


Figure 18. Water availability, capacity-yield, and yield-cost curves

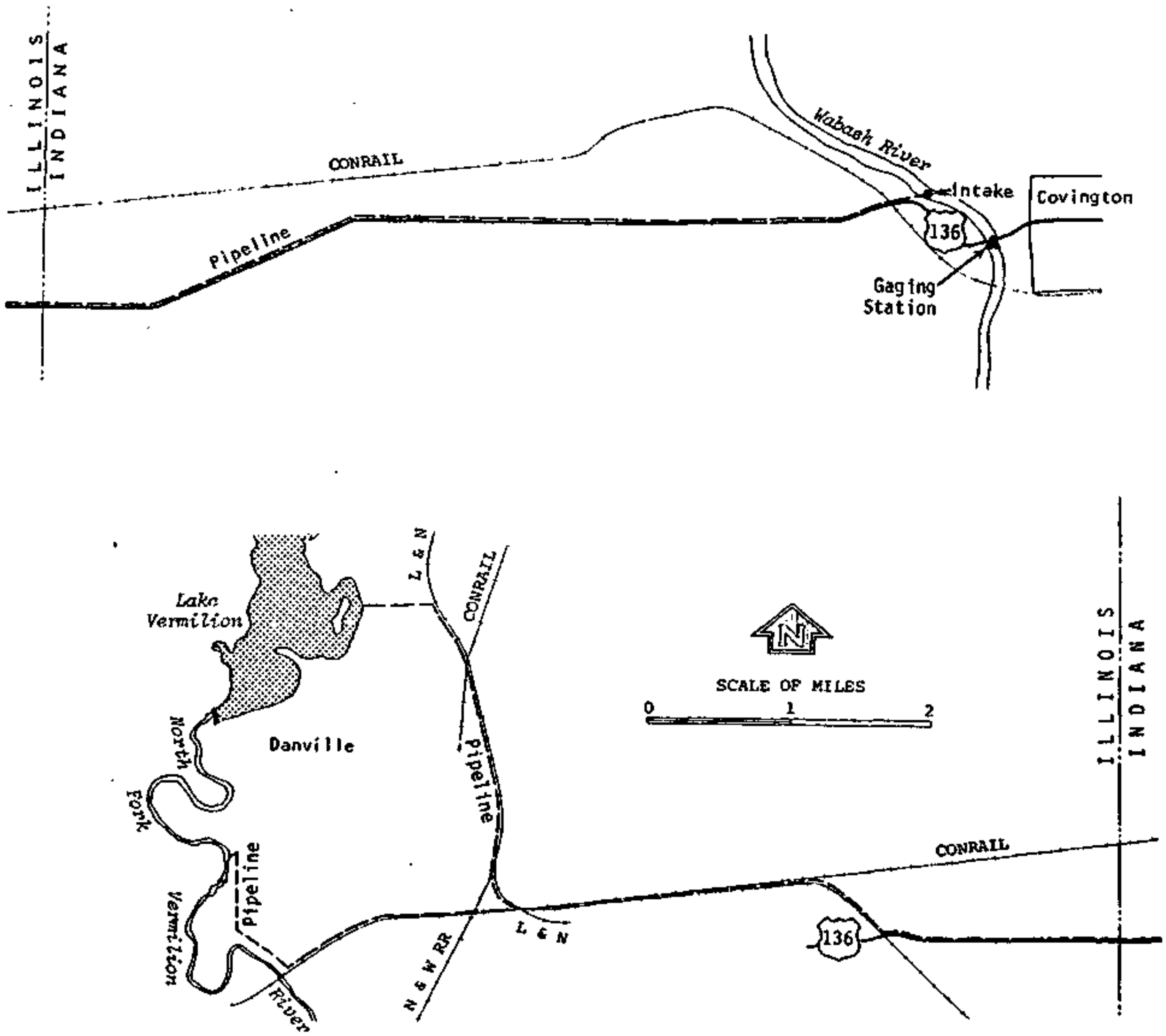


Figure 19. Pipeline for carrying water from the Wabash River to Danville

for pumping capacities of 2 through 24 mgd were computed with the cost functions in Appendix II. Cast iron, ductile iron, or steel pipe with cement lining would be used because of high head and water hammer pressures. Electric charges were computed considering pumping at the capacity rate for an average of 20 percent of the time as well as full time pumping. The intake tower is estimated to cost $250,000. The transmission line will be taken below the streambed of 8 minor and medium size drainages crossing the pipeline. An extra cost of $40,000, or an average of $5000 for a drainage crossing, is allowed. The pipeline will have three railroad crossings. Estimated extra cost for each crossing is $20,000. It is assumed that there are not too many utility lines buried in the right-of-way of the railroads along which the pipeline will be laid. The pipeline passes through part of the town. An extra cost of $50,000 is allowed for increased cost of pipeline construction in that area. The total annual costs are given in Table 13 for 20 percent and full-time pumping. The annual costs for different yields can be determined from the yield-cost curve in Figure 18b. This curve is based on the information in Table 13 and the capacity-yield relationship.

TABLE 13. Annual Cost of Water from the Wabash River

Pumping capacity (mgd)	*Yield (mgd)*	*Annual cost, full pumping (dollars)*	*Optimal diameter (inches)*	*Innual cost, 20% pumping (dollars)*	*Optimal diameter (inches)*
2	1.53	193,400	12	189,000	12
4	3.07	261,700	16	253,700	16
6	4.60	315,500	20	305,600	20
8	6.05	369,200	24	358,000	24
10	7.57	414,600	24	397,500	24
12	9.08	466,500	30	451,700	30
14	10.60	500,800	30	481,500	30
16	12.11	541,900	30	517,400	30
18	13.56	589,600	36	560,000	30
20	15.07	619,700	36	594,500	36
22	16.57	654,000	36	624,400	36
24	18.08	692,500	36	657,900	36

Note: Annual cost of intake tower, drainage railroad crossings, etc. (8% and 50 years) = $(250,000 +'40,000 + 60,000 + 50',000) x 0.08174 = $32,700

8. GROUNDWATER DEVELOPMENT ALONG THE WABASH RIVER

The groundwater development in sand and gravel aquifers associated with the Wabash River bottomlands in Indiana was evaluated as an alternative source for augmenting the Danville water supply. Several communities in Indiana including Covington, Perrysville, and Cayuga have developed groundwater supplies In these sand and gravel aquifers.

Kazmann (1946) reported on a groundwater development by the U.S. Army Ammunition Plant, also called the Newport Ordnance Works, shown in Figure 20. Kazmann described the Ordnance Well Field as probably the most productive well field in the country and probably the world during the early 1940s. The well field is located on a broad terrace of the Wabash River about 3 miles south of Newport. It consists of 6 Ranney-type wells or water collectors located adjacent to the river spaced at approximately 2000-foot intervals. The collectors tap an aquifer composed of sand, gravel, and boulders with a total depth of 100 feet. A typical Ranney collector consists of a reinforced concrete caisson, 13 feet Inside diameter and 16 feet outside diameter, from which horizontal screen laterals project radially near the bottom. The horizontal screen laterals consist of 8-inch slotted well-casings and vary In length from 50 to 250 feet. The laterals were projected in a half-radial pattern toward the Wabash River.

According to Kazmann, during the 24 months that the Ordnance Works was in full operation (August 1943 through July 1945) the field yielded an average of 72 mgd. He concluded that artificially induced infiltration from the nearby Wabash River was the principal source of water. On the basis of the performance of the Ordnance Well Field, the Wabash River bottomlands east of Danville appeared to be a promising site for location of a large groundwater development for Danville. The location of a well field along the Wabash River with potential for induced infiltration from the river has an added advantage of allowing close spacing of production wells.

The reach of the river selected for investigation extends from the confluence of Redwood Creek and the Wabash River south to Perrysville (Figure 20). The selection was made to keep the pipeline length to Danville within reasonable limits.

Groundwater Conditions in Study Area. The study area (Figure 20) includes the bottomlands along a 12-mile reach of the Wabash River in Warren, Fountain, and Vermillion Counties, Indiana, extending from the confluence of Redwood Creek with

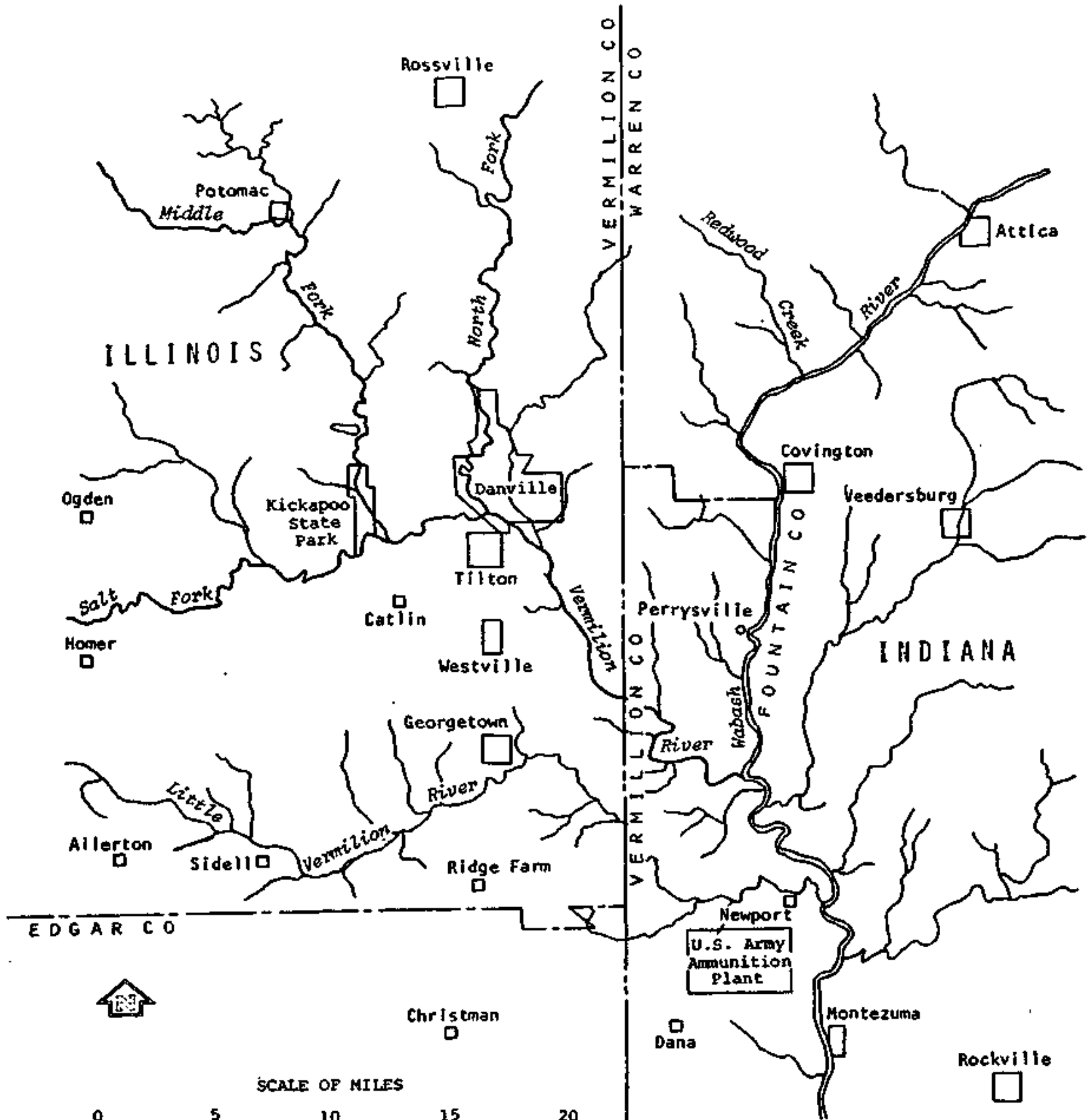


Figure 20. Map showing the Wabash River and Vermilion River drainage system

the Wabash River south to Perrysville. There are no published groundwater reports available for Warren County. Preliminary reports on the groundwater resources of Fountain and Vermillion Counties were published by the State of Indiana (Watkins and Jordan, 1965 a and b). Figure 21 is based on these reports and shows the probable yields from the sand and gravel aquifers. Except along a short reach of the river in section 11, T19N, R9W, groundwater conditions were not described as favorable for a large groundwater development in the bottomlands. It is interesting that Watkins and Jordan do not indicate groundwater conditions as favorable on the east side of the river in this section.

The chances of locating thick sand and gravel deposits suitable for a large groundwater development are more favorable In bedrock valleys. According to Burger et al. (1966) the present Wabash River valley is not associated with a bedrock valley along the full length of the study area. Narrow bedrock valleys trending east to west cut across the present river valley about 2 miles south of Covington and Just south of Perrysville. A broader bedrock valley is shown slightly to the east and parallel to the present valley from south of Redwood Creek to about 1 mile northwest of Covington. The more favorable areas for groundwater development along the river bottomlands shown In Figure 21 are associated with the bedrock valleys. Favorable groundwater conditions are not shown In the narrow bedrock valley south of Perrysville.

In Figure 21 an area west of the river from 1 to about 3 miles wide is shown as an area where large well yields may be available. According to discussions with the Indiana Department of Natural Resources, well yields may vary from 300-500 gallons per minute (gpm) to 50-150 gpm In this area. Because it would-be necessary to construct a large number of widely spaced wells to meet the water demand for Danville, this area was not considered.

Available groundwater information in the study area was obtained from the files of the Indiana Department of Natural Resources. Available data along the reach of the river from Covington to Perrysville were too sparse to make estimates of yields for a large groundwater development. The data do indicate, however, that aquifer materials are too thin for a large groundwater development. The bedrock is apparently near land surface along this reach. Available data did not support the presence of the two bedrock valleys shown by Burger et al. (1966).

Information from Drillers Logs. The available data on bottom elevations of wells or bedrock surface elevations, and saturated thickness of the sand and gravel aquifer obtained from wells drilled along the Wabash River bottomlands from

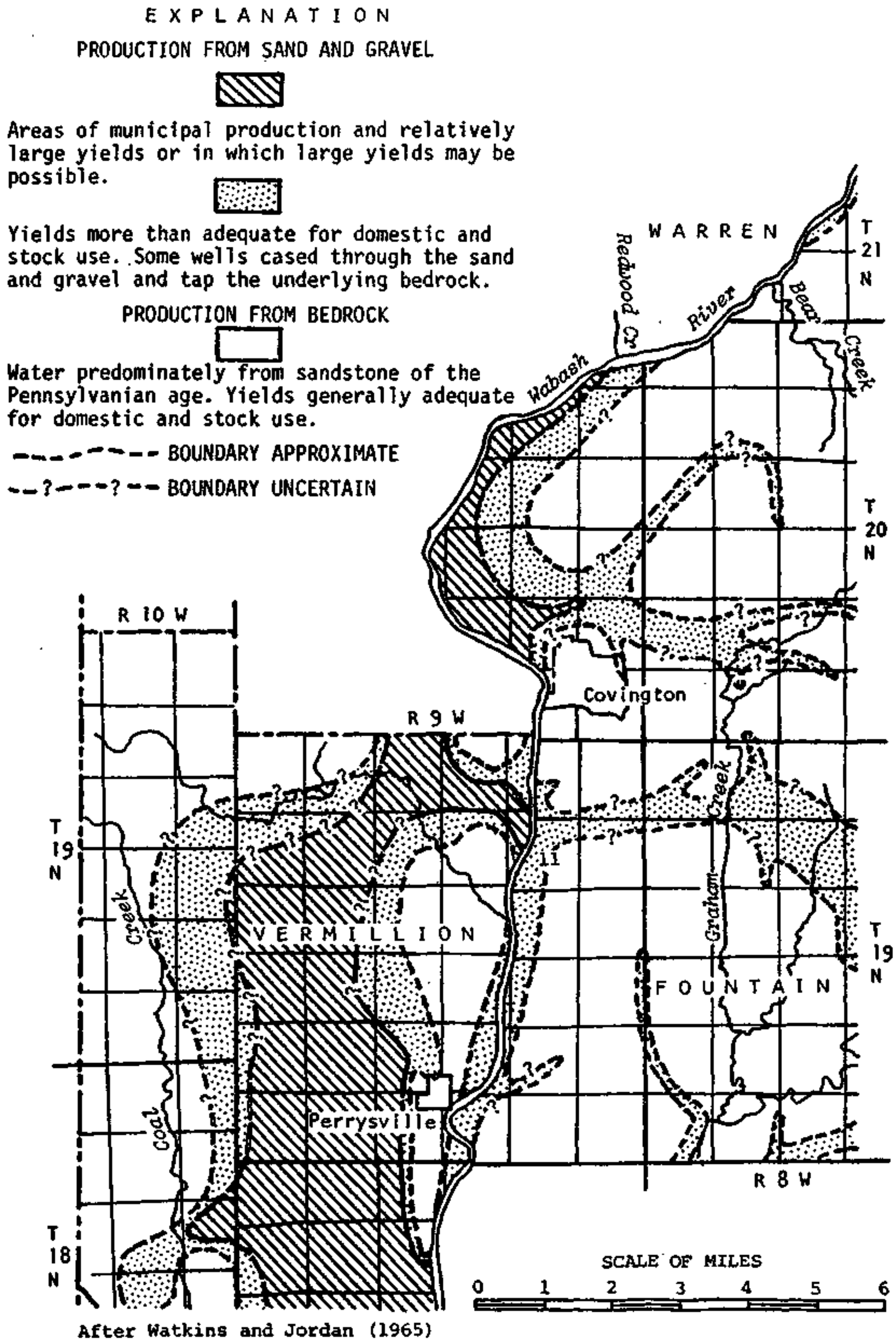


Figure 21. Availability of groundwater in the study area along the Wabash River

Covington north to Redwood Creek are plotted in Figure 22. These data confirm the existence of the bedrock valley (Burger et al., 1966), lying slightly east and parallel to the Wabash River near Redwood Creek and southwest to where the river bends to the southeast, crossing the river in sections 21 and 28, and then swinging west-northwest to sections 18 and 19 (not shown). The plotted data do not define the bedrock valley. Drillers logs are primarily for domestic or farm wells which yield sufficient water without penetrating the entire sand and gravel thickness. However, drillers logs for 3 wells in the northeast quarter of section 28 were drilled to bedrock. The low elevations are Indicative of a bedrock valley.

Reported sand and gravel thicknesses along the southern edge of section 21 and in sections 27 and 28 range from 52 to 106 feet. Four wells shown in the southwest quarter of section 27 were test pumped at rates of 1140, 1893, 1900, and 1900 gpm with 20, 9.9, 12.92, and 18.39 feet of drawdown, respectively. The data were not adequate to make determinations of well yields. However, they indicate that a large groundwater development is feasible In this area. However, an industry (Olin-Mathieson Company) has already developed a large groundwater supply in this area and owns the property along the river in sections 21, 27, and 28 where a well field could be constructed to induce recharge from the Wabash River.

None of the wells located east of the river were completed to bedrock. However, the drillers log for one well in section 22 indicated an aquifer thickness of 68 feet.

In summary, the most favorable location for a large groundwater development without excessive interference with existing supplies appears to be along and east of the Wabash River In sections 10, 15, 21, and 22. A test drilling and aquifer testing program would be necessary to define the resource for developing a 10-mgd field. A smaller capacity field would be uneconomical because of the long transmission pipeline to Danville and economies of scale. The chemical quality of the water would depend upon the quantity of water induced from the river and would probably approach the mineral concentrations in the Wabash River (see Table 19 in section 11).

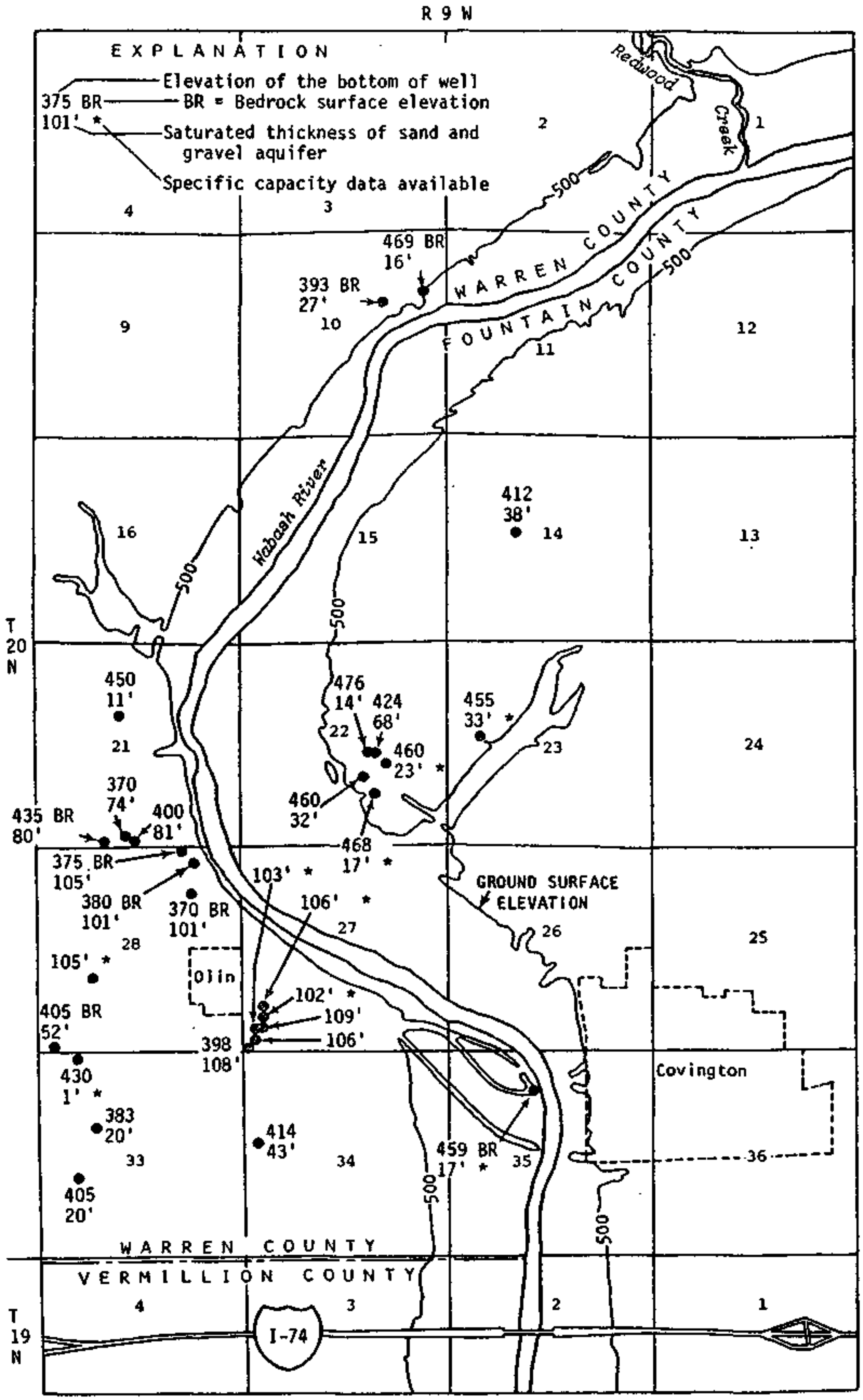


Figure 22. Information from well logs in the study area along the Wabash River

9. GROUNDWATER POTENTIAL NEAR DANVILLE

A survey of Information and literature was conducted to trace the development of groundwater aquifers in and around Danville for water supply purposes. The geohydrology of the general area was studied to confirm, deny, or modify the partial and sketchy information available. The potential for the future development of wells was Investigated for augmenting the water supply for Danville. Some pertinent Information for wells drilled in and around Danville, obtained from the State Water Survey files, is given in Table 14.

Winter Avenue Well Field. In 1913, seven wells were drilled in the north part of Danville In the lowlands near Stony Creek. These wells are referred to as the "Winter Avenue Well Field" shown in Figure 23. They ranged In depth from about 77 to 132 feet and were finished in water-bearing sand and gravel deposits. According to available drillers logs, the average elevation of the top and bottom of the sand and gravel is 554 and 512 feet above mean sea level, or about 42-feet-thick sand and gravel. Recorded nonpumping water levels were near an elevation of about 610 feet. The pumping rate from two of the production wells is recorded as 303 and 364 gpm. The pumpage from the well field is mentioned in Inter-State Water Company reports to have been about 1.5 mgd during dry weather periods in 1913. The wells were developed only as a temporary source of water.

Griffin Street Wells. Two wells were drilled about 1913 in the south part of Danville near Griffin Street as shown in Figure 23. These wells were about 105 feet deep. The drillers log for one of the wells shows sand and gravel from 46 to 105 feet. Yields are not recorded. These wells were never used.

Dam No. 2 Well Field. The Inter-State Water Company developed a well field about 1921 near Dam No. 2 (upstream of Denmark Road Bridge) that had been built in 1916. Eight wells were constructed and they tapped sand and gravel deposits lying between depths of about 30 to 90 feet. The average elevation of the top and bottom of the sand and gravel deposits is 520 and 475 feet above mean sea level. Nonpumping water levels ranged from just above land surface to a depth of 27 feet and were near an elevation of 54l feet.

Information in the Water Survey files indicates that the pumping rates of individual wells ranged from 180 to 480

TABLE 14. Record of Wells Drilled in and around Danville

Location	Land surface elevation (ft above msl)	Well depth (feet)	Principal sand and gravel interval (ft below lsd)	Nonpumping water level (ft below lsd)	Pumping rate/drawdown (gpm/ft)
Winter Avenue Well Field					
20N11W - 28.8a (1)	6l2	89	48-89	0	
- 28.8a (2)	614	77	54-77		303/
- 28.8a (3)	615	105	59-105		
- 28.8a (4)	618	96	54-96	0	364/
- 32.1h (5)	610	132	52-110 114-132	12	
- 28.8a (6)	618	126	69-126		
- 28.8a (7)	620	112	67-112		
Griffin Street Wells					
19N11W - 9.1a	555	105	46-105		
- 9.1a	555	105			
Dam No. 2 Well Field					
20N11W - 19.3a (1)	555	150	39-88		268/
- 19.3a (2)	568	151	45-86	27	180/54
- 30.3h (3)	550	151	30-55 58-71	14	357/49
- 19.2a (4)	562	85	33-85		
- 19.3a (5)	553	150	30-62	10	307/54
- 19.3a (6)	553	151	30-85		295/
- 30.3h (7)	551	151	38-80 82-90	10	480/47
- 30.3g (8)	551	150	70-99	0	375/
Industrial - Commercial Wells					
19N11W - 4.5a (Merlan)	630	91			60/
- 5.1a (Lauhoff)	570	109	9-107	31.5	1034/4.5
- 8.1h (Fecher)	580	97	14-97	9.5	500/6
- 8.-		104	0-104	21	1049/39
- 9.8f (Lauhoff)	565	115	13-118	33	1232/6
-14.7c (Vermilion Hills CC)	635	78		39.5	240/10.5
20N11W -19.6a (Danville CC)	650	116		32.5	165/20
-28.8b (Elks Golf)	635	88	67-88	1.7	203/9.7
Domestic Wells					
19N11W - 9.7e	560	50		20	
-14.5f	620	130	35-100 125-130	97	6/18

TABLE 14. (Continued)

Location	*Land surface elevation (ft above msl)*	*Well depth (feet)*	*Principal sand and gravel interval (ft below lsd)*	*Nonpumping water level (ft below lsd)*	*Pumping rate/drawdown (gpm/ft)*
19N11W - 15.6d	545	88	38-46 49-88	14	35/10
- 15.7g	555	119	85-119		
- 15.7f	550	155	77-97 147-152		
- 15.8e	540	57	0-57	30	12/
- 27.3a	530	48	32-47	29	10/
- 34.7e	540	87	44-86	27	5/30
20N11W - 6.1b	660	200	73-173 193-200	90	7/10
- 7.7e	660	160	144-160	70	5/30
- 8.5e		111	72-111	24	30/1
- 8.8f	625	108	80-108	25	20/5
- 17.4a	670	225	118-225	100	15/5
- 17.6a	650	113	100-110	57	15/11
- 17.6a	650	165	118-126 155-160		
- 17.7a	640	105	70-100	30	5/5
- 17.7h	650	148	133-148	85	15/3
- 17.8h	650	135	83-135		10/10
- 19.3e	640	148	99-146		5/
- 19.3e	640	144	91-141	53	5/3
- 19.3f	620	65	49-64	40	10/2.5
- 28.7a	622	108	88-108	32	12/1
- 29.4h	652	200	150-198	54	7.5/1
20N12W - 12.-		160	88-105 155-160	60	10/12
- 16.1e	690	126	97-126		
- 22.2h	700	85	64-85	30	20/5
- 23.7d	690	72	61-72	35	8/
- 25.5a	692	175	160-175	105	20/1
- 25.7h	690	185	75-80 168-185	115	24/1

Note: 1sd denotes land surface datum

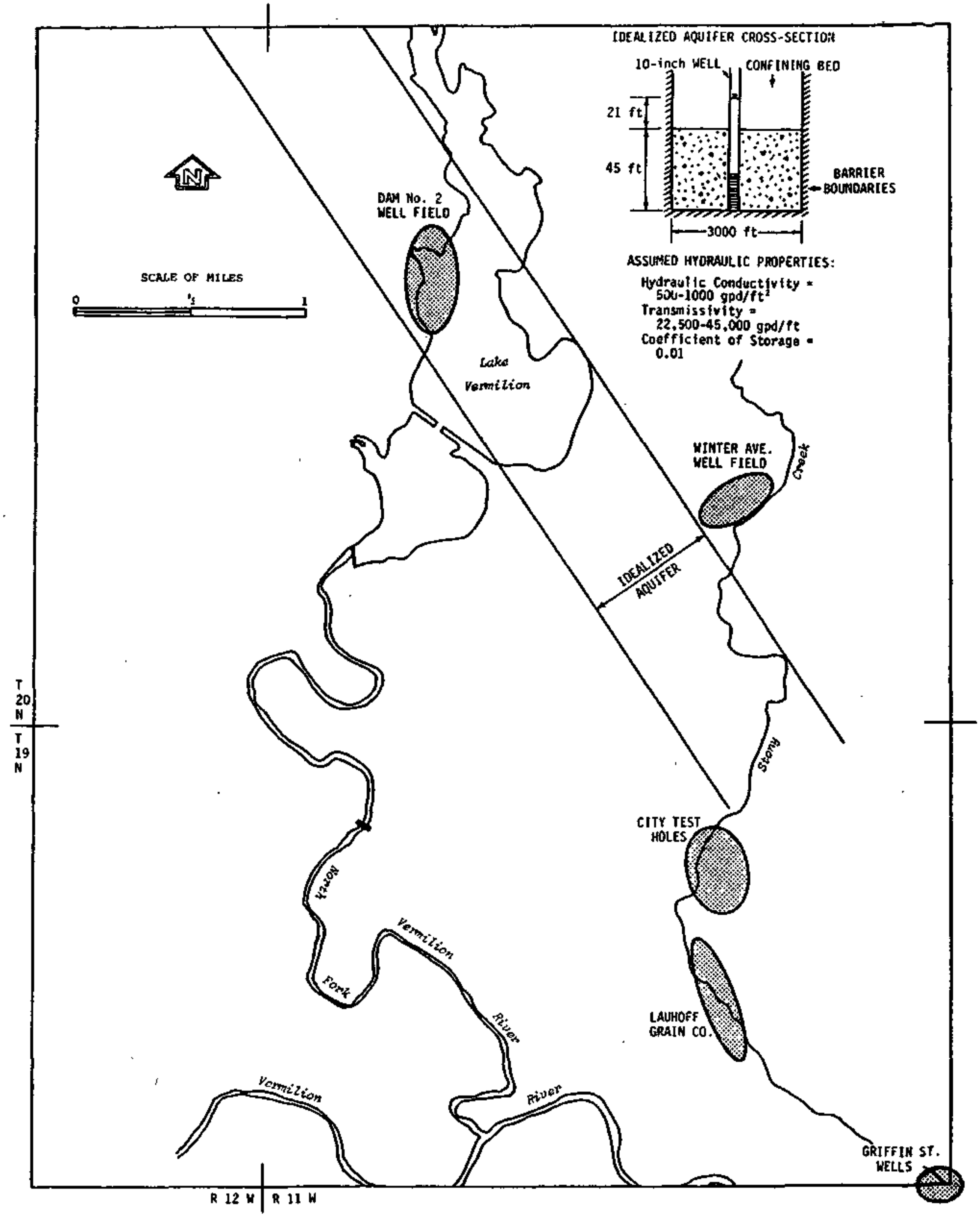


Figure 23. Location of well fields in and around Danville

gallons per minute. The combined discharge of all eight wells was estimated to be as much as 3 mgd. Available records at the Inter-State Water Company and the Water Survey do not show whether the wells were ever used on a regular basis. The site of this well field is now inundated by the present Lake Vermilion.

Geohydrologio Conditions at Danville. Geologic information available from the State Geological Survey indicates that the city of Danville is underlain by glacial drift material or till overlying the Pennsylvanian bedrock. The drift deposits vary greatly in thickness and texture within short distances. A deep, narrow preglacial valley carved into the bedrock trends south-southeast to north-northwest near the center of Danville. The greatest thickness of drift is contained in this bedrock valley. Away from the bedrock valley in the bedrock upland areas, the drift is thin, less than 50 feet, and water-bearing sand and gravel deposits are rarely encountered.

Drillers logs of the old well fields owned by the Inter-State Water Company, test holes drilled in 1955 by the city of Danville, and available information on the industrial and commercial wells suggest that fairly thick deposits of water-bearing sand and gravel tend to occur in the valley deposits. The wells developed by the Lauhoff Grain Company, the Griffin Street Wells, the Dam No. 2 Well Field, and the 1955 city test holes suggest sand and gravel deposits usually are present in a basal position within the bedrock valley. Sand and gravel deposits occurring at a slightly higher elevation have been tapped at the Danville Country Club and by the old Winter Avenue Well Field. These deposits also appear to be present in the Lauhoff Grain Company wells which show the greatest thickness of sand and gravel in the study area.

The Pennsylvanian bedrock that underlies the area consists principally of shale with only a few thin beds of water-yielding sandstone and creviced limestone. These rocks yield water in extremely limited quantities and have practically no potential for public water supply in this area.

Potential For Future Development. The geohydrologic information presently available is extremely limited but suggests that the groundwater resources within the immediate vicinity of Danville probably are not adequate to sustain long-term withdrawals to meet the future municipal and industrial water demands of the community. However, sand and gravel aquifers are present in the Danville area and are tapped for several commercial and industrial supplies. The possible development of these deposits for supplemental municipal use was investigated. A cost effective alternative would be to develop well(s) near Lake Vermilion and to discharge groundwater directly to the lake to augment the water stored during periods when inflows are insufficient to meet water demands and the lake level is falling.

The Dam No. 2 Well Field was completed in basal sand and gravel deposits associated with the buried preglacial bedrock valley which trends beneath Lake Vermilion. The limited data suggest that this sand and gravel aquifer is present at sites adjacent to the lake and could be developed for supplemental water supply.

The potential quantity of groundwater that might be available was estimated by calculating the quantity available from an idealized model aquifer having dimensions and hydraulic properties similar to the sand and gravel aquifer. The drillers logs, the locations of wells within the well field, and the modified map of the bedrock valley permitted a guess that the effective width of the aquifer was about 3000 feet and the thickness was about 45 feet. The length of the aquifer was assumed to extend for many miles. Data suitable for determining the hydraulic properties of the aquifer are not available but specific capacity data on the old wells at Dam No. 2 suggested that the hydraulic conductivity is fairly low.

Values for the hydraulic conductivity of 500 to 1000 gpd/ft^2 and 0.01 for the coefficient of storage appeared reasonable and were chosen for the purpose of calculation. The dimensions and hydraulic properties of the model aquifer are shown in Figure 23. It also was assumed that production wells would be 10 inches in diameter and equipped with 20 feet of commercial well screen.

Computational results suggest that as much as 1 mgd, depending on aquifer transmissivity, could be developed for a period of 3 to 6 months from two wells located about 5000 feet apart adjacent to Lake Vermilion. Although the model aquifer chosen needs to be validated from additional test drilling and aquifer testing, the possible yield can be incorporated in evaluating water supply alternatives pending further groundwater resource investigation.

The site of the Winter Avenue Well Field is approximately 1 mile from Lake Vermilion and might be considered as a site for development of a supplemental water supply. However, the data available are very limited and do not permit an estimate of the areal extent or hydraulic properties of this sand and gravel aquifer.

In an excerpt of a report on the supply source history of the Inter-State Water Company it is mentioned that "...a temporary pumping plant was installed on the Wilson land (Winter Avenue site) to furnish water during the drought of 1913. ...About 1 1/2 million gallons daily were obtained from this plant." The reliability of this information cannot be assessed. If it is desired to include the Winter Avenue site in the evaluation of water supply alternatives, it may be possible to develop about 1/2 to 1 mgd for a period of 3 to 6 months in a year. Test drilling and aquifer testing would be required to evaluate the potential yield of the site.

Associated Costs. The annual costs are estimated for two developments: 1) two wells, one on the east and the other on the west side of the lake, each of 0.5 mgd capacity, and 2) three wells, one a standby, in the Winter Avenue Well Field, each of 0.375 mgd capacity.

1. *Two wells near the lake:* total yield 1.0 mgd

Cost of test drilling and exploration	$15,000
Cost of two gravel-packed, 10-inch screen diameter, 100-foot deep wells	$'26,600
Cost of pumps and motors for 0.5 mgd at 100-feet head from each well	$12,550
Cost of about 600 feet of pipe or drain from the wells to the lake	$ 2,000
Annual cost of items above = (15,000 + 2000) x 0.0817 + (26,600 + 12,550) x 0.0937	$ 5,057
Annual electric charges for 20 percent time	$ 1,148
Annual OM&R cost on wells and pumps	$ 580
Total annual cost	$ 6,785
say	$ 6,800

2. *Three wells in Winter Avenue Well Field:* **yield 0.75** mgd

Cost of test drilling and exploration	$15,000
Cost of three gravel-packed, 8-inch screen diameter, 125-foot deep wells	$36,420
Cost of pumps and motors for 0.375 mgd at 125 head from each well	$18,580
Annual cost of items above = 15,000 x 0.0817 + (36,420 + 18,580) x 0.0937	$ 6,379

Annual cost of one mile 8-inch pipeline including OM&R and easement costs (cost increased 50% over that given in Appendix II to allow for higher construction cost in urbanized area)	$13,721
Annual electric charges for 20 percent time	$ 1,076
Annual OM&R cost on wells and pumps	$ 755
Total annual cost	$21,931
say	$22,000

Development of a 0.75 mgd supply from the three wells will cost $22,000 a year, a high price to pay for such a supply. Two wells near the lake will provide 1.0 mgd at an annual cost of $6,800. No standby is provided on the assumption that the two wells would be maintained in good condition.

10. GROUNDWATER DEVELOPMENT PROM NORTHERN VERMILION COUNTY

The feasibility of developing groundwater supplies for the city of Danville from the glacial drift deposits in the northern part of Vermilion County has been Investigated. Here we have summarized the geohydrologic setting for the general area, described the location, configuration, and design of some suitable well fields, and analyzed the effect of groundwater development in these well fields on the water level in the nearby town wells. Two alternatives were considered for transporting water from the well fields to Lake Vermilion: a pipeline, or the North Pork Vermilion River and its tributaries. Evaporation and seepage losses were estimated for the condition when water is transported via the river. Annual costs of groundwater development were computed for meeting different supply rates from the well fields and with pipeline or river transport.

Geohydvologic Setting. The description is based on a report prepared by Dr. John P. Kempton (1977) of the Illinois State Geological Survey. The geologic setting in Vermilion County is such that development of large groundwater supplies must be made from sand and gravel aquifers within the glacial deposits. These deposits are by far the thickest in the northern three tiers of townships of the county, and it is within this area that thick and extensive sand and gravel aquifers have been presumed to exist. Because the shallow, less than 200 feet in depth, sand and gravel aquifers have generally produced adequate groundwater supplies in the region., deeper exploration has not been undertaken and few data on the character of the deeper sand and gravel aquifers have been available.

Results are available, however, from numerous bedrock structure tests conducted over the last five years. Prom these and other existing information, a preliminary evaluation has been made of the probability of occurrence of a sand and gravel aquifer for large groundwater development. Based on a tentative, modified bedrock map, Figure 24 shows areas numbered 1 through 5 in decreasing order of magnitude of the potential for groundwater development. Within areas 1 and 2, two or possibly three sand and gravel aquifers greater than 20 feet thick may be present, except in the narrowest portion of the Danville Bedrock Valley In T19N, R11W, and T20N, R11W. The southeast-to-northwest trend of the Danville Valley Is evident from the contours shown in the figure. The valley joins the westward trending Mahomet Bedrock Valley at the northwest corner of Vermilion County. The explanation of the numbered categories, 1 through 5 is given on the next page.

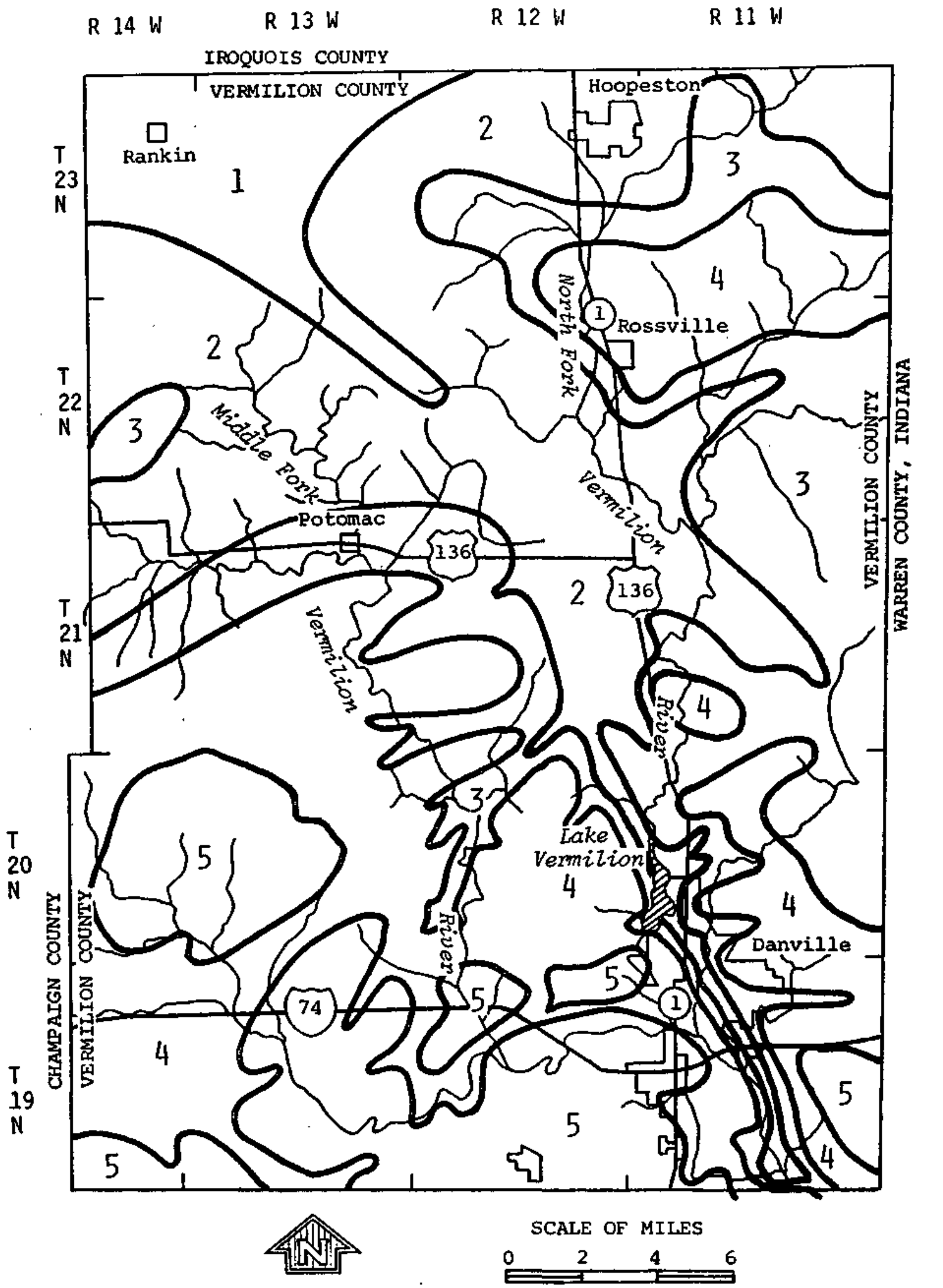


Figure 24. Probability of occurrence of sand and gravel aquifers in northern Vermilion County (after Kempton, 1977)

Category	*Bedrock elevation (ft msl)*	*Explanation*
1	<400	Two or more aquifers generally present which may be capable of yielding large groundwater supplies, often greater than .20 feet thick. The deeper aquifers occur at elevations below about 550 feet; locally, may be continuous from 550 feet to the top of bedrock (greater than 100 feet thick).
2	400–450	Same as 1 except that lowermost aquifer is often not as thick or portions may be absent. Some exploration necessary to find the best well sites for large municipal supplies.
3	450-500	Upper aquifer (at elevation of 600 - feet) and only upper portion of lower aquifer may be present. Locally, large municipal supplies may be available. Considerable exploration needed.
4	500-600	Upper aquifer generally present but probably capable of producing only small municipal supplies.
5	>600	Normally, only discontinuous, relatively thin sand and gravel aquifers present.

Some geophysical logs indicate the presence of continuous sand and gravel from about 550 feet msl to the top of bedrock. Others suggest two aquifers, one above 500 feet, and the other below 450 feet, both generally between 20 and 70 feet thick. A few logs suggest that only one or the other is present. The uppermost aquifer of potential significance for the additional development of large groundwater supplies occurs approximately at elevation 600 feet. Wells drilled for the village of Rankin appear to be finished in the lowermost aquifer below 450 feet. Wells within Hoopeston are finished in both the middle and upper aquifers, and wells in the village of Rossville appear to be finished in the upper aquifer.

In summary, the Kempton report indicates that a significant potential exists for the development of large groundwater supplies between 6 and 18 miles north and northwest of Danville from one or more of the three sand and gravel aquifers.

Location and Design of Well Fields. Locations of well fields capable of producing up to 10 mgd and even more were determined from the available geohydrologic information, the presence of effective aquifer boundaries, the distance to existing wells, and the availability of water transmission routes (roads or major tributaries to the North Pork Vermilion River). Sites were chosen, subject to the above considerations, to minimize the transmission distances. Three alternative sites — A, B, and C in Figure 25, were selected within the category 2 area of Figure 24. Site A was designed for supplying groundwater to Lake Vermilion by way of the North Fork, site B by way of a pipeline along U. S. Route 136, and site C by way of either a river or a pipeline route. In addition a fourth well field site, D, designed for up to 20 mgd production, was located in a category 1 area. The locations and transmission routes for the four proposed well fields are shown in Figure 25 in relation to the category areas.

Well field designs, pumping lifts, and interference effects were estimated by applying the leaky artesian well function and the concept of idealized model aquifers to the assumed geohydrologic setting. Aquifer hydraulic properties were hypothesized on the basis of the geology and hydraulic properties known in Vermilion County and within the Mahomet Valley in Champaign, Ford, and Iroquois Counties (Visocky and Schicht, 1969). For well fields within the category 2 area, the following aquifer properties were used: hydraulic conductivity = 1000 gpd/ft , transmissivity = 50,000 gpd/ft, and leakage coefficient = 0.002 gpd/ft^3. For field D in the category 1 area, the corresponding values were 1500 gpd/ft^2, 112,500 gpd/ft, and 0.002 gpd/ft3, respectively. Wells were assumed to penetrate the entire drift thickness, and nonpumping levels, extrapolated from the Mahomet Valley, were estimated to be near land surface.

Individual well yields in fields A, B, and C were assumed to be 700 gpm or 1 mgd, and for field D, 1000 gpm. Wells would be 12 inches in diameter with 35 foot screens in fields A, B, and C, and 50 foot screens in field D. An analysis of mutual interference effects indicated that a well spacing of 1000 feet would be adequate in all fields. Standby wells were allowed on the basis of the relationships (Singh et al, 1972)

$$N_{wt} = N_w + 1 \text{ for } N_w \leq 3$$

$$N_{wt} = N_w + 2 \text{ for } N_w > 3$$

in which N_w is the number of wells for required well field production and N_{wt} is the total number of wells including standbys.

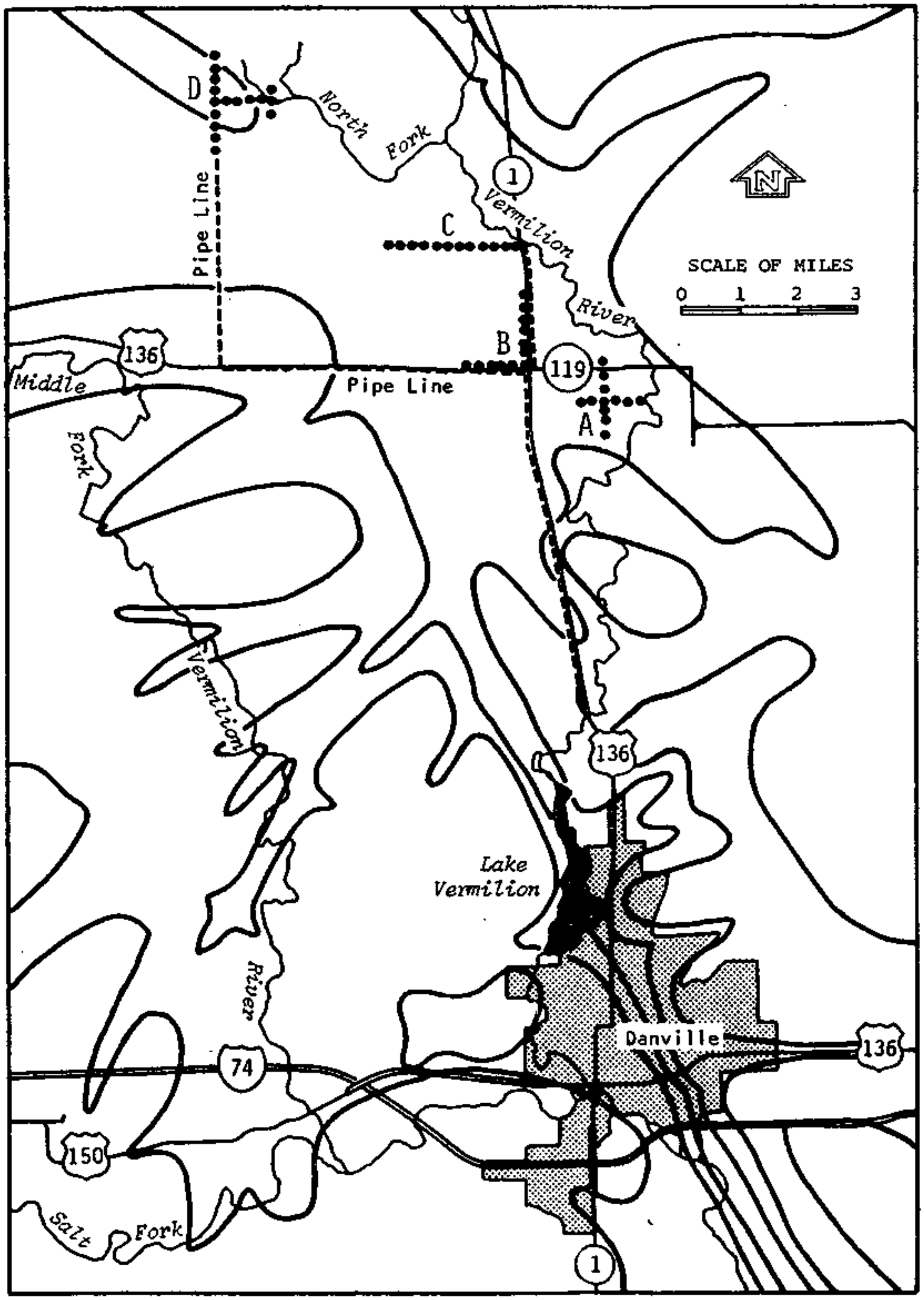


Figure 25. General location of proposed well fields

Fields A, B, and C were designed for yields of 2, 4, 6, 8, and 10 mgd, while field D was designed for either 10 or 20 mgd. Pumping lifts for pump design purposes were calculated to include drawdown, partial penetration, mutual interference, boundaries, turbulent well losses, head losses within the well field interconnecting pipelines, and transmission line loss varying from 0 to 40 feet. Figure 26 shows the schematics of design for the proposed well fields.

Field A. It would be located along intersecting secondary roads in sections 6, 7, 8, and 18, T21N, R11W. For a 10 mgd supply, 12 wells would be strung along these roads and the collected water discharged eastward into the North Fork Vermilion River, 9 miles upstream from Lake Vermilion. Wells would average 275 feet in depth and would have pumping lifts of about 105 feet.

Field B. It would be located along highways U. S. 136 and I11. 1 in sections 1 and 2, T21N, R12W, and section 36 T22N, R12W. Water pumped from the field would be collected and piped south along U. S. 136 or I11. 1 for 6 miles to the upper end of Lake Vermilion. Twelve wells, averaging 280 feet in depth, would be needed for developing a 10-mgd supply. The average pumping head including estimated net line losses, would be about 120 feet.

Field C. The site selected allows transport of water via the river or a pipeline. The field would be located along a secondary road at the bottom of sections 25 through 28, T22N, R12W. Water can be discharged directly eastward into the North Fork Vermilion River, 14 miles upstream of Lake Vermilion. Or it can be piped south along I11. 1 for 8 miles to the upper end of the lake. Twelve wells, averaging 265 feet in depth, would be required. Average pumping lift would be about 95 feet for the river route and 125 feet for the pipeline alternative.

Field D. It was designed for a production rate of as much as 20 mgd and, like field C, it is located for either a river or a pipeline route. Wells would be located along secondary roads in sections 7, 17, 18, and 19, T22N, R12W. For the river transmission, water would be pumped eastward to a tributary of the North Fork Vermilion River, 21 miles upstream from Lake Vermilion. For the pipeline route, water would be piped southward along the township road to U. S. 136 and along this highway to the upper end of the lake. The total pipeline length would be 15 miles. For a production of 20 mgd a total of sixteen 1000-gpm wells with average depth of 255 feet, would be needed. The average pumping lift would be 93 feet for the river and about 133 feet for the pipeline alternative.

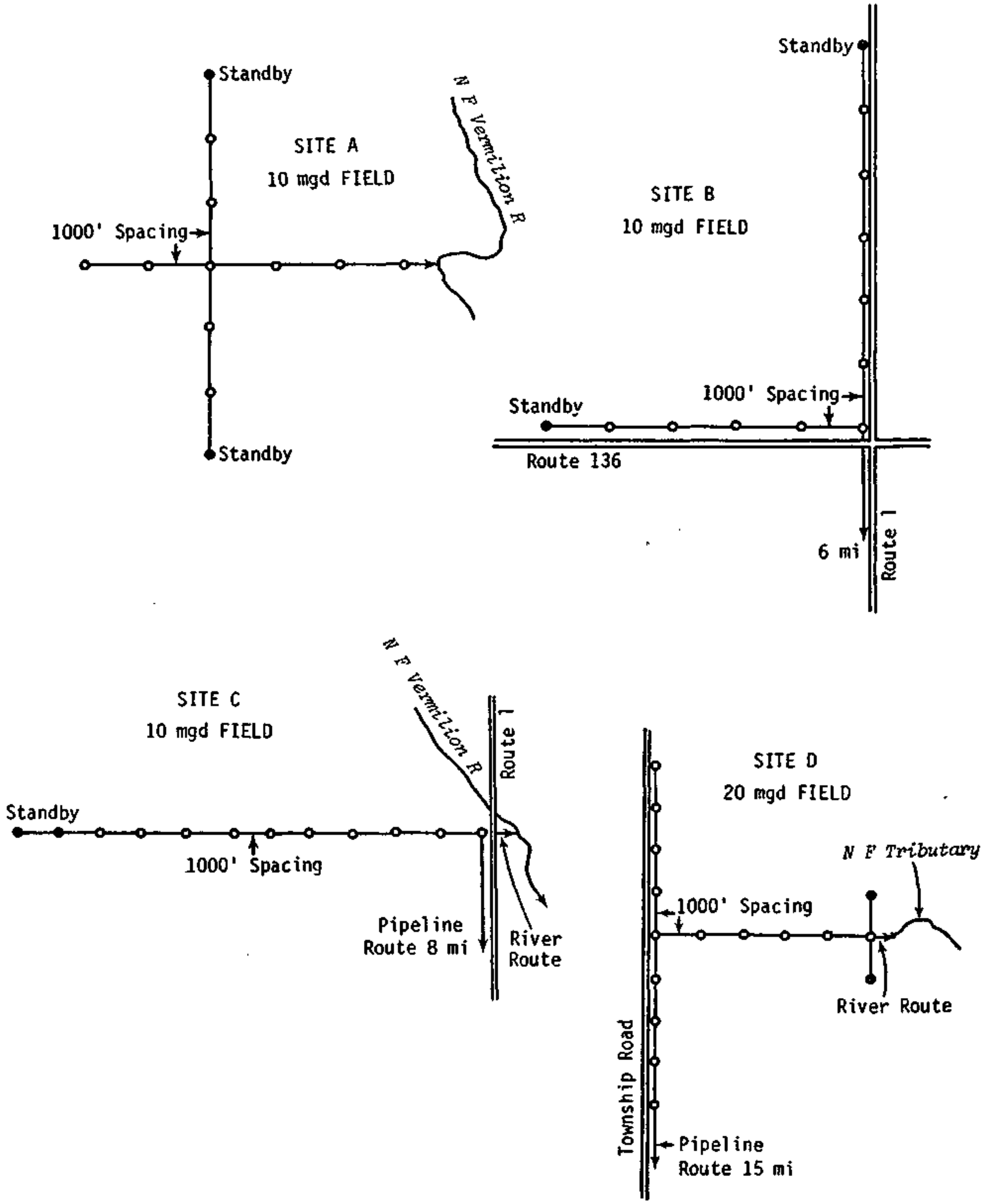


Figure 26. Details of proposed well fields

Effect on Surrounding Wells. The effect of full development of A, B, C, or D well fields on the wells of the surrounding communities in terms of increase in drawdown was investigated. The estimates of interference, or feet of drawdown, are listed below for the wells in these communities.

Town	Interference or feet of drawdown from development of Field A	Field B	Field C	Field D
Alvin	4.0	0.9	0.3	0.1
Armstrong	-	-	-	0.1
Bismarck	2.2	0.4	0.1	
Henning	0.4	2.5	2.5	0.5
Hoopeston	-	-	-	0.1
Potomac	-	-	-	0.7
Rossville	0.1	0.1	0.5	0.7

Though the above figures indicate only slight interference effects from any of the proposed well fields, the shape and extent of the cone of depression due to pumping is rather sensitive to the magnitude of leakage which occurs from the overlying formations. Greater interference would occur if leakage is less than that assumed in this study. One of the important factors to be determined by test drilling, therefore, is the nature and magnitude of recharge to the deep sand and gravel deposits.

Water Transmission Via River or Pipeline. The water pumped from a well field may be transported to Lake Vermilion either by the North Pork Vermilion River and its tributaries or by a suitable pipeline. The river alternative involves a consideration of evaporation and streambed infiltration losses.

Evaporation. In estimating evaporation losses, the maximum monthly rate of 6.98 inches of lake evaporation at Urbana (Roberts and Stall, 1967) was assumed. The stream condition is represented better by the pan evaporation when groundwater is discharged to the North Fork during low-flow conditions'. The lake evaporation was converted to pan evaporation by dividing 6.98 by the pan-to-lake coefficient of 0.79, yielding a maximum evaporation rate of 8.83 inches for the month of July. Average stream width for a flow of 10 mgd is estimated as 50 feet (Stall and Fok, 1968). The evaporation loss from the stream is

$$\frac{8.83 \times (5280 \times 50) \times 7.48}{30 \times 12} \text{ or } 48{,}435 \text{ gpd/mi}$$

Infiltration Loss. Leakage through a streambed may be calculated from

$$q = (K/m)\,\Delta h$$

in which q denotes the leakage, in gpd/ft^2; K is the vertical hydraulic conductivity of the streambed, in gpd/ft^2; m is the thickness of the streambed, in feet; and Ah equals the head difference across the streambed, or the depth of water in the stream, in feet. Typical values of K for glacial drift material in Illinois average 0.03 gpd/ft^2 (Walton, 1965). If we assume m to be about 1 foot and K to be as high as 0.1 gpd/ft^2, K/m equals 0.1 gpd/ft^3 which is thought to be on the high end of a typical range of such values. The maximum likely infiltration rate with 0.76 foot depth of flow (Stall and Fok, 1968) at 10 mgd is

0.1 x 0.76 x (5280 x 50) or 20,064 gpd/mi

The total loss from evaporation and streambed infiltration at a flow of 10 mgd Is, therefore, 48,435 + 20,064, or 68,500 gpd/mi. The estimated transmission loss between field A and Lake Vermilion, a distance of 9 miles, is 0.62 mgd, and between field C and the lake, a distance of 14 miles, is 0.96 mgd.

For a 20 mgd flow, the average flow width in the stream is 55 feet. The evaporation loss is calculated as 53,279 gpd/mi. The infiltration loss at an average 0.92 ft depth of flow comes to 26,717 gpd/mi. Total loss per mile is 79,996 gpd. When water is transported via the North Fork Vermilion system to Lake Vermilion, the travel distance Is 20.9 miles. Total transmission loss for this distance would be 1.67 mgd.

Test Drilling. Exploratory drilling and test holes will be needed for locating suitable capacity wells in and around the sites proposed for the four well fields. Based on information from Layne-Western, the cost per test hole can be approximated by

$$C_{th} = 400 + 10d$$

in which C_{th} is the cost per test hole in 1976 dollars and d is the depth of test hole, in feet. As suggested by Dr. Kempton, five exploratory holes would be required at each field site and one test hole would be drilled at each individual well site. The annual cost can be obtained by multiplying with a cost recovery factor for 50 years and 8 percent interest.

Wells and Pumps. The 12-inch diameter gravel-packed wells with a 6-inch thick annular gravel pack would be drilled to the desired depths. Vertical turbine pumps would be installed on these wells. The pump capacity and the average pumping lift have been defined already for B, C, and D fields. The size of the interconnecting pipeline from one well to the next increases with the increase in the number of wells or the cumulative flow. The interconnecting pipes were considered a part of the transmission cost and their cost was included with the main pipeline. The costs of wells, pumps, annual electric charges, and annual operation and maintenance cost of wells and pumps were computed as shown in Appendix III.

Pipeline Design. The interconnecting pipes within each well field and the transmission pipeline from each well field to Lake Vermilion were designed and their costs computed as explained in Appendix II. Transite or asbestos-cement transmission pipes would be used instead of cast iron, ductile iron, or steel pipe with a cement lining. The asbestos-cement pipes, available in 6- to 36-inch diameter, are cheaper and well-suited to situations where static head (because of downward land slope) balances most of the friction head and the pipe route is along a highway passing through sparsely inhabited areas. These pipes are a bit smoother and have, therefore, a bit higher conveyance than other pipes.

With the yield of Lake Vermilion for drought frequency varying from 2 to 40 years, groundwater pumpage from one of the four fields would be needed for about 10 percent of the time on a long-term basis. However, an average 20 percent pumping time at full field capacity was assumed to provide an ample margin of safety. The computer program developed to calculate the annual cost of the transmission line for various field capacities indicated a net head of less than 25 feet for the pipelines from B and C fields and about 40 feet for that from field D. These extra heads were included in estimating the average pumping head for the pumps and motors installed on the wells. The transmission line cost comprises, therefore, the pipeline construction cost, pipeline maintenance cost, and the easement cost.

Annual Costs. The costs of test drilling, wells and pumps, and pipelines for transporting water from each of the four well fields to Lake Vermilion via the river route and/or the pipeline are given in Table 15. These costs are given on an annual basis by means of the suitable cost recovery factors specified in Appendix II and III.

The supply rate to the lake when water is transported via the river equals the well field capacity minus the water loss from evaporation and infiltration, taken as 0.6, 1.0, and 1.7 mgd for 10, 10, and 20 mgd well fields A, C, and D, respectively. This loss can be applied to lower capacities of 2, 4, 6, and 8 mgd for fields A and C and 10 mgd for field D. The supply rate so obtained at lower pumpage is a little lower than that obtained when a reduced depth of flow in the stream from infiltration is allowed.

The annual cost and water supply information in Table 15 are plotted in Figure 27. It is evident that augmenting the water supply by transporting groundwater via the river is very economical. In the event that a trend is detected for gradually increasing water use by the riparian owners or by industry (if it develops along the river upstream of Lake Vermilion), the water can be piped or some suitable agreements for limiting the use can be worked out with the users.

TABLE 15. Annual Cost of Supplying Water to Lake Vermilion (in thousands of dollars)

Well field	*Supply rate (mgd)*	*Annual cost of test drilling*	*wells & pumps*	*trans-mission*	*total*	*Main line diam (inches)*	*Means of trans-port*
A	1.4	2.1	17.3	2.6	22.0	12	river
	3.4	2.8	34.4	7.4	44.6	16	
	5.4	3.3	46.4	10.5	60.2	18	
	7.4	3.9	58.4	13.6	75.9	20	
	9.4	4.4	71.5	17.1	93.0	21	
B	2.0	2.1	17.8	48.6	68.5	14	pipeline
	4.0	2.9	35.6	66.3	104.8	18	
	6.0	3.4	48.1	77.5	129.0	20	
	8.0	3.9	60.6	97.5	162.0	24	
	10.0	4.4	73.2	113.6	191.2	27	
C	2.0	2.0	17.6	62.7	82.3	14	pipeline
	4.0	2.7	34.9	87.9	125.5	18	
	6.0	3.2	47.2	99.3	149.7	20	
	8.0	3.7	59.6	128.9	192.2	24	
	10.0	4.2	72.0	150.5	226.7	27	
	1.0	2.0	16.8	2.5	21.3	12	river
	3.0	2.7	33.1	7.5	43.3	16	
	5.0	3.2	44.7	11.4	59.3	18	
	7.0	3.7	56.3	15.9	75.9	20	
	9.0	4.2	67.9	20.6	92.7	21	
D	10.0	3.4	58.2	264.5	326.1	27	pipeline
	20.0	5.1	106.0	376.6	487.7	36	
	8.3	3.4	52.9	10.4	66.7	21	river
	18.3	5.1	95.8	28.7	129.6	30	

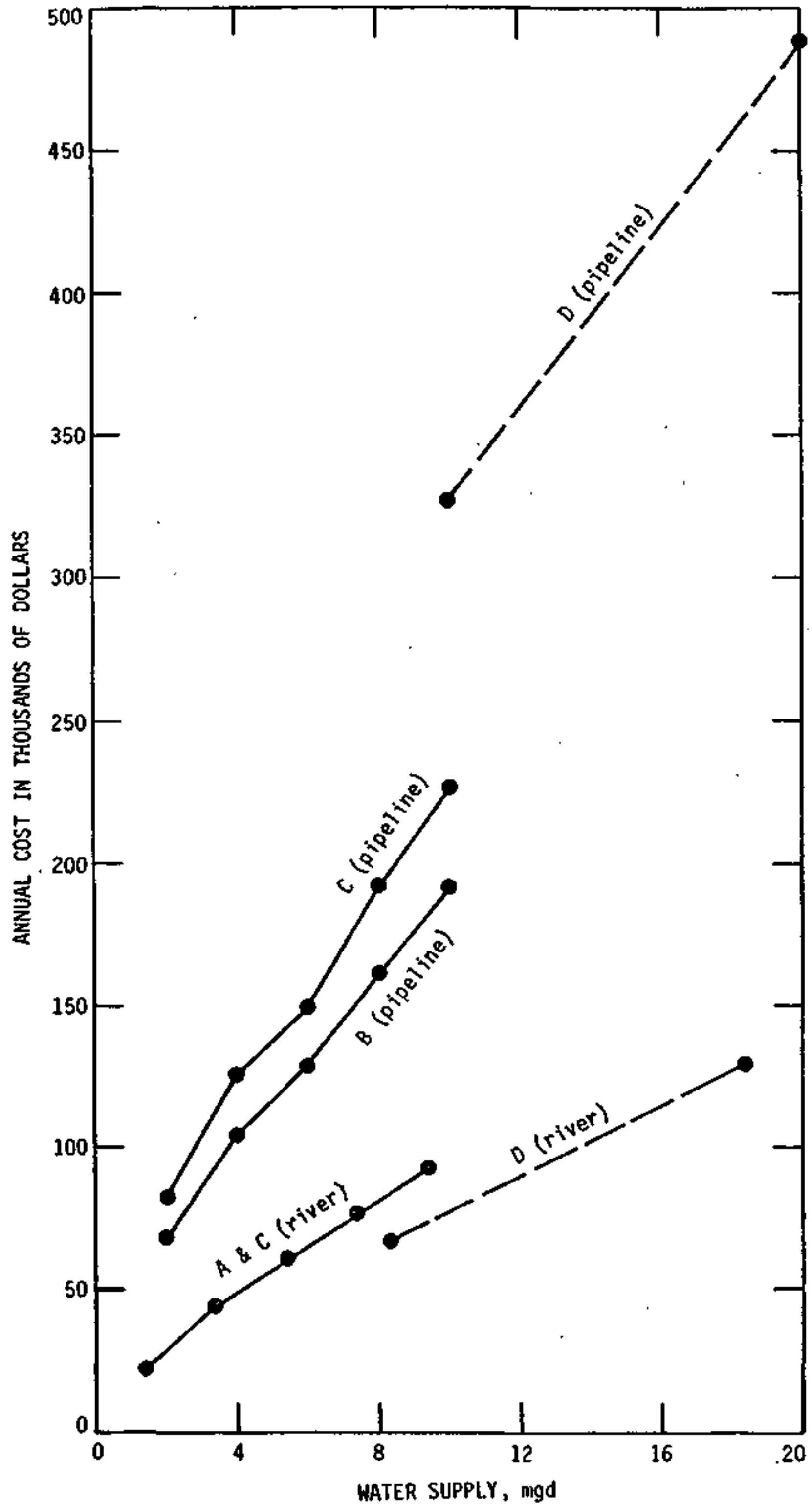


Figure 27. Annual cost of supplying water to Lake Vermilion from the four well fields via the river or pipeline

ASSOCIATED MATTERS

11. WATER QUALITY AND TREATMENT

The Inter-State Water Company supplies finished water to Danville and the surrounding communities. Water is pumped from the North Fork Vermilion River, upstream of the weir opposite the treatment plant and downstream of Lake Vermilion. Alum, potassium permanganate, chlorine, fluoride, and a coagulant aid are fed. The water is then mixed, settled, filtered, and passed to the clear wells. Water is then chlorinated, treated with ammonia, and passed to the 300,000 gallon elevated tank and to the 5.0 and 3.0 million gallon stand pipes.

The quality of raw and finished water from some recent samples is shown in Table 16 in terms of the observed concentrations. It is evident that concentrations of iron and manganese higher than allowed with no sequestering of these substances (Table 17) occur sometimes in both raw and finished water. These substances in higher concentrations stain laundry, corrode plumbing, and cause some economic problems. They do not adversely affect health. Though some fraction is removed during coagulation and filtration, other methods to ensure meeting the finished water standards would be 1) addition of chemicals, e.g., polyphosphates, to keep them in solution, 2) lime-soda softening process if it is desired to soften the water, or 3) oxidation before filtration. The raw water standards in Table 17 are from the Illinois Pollution Control Board Rules and Regulations, Chapter 3> Section 204, and Chapter 6, Section 307 and the standards for finished water are from Chapter 6, Section 304.

Water quality data from the Illinois State Water Survey files and other sources were used to assess the quality of water from alternative sources vis-a-vis that from the North Pork Vermilion River.

Vermilion River. Water samples from the Vermilion River upstream of the confluence with the North Fork, which would be near the intake tower for the alternative of water from the Vermilion River, were collected by the Central Foundry Division of General Motors, Danville, and analyzed in the Illinois Environmental Protection Agency laboratory in Champaign. The results of these analyses for seven samples collected in September, October, and November are shown in Table 18. The sample period does not cover the full year but the results may not be much different for the high flow months of April to July. The following differences in water quality are noted from Tables 16 and 18.

TABLE 16. Raw and Finished Surface Water Quality, Danville

Substance	Reported as	Observed concentrations (mg/l) Raw water 4/5/73	Raw water 12/9/74	Raw water 2/12/76	Finished water 4/5/73	Finished water 2/12/76	Finished water 12/6/76
Iron	Fe	0.1	0.1	3.1	0.6	0.0	0.0
Manganese	Mn	0.01	0.00	0.15	0.20	0.12	0.04
Calcium	Ca	64	76	54	6.3	47	58
Magnesium	Mg	27	31	15	26	15	34
Ammonium	NH_4	0.40	0.00	0.89	0.30	0.39	
Sodium	Na	5.0	7.0	5.0	5.0	6.4	15.0
Potassium	K	1.3	1.3	4.3	1.3	3.0	
Silica	SiO_2	8.0	9.0	5.0	9.0	,4.0	
Arsenic	As	0.00	0.00	0.00		0.00	0.00
Barium	Ba	0.3	0.10	0.00		0.00	0.00
Boron	B	0.01	0.10	0.30	0.10	0.30	
Cadmium	Cd	0.00	0.00	0.00	0.00	0.00	
Chromium	Cr	0.00	0.00	0.00	0.00	0.00	
Copper	Cu	0.00	0.00	0.05	0.12	0.00	
Lead	Pb	0.00	0.00	0.00	0.00	0.01	0.00
Mercury	Hg	0.00	0.00	0.00		0.003	0.000
Fluoride	F	0.7	0.30	0.20	0.20	0.50.	
Chloride	Cl	18	19	13	15	44	
Nitrate	NO_3	37	24.0	12.3	38.0	15.0	
Sulfate	SO_4	73	70	30	50	62	
Alkalinity	$CaCO_3$	134	206	160	170	72	
TDS		300	379	170	335	270	
pH	pH units		8.5	7.9		6.9	
Hardness	$CaCO_3$	271	318	194	264	178	
Nickel	Ni	0.00	0.00	0.00	0.00	0.00	0.00
Selenium	Se	0.01	0.00	0.00		0.00	0.00
Silver	Ag	0.00	0.00	0.00		0.00	
Zinc	Zn	0.02	0.00	0.00	0.02	0.00	0.00
Cyanide	CN		0.00	0.00		0.00	0.00
Phosphate	PO_4	0.00	0.21	4.45		0.06	

TABLE 17. Illinois Pollution Control Board Water Quality Criteria for Public Water Supplies

Substance	*Reported as*	Maximum concentration* (mg/l) *Raw water*	*Finished water*
Arsenic	As	0.01	0.01
Barium	Ba	1.0	1.0
Cadmium	Cd	0.01	0.01
Chloride	Cl	250	
Chromium	Cr		0.05
Copper	Cu		1.0
Cyanide	CN	0.01	0.2
Fluoride	F		2.0
Foaming Agents	MBAS	0.5	0.5
Iron	Fe	0.3	0.3(a)
Lead	Pb	0.05	0.05
Manganese	Mn	0.05	0.05(a)
Mercury	Hg		0.002
Nitrate	NO_3	45.5	45.5(b)
Selenium	Se	0.01	0.01
Silver	Ag		0.05
Sulphates	SO_4	250	
TDS		500	
Turbidity	JTU		1.0 JTU(c)
Zinc	Zn		5.0
Total coliform/100 ml		20,000†	
Fecal coliform/100 ml		2,000†	

Notes: * Maximum allowable 12-month-average concentrations.

(a) Excess concentrations allowable if sequestration is effective.

(b) Allowed up to 91 for 35 calendar days, no single period to exceed 15 consecutive days.

(c) Higher turbidity up to 5 may be allowed if certain conditions are met.

† Figures are the 12-month running geometric means of coliform densities.

TABLE 18. Water Quality at the Proposed Intake in the Vermilion River

		1977 Observed concentrations (mg/l unless indicated)							
Substance	*Reported as*	*9/28*	*10/6*	*10/13*	*10/19*	*10/27*	*11/2*	*11/9*	*Mean*
Iron	Fe	0.4	1.2	0.6	0.3	0.7	1.2	0.6	0.7
Manganese	Mn	0.05	0.09	0.07	0.04	0.05	0.13	0.06	0.07
Calcium	Ca	83	75	84	88	69	65	85	78
Magnesium	Mg	37	34	36	37	31	28	35	34
Ammonium	NH_4	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0.01
Sodium	Na	11	8	10	11	7	7	11	7
Potassium	K	1.2	1.9	1.7	1.1	2.1	1.8	0.9	1.5
Silica	SiO_2	12	12	11	11	11	9.1	10	11
Arsenic	As	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Barium	Ba	0.1	0.1	0.0	0.1	0.4	0.2	0.1	0.1
Boron	B	0.2	0.2	0.2	0.2	0.2	0.3	0.3	0.2
Cadium	Cd	0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00
Chromium	Cr	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Copper	Cu	0.01	0.01	0.01	0.00	0.00	0.00	0.01	0.01
Lead	Pb	0.00	0.01	0.03	0.00	0.00	0.01	0.01	0.01
Mercury*	Hg	0.0	0.3	0.1	0.0	0.0	0.05	0.0	0.06
Fluoride	F	0.3	0.3	0.3	0.3	0.3	0.3	0.2	0.3
Chloride	Cl	23	20	22	22	22	21	26	22
Nitrate	NO_3	29	29	31	32	29	7.1	30	27
Sulfate	SO_4	67	61	63	68	54	51	62	61
Alkalinity	$CaCO_3$	254	230	246	258	219	200	258	238
TDS		422	400	428	424	367	358	428	404
pH	pH units	8.4	8.1	8.3	8.4	8.3	8.1	8.2	8.3
Hardness	$CaCO_3$	358	323	362	394	322	279	360	343
Nickel	Ni	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Selenium	Se	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Silver	Ag	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Zinc	Zn	0.0	0.1	0.2	0.0	0.0	0.0	0.1	0.1
Cyanide	CN	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Phosphate	PO_4	0.23	0.22	0.13	0.13	0.13	0.31	0.22	0.20
Total Organic Carbon		11	5	11	1.5	6.4	8	6.6	7.1
Turbidity	JTU	21	39	22	9	32	108	15	35
Total coliform/ 100 ml		1700	24000	26000	17000	20000	18000	17000	14092†
Fecal coliform/ 100 ml		330	620	520	350	320	280	300	372†
Fecal strep./ 100 ml		160	220	270	160	680	430	110	242†

* In microgram/liter

† Geometric mean

	Average values (mg/l)			
	No. of Samples	Iron	Alkalinity	Hardness
North Fork	3	1.1	167	26l
Vermilion River	7	0.7	238	343

The quality of water from the Vermilion River at the proposed intake is very similar to that from the North Pork Vermilion River except for higher mineralization.

Wabash River. Water quality records published by the Indiana State Board of Health (1974) indicated only one station (a few miles downstream of Lafayette, Indiana, and upstream of the confluence with the Vermilion River) which had data for 15 water quality parameters. The drainage area of the Wabash River at Lafayette is 7267 square miles and at Covington, 8218 square miles. The respective river miles are 311.9 and 271.1 upstream of the Ohio River. Water quality information in Table 19 can be taken as applicable to that at the proposed intake for the alternative "water from the Wabash River in Indiana" a little upstream of Covington. The following differences in water q.uality are noted.

	Average values (mg/1)				
	No. of Samples	Chloride	Nitrate	Nickel	Zinc
North Fork	3	17	24.4	0.00	0.01
Wabash River	12	26	11.8	0.03	0.10

TABLE 19. Water Quality of the Wabash River, Station WB 292 (near Lafayette, Indiana) for 1974

Substance	Reported as	No. of Samples	Concentration (mg/l) Minimum	Maximum	Average	Geo.mean	Median
Ammonium	NH3-N	12	0.20	1.40	0.44	0.36	0.30
Cadium	Cd	12	0.01	0.01	0.01	0.01	0.01
Copper	Cu	12	0.02	0.03	0.022	0.021	0.02
Chloride	Cl	12	14	36	26	25	28
Nitrate	NO_3	12	1.8	20.9	11.8	10.0	12.7
Sulfates	SO_4	12	40.0	90.0	65.2	64.0	63.5
Suspended Solids		12	6	220	52	36	37
pH	pH units	12	8.1	8.5	8.3	8.3	8.4
Nickel	Ni	12	0.02	0.05	0.03	0.028	0.020
Zinc	Zn	12	0.03	0.29	0.104	O.O67	0.095
Phosphorus	P	12	0.16	0.45	0.24	0.23	0.20
BOD	BOD	12	2.0	8.0	4.1	3.8	4.3
DO	DO	12	8.0	13.0	10.5	10.3	9.8
Fecal coliform/100 ml		12	520	4600	2497	2061	2550

The Wabash River water has somewhat higher concentrations of chloride, nickel, and zinc than the North Fork, but the same are much lower than the permissible limits given in Table 17.

Groundwater near Danville. Groundwater quality from wells drilled in and around Danville are given in Table 20. The Winter Avenue Well Field water quality shows an average of 288 mg/l hardness and 392 mg/l total dissolved solids (TDS). These values are higher than the corresponding concentrations of 261 and 283 for the North Fork water, but less than the allowable standards. Because the potential of this well field is only 0.75 mgd, any future use of this source would affect the overall hardness and TDS concentrations only slightly because of dilution with other waters.

For the alternative of 2 wells, one on each side of Lake Vermilion in T20N, R11W, sections 19 and 31, the water quality is not much different from that for the North Fork. The potential of this source is 1.0 mgd

Groundwater from Northern Vermilion County. This alternative provides the possibility of developing 10 to 20 mgd or more from well fields A, B, C, and D (see Figure 25). These fields lie in T22N, R12W and in the northeastern and northwestern portions of T21N, R12W, respectively. Typical water quality is shown by the average of samples given in Table 21. Also given are the number of samples and the observed range of concentration for the substances or water quality parameters listed. The following differences are noted.

	Average values (mg/1)										
	NH_4	*Na*	*SiO_2*	*B*	*Cl*	*NO_3*	*SO_4*	*TDS*	*pH*	*Hardness*	*Phosphate*
North Fork	0.43	6	7	0.14	17	24.4	58	283	8.2	261	1.55
T22N, R11W	1.40	25	19	0.34	5	0.9	9.6	399	7.6	338	0.70

The groundwater has very low concentrations of nitrate and sulfate. It has more TDS and hardness, but the concentrations are lower than those allowed by the standards. If the Danville water supply were to be softened, the extra cost of chemicals to achieve a similar hardness in the finished water would be about 1¢ per 1000 gallons (Singh and Adams, 1977) more than that for the North Fork Vermilion River water.

The quality of water in wells of some nearby communities is given below.

			Concentrations (mg/1)				
	Well No.	*Date*	*Fe*	*Hardness*	*TDS*	*Cl*	*Alkalinity*
Bismarck	1	12/11/74	3.2	267	502	16	462
Hoopeston	5	6/7/77	1.5	294	370	3	330
Rankin	2	6/21/77	1.6	298	400	9	370
Rossville	1	8/10/72	T	324	380	0	364

TABLE 20. Groundwater Quality from Wells Drilled in and around Danville

Location	*Date of Sample*	Observed concentration (mg/l) *Fe*	*Mn*	NH_4	*Na*	*Ca*	SiO_2	*F*	NO_3	*Cl*	SO_4	*Alk*	*Hard-ness*	*TDS*
Winter Avenue Well Field (T20N, R11W, Sec. 28.8a)														
Well #6	11/11/13	0.4	0.0	0.4	47.9	62.4	19.4	-	0.9	1	16.0	-	290	415
#4	11/14/13	0.9	-	1.8	29.5	62.8	28.6	-	-	2	8.3	-	287	372
#7	11/11/13	1.5	0.0	1.8	38.6	69.9	32.2	-	0.2	2	13.1	-	304	432
#3	5/7/36	0.0	0.0	0.7	25.7	62.9	14.0	-	1.2	3	17.7	326	291	377
#3	10/22/45	1.4	-	-	-	-	-	-	-	3	13.8	332	270	366
Domestic-Industrial-Commercial Wells (T20N, R11W)														
8.1d	8/26/71	0.3	-	2.0	-	-	-	-	-	86	-	472	252	626
8.5c	7/23/70	2.0	0.01	-	-	-	-	0.5	12.8	6	-	440	366	458
15.5a	2/5/70	1.5	-	-	-	-	-	-	1.5	1	-	364	414	513
17.8c	4/25/56	3.2	-	-	-	-	-	-	-	8	-	396	296	527
20.1e	6/12/55	0.4	-	-	-	-	-	-	-	57	13.4	388	112	506
20.7a	7/26/63	1.5	-	-	-	-	-	-	0.1	8	-	396	272	438
20.7e	7/9/56	2.0	-	-	-	-	-	0.7	0.2	23	-	392	244	418
21.4g	8/3/57	1.8	-	-	-	-	-	-	-	5	-	364	460	497
21.4h	9/14/63	3.9	-	-	-	-	-	-	1.2	2	-	448	372	452
21.8h	6/18/60	0.7	-	-	-	-	-	-	-	8	-	316	400	452
21.8h	6/18/60	0.8	-	-	-	-	-		-	12	-	296	384	398
21.8h	6/18/60	15	-	-	-	-	-	-	-	7	-	252	238	308
26.2a	12/29/60	9.4	-	-	-	-	-	-	-	19	-	404	300	428
28.7a	3/23/73	1.5	-	-	-	-	-	0.8	9.7	1	-	368	298	424
29.5e	8/20/47	4.7	-	-	-	-	-	-	-	9	-	524	792	1032
31.2h	5/10/70	2.6	-	-	-	-	-	-	1.4	4	-	246	236	296
Domestic-Industrial-Commercial Wells (T20N, R12W)														
12.5c	7/4/67	7.9	-	30.7	-	-	-	-	-	5	-	552	370	555
13.4c	4/9/68	7.1	-	13.9	-	-	-	-	-	27	-	544	350	588
23.1f	2/15/72	5.0	-	-	-	-	-	-	0.9	7	-	436	436	619
24.1a	12/7/65	1.1	0.4	-	-	-	-	0.7	3.0	0	-	348	260	387
25.3a	10/29/74	0.6	-	-	-	-	-	-	2.8	24	-	388	446	670
27.1d	7/21/64	0.2	-	-	-	-	-	-	-	8	-	364	428	497

TABLE 20. (Continued)

Location	Date of Sample	Fe	Mn	NH_4	Na	Ca	Sio_2	F	No_3	Cl	So_4	Alk	Bard-ness	TDS
		Observed concentration in (mg/l)												
Domestic-Industrial-Commercial Wells (T19N, R11W)														
4.5g	12/9/69	6.1			-	-				11	-	324	368	457
4.6c	8/11/64	3.0	0.08	-	-	-			0.4	12		390	412	502
5.1c	12/21/55	1.0	-	-		97				7	-	348	424	497
5.3d	5/2/55	0.5			-	-	-	-	-	15	-	392	364	434
5.4c	3/18/29	1.0	0.00	2.1	11.0	102.5	11.0	-	0.4	20	110	364	477	610
14.7c	5/17/71	4.5	0.00	-	-	-	-	0.3	0.8	4	-	370	344	371
15.4d	9/19/63	5.6	-	-	-	-	-			4	-	480	375	493
15.8h	7/8/37	4.8	0.00	0.6	19.8	103.8	10.0	-	2.6	20	47.7	416	450	510

TABLE 21. Groundwater Quality Summaries for Wells Drilled in Northern Vermilion County

			Concentration (mg/l) T22N, R12W	
Substance	*Reported as*	*Number*	*Range*	*Average*
Iron	Fe	27	0.0-4.6	1.5
Manganese	Mn	11	0.00-0.04	0.01
Calcium.	Ca	10	57-71	65
Magnesium	Mg	11	33-42	39
Ammonium	NH_4	10	0.0-3.1	1.4
Sodium	Na	10	17-49	25
Potassium	K	5	1.9-2.5	2.3
Silica	SiO_2	11	16-22	19
Barium	Ba	5	0.0-0.1	0.08
Boron	B	7	0.2-0.8	0.34
Cadmium	Cd	5	Not detectable	
Chromium	Cr	5	Not detectable	
Copper	Cu	5	0.0-0.01	0.004
Lead	Pb	5	0.00-0.05	0.01
Mercury	Hg	4	Not detectable	
Fluoride	F	15	0.2-1.4	0.4
Chloride	Cl	26	0-29	5
Nitrate	NO_3	17	0.0-5.7	0.9
Sulfate	SO_4	14	0-13	9.6
Alkalinity	$CaCO_3$	26	282-388	365
TDS		27	350-506	399
pH	pH units	9	7.3-8.1	7.6
Hardness	$CaCO_3$	25	280-416	338
Nickel	Ni	5	0.00-0.05	0.01
Selenium	Se	4	Not detectable	
Silver	Ag	4	0.00-0.008	0.002
Zinc	Zn	5	0.00-0.03	0.006
Cyanide	CN	3	Not detectable	
Phosphate	PO_4	12	0.0-4.8	0.7

12. LEGAL, INSTITUTIONAL, AND POLITICAL ASPECTS OF WATER SUPPLY PROM THE WABASH RIVER IN INDIANA

Transporting water from the Wabash River above Covington in Indiana has been evaluated in section 7 as one of the several alternatives to augment the supply from Lake Vermilion to meet future water requirements of the city of Danville. Possible groundwater development along the Wabash River near Covington has been shown in section 8 to be restricted to an area where extensive test drilling is needed to determine if wells of sufficient capacity can be drilled to develop a sizeable water supply. Therefore, the legal, institutional, and political aspects of providing water from the Wabash River to Danville have been considered in detail in this section. Such considerations applicable to possible development of groundwater and its transport to Danville are described briefly.

The course of the Wabash River in Indiana and Illinois is shown in Figure 28. For about 100 miles from Terre Haute downstream to the Ohio River the Wabash River provides the state boundary between Illinois and Indiana. Where the Wabash River is a boundary water, it would seem reasonable for persons in Illinois to make use of water from the river if desired. For a reach of about 60 miles from Covington downstream to Terre Haute, the Wabash River flows within about 10 miles of the Illinois state line.

The route(s) that a water supply pipeline may take if water is pumped from the Wabash River near Covington, Indiana, and emptied into Lake Vermilion is shown in Figure 19, section 7. The total pipeline length is 13.8 miles. Ample quantity of water has been shown to be available from the Wabash River except for about 1.5 months during the low-flow period of August through November, on an average of once in 40 years.

The Problem. The pipeline route to transport water from Covington, Indiana, to Lake Vermilion or to the North Fork Vermilion River above the water treatment plant crosses the Illinois-Indiana state boundary. There would certainly be legal problems involved in obtaining rights from the state of Indiana to allow the construction o'f this pipeline and the U8e of water from the Wabash River. Various factors involved in the legal, institutional, and political setting regarding the grant of needed rights were considered. Extensive use has been made of a study by Waite (1968) who summarized Indiana's existing water laws, policies, and important court cases.

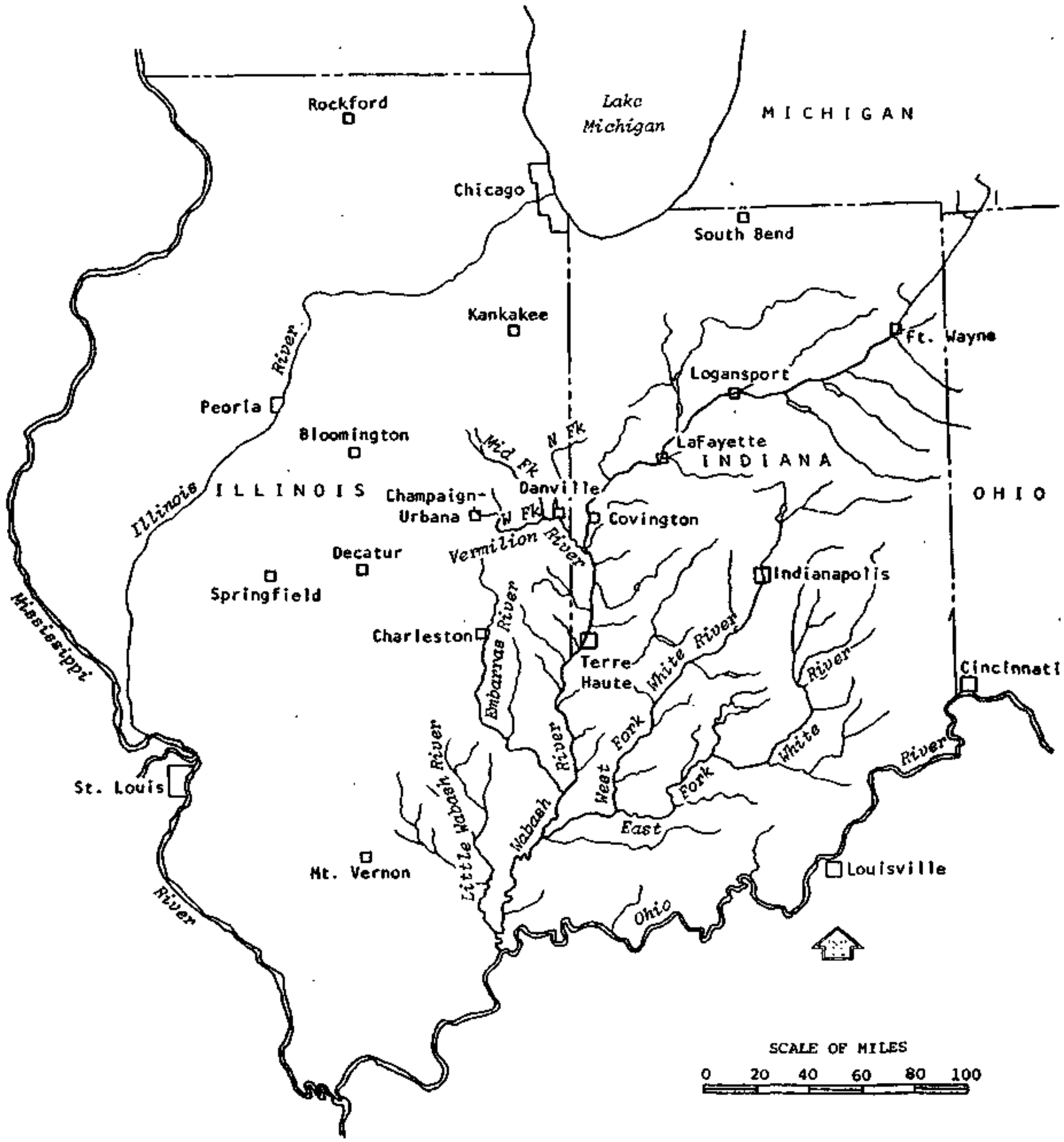


Figure 28. Map showing the states of Illinois and Indiana and the Wabash River drainage system

Riparian doctrine and Reasonable Use of Water. In Indiana, as in Illinois, an individual's right to use water from a natural stream is generally governed by the doctrine of *riparian rights.* This doctrine with modifications Is applied In most states east of the Mississippi River and In the humid states which border the Mississippi River to the west. The doctrine provides that landowners whose lands are contiguous to the water have rights to share in the use of the water. In the original English Common Law, for land to be riparian It was required that it must be *lapped by water.* In Indiana neither laws nor court cases have defined riparian land, and just what is meant by riparian landowner is still open to question. One definition of riparian land requires that It be located within the watershed of the stream from which the water is taken, and another requires that it be part of a tract that abuts the watercourse.

In Indiana, as In many other eastern states, the riparian owner's rights to use water are limited to uses which are *reasonable* under the circumstances. It seems that a riparian may consume as much water as Is necessary to satisfy his domestic needs. Indiana state law provides a special distinction for domestic water use and states that it "shall include but not be limited to water for household purposes, drinking water for livestock, poultry, and domestic animals." It further states that "the use of water for domestic purposes shall have priority and be superior to any and all other uses." Water uses which are often included In the term *domestic use* include drinking, cooking, laundry, and sanitation, the maintenance and subsistence of the proprietor and his family, and the watering of stock and other domestic animals.

The use of water for irrigation of agricultural land is a consumptive use of the water. It has never been decided in Indiana that Irrigation is or is not a reasonable use.

What comprises *reasonable use* of water in Indiana has not been determined. Theoretically, any use is permissible provided it is reasonable. In determining whether a use is reasonable or not, all the circumstances bearing on the use of the water are considered and it is not correct to say that use for a particular purpose will always be reasonable or unreasonable. Practically any use might be considered reasonable and many uses of water can easily be determined as not reasonable.

Regulation of Water Use from Navigable Streams. In 1977 the state of Indiana does regulate the use of water from navigable streams and this includes the Wabash River. It has been determined that the state of Indiana is the owner of the bed of the Wabash River. The United States Supreme Court has determined that the question of bed ownership of rivers Is determined by federal law. The bed of the river is divided from the upland by the location of the ordinary high-water mark. In general the federal test is that a particular stream or river is navigable if the water was susceptible of being used as a highway of commerce by boats at the time that state was admitted to the Union. Indiana became a state in 1816 and it has been determined that the Wabash River was navigable by boats in use at that time. Consequently, it has been determined that the title to the bed of the Wabash River passed to the state at that time.

It thus seems that the state of Indiana has ample legal authority to regulate use of water from the Wabash River. It is the present practice of the state of Indiana to require a permit from anyone who wishes to make use of water from the Wabash River. In order to bring water from the Wabash River to Danville, Illinois, it would be necessary for the city or some other legal entity to purchase riparian land along the Wabash River at the point of intake near Covington. It would then be possible to pump this water across the state line to Lake Vermilion. The city or legal entity would be required to obtain a water use permit from the state of Indiana. In order to obtain such a permit it would seem certain that public hearings would be held and the city of Danville would be required to show the public benefits of such a scheme. It seems possible that there would be opposition to such a scheme by Indiana residents.

Regulation of Structures Built in a Flood Plain. The Indiana legislature passed a law in 1955 which declared water, in any natural body of water, which may be applied to any useful and beneficial purpose to be a natural resource and under the ownership of the state. A flood control act was passed the same year and in 1965, the Department of Natural Resources was formed and took over all of the duties of regulation under this law.

In 1977, the Indiana State Division of Water within that department administers a flood control act which states that channels and portions of the flood plains of rivers shall be kept free and clear of obstructions which cause undue restrictions in the capacity of the floodways. In order to build any structure in a flood plain in Indiana, a permit must be obtained from the state of Indiana, Division of Water. It seems certain that any intake structure or pumping station which would be constructed near the Wabash River in Indiana would be occupying a portion of the flood plain of that river. Because of this the city of Danville or any legal entity which wishes to construct such an intake tower and pumping station would be required to obtain a permit from the state of Indiana. In the process of obtaining such a permit it would undoubtedly be necessary to hold a public hearing and obtain input from local residents as to the desirability, purpose, and use of such an installation.

Protected Flows in Indiana Streams. In 1977 the state of Indiana determined a policy, carried out by the Division of Water, which states that anyone making *consumptive use* of water from Indiana streams is required to obtain a permit. At present there is no state law requiring such a permit, but it has been the policy of the state to require it. During the past few years the principal category of water users who have obtained permits for consumptive use of water have been power plants which take water from Indiana streams for cooling purposes.

If the city of Danville or any associated legal entity were to construct an intake on the Wabash River near Covington, a permit to remove water from the river would be required because water which would be pumped to Danville would be considered a consumptive use of the water from the Wabash River. At present there exists no state of Indiana policy as to what conditions must prevail in order that such permits be issued. It would seem reasonable that the state of Indiana would have many reservations or doubts about Issuing such a permit for water to be exported from Indiana to Illinois.

One of the restrictions placed on the consumptive use of water from Indiana streams is that no user can remove water when the flow in the stream Is below the *7-day 10-year low flow.* This is the lowest flow expected over 7 *consecutive days* at an average of once In 10 years. This 7-day 10-year low flow is a value used extensively by the U.S. Environmental Protection Agency in processing permits for waste discharges into streams throughout the United States. As shown in section 7, water will not be available from the Wabash River for 1.5 months during August to November for a 40-year drought.

Indiana 1977 Water Resources Study Commission. On July 20, 1977, Otis R. Owen, Governor of the state of Indiana, signed an Executive Order which established the Governor's Water Resources Study Commission. The Commission consists of about 30 members and its purpose Is to develop recommendations "for an integrated system of policy, law, and management to provide the essential framework within which the human, social, and economic water needs of the people of Indiana may be satisfied in a timely and equitable manner." The Commission Is to recommend a water policy which "shall be based on a comprehensive examination of water availability, law, management, and present and projected human, social, and economic uses and needs." The Commission is to provide a report by January 1, 1980, incorporating Its findings and recommendations which may Include proposals for legislative or administrative action deemed necessary by the Commission.

In case the city of Danville, some legal entity representing the city, or even the state of Illinois were to approach the state of Indiana requesting the permits described above and approval to take water from the Wabash River for use at Danville in Illinois, it seems possible that the state of Indiana would be reluctant to make any major long-term commitments until this elaborate two-year study of its own water resources use and policies is completed.

The Indiana Water Conservancy District Act. This act was passed in 1957. It provides that a conservancy district may be organized for any of nine broadly stated purposes. One of the purposes is providing water supply including treatment and distribution for domestic, industrial, and public use. Since the passage of this act, there have been as many as 70 districts formed in the state of Indiana, a few of which provide public water supply. It might seem reasonable that such a district could be formed in the Covington region to provide water supply to the city of Danville, Illinois; however, there are legal aspects of the district that would seem to place heavy limits on its use in this manner.

A water conservancy district can be formed by petitioning the county court and by holding a public election by the persons within the boundaries of such a district. Naturally, the boundaries of such a district would have to be within the state of Indiana. In the event a conservancy district proposes to render the service of water supply to users outside of its boundaries, the district must petition the Public Service Commission of Indiana for territorial authority to provide such service. The district would be required to file its initial schedule of rates and charges to patrons of such a district with the Public Service Commission of Indiana. Thereafter, for the purpose of changing such rates, it would be subject to the jurisdiction of the Commission in the same manner as provided by law for the regulation of all rates and charges of municipal water utilities.

The Use of Groundwater. A test-drilling program is indicated to evaluate the groundwater potential in the area 2 to 4 miles northwest of Covington. If the test drilling were to show the possibility of developing an economical 5 to 15 mgd capacity well field, a groundwater supply could be developed for piping to Danville. Such a supply would not be subject to the laws and regulations governing the use of stream water. For this reason it is pertinent to examine the laws in Indiana governing the use of groundwater.

Generally speaking, the state of Indiana has only a few cases dealing with groundwater. Most groundwater problems that arise are dealt with on the assumption that the water is *percolating* and that it is not in an underground stream. Generally, the doctrine held is that the groundwater is part of the land in which it is found and that the owner of the land has the property right to do what he wishes with the water even if he deprives his neighbors of water. However, he may not deliberately and maliciously pump water away from his neighbor's use.

If a well field is constructed in the river bottoms of the Wabash River, 2 to 4 miles northwest of Covington, Indiana, it would be a structure within the flood plain of the Wabash River, and as such, would require a permit to build a structure in the flood plain. The construction of such a well field would, thus, be subject to the permit procedures, including a public hearing, that are required for the construction of any structure within a flood plain in Indiana.

Summary. The development of a water supply and its transport for the city of Danville from the state of Indiana would involve the following legal, institutional, and political factors:

1. The state of Indiana regulates the use of water from the Wabash River. A permit would be required.
2. The state of Indiana requires a permit for the construction of an intake or well structure within the flood plain of the Wabash River. A permit would be required for such a construction.
3. The existing policy of the State of Indiana is to require a permit for the consumptive use of any water from Indiana streams. A permit would be required.
4. A private corporation or a water conservancy district formed legally in Indiana to provide water to Danville, Illinois, would be subject to the regulations of the Indiana Public Service Commission.
5. For the next two years the Indiana Water Resources Study Commission will be reexamining all of Indiana's water development and control policies and regulations. It would seem the state of Indiana would hesitate to provide a permanent commitment to an adjoining state during this two-year interim period.
6. It is reasonable to expect resistance from the public in Indiana to a plan for providing water to the city of Danville.

Further consideration of the development of a water supply in Indiana should be contingent upon development of a better outlook for the legal and other factors that might be involved in transporting water from the Wabash River. Prior to such a study it seems reasonable to pursue Illinois sources for meeting the water supply requirements of the city of Danville.

13. LEGAL ASPECTS OF TRANSMISSION OF GROUNDWATER TO LAKE VERMILION VIA NORTH FORK VERMILION RIVER

Information on legal restrictions concerning transport of groundwater from potential well fields in northern Vermilion County to Lake Vermilion by way of the North Fork Vermilion River largely follows a memorandum prepared December 12, 1977, by Cheryl Sylvester of the Illinois Department of Transportation.

Liability. If groundwater from wells in northern Vermilion County were released into the North Fork Vermilion River, the city of Danville would be subject to liability for injury sustained by intervening riparian owners because of the increased flow (Clark, *Waters and Water Rights,* vol. 7 at 632 (§ 632)). In order to protect its interests, the city would need to purchase flowage or flood easements covering land reasonably expected to be flooded as a result of the increased flow. However, because the groundwater would be discharged to the North Fork only during low flow conditions, flooding would not occur and thus purchase of flowage or flood easements would not be necessary.

Withdrawal of Water by Riparian Owners. It is not clear what measures the city should take to ensure that no consumptive withdrawal of the well water (called "developed" water, as opposed to riparian water) is made by intervening riparians. Such legal control of the developed water after it mixes with the natural flow is essential in order for the city to ensure that it can withdraw as much water as it puts in upstream (Druley v. Adam, 102 Ill. 177 (1882)). The city would therefore be well advised to enter into binding agreements with intervening riparian owners, restricting their right to remove the developed water.

Condemnation of Flowage Easement. An easement is "property" and is protected by the constitutional provision that private property cannot be taken or damaged without Just compensation (City of Springfield v. Springfield Consol. Ry. Co., 295 Ill. 234 (1921), and Village of Bradley v. New York Cent. R. Co., 296 Ill. 383 (1921)). The city is authorized to establish waterworks and a waterworks system outside its corporate limits, under Ill. Rev. Stat. 1975, ch. 24 §§11-125-2 and 11-126-3. "Waterworks" is defined in the Ill. Rev. Stat. 1975, ch. 24 §11-130-2, as "a waterworks system in its entirety or any integral part thereof, including mains, hydrants, meters, valves, standpipes, storage tanks, pumping tanks, intakes, wells, impounding reservoirs, or purification plants." This definition is for the purposes of Division 130 of Article 11, concerning construction and purchase of waterworks, but it seems clear that the functions of Divisions 125, concerning construction of waterworks, and 126, concerning joint construction of water supply, are so nearly analogous that the definition of waterworks for Division 130 should be considered, by implication, as the referent intended by the Legislature for Divisions 125 and 126.

Both §11-125-2 and §11-126-3 authorize condemnation of property for waterworks purposes. Section 11-125-2 authorizes condemnation of necessary property "for the purpose of establishing or supplying waterworks and to...extend, improve and operate waterworks...." Under §11-126-3, a municipality may condemn necessary property "for the purpose of locating, constructing, maintaining, or supplying...a system of waterworks...."

Laws conferring powers of condemnation are strictly construed (Prather v. City of Springfield, 202 Ill. App. 406 (1917)). In light of this, the language of §§11-125-2 and 11-126-3 must be evaluated to determine whether they reasonably can be read to authorize a condemnation of flowage easements along a river, in order to obtain the right to increase flow by transmitting added well water to the downstream municipality. The proposition is unusual, so there is no legal precedent directly on this point in Illinois.

The first question is whether use of the river to transmit the water would bring such use within the definition of "waterworks." The answer is unclear, although an argument can be made that use of the river as a means of transmission makes it the functional equivalent or analogue of a water main.

If the river as a means of transmission does not qualify as a component of a waterworks, is use of the river essential to supply a waterworks, within the meaning of §11-125-2 and §11-126-3? Certainly a strong argument can be made that this is precisely the function intended to be filled by use of the river.

Assuming for the sake of discussion that the proposed condemnation would be for an authorized purpose, the municipality would then need to ensure that such condemnation would meet the requirements that apply generally to municipal exercise of eminent domain. The city's attorney can best advise Danville on these general requirements.

Strategy. The section on costs in this report shows that if a groundwater source were developed in northern Vermilion County, the groundwater might be emptied into the North Pork Vermilion River and its tributaries, from whence it would flow by gravity downstream into Lake Vermilion. It would flow from 9 to 21 miles in the stream, depending on the location of the well field. It has been shown elsewhere in this report that it would be much cheaper to use the stream route than to construct a pipeline. The city would, however, assume a liability for this *developed* water in the stream.

It might be considered reasonable to meet these legal problems by developing a system to use the stream for transmitting the water. If, as years go by, riparian owners begin to use the *developed* water, it would then be possible for the city to counteract this action by building the pipeline. The water would then be completely controlled by the city, after bearing the cost of building the pipeline.

FINDINGS

14. ALTERNATIVES FOR MEETING FUTURE WATER DEMANDS

Three future demand curves — SWS, HM&G, and Danville — have been identified in section 2 on future water supply demands. The various alternative sources to supplement the yield of Lake Vermilion for meeting future demands as well as the cost of these alternatives at various levels of development are detailed in sections 4 through 10. These costs are given in terms of July 1976 dollars and do not include the cost of engineering design. Water quality of these sources is not much different from that of the present source, Lake Vermilion. Various alternatives and their combinations to meet each of the three demand curves are analyzed and ranked in the order of increasing cost. However, all combinations are not listed because there would be many such combinations. Legal problems involved In transporting groundwater via the North Fork Vermilion River and its tributaries may change the order of preference derived on the basis of economics alone.

Many options are involved in raising the lake level and dredging. The options arise from questions such as: Will the rise in lake level be 3 or 5 feet? Will the annual dredging rate be 50 or 110 ac-ft? Will the dredged material deposited in spoil areas (low-lying land adjacent to Lake Vermilion and mostly owned by the Inter-State Water Company) be allowed to settle naturally and the reclaimed area given credit for use as agricultural land, or will the dredged material deposited in spoil areas be compacted mechanically and the reclaimed area allowed credit for prime urban land near a lake? Questions about the desirability and acceptability of some alternatives can only be answered after getting the views of groups and entities concerned with Danville water supply, including environmentalists, planners, and others. The impact of credit on lowering the dredging costs is dealt with at length in section 5. However, for the purpose of ranking various alternatives, the credits for agricultural and/or urban land created by suitable disposal of the dredged material in the spoil area are not considered.

The staging of water development to meet the increase In demand with years was not a part of this study. In transporting water by pipelines, the fixed costs vary from an average of 90, 82, and 70 percent of the total annual cost for the well fields in northern Vermilion County, the Wabash River, and the Vermilion River, respectively. The pipeline diameter is designed for the needed supply from the source in 2010. The pipeline construction cost depends on the pipeline diameter and forms a major portion of the total cost. The future demands are still tentative estimates. As such, the optimal staging of alternative source development may be investigated when the alternatives are narrowed down to a few promising ones and the future demands to be met have been determined.

The capacity of a well field can be increased with time by drilling more wells to keep up with the demand. Wells built a little later in response to an increase in demand will last more into the future because a certain useful life is assumed for these wells.

Alternatives for Meeting the SWS Demand Curve. The alternatives are ranked in Table 22 in order of their increasing annual cost. The SWS demand for 2010 is 13 mgd. The corresponding lake yield with half-capacity use under a 40-year drought is estimated as 6.3 mgd. <u>Annual cost is based on the 2010 demand</u>.

Single source alternatives in order of increasing cost are groundwater from well field A via river, water from the Vermilion River, groundwater from well field B by pipeline, and water from the Wabash River. Combined with raising lake level by' 5 feet, the alternatives in order of increasing cost are well field A and wells along the lake, well field A, Vermilion River and wells along the lake, Vermilion River, and dredging. Alternative 11, raising lake level by 5 feet and dredging 50 ac-ft per year, has an annual cost of $124,600. However if credit for agricultural land is given for uncompacted spoil area, this cost will be $114,600. If the dredged materials in the spoil area are compacted, the cost after credit for urban land is estimated as $72,300.

Costs of procuring easements along the North Pork to protect the pumped flow from being used by the riparian landowners is not included in the alternative of groundwater from well field A.

Alternatives for Meeting the HM&G Demand Curve. The alternatives are ranked in Table 23 in order of their increasing annual cost. The 2010 demand is 19.9 mgd. With a 6.3-mgd yield from Lake Vermilion in 2010 and half-capacity use under the 40-year drought condition, the needed capacity of the alternative source or sources would be 13.6 mgd in 2010. Deficits in 2010 with raising the lake level by 3 feet and dredging at 110 ac-ft/yr, raising the lake level by 5 feet and dredging at 50 ac-ft/yr, and raising the lake level by 5 feet and dredging at 110 ac-ft/yr are estimated at 6.7, 6.8, and 3.6 mgd, respectively. Raising the lake level and dredging cannot meet the 2010 demand and would have to be supplemented with water from well field A, the Vermilion River, or well field B. The 2010 demand can be met also by raising the lake level (but not dredging) combined with a supplementary source(s). Single sources capable of supplying 13.6 mgd in order of increasing cost are well field D via the river, well field A and C via the river, well field B and C with pipelines, well field D via the pipeline, and the Wabash River. The Vermilion River has a supply capacity of about 7 mgd and can meet the deficit of 13.6 mgd only in conjunction with another alternative(s).

Combined with raising the lake level by 5 feet, the alternatives in order of increasing cost are well field A, Vermilion River and well field A, Vermilion River and well field B, and well field B. The annual cost of raising the lake level by 5 feet, dredging at the rate of 50 ac-ft per year, and 6.8 mgd from well field A will be reduced from $195,800 to $185,800 and $143,500 with credit for agricultural and urban land, respectively.

TABLE 22. Alternatives for Meeting the SWS Demand Curve
(Annual costs in July 1976 dollars)

Rank	*Alternative*	*Raising Lake level (ft)*	*Dredging rate (ac-ft/yr)*	*Annual cost (dollars)*
1	*2*	*3*	*4*	*5*
1	Groundwater from well field A via river, 5.7 mgd; and 2 wells along the lake, 1.0 mgd			69,400
2	Raising lake level; 1.9 mgd from well field A via river; and 1.0 mgd from 2 wells along the lake	5		69,600
3	Groundwater from well field A via river, 6.7 mgd			70,400
4	Raising lake level; and 2.9 mgd from well field A via river	5		74,100
5	Raising lake level; 1.9 mgd from Vermi-lion River; and 1.0 mgd from 2 wells along the lake	5		83,200
6	Vermilion River, 5.7 mgd; and 2 wells along the lake, 1.0 mgd			85,400
7	Raising lake level; and 2.9 mgd from Vermilion River	5		86,800
8	Vermilion River, 6.7 mgd supply			89,700
9	Vermilion River, 4.95 mgd; wells along lake and in Danville, 1.75 mgd			99,700

TABLE 22. (Continued)

1	2	3	4	5
10	Groundwater from well field B by pipeline, 5.7 mgd; and two wells along the lake, 1.0 mgd			123,700
11	Raising lake level and dredging	5	50	124,600
12	Groundwater from well field B by pipeline, 6.7 mgd			140,600
13	Raising lake level and dredging	3	110	212,500
14	Wabash River, 6.7 mgd, 13.8-mile pipeline			374,900

Note: Annual cost is based on the 2010 demand.

TABLE 23. Alternatives for Meeting the HM&G Demand Curve
(Annual costs in July 1976 dollars)

Rank	*Alternative*	*Raising lake level (ft)*	*Dredging rate (ac-ft/yr)*	*Annual cost (dollars)*
1	*2*	*3*	*4*	*5*
1	13.6 mgd from well field D via the river			100,000
2	Raising lake level; and '9.8 mgd from well field A via river	5		131,600
3	9.4 mgd from well field A; and 4.2 mgd from well field C, both via river			145,900
4	Raising lake level; 7.2 mgd from Vermilion River; 1.0 mgd from • 2 wells along the lake; and 1.6 mgd from well field A via river	5		162 600
5	Raising lake level; 7.2 mgd from Vermilion River; and 2.6 mgd from well field A via river	5		167,100
6	Raising lake level; 7.2 mgd from Vermilion River; 1.0 mgd from 2 wells along the lake; and 1.6 mgd from well field B by pipeline	5		193 100
7	Raising lake level and dredging; and 6.8 mgd from well field A via river	5	50	195,800
8	Raising lake level; 7.2 mgd from Vermilion River; and 2.6 mgd from well field B by pipeline	5		210,900
9	Raising lake level; 8.8 mgd from well field B with pipeline; and 1.0 mgd from 2 wells along the lake	5		215,600

TABLE 23. (Continued)

1	2	3	4	5
10	Raising lake level and dredging; and 6.8 mgd from Vermilion River	5	50	215,600
11	Raising lake level; and 9.8 mgd from well field B with pipeline	5		223,400
12	Raising lake level and dredging; and 3.6 mgd from well filed A via river	5	110	260,300
13	Raising lake level and dredging; and 6.7 mgd from well field A via river	3	110	282,900
14	Raising lake level and dredging; 5.7 mgd from - Vermilion River; and 1.0 mgd from 2 wells along the lake	3	110	297,900
15	Raising lake level and dredging; and 6.7 mgd from Vermilion River	3	110	302,200
16	13.6 mgd from B and C fields via pipeline			308,100
17	13.6 mgd from well field D via pipeline			384,300
18	13.6 mgd from the Wabash River			560,900

Note: Annual cost is based on the 2010 demand.

Alternatives for Meeting the Danville Demand Curve. The alternatives are ranked in Table 24 in order of their increasing cost. The 2010 demand is 17.3 mgd. With the previously described 6.3-mgd yield from Lake Vermilion, the needed capacity of alternative sources would be 11.0' mgd. Deficits in 2010 with raising the lake level by 5 feet and dredging at 50 ac-ft/yr, and raising the lake level by 5 feet and dredging at 110 ac-ft/yr are estimated as 4.2 and 1.0 mgd, repsectively. The supplemental sources could be two wells along the lake, the Vermilion River, or the well fields in northern Vermilion County. Single sources to meet the total deficit of 11.0 mgd in 2010 are the well fields in northern Vermilion County as described for the HM&G curve and the Wabash River. The demands can be met by well field A, the Vermilion River, and well field B in conjunction with raising the lake level by 5 feet.

The annual cost of raising the lake level 5 feet, dredging at the rate of 50 ac-ft per year, and 4.2 mgd from well field A will be reduced from $ "175,400 to $ 166,400 and $ 123,200 with credit for agricultural and urban land, respectively.

Investments for Various Alternatives. The magnitude of investment is a major consideration in choosing one or the other alternative when the capital resources are. limited. Therefore, the capital investments were calculated for each of the alternatives for the three demand curves, as ranked in Tables 22 through 24 in the order of increasing annual cost, and these are given in Table 25. The capital investment, even when spread over the 30-year period of 1980-2010, is not discounted to present value because the staging of various alternatives was not a part of this study. Some indication is given in the 'Remarks' column regarding the magnitude of initial and subsequent investments.

TABLE 24. Alternatives for Meeting the Danville Demand Curve
(Annual costs in July 1976 dollars)

Rank	*Alternative*	*Raising lake level (ft)*	*Dredging rate (ac-ft/yr)*	*Annual cost (dollars)*
1	*2*	*3*	*4*	*5*
1	11.0 mgd from well field D via river			83,700
2	Raising lake level, and 7.2 mgd from well field A via river	5		99,400
3	11.0 mgd from well field, A via river			107,400
4	Raising lake level; 6.2 mgd from the Vermilion River; and 1.0 mgd from 2 wells along the lake	5		125 300
5	Raising lake level; and 7.2 mgd from the Vermilion River	5		131,500
6	Raising lake level and dredging; and 4.2 mgd from well field A	5	50	175,400
7	Raising lake level; and 7.2 mgd from well field B via pipeline	5		183,900
8	Raising lake level and dredging; and 4.2 mgd from the Vermilion River	5	50	187,700
9	11.0 mgd from well field B via pipeline			206,000
10	Raising lake level and dredging; and 1.0 mgd from 2 wells along the lake	5	110	220,900
11	Raising lake level and dredging; and 4.2 mgd from field B via pipeline	5	50	231,800

TABLE 24. (Continued)

1	2	3	4	5
12	Raising lake level and dredging; and 4.1 mgd from well field A	3	110	262,600
13	Raising lake level and dredging; and 4.1 mgd from the Vermilion River	3	110	274,600
14	Raising lake level and dredging; and 4.1 mgd from well field B	3	HO	319,700
15	11.0 mgd from the Wabash River			491,000

Note: Annual cost is based on the 2010 demand.

TABLE 25. Investment Costs for Various Alternatives

(Annual and investment costs in thousands of July 1976 dollars)

Rank of alternative	Annual cost	Investment cost	Remarks
A. Alternatives for meeting SWS demand curve (Table 22)			
1	69.4	656.4	Initial about 50%
2	69.6	751.4	Initial 430.0; remaining 15 years later
3	70.4	673.5	Initial about 50%
4	74.1	806.3	Initial 430.0; remaining 15 years later
5	83.2	883.9	Initial 430.0; remaining 15 years later
6	85.4	755.0	Initial 699.0; remaining 20 years later
7	86.8	901.7	Initial 430.0; remaining 15 years later
8	89.7	742.6	
9	99.7	898.8	Initial 646.6; remaining 20 years later
10	123.7	1396.3	Initial 1340.1; remaining 25 years later
11	124.6	850.0	Initial 570.0; 140.0 each at 10 and 20 years for a new dredge
12	140.6	1499.0	Initial about 70%
13	212.5	1250.0	Initial 690.0; 280.0 each at 10 and 20 years for two new dredges
14	374.9	3712.7	Initial about 90%
B. Alternatives for meeting HM&G demand curve (Table 23)			
1	100.0	920.6	Initial about 40%
2	131.6	1334.7	904.7 in 20 years; then 430.0
3	145.9	1313.0	874.7 in 20 years; then 438.3
4	162.6	1489.5	Initial 771.4; after 10 years 288.1; after 20 years 430.0

TABLE 25. (Continued)

Rank of alternative	Annual cost	Investment cost	Remarks
5	167.1	1544.4	Initial 771.4: after 10 years 343.0; after 20 years 430.0
6	193.1	1861.9	Initial 771.4: after 10 years 430.0; after 20 years 660.5
7	195.8	1530.8	Initial 570.0; 140.0 each for a dredge after 10 and 20 years; 680.8 after 13 years
8	210.9	2070.0	Initial 771.4: 430.0 after 10 years; 868.6 after 20 years
9	215.6	2333.5	Initial 486.2; after 10 years 1847.3
10	215.6	1598.0	Initial 570.0; 140.0 each for a dredge after 10 and 20 years; 748.0 after 13 years
11	223.4	2428.9	Initial 1998.9; after 20 years 430.0
12	260.3	1716.4	Initial 710.0; 280.0 for 2 dredges each at 10 and 20 years; 446.4 after 20 years
13	282.9	1923.5	Initial 690.0; 280.0 for 2 dredges each at 10 and 20 years; 673.5 after 12 years
14	297.9	2005.0	Initial 690.0; 280.0 for 2 dredges each at 10 and 20 years; 755.0 after 12 years
15	302.2	1992.6	Initial 690.0; 280.0 for 2 dredges each at 10 and 20 years; 742.6 after 12 years
16	308.1	3314.3	Initial and first 20 years 2029.2; thereafter 1285.1
17	384.3	4248.8	Initial about 80%
18	560.9	5050.1	Initial about 90%

TABLE 25. (Continued)

Hank of alternative	Annual cost	Investment cost	Remarks
C. Alternatives for meeting Danville demand curve (Table 24)			
1	83.7	766.6	Initial about 40%
2	99.4	1140.1	Initial 710.1; later 430.0
3	107.4	994.6	Initial about 40%
4	125.3	1204.0	Initial 774.0; after 10 years 430.0
5	131.5	1201.4	Initial 771.4; after 10 years 430.0
6	175.4	1340.4	Initial 1060.4; 140.0 for a dredge each after 10 and 20 years
7	183.9	2016.3	Initial 1586.3; after 10 years 430.0
8	187.7	1427.1	Initial 1147.1; 140.0 for a dredge each after 10 and 20 years
9	206.0	2180.8	Initial about 80%
10	220.9	1326.2	Initial 710.0; 2 dredges after 10 years 280.0; 2 dredges and 2 wells after 20 years 336.2
11	231.8	2007.0	Initial and first 10 years 1727.0; 140.0 for a dredge each after 10 and 20 years
12	262.6	1733.0	Initial and first 10 years 1173.0; 280.0 for 2 dredges each after 10 and 20 years
13	274.6	1818.8	Initial and first 10 years 1258.8; 280.0 for 2 dredges each after 10 and 20 years
14	319.7	2394.8	Initial and first 10 years 1834.8; 280.0 for 2 dredges each after 10 and 20 years
15	491.0	4763.7	Initial about 95%

APPENDICES

APPENDIX I - REFERENCES

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APPENDIX II - COST OP TRANSPORTING WATER BY PIPELINES

Water from the Vermilion and Wabash Rivers and the groundwater aquifers near Danville and in northern Vermilion County needs to be transported from the source to Lake Vermilion or the city of Danville. The transport of water is achieved by transmission pipelines and suitable pumping stations to overcome the frictional and other head losses. The size and scope of the transmission network may differ from one supply system to the other.

Six components of transmission cost (Singh, 1971; Singh et al, 1972) are: 1) pipeline construction cost, 2) pipeline maintenance cost, 3) easement cost, 4) pumping station cost, 5) pumping cost, and 6) pumping station OM&R cost. The cost functions for these components were adjusted to July 1976 dollars with the Handy-Whitman Indexes. The interest rate for calculating the capital recovery factor, CRF, is taken as 8 percent.

To keep corrosion troubles to a minimum, all pipes whether cast iron, ductile iron, or steel should have a cement lining. The useful life of these pipes is 100 years or more, but a 50-year useful life has been assumed for computing the cost recovery factor. Pumping stations and pumps will be amortized over a useful life of 30 years.

Pipeline Construction Cost, C_l . This covers the cost of pipe, transportation, installation, valves and other appurtenances that are an integral part of a transmission line. The cost C_l in July 1976 dollars is given by

$$C_l = 4510\ D^{1.2}L$$

in which D is the inside diameter of the pipe in inches and L is the length of pipeline under consideration in miles.

Pipeline Maintenance Cost, C_2. The major portion of this annual cost

$$C_2 = 21\ D\ L$$

is for repairing any leaks or breaks in the pipeline.

Easement Cost, C_3. The pipeline is usually laid in the right-of-way of state or county roads, highways, or rail lines. A 15-foot wide permanent easement and a 30-foot wide construction easement will generally be needed for laying a pipeline. The easement cost of $1700 per mile (Singh, 1971) in 1964 dollars has been adjusted to 1976 dollars using Index Numbers of Illinois Farmland Values.

$$C_3 = 6250\ L$$

Pumping Station Cost, C_4. This includes the cost of pumping equipment and the station. Installed horsepower, P., of the pumping equipment is obtained from

$$P_i = 0.1756 \; Q \; H \; J/E$$

in which Q is in mgd, H denotes the total head including friction head and bend loss and any static head difference between the two ends of the pipeline, J represents the firming factor or the standby factor, and E is the overall efficiency at peak load and associated H. The value of J is obtained from Koenig (1964):

$Q \leq 2.0$	$J = 2.08 - 0.18\,Q;\ J \not> 2.0$
$2.0 < Q \leq 5.0$	$J = 1.967 - 0.123\,Q$
$5.0 < Q \leq 10.0$	$J = 1.42 - 0.014\,Q$
$10.0 < Q \leq 20.0$	$J = 1.30 - 0.002\,Q$

It is assumed that the pumping station will boost the pressure a maximum of 300 feet of water. If the total head, H, exceeds this limit, two or more pumping stations will be needed. The cost of pumping stations, C_4, in July 1976 dollars is expressed by

$$C_4 = 43{,}200\ (H/_{300}) + 325\ P_i$$

Pumping Cost, C_5. The annual cost of energy depends on the horsepower actually expended (varying with the varying pumpage demand) integrated over the year and the prevailing energy rate schedule. Considering uniform pumpage demand, the total energy consumption in kwh per year is given by

$$\text{Total kwh per year} = k\ Q\ (H_O + H_S)\ t$$

in which Q is in mgd pumped over the fraction of the year, t; H_o and H_s are the head loss in friction and the static head, respectively; and k is a factor for yielding the result in kwh. The value of k is calculated from

$$k = (0.1337 \times 365.21 \times 62.1 \times 0.7157 \times 10^6)\ /\ (550 \times 3600 \times E_a)$$

in which E_a is the overall efficiency during the pumping over the year. The kwh per year is converted to annual pumpage cost, C_5, by the present rate schedule. An average rate of 2.5¢ per kwh is assumed.

Pumping Station OM&R Cost, C6. The cost includes oiling, painting, routine checking, servicing, and repairs to or renewal of worn-out parts

$$C_6 = 2500\ (H/300) + 20\ P_i^{1.05}$$

The total annual cost of transporting water, TCT, through a given pipeline, is

$$TCT = (C_1 + C_3\ (CRP)_{50} + C_4\ (CRF)_{30} + C_2 + C_5 + C_6$$

The computer program based on the methodology described for the cost of transporting water calculates the optimum pipe diameter, installed horsepower, and the total annual cost of transporting Q mgd water.

Asbestos-Cement Pipe. Economies can be achieved by using asbestos-cement pipes designed to withstand moderate internal hydrostatic design pressure, (operating pressure plus surge pressure) x safety factor, and external crush design load. Such pipes can be used for transporting groundwater from the well fields in northern Vermilion County because the static head balances most of the friction head for optimal size pipelines and the route passes along the highways through rather sparsely inhabited areas. These pipes are somewhat smoother than ductile iron, steel, or cast iron pipes. The pipeline cost, C_1 is obtained from the following

$$C_1 = 4510\ (D^{1.2} - y)\ L$$

in which the value of y for asbestos-cement pipe diameters varying from 8 to 36 inches are:

D, inches	8	10	12	14	16	18	20	24	27	30	36
y	3.0	3.8	4.6	5.4	6.2	7.1	7.9	9.5	10.8	12.0	14.5

APPENDIX III - COST OP WELLS AND PUMPS

The cost of constructing a well depends on the type of aquifer, the need for a well screen and/or gravel pack, and the diameter and depth of a well. The diameter of a well depends on the expected well capacity and the size of the pump required. Well diameters for various pumping rates or well capacities (Smith, 1961) used in Illinois are given below.

Pumping rate (gpm)	Diameter of well (inches)
125	6
300	8
600	10
1200	12

For intermediate pumping capacities, the next larger diameter is used.

The cost of a pump includes the pump and motor, their installation, electrical wiring, meters, and connections, etc. The two types of pumping installations in use are: the vertical turbine pump and the submersible turbine pump. The choice of one or the other depends on the preferences of the engineering consultant, well driller, and the municipal authorities who are guided by their past experience.

The useful life of a well in a sand and gravel aquifer can be taken as 25 years. Well pumps are assumed to have a useful life of 10 years with full-time pumping and 25 years with 20% time pumping with normal operation and maintenance. At an 8 percent interest rate, the capital recovery factor, CRF, equals 0.0937 and 0.1490 for 25 and 10 years, respectively.

Cost of Wells. Wells for various alternatives of supplementing water supply to Danville will be drilled in sand and gravel aquifers occurring in the drift or buried valleys. In July 1976 dollars, the cost of a gravel-packed well, WC, is

$$\begin{aligned} WC &= 4600 + 290\,D + 5.8\,dD && \text{rotary drilling, well capacity} > 0.5 \text{ mgd} \\ &= 1.6(4600 + 290D + 5.8\,dD) && \text{reverse rotary drilling, higher capacity wells} \end{aligned}$$

in which D is the well screen diameter in Inches (6 to 12 Inches) and d is the well depth in feet. A 6-inch thick annular gravel-pack is assumed in the above equation. Wells will be drilled by reverse rotary process in northern Vermilion County well fields A, B, C, and D.

Cost of Pumps. Vertical turbine pumps (Singh and Adams, 1977) will be used for the capacity and head requirements for wells in the proposed alternative well fields. The cost of these well pumps in July 1976 dollars can be estimated from

$$PC = 2000 + 16.4 \ Q^{0.45} \ H^{0.64}$$

in which PC is the pump cost and Q and H are the pump capacity in gpm and total dynamic head in feet, respectively.

Annual Electric Charges: The total kwh per year (Singh, 1971) is calculated from

$$kwh = 1147.6 \ QHt/E$$

in which Q is the pumping rate in mgd, t is the average fraction of time pumped, and E is the average overall efficiency during the year for pumping groundwater, taken as 0.6. The annual electric charges are obtained by multiplying kwh by 0.025; the rate per kwh is taken as 2.5*.

Annual OM&R Cost: According to Singh et al (1972), the annual OM&R cost for a well field in terms of July 1976 dollars is

$$\text{Annual OM\&R cost} = 230 + 175 \ N_{wt}$$

in which N_{wt} is the total number of wells in a well field, including the standby well.

Annual Cost of Raw Groundwater: The well and pump costs are reduced to annual costs by multiplying with appropriate capital recovery factors. Added to these annual costs are the annual electrical charges and OM&R costs. Total annual cost for pumping at an average of 20% of time equals:

$$(WC + PC) \ (CRF)_{25} + \text{Annual electric charges} + \text{Annual OM\&R}$$