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HYDROLOGIC AND HYDRAULIC INVESTIGATION OF THE CULVERT #4 WATERSHED ON THE HENNEPIN CANAL, BUREAU COUNTY, ILLINOIS

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INTRODUCTION

The Illinois Department of Conservation (DOC) has been primarily responsible for the operation and maintenance of the Hennepin Canal since 1970, when the State of Illinois assumed full ownership of the canal from the U. S. Army Corps of Engineers (COE). Since acquiring the canal, the Department of Conservation has been faced with numerous problems along the canal associated with levee breaks and siltation of culverts designed to carry drainage water under the canal to nearby streams. This report summarizes the results of a study of one segment of the canal.

Study Area

The study area is part of the Hennepin Canal Parkway, which is described in an Illinois Department of Conservation leaflet (1978) as follows: "Hennepin Canal Parkway is a linear recreation area -- 104.5 miles long and from 380 feet to one mile wide. Shaped like a T, the Parkway is located in Rock Island, Bureau, Henry, Lee and Whiteside counties and includes approximately 3,000 acres of land and over 3,500 acres of water. Its northernmost area is Lake Sinnissippi, a 2,400 acre pool in the Rock River at Sterling-Rock Falls. From Lake Sinnissippi, the Parkway extends almost due south 29.3 miles along the feeder canal. Just north of Interstate 80, about midway between Routes 78 and 88, the feeder meets the main canal. From this point the Parkway runs southwest 46.9 miles to the Mississippi River near Rock Island and southeast 28.4 miles to the Illinois River near the town of Hennepin. At its southeastern end, it encompasses Lake DePue."

History of the Hennepin Canal*

In 1834 the idea of the Hennepin Canal was conceived. It was proposed to be an extension of the Illinois and Michigan Canal, which was a canal version of Interstate 80. The canal was to be located in a natural pass for a canal, since there was a depression along the entire proposed route with high land on either side. Due to a lack of support and funds, however, the canal was not built, and in 1860 the Chicago, Rock Island and Pacific Railroad was constructed over the original canal route. But the idea for a canal was not abandoned. The first survey for the proposed canal was performed in 1866, and the first federal survey was made in 1870. From 1886 through 1889 Congress repeatedly considered the proposed canal, but no construction appropriation was made. The main objections to constructing the canal were centered around the fact that without enlargement of the Illinois and Michigan Canal, the Hennepin Canal would be of only local importance. To counter this objection by stressing the national significance of the canal, the name was officially changed in 1889 from the Hennepin Canal to the Illinois and Mississippi Canal, although it is still commonly referred to as the Hennepin Canal.

*The materials in this section come primarily from articles written by M. Yeater (1978).

In 1890, with the passage of the River and Harbor Act, Congress appropriated money for purchase of the right-of-way and for construction of the canal. The Hennepin Canal marked the beginning of the use of concrete in canal construction in the United States.

As completed in 1907, the canal ascended 196 feet from the Illinois River to the summit level in a distance of 18 miles and descended 93 feet to the Mississippi River in 46 miles. The total length of the main line was 75 miles, and the feeder canal was 29.3 miles long. The canal was 52 feet wide at its bottom and 80 feet at the water line; the depth of the water was 7 feet. Where the canal was carried entirely above the natural surface of the ground, the banks were 10 feet wide on the top. There were 33 locks on the canal: 1 at the head of the feeder and 32 on the mainline. All the locks were 170 feet long and 35 feet wide and were capable of passing barges with at least 140-foot lengths, 34-foot beams, and gross tonnages of 840.

The Hennepin Canal was operated by the United States Army Corps of Engineers (C0E) as a navigable waterway from October 24, 1907, until July 1, 1951. The canal was used very little but during its operation the C0E employed at least 50 (and often more) full-time workers throughout the year to operate and maintain the canal. The total cost of operations from 1908 through 1951 was $6,900,653, or an average of $160,480 per year. The high cost was due in part to a series of circumstances involving the farmers along the banks of the canal. Construction of the canal had drained the swampland adjacent to the canal right-of-way, and during the period when the canal had been constructed but had not yet been watered, farmers began reclaiming and cultivating the very fertile land. When water was turned into the canal, the under-draining ceased and the land reverted to swamp. Reluctant to forego profits, which they had been collecting for as long as 13 years, many landowners blamed canal seepage for the wet conditions on the land adjacent to the canal and demanded that the COE construct drainage systems. Despite the lack of validity of the farmers' contentions that the canal was seeping, the COE built the drainage ditches at a cost of about half a million dollars. During 1951-1970 the canal was not used as a navigable waterway because of excessive maintenance costs.

On August 1, 1970, the State of Illinois assumed full ownership of the canal. The state has operated the canal, primarily under the jurisdiction of the Department of Conservation (DOC), as a recreational corridor affording a variety of water and trail related outdoor recreational opportunities.

Study Objectives

The portions of the Hennepin Canal that were constructed aboveground had a tendency to block the natural drainage. To solve this problem, culverts were placed beneath the canal, restoring natural drainage. Since the canal has come under the control of the Department of Conservation it has been brought to their attention that several culverts have silted up frequently and subsequently have blocked the surface drainage of the upstream land.

This report addresses one such case, Culvert #4. The location map of the Culvert #4 watershed is shown in figure 1. This project was initiated to provide the State of Illinois with more detailed information and to aid the DOC in resolving the drainage problems at Culvert #4.

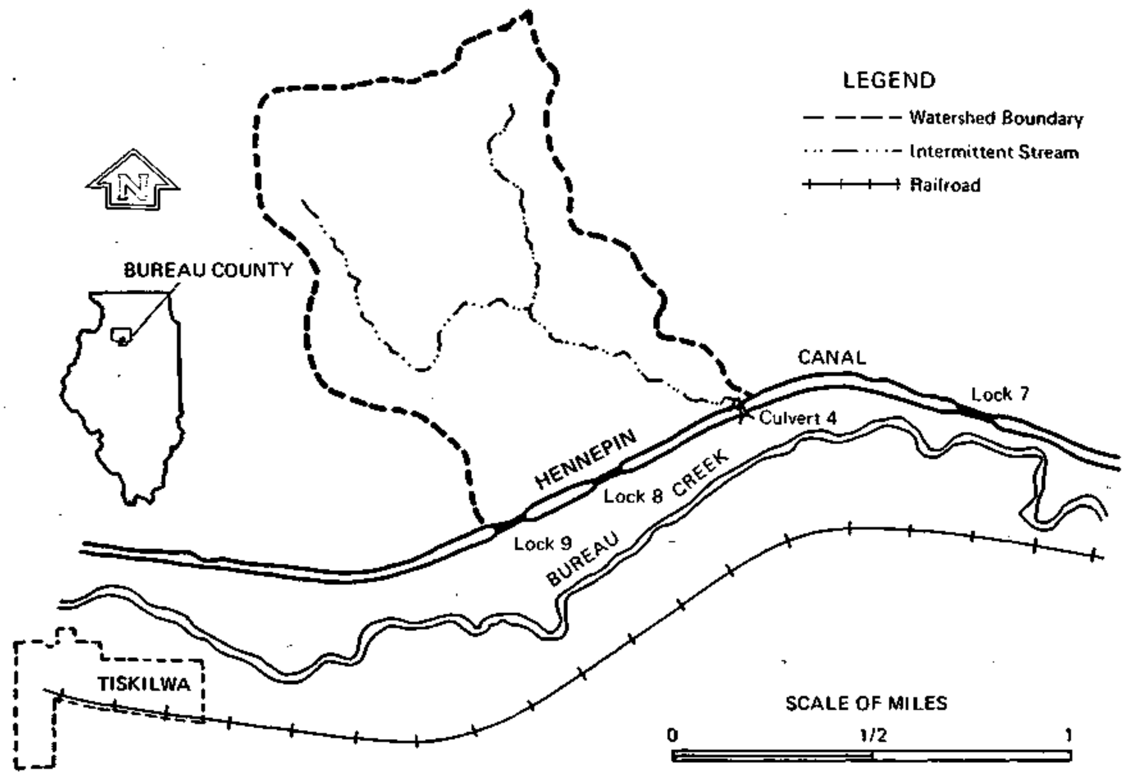

Figure 1. Location of the Culvert #4 watershed

The main objectives of the study were as follows:

1) Perform a needed survey in the vicinity of Culvert #4.
2) Do a hydrologic analysis of nearby Bureau Creek.
3) Analyze precipitation.
4) Investigate the rainfall-runoff processes in the area.
5) Perform a soil loss assessment as it relates to land use.
6) Draw conclusions from the results and recommend alternative drainage plans.

Acknowledgments

This project was conducted under the administrative guidance of Stanley A. Changnon, Jr., Chief, Illinois State Water Survey, and Michael L. Terstriep, Head, and Nani G. Bhowmik, Assistant Head, Surface Water Section.

The authors wish to thank William P. Fitzpatrick of the Surface Water Section and Barry Klepp, an undergraduate student employee of the Water Survey, for their help in the data collection. Maureen Kwolek, a graduate student at the University of Illinois, computed the soil loss from the watershed. John Brother, Jr., William Motherway, and Linda Riggin prepared the illustrations. The camera ready copy was prepared by Kathleen Brown and Pamela Lovett, and Gail Taylor edited the report.

DATA COLLECTION AND ANALYSIS

Field Surveys

Culvert #4 was visited on two separate occasions, July 20-22, 1982, and June 13-16, 1983. On each visit a field survey was conducted to obtain accurate and up-to-date information. Available topographic information consisted of 1901 and 1930 maps prepared by the U.S. Army Corps of Engineers and a 1966 U.S. Geological Survey map.

The portion of Bureau Creek that is near Culvert #4 may be seen in the aerial photograph in figure 2. Historical aerial photos show that Bureau Creek has meandered a great deal and changed its location quite often. Between 1958 and. 1964, the stream was relocated from its original location and straightened. The locations and elevations for Bureau Creek that are described in this report were collected in June 1983.

Figure 3 depicts the surveyed area near Culvert #4. The location of the survey on the south side of the canal was determined in part on the basis of the 1930 COE topographic survey (COE, 1937), which is shown as figure 4. The natural drainage pattern from the area adjacent to Lock 8 appears to have been to the east. In the 1983 survey most of the elevations on the south side of the canal were found to be higher than those on the north side. Since most of the land in this area was disturbed during the construction of the canal and was further disturbed by farmers after the construction of the canal, it is difficult to determine the original natural drainage patterns. A minor depression on the north side of the canal was located, which had standing water in it. This survey showed that the land is fairly flat with only minor undulations, which indicates that surface drainage will be poor and some areas probably will drain by seepage. The banks of Bureau Creek are

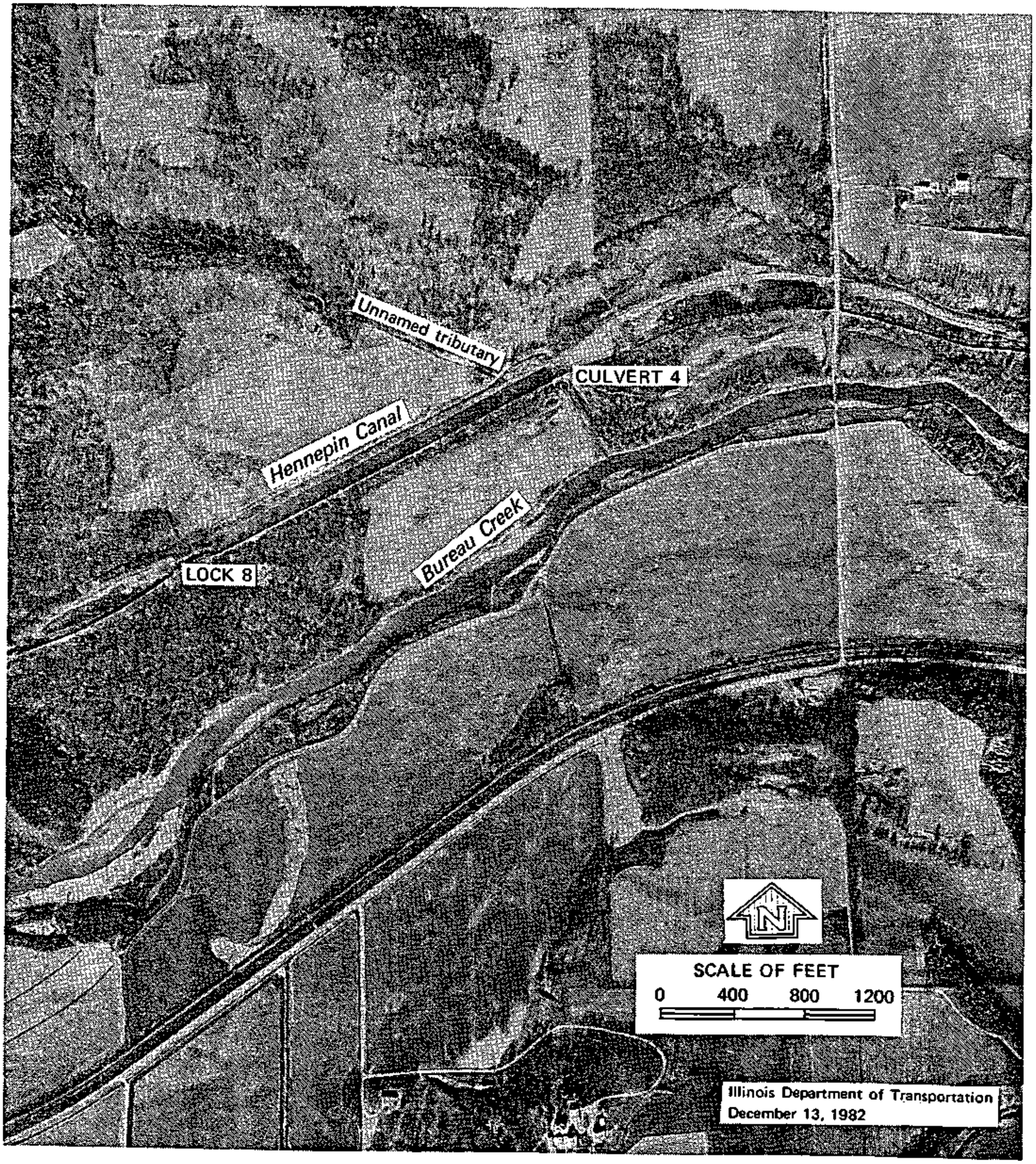

Figure 2. Aerial photograph of the area around Culvert #4

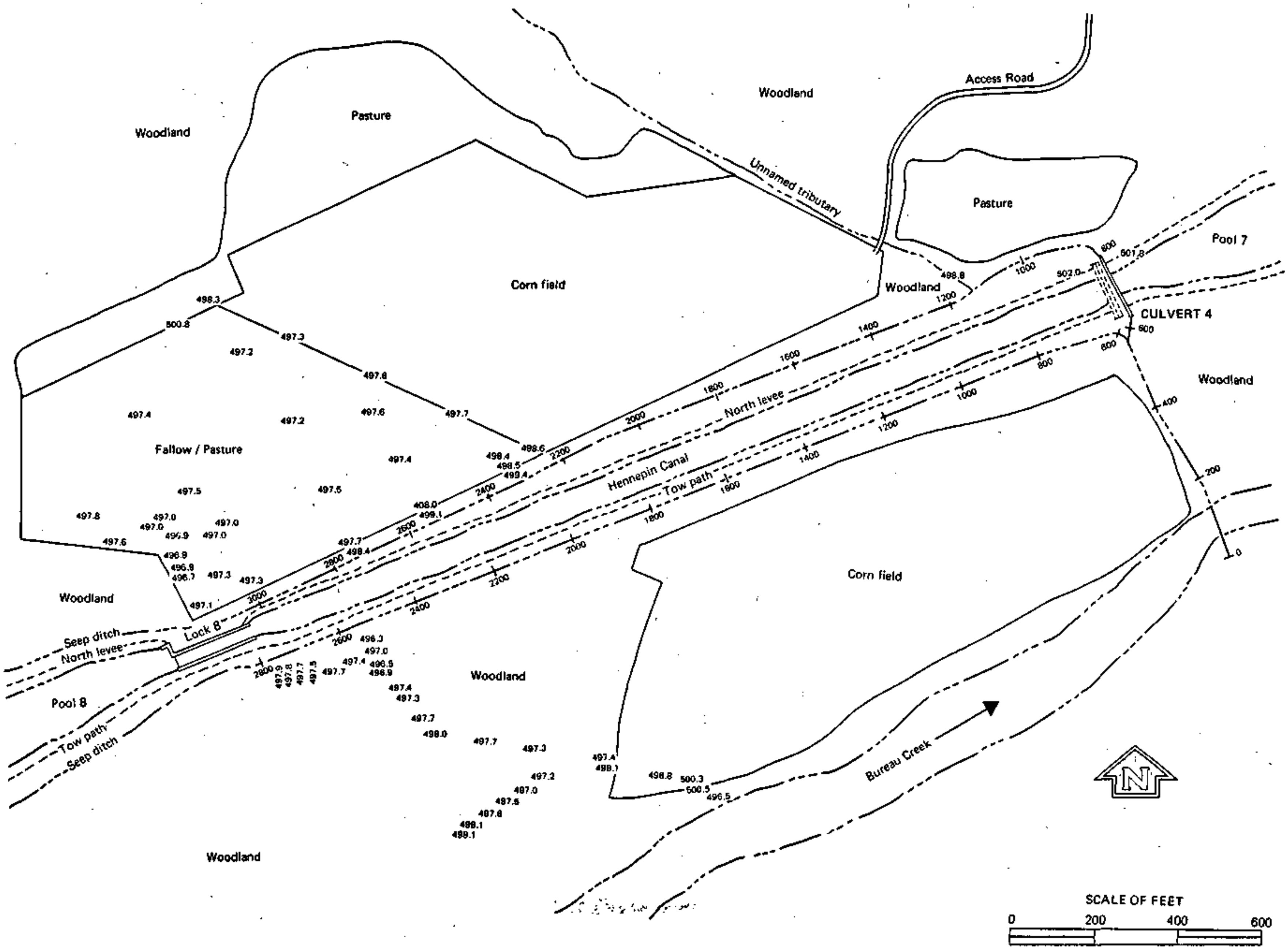

Figure 3. Point elevations around Culvert #4, surveyed June 1983

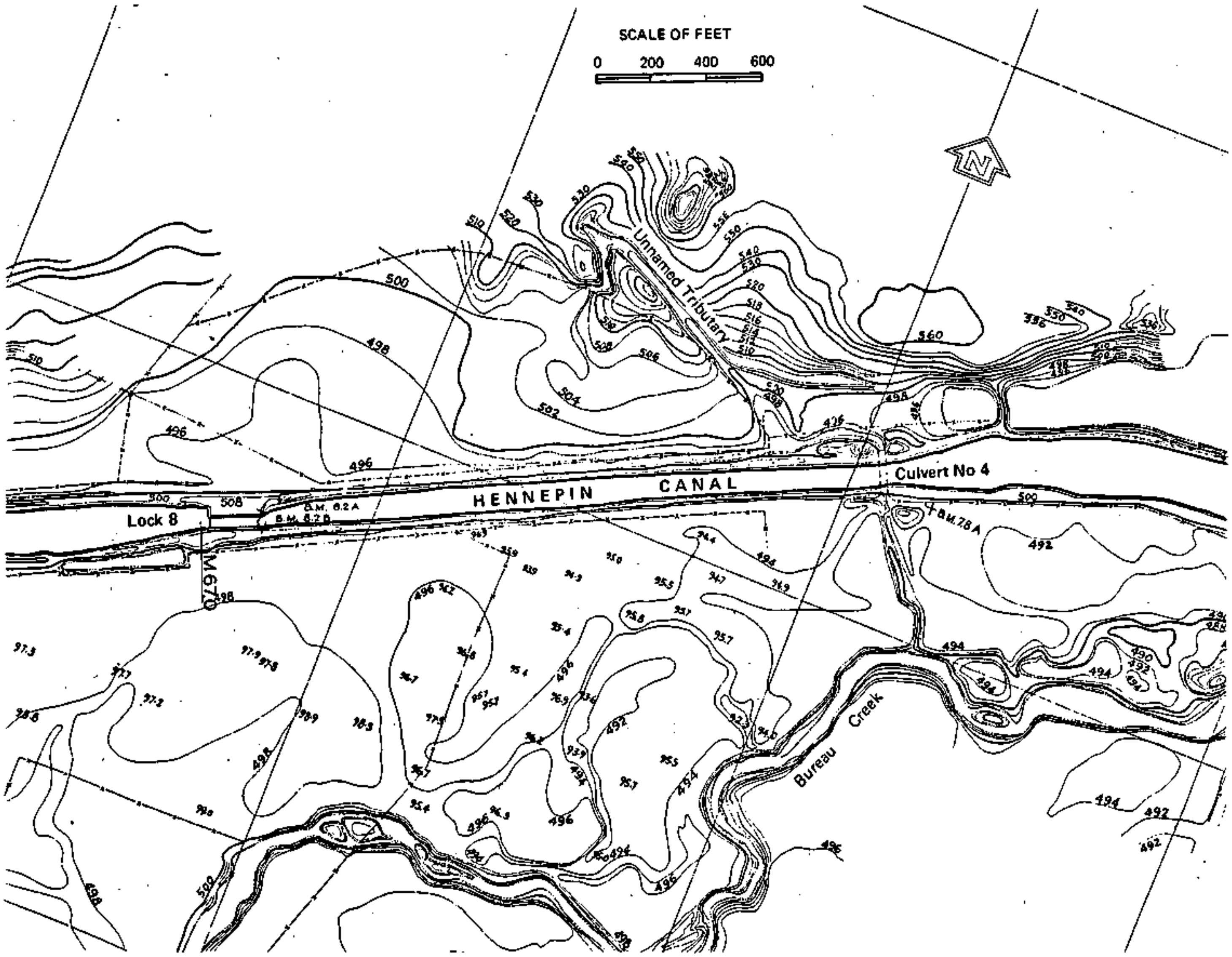

Figure 4. Topography around Culvert #4, based on a 1930 survey (U.S. Army Corps of Engineers, 1937)

quite steep, rising 10 feet above the normal water surface. The land surface on the floodplain decreases in elevation from the banks to the canal. Natural levees occur because during times of high flow the coarse sediment material quickly drops out of suspension close to the channel. Further away from the channel in the floodplain the velocity of the flow is low, allowing the finer materials to drop out of suspension (Simons and Senturk, 1977). As a result the elevation decreases away from the channel on the floodplain.

Figure 5 shows a profile across Bureau Creek through Culvert #4, including the north seep ditch. From this figure, it can be seen that the two original cast iron (CI) culverts were placed quite low compared to the bed elevation of Bureau Creek. In fact the water surface elevation on Bureau Creek on June 13, 1983, was 1 foot above the inverts of the cast iron culverts. There are approximately 2 feet of sediment above the top of the cast iron culverts. No information on the hydraulic design of the culverts is available. The inverts of the culverts were probably controlled by the canal bottom; that is, the tops of the culverts were placed just below the bottom of the canal.

Sediment has filled the replacement corrugated metal pipe culvert (CMP) to half of its depth. The water surface elevation in Bureau Creek on July 22, 1982, partially filled the corrugated metal pipe, which reduced the carrying capacity of the pipe.

As shown in figure 5, there is a high mound where the unnamed tributary enters the seep ditch from the north. The seep ditch continues to run southwesterly towards Lock 8. As can be seen in figure 5, the

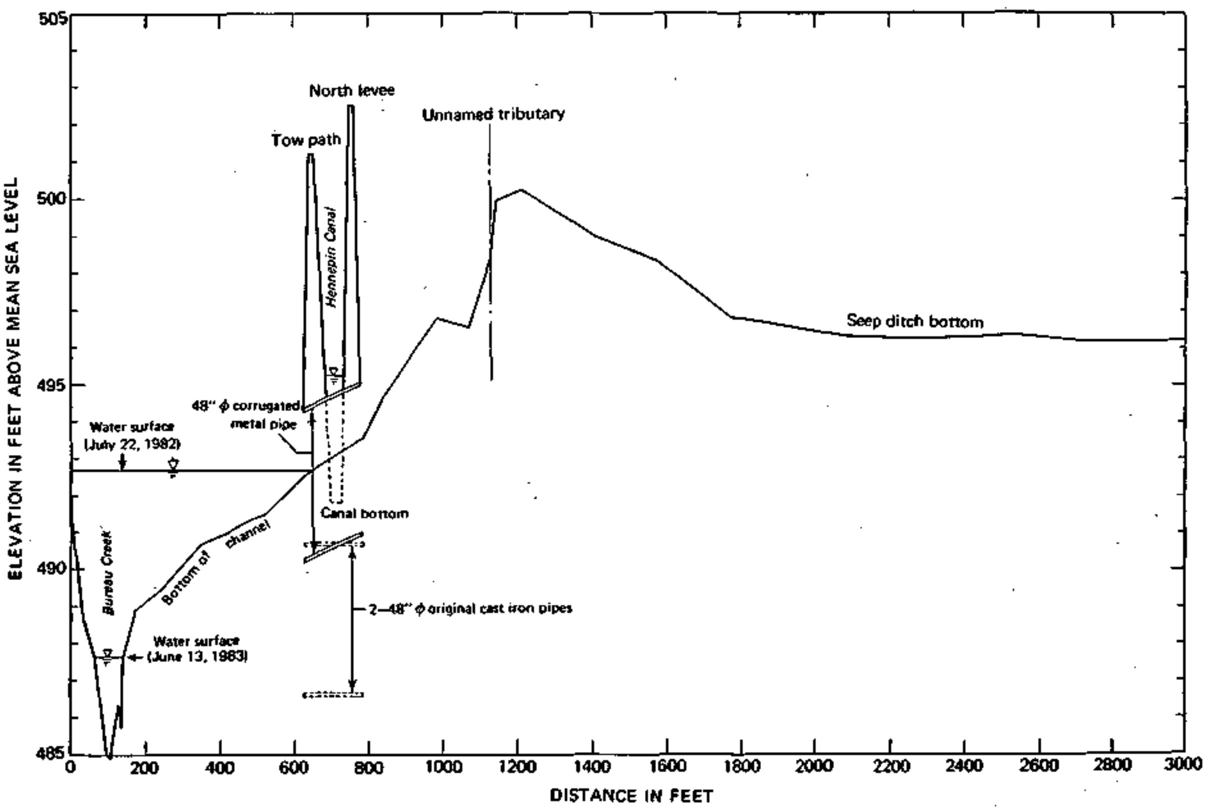

Figure 5. Profile from Bureau Creek to Look 8 along the north seep ditch

seep ditch cannot convey surface drainage from the area north of Lock 8 to Culvert #4 due to the mound in the seep ditch. More detail on this subject may be found in the section on soil loss.

Figure 6 shows a profile across Bureau Creek along the south ditch and then along the seep ditch to Lock 8. The overall slope in the south seep ditch from Lock 8 to Culvert #4 is fairly constant but contains a number of undulations, although the overall slope is negative. The downstream control of the south seep ditch is a culvert installed under a field access road, which conveys the south seep ditch flow. There was some organic deposition observed within this culvert, but it contained no sediment. A profile of the tow path is also presented in figure 6.

Precipitation Data

The available precipitation data were related to return intervals presented in a rainfall atlas. The frequency of occurrence of precipitation events with various return intervals was obtained as were the total monthly precipitation and departures from normal.

Methodology

Rather than performing a regression analysis for precipitation, it was decided that it would be more useful to use a rainfall frequency atlas (ISWS, 1970) to determine recurrence intervals. The Illinois State Water Survey (ISWS) atlas was chosen over the HYDRO-35 and TP-40 atlases upon the recommendation of professionals from the Water Survey. The utilization of a rainfall atlas avoids any error due to spatial distribution, gage malfunction, missing data, etc. The NOAA rain gage

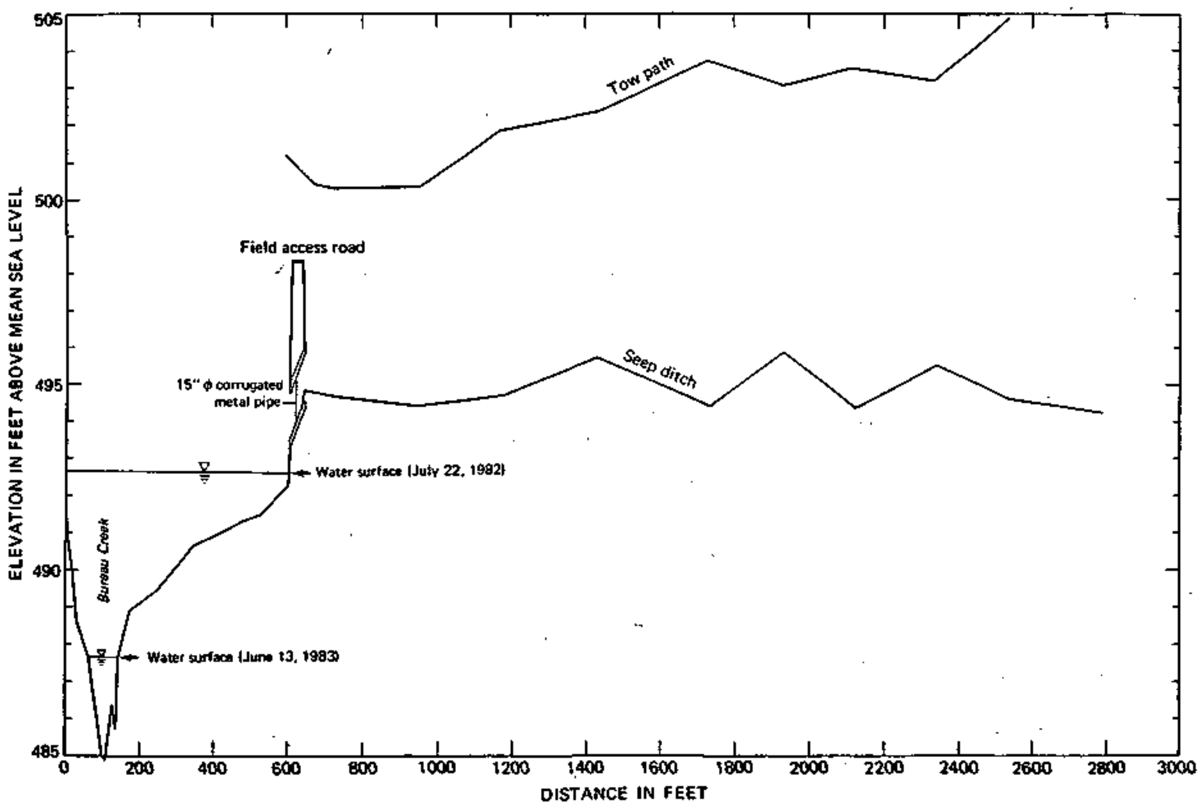

Figure 6. Profile from Bureau Creek to Look 8 along the south seep ditch

nearest to the Culvert #4 watershed is at Tiskilwa. This gage is read every 24 hours. Where data were missing, data from the NOAA rain gage at the Hennepin Power Plant were used.

Results

Table 1 depicts the total monthly precipitation and departures from normal at Tiskilwa in 1978-1982. The annual departures from normal ranged from -1.74 inches in 1978 to +8.77 inches in 1979. The monthly maximum negative departure was 3.37 inches while the maximum positive departure was 8.09 inches, which occurred in consecutive months in 1979.

For this report, a water year is considered to start on October 1 and continue through September 30 of the following year. The water year system was designed to roughly follow the growing season and to begin and end during a period of generally low flow.

Table 2 presents the frequency of occurrence of several ranges of daily precipitation in 1970-1982, along with the average return intervals. It is unlikely that an intense rainfall will occur entirely during fixed observation times. Analyses similar to this type give underestimates of true maximum amounts for the specified durations. The daily precipitation is not necessarily the maximum 24-hour precipitation. Thus, the daily precipitation is usually increased by 13 percent to obtain the maximum daily precipitation (Linsley et al., 1975). This was not done for table 2.

The information that may be obtained from table 2 includes the number of occurrences of various amounts of rainfall at Tiskilwa in 1970-1982 that correspond to various return intervals. There were nine occurrences of precipitation that exceeded a return period of 2 years. One extreme amount was 5.72 inches which was observed on August 18, 1979.

Table 1. Total Monthly Precipitation and Departures from Normal for Tiskilwa, Illinois, 1978-1982

Month		1978	1979	1980	1981	1982
October	Precipitation	3.83	1.61	1.56	1.99	1.60
	Departure	1.01	-1.21	-1.26	-.83	-1.22
November	Precipitation	1.96	3.01	2.54	.61	1.80
	Departure	.07	1.12	.65	-1.28	.09
December	Precipitation	.91	3.05	2.73	2.60	.96
	Departure	-.83	1.31	.99	.86	-.78
January	Precipitation	.53	2.25	. .36	.16	1.81
	Departure	-1.12	.60	-1.29	-1.49	.16
February	Precipitation	.71	.86	1.58	2.95	.96
	Departure	-.67	-.52	.20	1.57	-.42
March	Precipitation	.98	4.02	1.92	.39	4.12
	Departure	-1.66	1.38	-.72	-2.25	1.48
April	Precipitation	4.55	5.28	3.23	7.90	3.59*
	Departure	.49	1.22	-.83	3.84	-.47
May	Precipitation	6.19	2.99	1.45	2.98	3.58
	Departure	2.24	-.96	-2.50	-.97	-.37
June	Precipitation	4.08	. 5.23	5.13	6.80	3.31
	Departure	.08	1.23	1.13	2.80	-.69
July	Precipitation	2.74	3.68	2.11	5.33	8.60
	Departure	-1.06	-.12	-1.69	1.53	4.80
August	Precipitation	1.84	11.26	7.55	7.82	2.26*
	Departure	-1.33	8.09	4.38	4.65	-.91
September	Precipitation	4.41	0	4.43-	3.03	1.29
	Departure	1.04	-3.37	1.06	-.34	-2.08
Annual	Precipitation	32.73	43.24	34.59	42.56	33.88
	Departure	-1.74	8.77	0.12	8.09	-0.41

*Hennepin Power Plant data used

Table 2. Precipitation at the NOAA Tiskilwa Raingage for the Period 1970-1982

Average return interval (years)	Daily precipitation (inches)	Number of occurrences
	1.00 - 1.99	97
< 2	2.00 - 2.59	20
2 - 5	2.60 - 3.59	6.
5 - 10	3.60 - 4.39	2
10 - 25	1.40 - 5.69	0
25 - 50	5.70 - 7.19	1

One difficulty with using the Tiskilwa rainfall data was that the rainfall data were collected at 24-hour intervals and no smaller duration data can be estimated. The closest station at which hourly data were obtained was Kewanee, 28 miles west of Tiskilwa. Short-duration high-intensity rainfall may be quite localized, but the rainfall data from Kewanee should provide an indication of the magnitude of precipitation on the Culvert #4 watershed.

Table 3 presents the average recurrence intervals for various durations of precipitation at Kewanee. Water Year 1978 had three precipitation events with recurrence intervals greater than 2 years; 1979 had two such events; 1980 had one; and 1981 had two. In 1982 there were no precipitation events that were considered major events. ' The recurrence interval may change with the duration. For example on August 18, 1979, 1.2 inches of r'ain fell in 1 hour, which is considered a 2-year rain, but 2.1 inches fell in 2 hours, which places the rainfall in the 5-year rain category. So even though 1978 was the driest year for the 5-year period investigated, the year had the highest instantaneous flow in Bureau Creek. The temporal distribution is a more reliable indication of runoff potential than how much rain occurred during a year. The 1981 water year had 9.83 inches more rain than 1978, but as will be seen later (in table 6), the yearly runoff was higher in 1978 than in 1981. There are a number of factors that affect the amount of runoff, one of which is rainfall intensity. A short-duration high-intensity rain will cause more runoff than a rain of long duration and low intensity. Other factors that may vary over time are vegetation, infiltration rate, ice cover, and antecedent moisture condition. Detailed rainfall-runoff relationships will be discussed in a subsequent section.

Table 3. Average Recurrence Intervals at the NOAA Kewanee Precipitation Gage for the 1978-1982 Water Years

Date of event	Depth of precipitation (inches)	Depth of precip. required to be considered an event (inches)	Duration	Average recurrence interval (years)
October 25, 1977	1.7	1.5	3 hr	2
	1.8	1.8	6 hr	2
November 1, 1977	1.6	1.5	3 hr	2
	2.6	2.6	6 hr	5
	2.7	2.2	12 hr	2
	3.0	2.4	18 hr	2
	3.1	2.6	24 hr	2
	3.1	2.9	2 day	2
	3.1	3.1	3 day	2
May 13, 1978	2.4	2.2	12 hr	2
	2.7	2.4	18 hr	2
	3.0	2.6	24 hr	2
	3.5	2.9	2 day	2
	3.5	3.1	3 day	2
	3.5	3.5	5 day	2
August 18, 1979	1.2	1.2	1 hr	2
	2.1	2.0	2 hr	5
	2.4	2.2	3 hr	5
	2.8	2.6	6 hr	5
	2.8	2.2	12 hr	2
	2.8	2.4	18 hr	2
	3.3	2.6	24 hr	2
	3.6	2.9	2 day	2
	5.3	5.3	.3 day	10
	5.9	4.8	5 day	5
	5.9	5.8	10 day	5
August 20, 1979	1.5	1.4	2 hr	2
	1.7	1.5	3 hr	2
July 5, 1980	1.2	1.2	1 hr	2
	1.6	1.4	2 hr	2
	1.7	1.5	3 hr	2
August 5, 1981	5.8	5.8	10 day	10
August 14, 1981	2.1	2.0	1 hr	10
	2.3	2.0	2 hr	5
	2.4	2.2	3 hr	5
	2.4	1.8.	6 hr	2
	2.5	2.2	12 hr	2
	3.1	2.4	18 hr	2
	3.1	2.6	24 hr	2
	3.2	2.9	2 day	2
	3.2	3.1	3 day	2

Hydrologic Data for Bureau Creek

The hydrologic analysis consisted of an investigation of streamflow data within the Bureau Creek watershed. A flood-frequency analysis was performed on the basis of available data so that several flood return intervals might be obtained. These return intervals were obtained for the Bureau Creek basin at Culvert #4. Several backwater profiles were calculated from known stage-discharge relationships so these relationships might be obtained at Culvert #4. In the hydraulic analysis of Culvert #4 the water surface elevation of Bureau Creek will be referred to as the tailwater.

Flood Frequency

There are several stream flow gaging stations in the Big Bureau Creek basin. (Bureau Creek was renamed Big Bureau Creek in 1975, although its original name is still commonly used.) For this report four gages were used: Bureau Creek at Princeton, West Bureau Creek at Wyanet, East Bureau Creek near Bureau, and Bureau Creek at Bureau.. Their drainage areas are 196, 86.7, 99.0, and 485 square miles, respectively. The drainage area of Bureau Creek at Culvert #4 is 356 square miles. Since there is no stream gaging station at Culvert #4 the stream flow data from the other stations must be used to estimate flow in Bureau Creek at Culvert #4.

Flood-frequency analyses were performed for the four stations on the basis of the annual maximum series (the instantaneous maximum flow rates for each year). The series consist of annual maximums for the number of years under consideration. A flood-frequency relation defines the relation of flood-peak magnitude to exceedance probability or recurrence interval. Exceedance probability is the percentage chance that a flood peak of a given magnitude will be exceeded in any given year. Recurrence interval is the reciprocal of the exceedance probability multiplied by 100, and is the average time interval between occurrences of a flood peak of a given or greater magnitude. Probability describes only the likelihood of a random event occurring, and a flood magnitude of a given recurrence interval may actually be exceeded in a much shorter period of time, such as successive weeks or months (Curtis, 1977). Flood-frequency relations for gaging stations were defined on the basis of the U. S. Water Resources Council (1976) guidelines, which recommend the use of the log-Pearson Type III distribution and which outline procedures to fit observed annual peak data to the log-Pearson Type III distribution. A description of this theoretical distribution of floods may be found in most hydrology textbooks such as that by Linsley et al. (1975).

There were 11 years of data (1941-1951) for the gaging station on Bureau Creek at Bureau and 46 years of data (1936-1981) for the other three gaging stations. The drainage area of the gaging station at Bureau is larger than the Bureau Creek drainage at Culvert #4, while the other gaging stations possess smaller drainage basins. The discharge in Bureau Creek at Culvert #4 can be interpolated for various flood return periods obtained from a flood-frequency analysis performed on the flow rate data for the four drainage basins. The period of record (11 years) for the gaging station at Bureau must first be related to the period of record (46 years) for the other three gaging stations. The periods of record were related by performing a flood-frequency analysis on the data from all four gaging stations for the years 1941-1951. In addition, a flood-frequency analysis was performed on the data from the three gaging stations that had a period of record from 1936 through 1981. Eight return intervals were obtained for both the 11- and 46-year periods of record: 1.0526, 1.25, 2, 5, 10, 25, 50, and 100 years.

The results for the three gaging stations from the frequency analyses done on the 11- and 46-year records were compared for each return interval, and consistency was found between the two sets of data. Therefore, the data for the 11-year period (1941-1951) were plotted on a graph for all four gaging stations in terms of drainage area versus discharge. A smooth curve was then drawn through the points. This was done for each of the eight return intervals so that there were eight curves plotted. The size of the drainage area on Bureau Creek at Culvert #4 was used to obtain the discharge for the eight return intervals for that site.

Since consistency of the data among the three gaging stations was demonstrated for the 11- and 46-year periods of analysis, it was necessary to examine only one station. The gaging station on Bureau Creek at Princeton was selected, and the results from that gage were used to modify the discharges on Bureau Creek at Culvert #4. The results were modified by dividing the discharge at Culvert #4 by the discharge at Princeton for each of the eight return intervals derived from the data for 1941 through 1951. These ratios were then multiplied by the discharges obtained at Princeton for each return period so that eight discharges were obtained that corresponded to the return periods desired at Bureau Creek, Culvert #4.

Stage-Discharge Relationships

After the discharges were calculated, the corresponding stages at Culvert #4 for various return intervals were obtained. To accomplish this, the U.S. Army Corps of Engineers (1979) HEC-2 computer program was used to calculate the stage-discharge relationship. A stage-discharge relationship is available for Big Bureau Creek at the Illinois Route 29 bridge where a backwater computation was begun. On the basis of the stream cross-sectional data, a study by Stanley Consultants (1975), USGS topographic maps, and a field survey, eight water surface profiles were calculated that corresponded to the eight flood return intervals mentioned earlier. The results are shown in table 4.

Also shown in table 4 are the water surface elevations with and without an agricultural levee on the south side of Bureau Creek. Levees can increase the stage for a particular flood event since the floodplain storage and conveyance are eliminated. Therefore, the same amount of water may be higher with levees than without levees. A number of agricultural levees are present in this area. The dates of construction of these levees are unknown since no permits were obtained, so the effects of the levees from 1978-1982 are unknown.

Results

Figure 7 depicts the elevation of the water surface of Bureau Creek at Culvert #4 corresponding to various discharges. Also indicated in this figure are several reference points of interest, floods of various return intervals, and the instantaneous maximum flow rate for the years 1970-1982. All but the smallest flood flows in Bureau Creek are above

Table 4. Water Surface Elevations at Culvert #4
for Various Return Intervals in Bureau Creek

Return interval (years)	Flow (cfs)	Water surface elevation (ft) With levee	Water surface elevation (ft) Without levee
1.0526	94.5	491.24	491.24
1.25	2,480	493.45	493.44
2	5,680	495.22	495.01
5	10,910	497.37	496.80
10	14,300	498.56	497.87
25	18,310	499.97	499.31
50	21,160	500.72	500.08
100	23,460	501.30	500.70

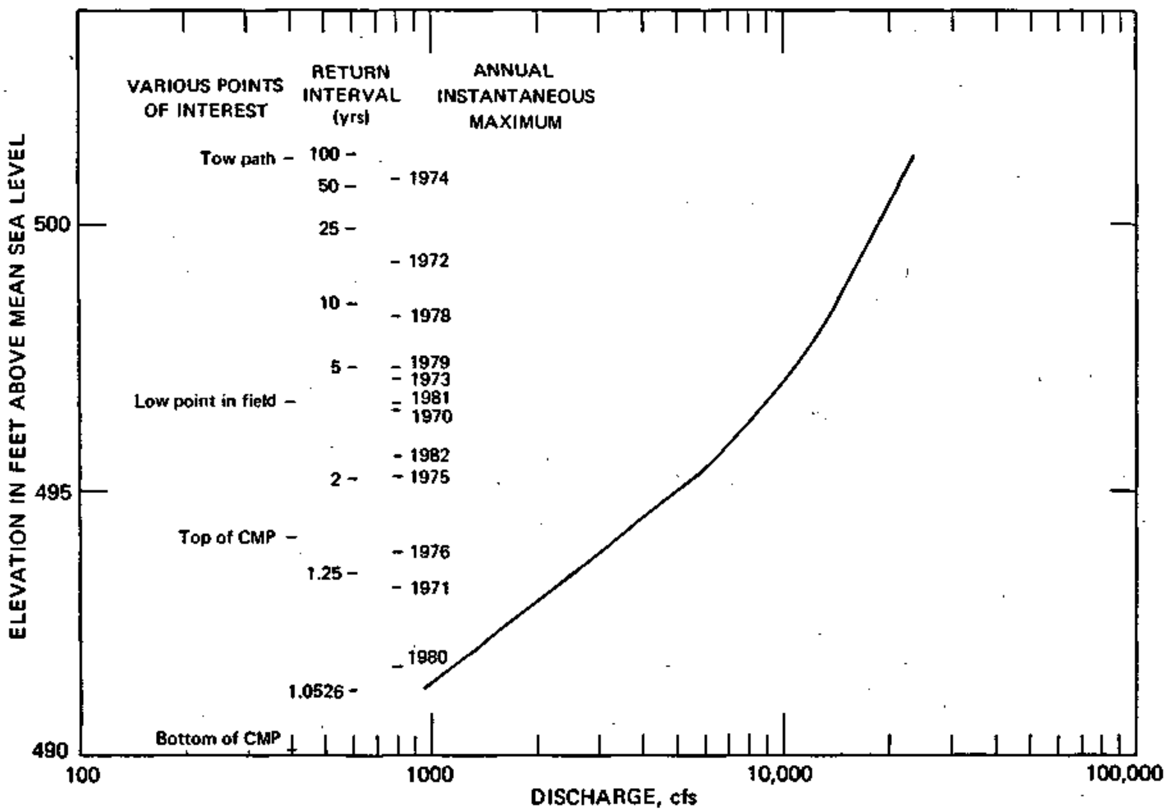

Figure 7. Discharge and water surface elevations in Bureau Creek at Culvert #4

the invert of the new corrugated metal pipe culvert. All flood flows above 2-year recurrence intervals submerge the crown of the corrugated metal pipe. Flows above the 5-year return period would inundate the low portion of the field north of Lock 8. During the period 1970-1982 there were five occurrences in which the instantaneous annual maximimum flood elevations were higher than that of the field. There were twelve occurrences in which the annual maximums reduced the conveyance capacity of the culvert since the water surface elevations were above the corrugated metal pipe invert.

Since the flows at the Princeton gage are directly related to those at Culvert #4, the Princeton gage will be used when ranking is involved. Table 5 presents the ten largest instantaneous maximum flows between the years 1936 through 1982 as well as the rankings of all the flows from the period 1970-1982. Of the top ten maximum flows five occurred in the period 1970-1982. Table 6 presents the annual mean flows and their rankings. The four years with the highest mean flow occurred between 1970 and 1981. Six of the ten years with the most runoff occurred between the years 1970 and 1981.

Table 7 presents the days on which the average daily flow exceeded • various return intervals between the years 1978-1982. The flows presented in the table existed for 24 hours, so the corresponding stages also were maintained for 24 hours. The average daily flow is an average flow rate for one day which is equal to the sum of the observed flow rates for the same day divided by the number of observations.

Table 8 presents the highest mean values and rankings for various numbers of consecutive days in 1937-1981. Although in 1974 the instantaneous flow rate reached its maximum value for the period of record

Table 5. Largest Instantaneous Maximum Flows at Bureau Creek at Princeton, Illinois, 1936-1982

Year	Discharge, cfs	Rank
1974	12,500	1
1938	11,800	2
1972	10,000	3
1978	8,620	4
1945	8,370	5
1937	8,300	6
1966	8,020	7
1979	7,160	8
1969	6,980	9
1973	6,870	10
1981	6,070	14
1970	5,990	15
1982	4,690	21
1975	4,240	26
1976	2,460	35
1971	1,930	41
1980	1,070	44
1977	554	46

Table 6. Annual Mean Flows and Rankings at Princeton Gage, 1937-1981

Year	Flow, cfs	Ranking
1973	301	1
1979	259	2
1974	255	3
1970	222	4
1960	219	5
1972	192	6
1966	190	7
1962	189	8
1955	188	9
1978	185	10
1981	171	14
1975	145	21
1971	107	27
1980	93	29
1976	91	31
1977	15	45

Table 7. Average Daily Flows at Culvert #4, 1978-1982

Return period (years)	Flow (cfs)
1.0526-1.25	945-2,479 (981-2,097)*
	1978: 5/14, 6/27, 7/3, 7/22
	1979: 3/18, 3/24, 3/25, 3/29, 3/31, 4/1, 4/2, 4/12, 4/27, 4/28, 8/20
	1980: none
	1981: 2/22, 2/23, 4/14, 6/15, 6/16, 8/16
	1982: 2/21, 2/22, 3/13, 3/16, 3/19, 3/20, 6/15, 6/16, 7/8, 7/13, 7/22, 7/23
1.25-2	2,480-5,679 (2,098-4,216)*
	1978: 7/2
	1979: 3/21, 3/22, 3/23, 3/30, 4/26
	1980: none
	1981: 6/13, 6/14, 8/5
	1982: 7/7
2-5	5,680-10,909 (447-7,208)*
	1978: 6/26
	1979: 3/19, 3/20
	1980: none
	1981: none
	1982: none

*Flow at Princeton gage

Table 8. Highest Mean Values and Rankings for Various Numbers of Consecutive Days, 1937-1981 (Year Ending September 30)

(Table provided by the U.S. Geological Survey)

DISCHARGE-(CFS)
MEAN
BIG BUREAU CREEK AT PRINCETON, IL

YEAR	1		3		7		15		30		60		90		120		183	
1937	7370.00*	2**	2980.00	9	1420.00	17	798.00	19	509.00	24	432.00	15	417.00	9	359.00	11	289.00	10
1938	7600.00	1	3620.00	4	1690.00	9	952.00	12	534.00	21	319.00	28	301.00	22	262.00	22	256.00	21
1939	6150.00	7	2390.00	17	1110.00	23	569.00	32	393.00	31	305.00	30	280.00	29	238.00	29	217.00	24
1940	2480.00	27	1530.00	27	800.00	32	415.00	35	242.00	39	131.00	39	94.00	42	75.00	43	53.00	43
1941	2280.00	28	1560.00	26	779.00	34	404.00	37	251.00	38	231.00	35	194.00	36	168.00	36	138.00	36
1942	1690.00	34	1100.00	36	982.00	26	745.00	23	563.00	18	393.00	20	305.00	21	253.00	24	271.00	15
1943	2660.00	24	1970.00	22	1290.00	20	902.00	15	647.00	12	445.00	12	354.00	18	333.00	16	270.00	16
1944	4480.00	15	2330.00	21	1230.00	22	657.00	28	396.00	30	338.00	26	292.00	24	249.00	25	170.00	30
1945	5360.00	10	3200.00	7	1530.00	14	783.00	21	474.00	26	418.00	17	316.00	20	254.00	23	188.00	26
1946	4840.00	11	3090.00	8	1960.00	7	1040.00	11	564.00	17	342.00	25	345.00	19	279.00	19	243.00	22
1947	2740.00	23	1720.00	24	994.00	25	609.00	30	590.00	15	431.00	16	391.00	13	319.00	17	234.00	23
1948	2810.00	22	1760.00	23	1270.00	21	730.00	24	527.00	22	310.00	29	228.00	32	185.00	33	144.00	33
1949	3620.00	19	2950.00	10	2390.00	2	1550.00	2	838.00	6	656.00	4	474.00	6	390.00	6	270.00	17
1950	4730.00	12	2790.00	13	1570.00	12	926.00	14	733.00	9	492.00	8	393.00	12	366.00	10	283.00	11
1951	4560.00	14	2360.00	18	1410.00	18	873.00	16	597.00	14	414.00	18	403.00	11	353.00	12	335.00	7
1952	2170.00	29	1520.00	28	929.00	28	622.00	17	585.00	16	436.00	14	363.00	16	337.00	15	282.00	12
1953	847.00	39	459.00	42	313.00	41	181.00	42	129.00	43	107.00	42	102.00	41	99.00	39	84.00	39
1954	1450.00	36	1160.00	35	841.00	31	493.00	33	283.00	36	220.00	37	209.00	35	170.00	34	141.00	34
1955	4640.00	13	2500.00	16	1410.00	19	796.00	20	452.00	29	328.00	27	289.00	25	269.00	20	262.00	18
1956	736.00	40	453.00	40	283.00	42	255.00	40	169.00	40	112.00	41	106.00	39	92.00	41	80.00	40
1957	914.00	38	705.00	38	490.00	38	327.00	39	312.00	33	257.00	33	210.00	33	188.00	32	138.00	35
1958	1970.00	32	1170.00	34	755.00	35	687.00	26	471.00	27	389.00	21	280.00	30	226.00	31	202.00	25
1959	1600.00	35	1340.00	31	949.00	27	694.00	25	555.00	20	370.00	24	299.00	23	248.00	26	169.00	32
1960	4390.00	16	3650.00	3	2040.00	5	1160.00	8	938.00	2	653.00	5	525.00	4	424.00	5	374.00	4
1961	417.00	44	316.00	44	219.00	44	148.00	44	115.00	44	100.00	44	82.00	44	68.00	44	52.00	44
1962	4080.00	17	2770.00	14	1800.00	8	1260.00	4	828.00	7	580.00	7	450.00	7	369.00	9	291.00	9
1963	2500.00	26	1370.00	30	781.00	33	463.00	34	262.00	37	154.00	38	122.00	38	94.00	40	64.00	41
1964	733.00	41	452.00	41	264.00	43	177.00	43	130.00	42	102.00	43	85.00	43	82.00	42	62.00	42
1965	2000.00	31	1270.00	32	748.00	36	683.00	27	486.00	25	273.00	32	264.00	31	236.00	30	170.00	31
1966	6240.00	6	3590.00	5	2110.00	4	1250.00	5	780.00	8	488.00	10	360.00	17	373.00	8	320.00	8
1967	1890.00	33	1490.00	29	884.00	29	636.00	29	467.00	28	372.00	23	287.00	27	245.00	28	176.00	28
1968	421.00	43	331.00	43	314.00	40	216.00	41	138.00	41	117.00	40	105.00	40	102.00	38	97.00	38
1969	4000.00	18	2830.00	12	1560.00	13	1250.00	6	660.00	11	377.00	22	284.00	28	267.00	21	259.00	19
1970	3290.00	21	2370.00	19	1440.00	15	935.00	13	849.00	3	594.00	6	505.00	5	425.00	4	363.00	5
1971	1400.00	37	767.00	37	572.00	37	375.00	38	290.00	34	250.00	34	209.00	34	168.00	35	178.00	27
1972	7230.00	3	3590.00	6	1980.00	6	1270.00	3	723.00	10	474.00	11	429.00	8	387.00	7	341.00	6
1973	5640.00	5	2860.00	11	1640.00	10	1090.00	9	839.00	5	721.00	2	611.00	2	523.00	2	444.00	3
1974	6560.00	4	3970.00	2	2160.00	3	1240.00	7	848.00	4	708.00	3	531.00	3	473.00	3	447.00	2
1975	2530.00	25	1640.00	25	1020.00	24	758.00	22	557.00	19	491.00	9	405.00	10	343.00	13	276.00	13

Table 8. Concluded

DISCHARGE-(CFS)
MEAN
BIG BUREAU CREEK AT PRINCETON, IL

YEAR	1		3		7		15		30		60		90		120		183	
1976	2100.00	30	1230.00	33	847.00	30	575.00	31	380.00	32	303.00	31	289.00	26	248.00	27	173.00	29
1977	320.00	45	217.00	45	131.00	45	73.00	45	41.00	45	33.00	45	28.00	45	25.00	45	23.00	45
1978	6030.00	8	2570.00	15	1630.00	11	1040.00	10	635.00	13	437.00	13	374.00	14	342.00	14	258.00	20
1979	6550.00	5	5140.00	1	3600.00	1	2420.00	1	1570.00	1	1100.00	1	819.00	1	650.00	1	463.00	1
1980	690.00	42	503.00	39	483.00	39	413.00	36	287.00	35	228.00	36	183.00	37	156.00	37	136.00	37
1981	3430.00	20	2350.00	20	1420.00	16	812.00	18	520.00	23	397.00	19	368.00	15	310.00	18	272.00	14

* Highest mean value
** Ranking

(table 5), the year 1938 had the maximum 24-hour flow, and 1979 had the highest 3, 7, 15, 30, 60, 90, 120, and 183 consecutive days of highest mean flow. The year 1979 had the second highest yearly mean.

Table 9 gives information on average daily discharge for the years 1937-1981. For each year, it shows the number of days on which the flow fell in each of 34 ranges of values, or "classes." The discharge values for each class are printed at the end of the table. For example class "0" represents discharge values ranging from 0.00 to 0.10 cfs. The numbers in the body of the table represent the number of days on which the flow was in the range corresponding to the class. In the list of classes at the end of the table, "Value" is the corresponding discharge value; "Total" is the total number of days that are in the respective class for the period 1937-1981; "Accum" is the accumulated days from the highest class to the lowest; and "Perct" represents the percent of time that class is represented. Obviously class 0 is represented 100 percent of the time because there must always be 0 to 0.10 cfs flowing in Bureau Creek.

Table 9 may be used to determine if the amount of discharge in Bureau Creek is above normal. There are 45 years or 16,436 days of data. The period 1978 through 1981 represents 4 years of data or 1161 days, which is 8.9 percent of 45 years. The flows for these 4 years are all contained within class 12 or above and account for 10.7 percent of all the days in this class. Similarly, they account for 12.6 percent of the days in class 28, 19.2 percent of the days in class 31, and 20.0 percent of the days in class 33 and above. For the period from 1978 to 1981, the flow in Big Bureau Creek was above normal most of the time.

Table 9. Duration Table of Daily Discharge Values, 1937-1981 (Year Ending September 30)

(Table provided by the U.S. Geological Survey)

DISCHARGE-(CFS)
MEAN
BIG BUREAU CREEK AT PRINCETON, IL

CLASS*	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34
YEAR													NUMBER OF DAYS IN CLASS																						
1937								1	3	8	26	14	6	3	7	8	6	19	17	34	40	44	31	34	27	20	6	5	3	2				1	
1938								4	3	1	6	34	44	6	8	6	7	5	19	47	33	34	24	30	18	13	10	5	5	1		1			1
1939												10	4	17	13	15	34	45	37	31	35	48	31	19	8	6	7	1		1	1		1	1	
1940	31			1	2	13	28	9	12	21	9	47	24	26	18	32	29	22	14	7	5	4	4	1	3	1		1	1		1				
1941							10	21	25	31	14	10	12	13	15	14	25	18	13	35	36	30	14	12	5	4	3		1	1	3				
1942												3	7	6	11	12	12	12	10	23	65	51	48	31	28	20	9	10	4	3					
1943									9	9	20	24	10	9	7	14	14	24	17	38	33	40	19	19	17	14	13	10	2		3				
1944								2	24	41	30	78	25	7	11	5	8	6	6	12	9	26	20	26	12	10	4	1	1	1			1		
1945							2	9	29	21	11	55	15	6	10	9	18	8	18	13	41	36	18	16	11	9	3	4	1			1		1	
1946											7	12	10	14	10	6	8	7	24	57	58	56	28	19	16	17	5	5	1	2	1	1	1		
1947										11	28	22	28	40	33	14	14	17	15	13	18	11	27	23	15	14	11	6	1	3		1			
1948								1	7	22	30	35	25	36	18	20	23	30	26	20	15	19	13	6	7	3	3	1	2	1	1	2			
1949								9	14	12	23	35	35	17	16	13	16	10	31	23	23	25	13	7	8	11	4	7	3	5	2	3			
1950									24	39	26	14	7	11	4	11	7	11	29	30	36	23	17	15	16	16	15	6	2	5			1		
1951											22	63	4	4	6	17	9	7	10	23	23	49	46	28	15	13	7	5	7	3	1	2	1		
1952								3	3	8	5	6	6	5	12	7	9	5	12	21	70	57	38	28	21	19	9	14	6	1	1				
1953							4	2	7	22	45	37	26	27	8	12	15	27	24	36	29	19	8	5	4	5	2	1							
1954	24				8		11	8	5	17	24	14	8	3	5	22	48	20	25	24	21	19	19	14	11	5	5	1	2	2					
1955									1	4	9	6	4	4	3	15	9	13	11	10	71	54	49	36	26	16	9	6	3	3	1	1	1		
1956									12	8	2	3	2	20	40	38	67	52	28	25	19	16	12	6	7	5	2	2							
1957									4	44	48	19	14	24	22	10	19	11	17	10	14	38	28	16	15	5	5	2							
1958										10	10	6	18	19	17	15	30	30	29	35	41	28	21	18	14	7	7	3	2	4	1				
1959								6	13	34	23	26	31	25	26	22	22	7	5	10	26	26	19	11	10	8	5	5	2	3					
1960													6	7	9	14	17	11	16	25	55	50	35	29	34	32	9	7	2	3	2	1	2		
1961									5	5	9	41	17	39	36	46	35	21	21	24	21	20	13	9	1	2									
1962											4	22	11	6	7	2	8	10	17	10	33	89	44	30	24	24	10	8		3	1	1	1		
1963									1	27	63	43	72	25	15	19	26	15	16	11	6	10	4	1	4	4	1		1		1				
1964								1	32	40	46	23	39	20	11	17	18	22	22	21	20	10	10	7	3	3		1							
1965							2	5	10	23	14	13	19	36	25	11	10	17	21	22	18	22	18	21	18	20	6	4	5	3	2				
1966										13	19	10	4	7	7	10	6	6	8	33	53	67	44	19	23	16	5	6	4	1		2	1	1	
1967										12	26	46	34	24	16	25	21	17	10	12	26	17	16	27	16	6	8	3	1	2					
1968									6	3	9	21	14	12	5	4	14	33	48	38	57	53	23	16	8	2									
1969										4	14	28	13	15	3	33	22	24	18	26	20	28	35	24	19	15	13	1	5	1	2	1	1		
1970												2	3	1	1	1	17	48	48	39	28	24	24	28	35	26	18	10	4	2	4	2			
1971									5	16	5	10	6	17	11	18	6	37	25	36	33	44	29	31	16	8	7	3	1	1					
1972									3	19	33	11	12	29	12	7	11	8	8	4	18	37	31	33	33	26	15	10	3	1	1			1	
1973													2	14	17	9	6	5	3	13	10	44	40	38	62	46	30	15	3	4	2	1		1	
1974															15	24	33	17	25	16	12	36	38	43	41	24	15	14	2	5	1	3		1	
1975												3	29	11	48	47	26	10	14	11	19	27	21	26	27	24	13	3	4	1	1				
1976									17	19	7	13	56	24	15	39	15	9	15	11	20	26	24	19	15	10	5	4	2		1				

Table 9. Concluded

DISCHARGE-(CFS)
MEAN
BIG BUREAU CREEK AT PRINCETON, IL

CLASS*	0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34
YEAR													NUMBER OF DAYS IN CLASS																						
1977								1	25	74	47	22	23	33	31	27	29	12	16	9	6	5	2	2	1										
1978													2	8	7	7	22	38	19	25	33	47	44	34	36	19	14	4	3	1	1			1	
1979														7	4	4	16	21	42	85	41	32	20	12	17	13	18	15	7	3	2	3	2	1	
1980														40	32	9	15	53	35	26	37	33	38	14	12	13	8	1							
1981																	22	33	30	61	52	50	30	27	21	18	7	5	6			3			

*CLASS	VALUE	TOTAL	ACCUM	PERCT	CLASS	VALUE	TOTAL	ACCUM	PERCT	CLASS	VALUE	TOTAL	ACCUM	PERCT
0	0.00	55	16436	100.00	12	4.2	727	13706	83.39	24	250.0	780	2196	13.36
1	0.10	0	16381	99.67	13	5.9	717	12979	78.97	25	350.0	592	1416	8.62
2	0.14	0	16381	99.67	14	8.3	647	12262	74.60	26	490.0	346	824	5.01
3	0.20	1	16381	99.67	15	12.0	710	11615	70.67	27	680.0	215	478	2.91
4	0.28	10	16380	99.66	16	16.0	844	10905	66.35	28	960.0	102	263	1.60
5	0.39	13	16370	99.60	17	23.0	873	10061	61.21	29	1300.0	72	161	0.98
6	0.55	57	16357	99.52	18	32.0	914	9188	55.90	30	1900.0	37	89	0.54
7	0.77	82	16300	99.17	19	45.0	1135	8274	50.34	31	2700.0	29	52	0.32
8	1.1	299	16218	98.67	20	63.0	1379	7139	43.44	32	3700.0	13	23	0.14
9	1.5	618	15919	96.85	21	89.0	1524	5760	35.05	33	5200.0	9	10	0.06
10	2.1	714	15301	93.09	22	130.0	1130	4236	25.77	34	7400.0	1	1	0.01
11	3.0	681	14587	88.75	23	180.0	910	3106	18.90					

VALUE EXCEEDED 'P' PERCENT OF TIME
P95 = 1.8
P90 = 2.7
P75 = 8.1
P70 = 12.5
P50 = 45.9
P25 = 136.4
P10 = 321.9

Note: VALUE = discharge value for class; TOTAL = total number of days in class; ACCUM = accumulated days from highest to lowest class; PERCT = percent of time class is represented

Hydrologic and Hydraulic Data for Culvert #4

The ability of Culvert #4 under the Hennepin Canal to convey flow was investigated. A computer program was used to route various frequencies of rainfall through the culverts with water surface elevations in Bureau Creek corresponding to several flood recurrence intervals. Based on the calculated hydrographs from the Culvert #4 watershed the flow carrying capacity of the culverts was determined, along with the extent of the ponding upstream of the culvert.

From the field surveys conducted as part of this study the condition of the culverts was assessed. There are presently three culverts under the canal at Culvert #4. There are two 48-inch-diameter cast iron pipes which were placed under the canal during its construction. Little design information was available concerning these culverts. The controlling criterion for the placement of the 48-inch cast iron pipes was the bottom of the canal. The tops of these pipes are about 1 foot below the present bottom elevation of the canal, as shown in figure 5. As a result of the elevation of the bottom of the canal, the inverts or bottoms of the culverts were placed at an elevation of 486.66 feet msl. This places the inverts partially under water by approximately 1 foot. These two culverts are therefore highly susceptible to sedimentation. During the surveys the 48-inch cast iron pipes were not visible. The downstream crowns, or tops of the pipes, were not seen since they were under 2 feet of sediment, although the headwall can be seen to the left (west) of the corrugated metal pipe. The upstream ends of these pipes could not be located since even the headwall was buried by sediment. For the hydraulic and hydrologic analyses, it was assumed that the two 48-inch cast iron pipes would not convey any part of the runoff from the watershed which they presently do not convey. It was also assumed that the seep ditch was free of sediment deposition in the area of the confluence with the unnamed tributary.

In 1977, to remedy the ineffectiveness of the original culverts, the DOC installed a 48-inch corrugated metal pipe above the original culverts. During the field inspections both the upstream and downstream ends of this culvert were visible, as seen in figures 8 and 9. There was deposition in this pipe also, with approximately one-half the cross-sectional area filled with sediment. For the hydraulic analysis it was assumed that the entire cross section was available to convey the runoff from the watershed to Culvert #4.

The measured length of the 48-inch corrugated metal pipe was 159.5 feet, and it had a slope of 0.0048 foot per foot. The culvert was found to be hydraulically long for the whole range of flows so that the control section was located at the outlet at all times. A culvert which is hydraulically long flows full due to friction losses within the pipe so flow is governed at the outlet. Therefore, downstream factors such as the culvert geometry and tailwater, as well as the headwater, govern the quantity of flow that may pass through the culvert.

The results from the precipitation and hydrologic analyses of Bureau Creek (discussed previously) were used, in the computer routing of the flow through the culvert.

Methodology

The analysis follows the general principle of hydrologic routing, or more specifically, reservoir routing. Reservoir routing depends on inflow, outflow, and storage. For this report, the storage component is the volume of ponding behind the culvert.

Figure 8. Inlet of the replacement 48-inch-diameter corrugated metal pipe

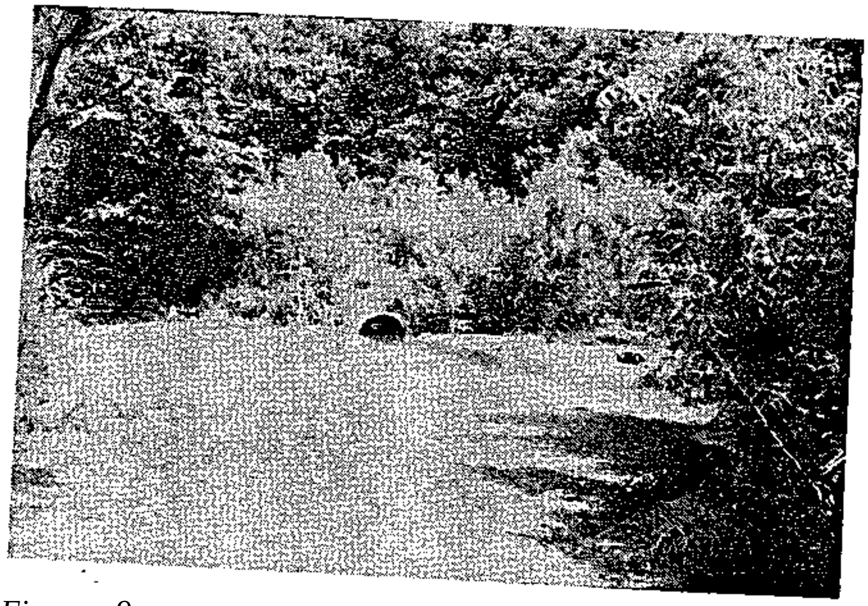

Figure 9. Outlet of the replacement 48-inch-diameter corrugated metal pipe

The computer program that was used to route the flows will be described briefly here; a more detailed description has been given by Makowski (1981).

The program attempts to take into account any changes in the inflow hydrograph caused by ponding. Ponding affects the inflow hydrograph because as the ponding depth increases, the pond occupies more surface area. This increased surface area causes a decrease in the overland and channel flow length and a corresponding decrease in the time of concentration (taken to be the time required for the runoff to flow from the remotest part of the drainage basin to the point of design). In the Rational Method (Chow, 1964), used in this program, the intensity of rainfall is related to the time to peak so the intensity of rainfall changes with the decrease in the time of concentration.

The intensity of rainfall was determined from an intensity-duration-frequency curve obtained from the Illinois State Water Survey (1970). A rainfall duration equal to the time of concentration was assumed. When rainfall duration and storm frequency are known, the intensity of rainfall may be determined. Therefore, since the time of concentration decreases as a result of ponding, the intensity of rainfall would increase since these two parameters are related in the Rational Method. The larger rainfall produces a greater amount of runoff, which has the effect of altering the inflow hydrograph.

The Rational Method is based on the following equation (Chow, 1964):

$$Q_p = CiA$$

where: Q_p = peak discharge in cfs
C = runoff coefficient
i = uniform rainfall intensity in inches per hour
A = area of the drainage basin in acres

The value for the duration of the storm intensity, i, is equal to the time of concentration for the watershed. The runoff coefficient, C, is determined from a combination of land use and watershed slope. Since the Culvert #4 watershed has a variety of land uses, a composite value was used. Four types of land uses were identified for the watershed. To aid in runoff calculations, the watershed was divided into four subwatersheds. The drainage area of Culvert #4 is 560.6 acres. This excludes the ponds just east of Culvert #4 and north of the canal. This area drains east under the county road.

The Rational Method is used to estimate the peak runoff rate. This method stems from the concept that a steady, uniform intensity of rainfall applied to a drainage basin will cause runoff to reach its maximum rate when all parts of the basin are contributing to the outflow at the point of investigation. The point of investigation for this report is the culvert. To produce the maximum flow, the design storm must have a duration greater than or equal to the time of concentration.

The major disadvantage to the Rational Method is that only the peak discharge is calculated; no runoff hydrograph is generated. This may be aided by using the peak flow and the time of concentration (time to peak) to generate an artificial hydrograph.

The volume of the ponding behind the culvert for a particular stage was computed by planimetering the area within successive contours on a topographic map and then multiplying this area by the contour interval. A stage-volume relationship was developed that was used to convert volume into a corresponding stage or elevation.

The initial values of inflow, outflow, and storage in the ponding area were known, as was the inflow after one time increment, which was obtained from the Rational Method. These flows were then averaged, and the inflow volume was found by multiplying the flow rate by the time increment. This average inflow, together with the geometry of the ponding area, was used to find the increases in depth and, therefore, the average depth in the ponding area. This averaged depth is the average head on the culvert during the specified time interval.

The average flow out through the culvert of the watershed was computed on the basis of this averaged depth, and this average flow out of the basin was then converted into a volume. This volume leaving the watershed was subtracted from the ponding volume previously found after one time increment. From this resulting ponding volume a depth in the pond was found using the stage-volume relationship which corresponds to the depth after one time increment. This process was repeated until the outflow was zero. Time averaging was used so that steady flow might be assumed.

The water surface elevation of Bureau Creek will be referred to as the tailwater of the culvert. Since the response time of the watershed above the culvert is so much faster than that of Bureau Creek, the tailwater may be assumed to be constant during a storm event on the Culvert #4 watershed. Upstream of the culvert, the water surface elevation is designated as the headwater.

There are four possible flow conditions that may exist within a culvert that is hydraulically long, which result from a combination of high and low headwater and tailwater. A description of the flow conditions may be found in Bodhaine (1969) or Chow (1959).

In the analysis, water from Bureau Creek was not allowed to flow upstream into the culvert. Any ponding of water at the upstream end of the culvert was caused by rainfall *over* the watershed. Flow through the culvert, therefore, occurred only when the elevation of the upstream ponding exceeded that of Bureau Creek. If the ponding level did not rise above the level in Bureau Creek, the outflow would be zero. Obviously if Bureau Creek was allowed to flow upstream in the culvert, the maximum ponding depth would increase.

Results

Fifty-four cases were investigated. These cases involved six different rain return intervals and nine levels in Bureau Creek corresponding to various flood recurrence intervals. The results are summarized in tables 10 through 12.

Table 10 presents the results for various rain return intervals with no tailwater from Bureau Creek. No tailwater exists at the 48-inch corrugated metal pipe when the water surface elevation in Bureau Creek is below 490.24 feet msl. This elevation corresponds to a flood recurrence interval of less than 1 year. The peak inflows vary from 284 cfs for a 2-year rain to 790 cfs for a 100-year rain. For comparison, if uniform flow in the culvert were to prevail, the culvert would have a maximum capacity of 60 cfs with the water surface at 93 percent of the diameter of the culvert. Greater capacity may be obtained by increasing the upstream head on the culvert, but this results in flooding of the nearby lower areas. The inflow time to peak is about 30 minutes. The time needed for the culvert to drain the entire watershed of the rain would range from 2.7 hours to 5.4 hours depending on the magnitude of the

Table 10. Summary of Peak Inflow, Time to Peak, and Time to Drain Watershed with No Tailwater, for Culvert #4 Watershed, Hennepin Canal

Rain return interval (yrs)	Peak inflow (cfs)	Time to peak (hrs)	Time to drain watershed (hrs)
2	281	0.5	2.7
5	372	0.5	3.1
10	447	0.5	3.6
25	583	0.5	4.3
50	700	0.5	5.0
100	790	0.5	5.4

rainfall, assuming no tailwater from Bureau Creek, as seen in table 10. So assuming the level in Bureau Creek is down and the seep ditch and culvert are clean, the duration of flooding upstream is minimal.

Table 11 presents a summary of maximum ponding elevations for various combinations of rainfall and flooding in Bureau Creek. With no tailwater, all rainfall return intervals cause some ponding, however short the duration. The 1.0526-year Bureau Creek flood does nothing to increase the maximum ponding depth, and the 1.25-year Bureau Creek flood does little to increase the ponding. The 2-year Bureau Creek flood and above begin to increase the ponding depths. Depending on the return interval of the rain, floods of 10- to 50-year recurrence intervals in Bureau Creek cause outflow from the culvert to cease. A 100-year rain is of substantially more volume than a 2-year rain of equal duration. Therefore, with a greater volume of rain there is a higher upstream head with which to convey flow. This is the reason a maximum ponding depth exists for a 25-year flood in Bureau Creek for a 100-year rain and not a 2-year rain.

From table 12 the peak outflow through the culvert may be seen. The increasing depths in Bureau Creek represented by the high (less frequent) flood recurrence interval decrease the carrying capacity of the culvert substantally. With no backwater from Bureau Creek the watershed is drained within a reasonable amount of time by the 48-inch corrugated metal pipe culvert.

The situation that existed in the field during the 1983 survey was not investigated, primarily because the topography is such that the land near Lock 8 cannot drain by overland flow. Since the culvert is approximately half full of sediment, the carrying capacity would be

Table 11. Results of Runoff Computations for Maximum Ponding Elevations (ft,msl), Culvert #4 Watershed, Hennepin Canal

Bureau Creek flood recurrence interval (yrs)	Rain return interval (yrs) 2	5	10	25	50	100
No tailwater	497.2	497.5	497.8	498.3	498.8	499.1
1.0526	497.2	497.5	497.8	498.3	498.8	499.1
1.25	497.1	497.4	497.7	498.3	498.8	499.1
2.0	497.4	497.7	498.0	498.6	499.0	499.4
5.0	497.7	498.0	498.3	498.9	499.3	499.7
10.0	497.9[1]	498.3[1]	498.6	499.1	499.5	499.8
25.0	*	*	498.6[1]	499.3[1]	499.8[1]	500.0
50.0	*	*	*	*	*	500.0[1]
100.0	*	*	*	*	*	*

[1]No outflow occurred; elevation results from storage of rainfall only; tailwater is greater than this elevation.

*Tailwater is above upstream ponding elevation so flow will not drain until Bureau Creek level subsides.

Table 12. Results of Runoff Computations for Peak Outflows (cfs), Culvert #4 Watershed, Hennepin Canal

Bureau Creek flood recurrence interval (yrs)	Rain return interval (yrs) 2	5	10	25	50	100
No tailwater	113.7	115.9	117.9	121.5	124.5	126.8
1.0526	113.7	115.9	117.9	121.5	124.5	126.8
1.25	113.4	115.6	117.5	121.4	124.5	126.6
2.0	94.1	76.3	80.5	88.3	94.0	98.5
5.0	28.0	39.2	47.0	58.8	67.4	73.4
10.0	0.0[1]	0.0[1]	8.7	34.1	46.8	54.9
25.0	*	*	0.0[1]	0.0[1]	0.0[1]	8.4
50.0	*	*	*	*	*	0.0[1]
100.0	*	*	*	*	*	*

[1]No outflow occurred since level in Bureau Creek is higher than headwater.

*Tailwater is above upstream ponding elevation so flow will not drain until Bureau Creek level subsides.

reduced by one-half as would the time to drain the watershed, providing that the water west of the culvert could be drained and would not be obstructed by the topography.

Soil Loss and Sedimentation Data

Land Use

Land use patterns were determined from aerial photographs of the watershed from 1941, 1951, 1958, and 1970, which were obtained at the map library at the University of Illinois, Champaign-Urbana campus. Aerial photographs taken of the watershed in 1982 were obtained from the Illinois Department of Transportation.

From 1941 through 1964 few changes were observed in the watershed. For the most part, the changes consisted of the pastures being overgrown. The most noticeable area of overgrowth was in the area of Lock 8. During the period from 1964 to 1970 this area was completely overgrown. The division between meadow and cropland in this area apparently followed the contours. The lower land was used as pasture.

The 1970 aerial photo shows that the area along the eastern boundary had been cleared of woodland and converted into cropland and meadow. The lower portion of the watershed and the area that was cleared are shown in figure 10.

The most dramatic alteration of land use occurred between the years 1970 to 1982. An area 2000 feet north of Lock 9 and an area 2400 feet northwest were cleared as well as the area near Lock 8, as shown by the shaded areas in figure 10.

The area cleared on the bluff will tend to increase the sediment load. In addition the clearing will increase both the volume of runoff and the peak flow rate. The clearing near Lock 8 has little impact on

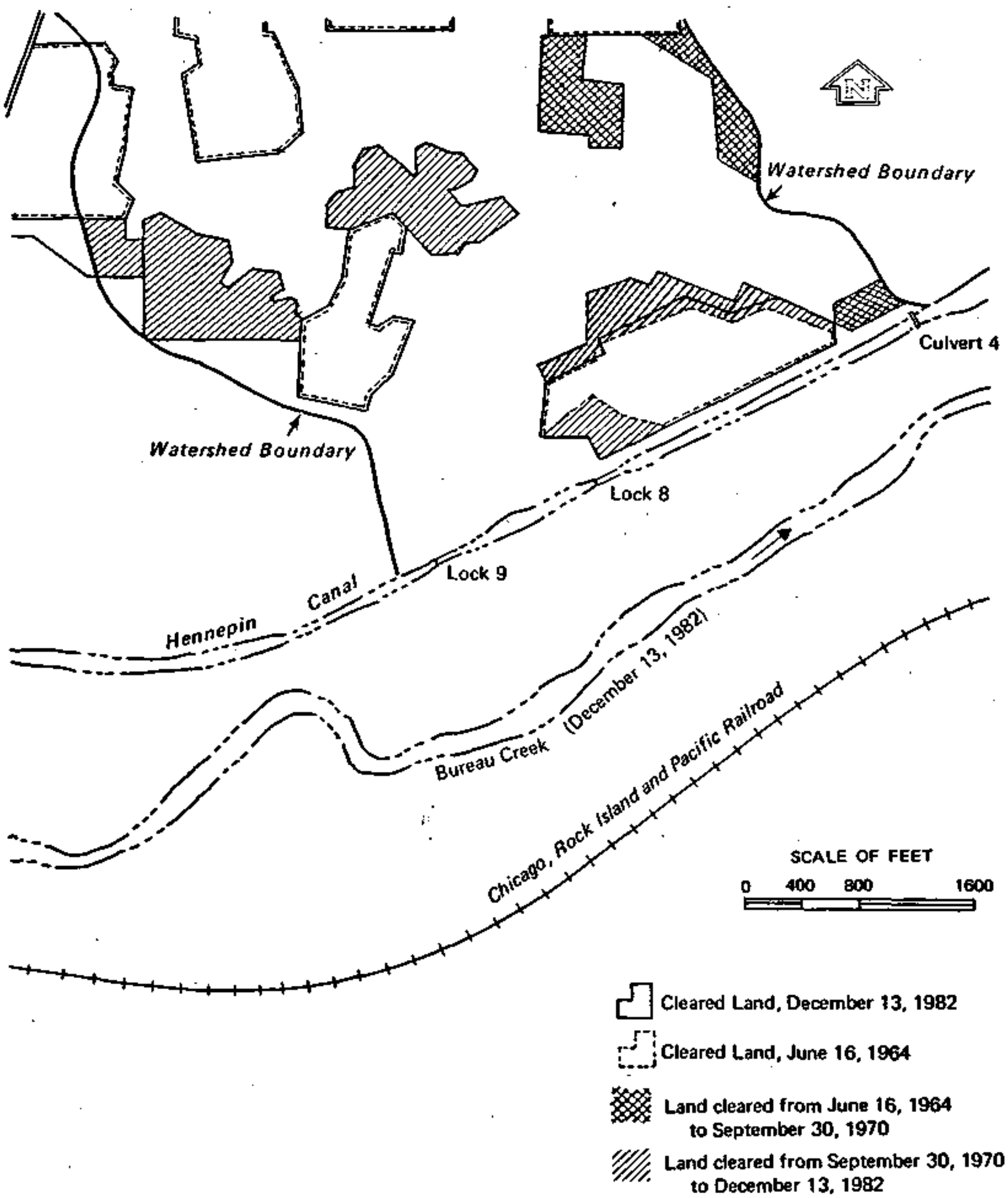

Figure 10. Land use changes within the Culvert #4 watershed

runoff and sediment load, but about half of the area that cannot sustain row crops was cleared after 1970. The five acres cleared near Lock 8 had not been previously cropped, or at least not since 1941. Prior land uses for this area consisted of meadow, pasture, and woodlands.

Soil Loss Rates

An estimation of soil loss rates for the Culvert #4 watershed was made by using the Universal Soil Loss Equation, USLE (Wischmeier and Smith, 1978). The USLE is an erosion model designed to predict the long-term average soil losses in runoff from specific field areas in specific cropping and management systems. The USLE is as follows:

$$A = RKSLCP$$

where A is the average annual soil loss rate in tons per acre per year, R is the rainfall factor, K is the soil erodibility factor, S is the slope-steepness factor, L is the slope-length factor, C is the cropping factor, and P is the support practice factor. A more detailed description of the USLE may be found elsewhere (Wischmeier and Smith, 1978; Walker and Pope, 1979; Peterson and Swan, 1979).

The assessment of several of the soil parameters involves land use and soil type data. Figure 11 shows the land use map of the Culvert #4 watershed that was developed from aerial photos taken on September 30, 1970. These photos were selected because these were the most recent photos that showed the entire watershed. The additional clearing done since this time would add little to the estimated soil loss. The land uses are divided into woodland, cropland, meadow, and farmstead.

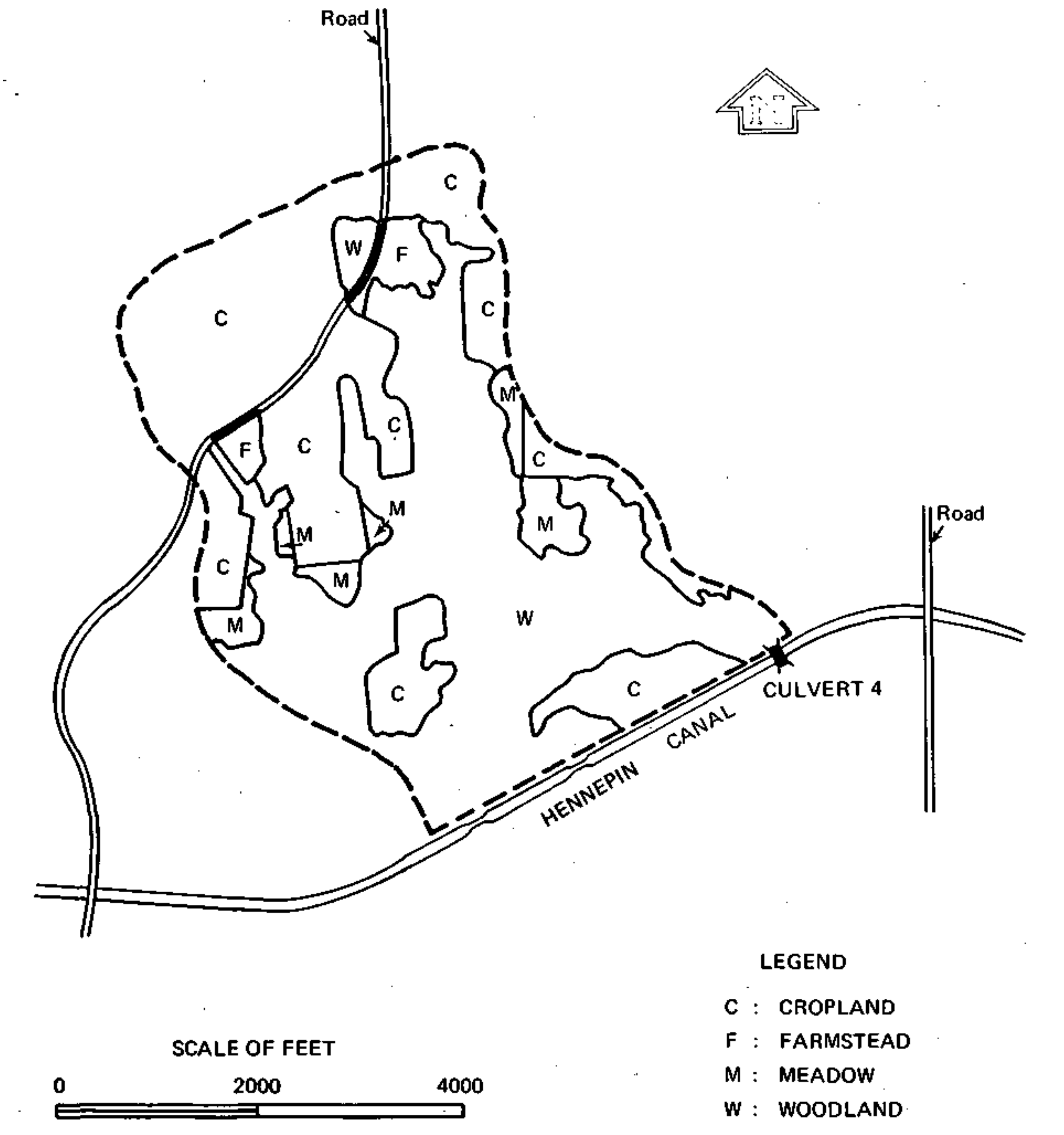

Figure 11. Land use in the Culvert #4 watershed, September 30, 1970

A 1950 soil map of the Culvert #4 watershed was obtained from the Soil Conservation Service, USDA, and the soil survey and aerial photos were combined to develop the soil map shown in figure 12. The soil types and their acreages are tabulated in table 13. It can be seen that strawn silt loam is the major soil type, covering about 41 percent of the watershed. Several types of Fayette silt loam cover another 39 percent of the area, and other silt loams and some silty clay loams make up the remaining acreage.

Table 14 shows the erosion parameters for the USLE for 26 soil samples, listed according to the sample numbers assigned to the tracts on the soil map in figure 12. The boundaries of each tract were measured for acreage, and the dominant appropriate land use was assigned from the land use map. The rainfall factor, R, for Bureau County is 175. The soil erodibility factor, K, of each soil type was obtained from soil interpretation records provided by the Bureau County Soil Conservation Service. The slope-steepness and slope-length factors were determined from soil slope symbols and topographic map measurements. The cropping factor, C, was assigned as 0.4 on all cropland indicating conventional tillage and corn-soybean rotation. Other cropping factors (conservation practice factors, P) were all specified to be 1.0 since no significant contouring and terracing practices are used on the watershed.

Based on the compilation of this information, the soil loss rates of each of the 26 tracts were computed and may be found in table 14. The total amount of soil loss for each sample was obtained through multiplication of the soil rate and soil acreage. The results indicate that the total gross erosion from the Culvert #4 watershed amounts to 3898 tons per year. Table 15 shows the breakdown of soil loss on the

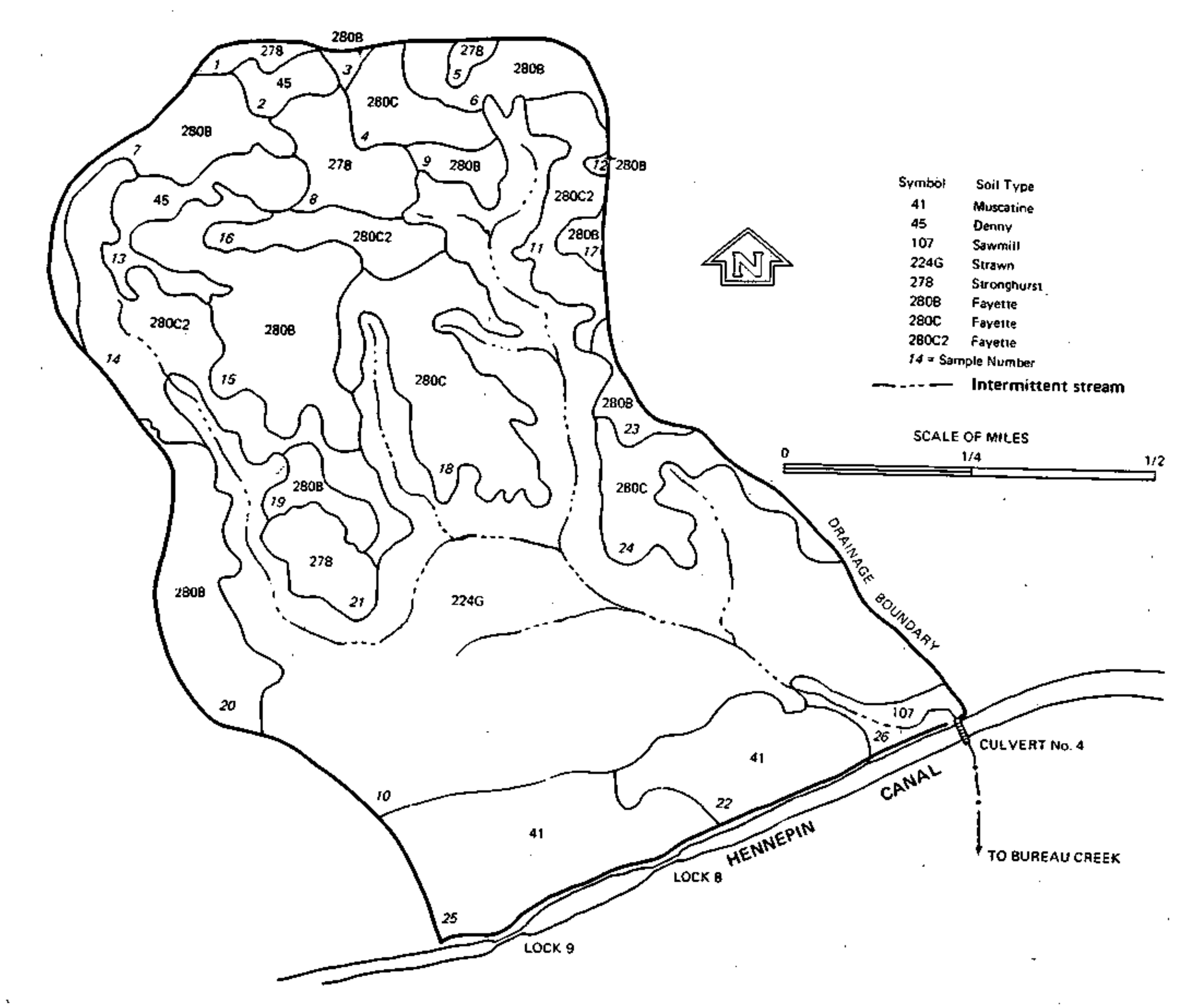

Figure 12. Soil types in the Culvert #4 watershed

Table 13. Culvert #4 Watershed Soil Types

Soil type		Acreage	Percent of total acreage
41	Muscatine silty clay loam	60.4	10.77
45	Denny silt loam	19.2	3.43
107	Sawmill silty clay loam	6.6	1.18
224G	Strawn silt loam	227.9	40.65
278	Stronghurst silt loam	27.5	4.91
280B	Fayette silt loam	103.4	18.44
280C	Fayette silt loam	57.9	10.33
280C2	Fayette silt loam	57.7	10.29
	TOTALS	560.6	100.00

Table 14. Soil Loss Assessment for Culvert #4 Watershed, Hennepin Canal, near Tiskilwa, Illinois

Sample no.	Soil type	Land use*	Acre- age	Erodi- bility K	Slope (%)	Slope length (ft)	Cropping factor C	R x P	Soil loss rate, A (tons/ac/yr)	Total amt. gross erosion (tons/yr)
1	S'hurst**(273)	Crop	2.9	.37	1	160	0.4	175	3.8	11.1
2	Dennv (45)	Crop	5.1	.37	1	270	0.4	175	4.5	23.0
3	Fayette (280B)	Crop	1.1	.37	3	180	0.4	175	8.9	9.7
4	Fayette (280C)	Wood	10.5	.37	6	520	0.003	175	0.3	3.1
5	S'hurst (278)	Crop	1.6	.37	1	310	0.4	175	4.7	7.5
6	Fayette (280B)	Crop	11.9	.37	3	430	0.4	175	11.5	136.8
7	Fayette (280B)	Crop	15.9	.37	3	330	0.4	175	10.6	169.3
8	S'hurst (278)	Crop	13.2	.37	1	660	0.4	175	5.9	77.6
9	Fayette (280B)	Farm	4.7	.37	3	270	0.20	175	5.0	23.6
10	Strawn (224G)	Wood	227.9	.37	20	560	0.001	175	0.6	147.3
11	Fayette(280C2)	Crop	14.2	.37	6	330	0.4	175	31.6	448.7
12	Fayette (280B)	Crop	0.4	.37	3	130	0.4	175	8.0	3.2
13	Denny (45)	Crop	14.1	.37	1	240	0.4	175	4.4	61.4
14	Fayette(280C2)	Crop	32.9	.37	6	360	0.4	175	33.2	1090.7
15	Fayette (280B)	Crop	34.2	.37	3	400	0.4	175	11.3	385.3
16	Fayette(280C2)	Crop	10.6	.37	6	340	0.4	175	32.1	340.4
17	Fayette (280B)	Crop	2.0	.37	3	330	0.4	175	10.6	21.3
18	Fayette (280C)	Mead	31.0	.37	6	360	0.08	175	6.6	205.5
19	Fayette (280B)	Crop	5.9	.37	3	360	0.4	175	10.9	64.3
20	Fayette (280B)	Crop	23.2	.37	3	490	0.4	175	12.0	277.6
21	S'hurst (278)	Head	9.8	.37	1	400	0.20	175	2.5	24.7
22	Muscatine (41)	Crop	23.4	.28	1	560	0.4	175	4.2	99.1
23	Fayette (280B)	Crop	4.1	.37	3	220	0.4	175	9.4	38.7
24	Fayette (280C)	Mead	16.4	.37	6	250	0.20	175	13.9	227.2
25	Muscatine (41)	Wood	37.0	.28	1	810	0.003	175	0.04	1.3
26	Sawmill (107)	Wood	6.6	.28	1	140	0.001	175	0.01	0.0
	TOTALS		560.6							3898.4

* Crop = cropland, Wood = woodland, Farm = farmstead, Mead = meadow
**Stronghurst

Table 15. Soil Loss Assessment Based on Land Use in Culvert #4 Watershed, Hennepin Canal, near Tiskilwa, Illinois

Land use	Acreage	Total amount of gross erosion (tons/yr)	Average soil loss rate (tons/ac/yr)
Woodland	282.0	151.7	0.5
Cropland	216.7	3265.7	15.1
Meadow	57.2	457.4	8.0
Farmstead	4.7	23.6	5.0
Total	56-0.6	3898.4	7.0 (average)

basis of land use. As is to be expected, the greatest soil loss rate is from cropland at 15.1 tons per acre per year or a gross amount of 3265.7 tons per year. The average soil loss rate for the entire watershed is 7.0 tons per acre per year.

As described in the State Water Quality Management Plan, control of erosion from cropland should be designed to reach the ultimate goal that no lands have erosion losses exceeding the soil loss tolerance levels ("T" values) established to maintain soil productivity. It is assumed that if the planned objective of "T" values is achieved on all lands, then actual soil loss reduction will result as indicated by Lee et al. (1983). In this case both the cropland and the watershed as a whole fail to meet the ultimate "T" values for the state, which are 5 tons per acre per year for the soil types found on the watershed. The soil loss rate from the cropland, which is the second most dominant land use, is about three times the recommended "T" value. The most dominant land use within the Culvert #4 watershed is woodland, which easily meets the recommended "T" value. The third most dominant land use is meadow, which has a value somewhat above the recommended "T" value. Farmstead land use meets the recommended "T" value. Therefore 51 percent of the watershed meets the recommended "T" values. Conservation practices must be applied to the rest of the watershed so that it meets the recommended soil loss tolerance value.

Sediment Movement and Deposition

On several occasions, surveys of the lower portion of the Culvert #4 watershed were made by Illinois State Water Survey personnel. Sediment conditions above as well as below the 48-inch corrugated metal pipe were investigated. In the portion upstream of the culvert, the north seep ditch was examined, along with an unnamed tributary which flows on the northeast side of the cornfield. This stream runs from the northwest to the southeast where it enters the north seep ditch and then flows to the culvert. From the culvert a drainage ditch carries the flow to Bureau Creek. The area may be seen in figure 3.

The original purpose of the seep ditch was to convey any seepage from the canal to a point where the flow may be directed to a natural drainage path. The ditch is not supposed to receive any surface flow and is constructed of sand to allow subsurface conveyance. The north seep ditch is about 2200 feet in length (from Lock 8 to Culvert #4), the upstream end (near Lock 8) is at an approximate elevation of 496 feet msl, and the downstream end (near Culvert #4) is either 486.7 feet msl (upstream end 48-inch cast iron pipe), 491.2 feet msl (upstream end 48-inch corrugated metal pipe), or 493.5 feet msl (upstream sediment elevation).

Three sediment samples were obtained from the Culvert #4 watershed. Two of the sampling locations were in the seep ditch above the upstream end of the 48-inch corrugated metal pipe. One sample, A, was taken 6 feet from the end and the other, B, was taken 20 feet from the end. The third sample, C, was taken within the downstream end of the 48-inch corrugated metal pipe. These samples were obtained by driving a shovel at a low angle (with respect to the channel bed) into the sediment to scoop out the top 1 inch of material. Samples were placed into plastic bags for transport to the Illinois State Water Survey Sediment and Materials Laboratory for particle size analysis. The results are presented in table 16.

Table 16. Particle Sizes of Samples Taken within the Culvert #4 Watershed

	A - Upstream end of Culvert #4 (6 feet)	B - Upstream end of Culvert #4 (20 feet)	C - Downstream end of Culvert #4
Gravel (%)	11.74	0	38.71
Sand (%)	85.28	12.62	60.72
Silt and clay (%)	2.98	87.38 (Silt-77.05, Clay-10.33)	0.57
Mean size (mm)	0.31	0.043	1.20
Size classification	Medium sand	Coarse silt	Very coarse sand

There is deposition of silt in the upper portion of the north seep ditch, toward Lock 8. There is no deposition in the unnamed tributary. The tributary's bed is mostly coarse gravel and appears to be armored as can be seen in figure 13. The armoring process occurs because the fine particles of the bed material are most easily transported by the flow, while the coarse particles tend to remain on the bottom (Simons and Senturk, 1977).

Any lateral inflow of suspended sediment to the unnamed tributary should remain in suspension due to the high slope (2 percent) and the resulting high velocity within the unnamed tributary. There appears to be a problem with the extent of the channel erosion. The channel is entrenching itself and is exposing and transporting very large sediment particles. Particles of 6-inch diameters and larger were not uncommon. These particles gradually move toward the confluence with the seep ditch. At the confluence the slope of the unnamed tributary flattens out and there is a deposition of material. Adding to the conditions contributing to the deposition in this area is the abrupt change in direction that the flow must make as the water enters the seep ditch. There is significant energy lost at this point, as evidenced by the 15-foot shear

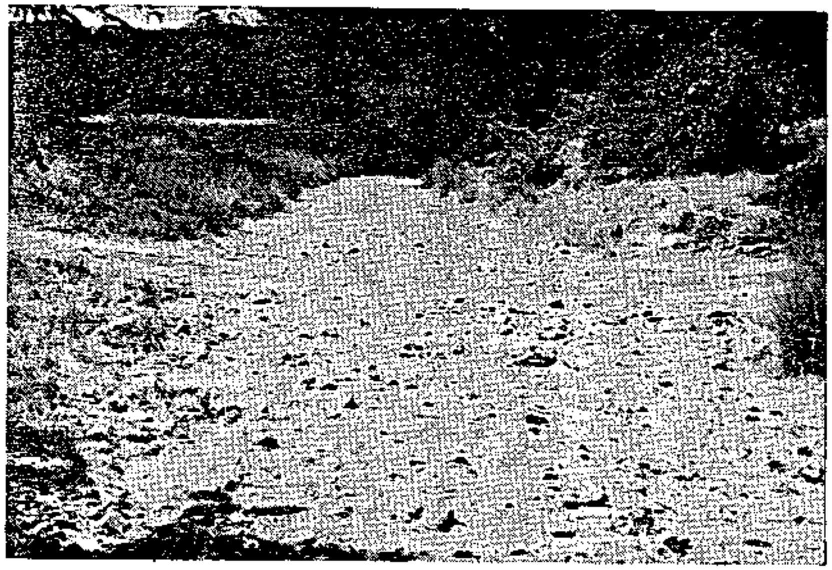

Figure 13. Looking upstream (northwest) in the unnamed tributary

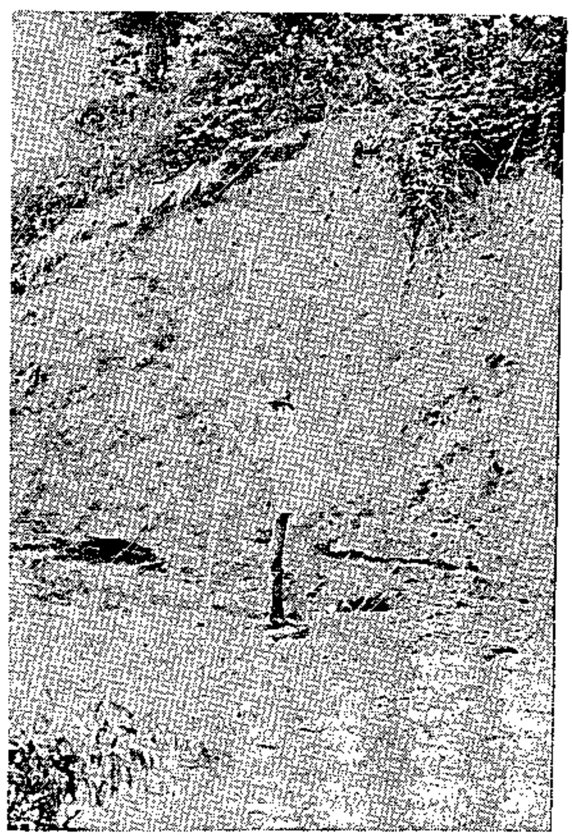

Figure 14. Confluence of the unnamed tributary and the north seep ditch

face in the north levee of the canal as seen in figure 14. There are many large cobbles deposited at this point. The high point in the seep ditch is found at this location, as was seen in figure 5.

With the seep ditch in its present state, the runoff from the watershed flows both northeast and southwest in the seep ditch. Each event brings with it more sediment. As the runoff decreases it wears away the newly deposited sediment within the channel to the culvert. Since the seep ditch heading southwest does not receive the lower flows, a natural embankment has formed that is about 1 to 2 feet higher than the portion of the seep ditch that flows to the culvert. This ridge is located about 50 feet southwest from the confluence.

The flow that runs to the southwest to Lock 8 does not have sufficient velocity to carry large sediment but does carry the fine sand as evidenced by the vegetation in the seep ditch that was pushed "upstream" (towards Lock 8). The present slope of the north seep ditch from the confluence with the unnamed tributary to Lock 8 is contrary to original construction. Each precipitation event will contribute a portion of its flow to the area near Lock 8 in addition to the amount of rain that falls on this subwatershed.

At the end of the event the runoff has no surface route out; it enters the ground and travels as subsurface runoff or enters the tile drainage systems. Either way, the amount of time for the ponding to vanish and the land to dry out is significantly greater than if there were surface runoff.

From the confluence of the unnamed tributary and its north seep ditch to Bureau Creek there is negligible sedimentation of small particles. The sediment samples indicate that some deposition of fine particles occurs upstream of the culvert (sample B). Closer to the culvert, less deposition of silts and clays (fine particles) occurs (sample A). This is due to the increased velocities near the culvert entrance. The velocities within the culvert itself move all sizes of particles except for gravel and some sand (sample C).

The results of the particle size analysis are a bit misleading. There does not appear to be much deposition occurring within the portion of the seep ditch from its confluence with the unnamed tributary to Bureau Creek. The sediment elevation in the downstream end of the culvert increased 0.76 feet between the 1982 and 1983 surveys. This portion of the ditch is rising gradually. During the higher flows caused by a rainfall, the sediment moves downstream. As the rain stops and flows decrease, so do the velocities. The lower velocities cannot continue to transport the sediment particles and they therefore deposit them, with the larger particles dropping out earlier, followed by the smaller particles. There are large particles both upstream and downstream of the culvert. The suspended sediment comprised of silt and clay does not adversely affect the operation of the culvert since these particles are passed downstream.

The main difficulty with respect to the culvert operation occurs with the larger sized particles. The culvert conveys sediment with some deposition. Problems occur downstream of the culvert because of the lack of slope from its invert to Bureau Creek, and this situation is further aggravated by high surface water conditions in Bureau Creek. The sediment-carrying capacity of the water is greatly reduced once the flow leaves the downstream end of the culvert due to a decrease in velocity if Bureau Creek is in flood elevation. Therefore, there is a buildup of larger particles downstream of the culvert which serves to raise the bottom of the channel. The deposition proceeds upstream until, as it does presently, it reduces the cross-sectional area of its culvert which lowers the carrying capacity. The loss of conveyance allows more ponding to occur upstream which permits particles upstream of the culvert to settle out.

The decreased cross-sectional area of the culvert should increase the velocity, which should clear out the culvert. To a certain degree this occurs, but the deposition downstream of the culvert curtails the cleaning. It was observed that the ditch downstream also suffers from deposition from Bureau Creek. When Bureau Creek rises into the ditch, the flow in the ditch becomes negligible. Sediment from Bureau Creek therefore settles out in this area as do the coarser particles from the Culvert #4 watershed. Therefore, there is buildup of the bottom of the ditch. In the field reconnaissance, 18 or more inches of deposition was observed (see figure 15). This deposition occurs in layers of fine and coarse particles.

As the water level in Bureau Creek falls, the water in the ditch begins to move again. As the velocity increases so does the capacity to move sediment. Since the sediment deposited from Bureau Creek may be classified as sand, the water in the ditch begins to scour away the deposition until an equilibrium point is reached once again. It was observed in the 200 feet downstream of the culvert that the sediment was comprised of coarse materials. In the remaining portion of the ditch to Bureau Creek, finer sediment was observed. This probably occurred because the stage of Bureau Creek rose and fell with only a little runoff coming from the watershed to move the sediment along. At the confluence

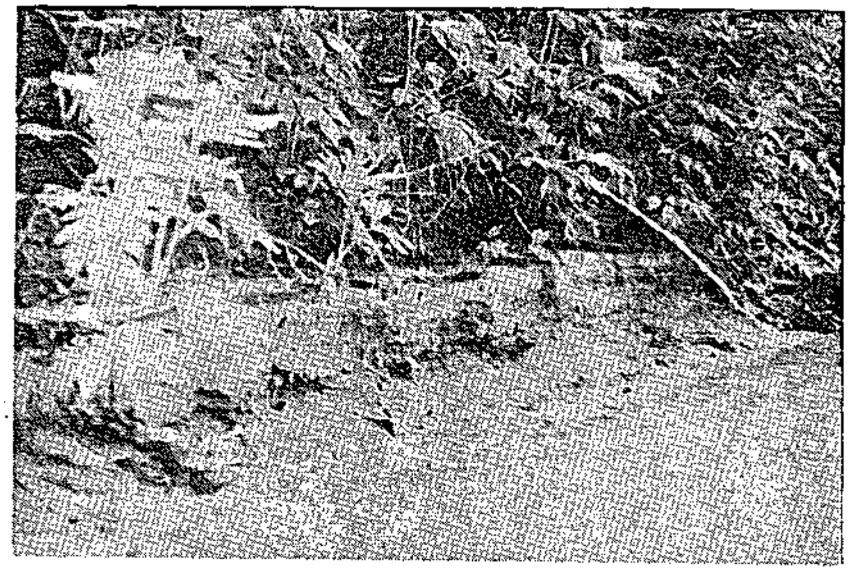

Figure 15. Deposition in the ditch between Culvert #4 and Bureau Creek

Figure 16. Area north of Lock 8 looking northeast

of Bureau Creek and the ditch there is a delta extending about 15 feet into Bureau Creek from the bank. The sediment is then carried downstream in Bureau Creek.

ALTERNATIVE DRAINAGE PLANS

The alternatives presented are possible solutions to the problem of the lack of drainage in the Culvert #4 watershed. Each alternative may be used separately or in combination with another alternative. Whatever alternative is selected, crest gages should be installed on the north and south sides of the canal. The crest gages will indicate the maximum water surface elevation.

Return of Land to Pasture

The limitations of the land near Lock 8 should be examined carefully. It might be best to take this area out of crop production and leave it to pasture or meadow. In the June 1983 survey and a subsequent visit on July 28, 1983, this area was seen to be used for grazing (figure 16). This might be the way the land was used previously. The local Soil Conservation Service (SCS) personnel can design a land management plan to prevent erosion on the upland and within the channel of the unnamed tributary.

Alteration of Bureau Creek

One alternative to decrease flooding would be to decrease the levels in Bureau Creek; however, the water surface elevations within Bureau Creek cannot be altered without great expense, and a study by the U.S. Army Corps of Engineers (1975) suggests that it is not economically justified. So no consideration will be given to this alternative. Instead, the recommendations will center on possible solutions to relieve the ponding near Lock 8.

No Change in Present Drainage System

Another alternative would be to do nothing. The advantage is that there would be no initial cost. However, the problem would recur and continue for the foreseeable future.

Purchase or Rental of Flooded Land

A second alternative would be to buy or rent the flooded land. This is a nonstructural alternative and would require a relatively low initial cost or low annual payments. The land might be used as part of the Hennepin Canal Parkway. This area would then be left natural for wildlife habitats. However, as with the above alternative, the problem would recur.

Dredging of North Seep Ditch to Bureau Creek

This, in itself, is not a new action. The north seep ditch from Lock 8 to the upstream invert of the 48-inch corrugated metal pipe, the pipe itself, and the ditch from the pipe to Bureau Creek would all be cleaned. It would not be practicable to dredge to the 48-inch cast iron culverts since they are placed too low to ever be effective. The unnamed tributary could be excavated to the property line. The spoil material should either be trucked off-site or placed where it would not be washed into the seep ditch in the next rain. Currently, placement of the spoil along the sides of the seep ditch may explain the "siltation" around the concrete property markers.

Protection should be to the north side of the north levee to protect it from the erosion it is experiencing that is caused by the unnamed tributary. A trash rack should be provided upstream of the culvert so that oversized rocks, tree limbs, tires, etc., do not block the culvert.

To try to arrest the onslaught of bed material a basin should be installed on the unnamed tributary on the canal property. This would have to be cleaned quite often as would the trash rack.

The June 1983 survey showed that the 48-inch corrugated metal pipe was a bit bowed upward. This should be corrected and perhaps the culvert might be inclined. The added slope would then provide additional velocity so that the culvert would rid itself of the sediment more effectively. The culvert might be raised on both ends to put the inverts at the sediment elevation presently within the culvert. This would help the culvert drainage but reduce the slope of the seep ditch. The seep ditch is in equilibrium and if a change is made, the equilibrium might shift upward.

For additional potential conveyance, a more efficient pipe could be used. A concrete pipe could carry 84 percent more flow than a similar corrugated metal pipe assuming all other parameters remained the same.

Allowing additional capacity under the canal would pass the runoff quickly, allowing less pondage. If it is assumed that the existing culvert, seep ditch, and ditch to Bureau Creek are in a clean condition, calculations have shown that the ponding levels would not stay high excessively long.

The advantage to this alternative is that little new construction would take place, depending on the degree of improvement selected. The disadvantage is that maintenance would be frequent. If no sediment trap were used, quite probably the next major precipitation event would deliver a significant amount of sediment to the seep ditch. This would deter flow from the area near Lock 8, and the ponding situation would remain. If a sediment trap were used it would have to be cleaned quite regularly so it would effectively remove the sediment and little deposition would occur in the seep ditch. If no sediment trap were used, the north seep ditch would require continuous dredging to prevent deposition within the seep ditch.

If a new culvert or the present culvert were installed at a higher elevation, there might be interference with the traffic within the canal prism. An inverted siphon is out of the question here due to the high sediment load. Even with a flush box the siphon would be very difficult to maintain and operate.

Installation of a Canal Siphon at Culvert #4

This alternative is separate from the previous alternative because the waters of the canal would be routed through the siphon and the runoff would pass unrestricted through the canal prism. This alternative could be coupled with parts of the previous alternative such as cleaning the north seep ditch and the ditch leading to Bureau Creek, installing a sediment trap, etc.

The advantage to this alternative is that the restriction of the culvert would be removed so that this part of the watershed's drainage would be "natural." The disadvantage is that continual maintenance would be required to drain the area near Lock 8. There is no way to determine a priori what the final slope of the seep ditch channel would be.

Renovation of Drainage Tile

This alternative considers the use of the drainage tile within the north seep ditch. On each visit to the canal this tile was observed to carry flow. The condition of this tile is unknown as are the design parameters.

It appears that this tile originally exited at the upstream end of the buried 48-inch cast iron pipes. The outlets to the tiles are also buried but have eroded a small channel to the 48-inch corrugated metal pipe. The use of these tiles could be combined with one of the previous alternatives discussed, such as cleaning out the seep ditch.

The renovation would entail finding the upstream and downstream portion of the pipe and providing suitable sumps for these. The upstream portion should be low enough to drain the field near Lock 8, and the water in this area must have ready access to the tile, which could be accomplished by drainage tiles or an overland route. The downstream end would have to be kept open and free from the sediment buildup that presently occurs.

The advantage is that this alternative is readily available and should keep the ponding level down in the area of Lock 8. The disadvantages are the unknowns, mainly the condition of the pipe. The slope of the pipe would have to be quite low. The diameter is thought to be 10 inches. If 1 foot of cover is over the upstream end, the invert should be 494 feet msl. This is quite adequate to drain the fields, but if the downstream invert is about 492 feet msl and the distance is 2000 feet, the slope is, at best, 0.0002 foot per foot. This is very flat so deposition and rate of discharge might be problems. This alternative would drain the surface waters very slowly. Presently this tile appears to be draining this area.

Installation of New Tile

This alternative and the next several alternatives consider separating the Culvert #4 watershed. There is a tendency for the unnamed tributary to do this on its own. The separation occurs at the confluence of the unnamed tributary and the north seep ditch. The advantage to subdividing the watershed is that the high concentration of sediment would pass at Culvert #4 and the balance of the watershed, about 82 acres, would be dealt with separately.

In addition the advantage to separating the high sediment area from the low sediment area is that all the runoff would go under the canal at Culvert #4. Obviously, this would reduce the amount of runoff to be handled near Lock 8. Not as obvious is that the extra discharge at Culvert #4 would mean higher velocities that should allow less deposition to occur within the channel from the unnamed tributary to Bureau Creek.

The idea of separating the drainage areas appears to be what nature is trying to accomplish. If it were not for the canal, the drainage near Lock 8 would find a path to Bureau Creek, perhaps joining the unnamed tributary further downstream. This is not to imply that the area near Lock 8 would be totally drained or farmable.

The watershed would be separated by a berm at the west end of the confluence of the unnamed tributary and the north seep ditch in the approximate location of the high point found in the survey. The north levee is quite high through this area, and the height of the berm would be above that of the east end of the cornfield. This would allow any pondage caused by the culvert to back up through the cornfield and run west and/or east overland. The overland flow should allow the runoff to slow down providing that no gullies form. This alternative and the following alternatives deal with the west subwatershed, and might be combined with some of the alternatives discussed previously.

This alternative (installation of new tile) would be similar to the last alternative presented, but a new tile would be used to drain the 82-acre subwatershed. There is a possibility of using the original tile for this alternative. The main difference between this alternative and the last is that the amount of runoff would be less with the new tile and there would be less chance of sediment entering the tile since the watersheds would be separated. The grass border between the field and the seep ditch must be left in place to trap any sediment that might potentially clog the tile.

The advantage to this alternative is that the canal prism would not be disturbed (unless an alteration were made at Culvert #4). A disadvantage is that even though the two subwatersheds would be separated they still would be connected at the upstream portion of Culvert #4. High levels in Bureau Creek could push large sediment particles into the outlet of the tile, so a flap gate might be used.

Placement of a Culvert at Lock 8

This alternative would further decrease the interdependence of the watersheds. A culvert, either straight or inverted, would be placed just downstream (east) of Lock 8. The south seep ditch would have to be cleared and sloped to drain to the east. The seep ditch would join the ditch from Culvert #4 to Bureau Creek just downstream of Culvert #4, following the perceived original drainage. It would not be practical to go straight to Bureau Creek from Lock 8 due to the adverse grade and the fact that the state does not possess an easement for this route.

The downstream invert would be 495 feet msl while its upstream would be 496 feet. This would allow a 3-foot drop in the south seep ditch and a 1-foot drop in the culvert under the canal for a slope of about 0.00625 foot per foot for the culvert. A 2-year precipitation return interval would develop an approximate peak runoff rate of 95 cfs, depending on the land use. Calculations indicate that 24-, 36-, and 48-inch diameter corrugated metal pipes would provide maximum discharges of 10, 30, and 70 cfs, respectively, with no ponding.

The straight culvert would have the advantage of ease of installation and ease of maintenance, while the inverted siphon would not obstruct the prism.

The advantage of this solution is that the watersheds would be separated for the most part. The disadvantages are that this alternative might be expensive and would involve maintenance of two culverts. (There is a culvert under a farm access field in the south seep ditch near Culvert #4 that would have to be maintained.) Maintenance must also be extended to the south seep ditch. If cost savings are opted for, the straight culvert would interfere with the prism but since it is near Lock 8 it should pose little hazard.

Dewatering of the Canal

In this alternative, the canal would be dewatered below Aquaduct 2 or Lock 12. The prism could be levelled and the area returned to the natural drainage.

The advantage of this alternative is that in the area that contains numerous problems such as drainage and the erosion of the prism by Bureau Creek, there would be a permanent solution that would not require any maintenance. The disadvantage is clear: the loss of a recreational area, which may be deemed unacceptable by the public.

Placement of a Pump at Lock 8

The final alternative would involve placement of a pump in the area north of Lock 8. The pump would discharge into the south seep ditch. The pipes would be closed conduits so no grade would have to be maintained and the canal prism would be undisturbed.

The main disadvantage would be the initial cost of the pump(s) and appurtenant items and the high operation and maintenance costs. Also, if no modification was made to the south seep ditch, there would be excessive ponding. Depending on the condition of the culvert under the access road to the field, flooding might occur in the south field.

SUMMARY AND CONCLUSIONS

This report summarizes an investigation of drainage, soil erosion, and sedimentation in the close proximity of Culvert #4 along the Hennepin Canal. The field investigation indicated that the present condition of the seep ditch prohibits the drainage of the low area near Lock 8. The north seep ditch from Lock 8 to Culvert #4 has a negative slope, which effectively reduces the flow of water from Lock 8 to Culvert #4.

Original topographic maps for the area around Culvert #4 were not available. The oldest available data consisted of 1901 and 1930 surveys by the U.S. Army Corps of Engineers. The 1930 data correspond to the period after the construction of the canal. In all probability, the surrounding areas may already have been altered. The data suggest that drainage from Lock 8 roughly followed the present location of the canal northeast. The drainage might have intersected a channel where the present ditch from the culvert to Bureau Creek is located, or it might have continued directly to Bureau Creek. Present survey data seem to confirm the fact that the area near Lock 8 flowed northeast. The sediment contributed by the unnamed tributary may have separated the drainage from the area near Lock 8 from the rest of the Culvert #4 watershed. The elevations obtained from the field survey indicate that certain portions of the land near Lock 8 are low and drain by subsurface flow. This land may not be a natural area and may have been altered by human activities.

The clearing of the land within the drainage area did not significantly alter the sediment load. However, due to changes in land use, the runoff will have a shorter time of concentration, which increases the velocity and the erosiveness of the water as it cascades down from the bluff area in the unnamed tributary. The amount of sediment contributed by the unnamed tributary is significant. The sediment particles are very large and reduce the effectiveness of the culverts by depositing in the slower moving water at the foot of the bluff area.

A portion of the land near Lock 8 has been cleared of timber since 1970 and is currently being used for row crops. The poor drainage conditions near Lock 8 are much more obvious now than when the land was in timber or pasture. This land is being drained by the drainage tile within the seep ditch or within the seep ditch itself. These mechanisms will in time drain the land, but very slowly. The unnamed tributary has delivered and continues to deliver sediment that is very large. Without continuous maintenance of the north seep ditch, the area near Lock 8 will not drain by a surface route. Instead, it will drain by a subsurface route, but more slowly.

From 1970-1982 Tiskilwa had nine occurrences of daily precipitation exceeding the 24-hour duration for average return intervals of more than 2 years. At Kewanee there were three precipitation events of various durations equaling or exceeding a 10-year return interval. As seen in routing precipitation through the 48-inch corrugated metal pipe (CMP), it has been determined that a heavy (100-year return interval) rain causes some ponding upstream of the culvert but its duration is short provided that the water level in Bureau Creek is below the downstream invert of the culvert. Precipitation does not cause flooding in the area of Lock 8 provided that the north seep ditch has a positive gradient from Lock 8 to Culvert #4, which is not practical given the nature of the sediment coming down in the unnamed tributary. Flood flows in Bureau Creek with a frequency of a 5-year recurrence interval or above would flood the land north of the canal at Lock 8. There were at least three occurrences after 1970 when the instantaneous maximum discharge would have flooded the area near Lock 8. The area near Lock 8 is in the floodplain of Bureau Creek. In one sense, the canal affords a degree of protection to this area by slowing the flood flow onto the land. On the other hand, the canal impedes outflow from the north seep ditch by acting as a low dam.

The flood frequency of Bureau Creek may have increased in the last several years due to changes in land use on the watershed and channelization of the creek itself. Flood stages in Bureau Creek have increased due to the construction of agricultural levees. Even though the channelization should decrease the flood stages by making the creek more efficient, it also increases the flood peak, thus offsetting the gain derived from channelization.

Natural drainage of the area north of the canal is not possible under the present conditions. This area will be wet during the period of flooding in Bureau Creek since it is part of the floodplain of the creek.

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