HYDROLOGIC AND HYDRAULIC ANALYSES OF THE HENNEPIN CANAL

FROM LOCK 27 TO THE ROCK RIVER

by Misganaw Demissie and Nani G. Bhowmik

Prepared for the
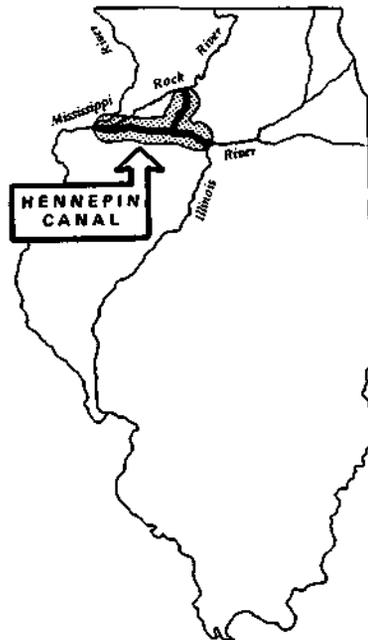
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INTRODUCTION

The Illinois Department of Conservation (DOC) has been responsible
for the operation and maintenance of the Hennepin Canal, also known as the
Illinois-Mississippi Canal, since 1970, when the U.S. Army Corps of
Engineers transferred jurisdiction of the canal to the State of Illinois.
Since acquiring the canal, the Department of Conservation has been faced
with numerous water-related problems along the canal, with some of the
problems resulting in court cases in which the state was asked to provide
compensation for flood damages. Many of the problems are associated with
canal levee breaks and siltation of culverts designed to carry drainage
water underneath the canal to nearby streams at the time of canal con-
struction.

The canal segment under investigation in this report starts from
Lock 27, where flow in the canal is controlled by a siphon which passes
under the Green River channel, and ends at the Rock River, where the canal
terminates. The major problems in this segment of the canal are located
on both sides of Interstate 80. During the construction of I-80, the
Department of Transportation (DOT) lowered the elevation of the south side
levee of the canal at the I-80 crossing. This action allowed flood waters
from the Green River to enter the canal.

At the same time, DOT built a drainage ditch to carry storm water
from the I-80 interchange and the surrounding areas into an existing
culvert (Culvert 40), which used to carry natural drainage from the north
under the canal to the Green River. When excessive sediment coming down
from the I-80 interchange drainage ditch completely filled Culvert 40, the
discharge from the drainage ditch broke the north canal levee. DOT and
DOC then placed a 36-inch corrugated metal pipe through the north levee of
the canal so that DOC could travel the north bank and so that drainage
water from the I-80 interchange area could flow into the canal even though

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the canal was not designed to carry any drainage water from the surrounding areas.

At present, the north and south levees of the canal are completely broken and the 36-inch corrugated metal pipe is out of use. This permits water from the drainage ditch and flood waters from the Green River to enter the canal freely. This excess water in the canal could possibly cause flooding problems to the areas both east and west of I-80 and threaten the stability of the canal levees further downstream. The canal just west of I-80 is also filled with excessive amounts of sediment from the I-80 interchange area, resulting in the rerouting of the flow of water from the canal to the Green River through the breach in the south levee during low canal flows.

Further downstream from the I-80 crossing, just to the east of the Burlington Northern Railroad crossing, there is a break in the south levee of the canal. It is the contention of the adjacent private property owner that this break has allowed the canal water to enter his property to repeatedly flood it. If this adjacent property is flooded by water from the canal, the Green River and the I-80 interchange water have to enter the canal at the breaks just west of I-80 and over the canal banks lowered by DOT.

The problems are interconnected, since fixing the levee break at the Burlington Northern location might not prevent future Green River flooding or further levee breaks at other locations unless the Green River water and surface runoff from the I-80 interchange area entering the canal just west of I-80 are controlled.

Objectives of the Study

The main objective of this study was to make a hydrologic and hydraulic investigation of the Hennepin Canal from Lock 27 to the canal's termination point at the Rock River. The two specific objectives were:

1) To make a recommendation as to the proper levee elevations to prevent Green River flood flows from entering the canal if it is found to be desirable to do so.

2) To make recommendations for the proper handling of the water from the I-80 interchange so that the canal levees will not be damaged by the water. As part of this effort the study attempted to:

a) Determine the feasibility of reactivating Culvert 40.

b) Evaluate the flow requirements of the pipes under a sewer
    crossing at Colona and the Route 84 bridge to determine if
    the drainage from the I-80 area can be accommodated by the
    canal under existing conditions.

c) Evaluate other options such as routing the drainage under the
    canal.

Plan of the Report

The report is organized into five main sections: 1) a background
section with a general discussion of the Hennepin Canal; 2) a section on
flood flow analyses, which discusses hydrologic and hydraulic analyses of
the Green River in the study area, flood elevations in the Rock River, and
flood and levee elevations along the Hennepin Canal; 3) a capacity
analysis of Culvert 40, including surface runoff calculations around the
I-80 interchange and a hydraulic analysis of Culvert 40; 4) a presentation
of alternative solutions to the flooding problem in the study area; and
5) recommendations to solve the problems in the study area. A brief final
section presents an investigation of drainage from the private property
east of I-80.

Acknowledgments

Department at the University of Illinois, assisted in the analyses and in the preparation of this report.

Illustrations for the report were prepared by William Motherway, Jr., and John W. Brother, Jr. Pamela Lovett typed the rough draft of the report and the camera ready copy, and Gail Taylor edited the report.

BACKGROUND

This section presents brief background information about the Hennepin Canal. Two publications, one by Yeater (1978) and another by Scruggs and Hammond (1959), discuss the historical and recreational aspects of the Hennepin Canal in much more detail.

The Hennepin Canal was built by the United States Army Corps of Engineers as a navigation canal to link the Illinois and Mississippi Rivers. It was conceived as a continuation of the Illinois and Michigan Canal for providing water navigation from Lake Michigan all the way to the Mississippi River in the Rock Island area. Because of the topography of the area for which it was proposed, a feeder canal from Lake Sinnissippi on the Rock River had to be built to provide water to the canal. The resulting canal system is shaped like an inverted T, as shown in figure 1.

The main canal from the Illinois River to the Mississippi River is 75 miles long. The canal ascends 196 feet going northwest from the Illinois River to the summit level in a distance of 18 miles. The summit level, which is 11 miles long, passes through level land. The canal then descends 93 feet going southwest from the summit level to the Mississippi River in a distance of 46 miles. The feeder canal from Lake Sinnissippi to– the main canal is 29.3 miles long. To control the water levels for navigation, 33 locks were built, one of which is at the head of the feeder canal. All the locks are 170 feet long and 35 feet wide. The canal was designed to have a bottom width of 52 feet and a width of 80 feet at the water surface, which was maintained 7 feet above the canal bottom.

The important dates in the development of the Hennepin Canal were:
1834 - The idea of the Hennepin Canal as an extension of the Illinois and Michigan Canal was initiated.
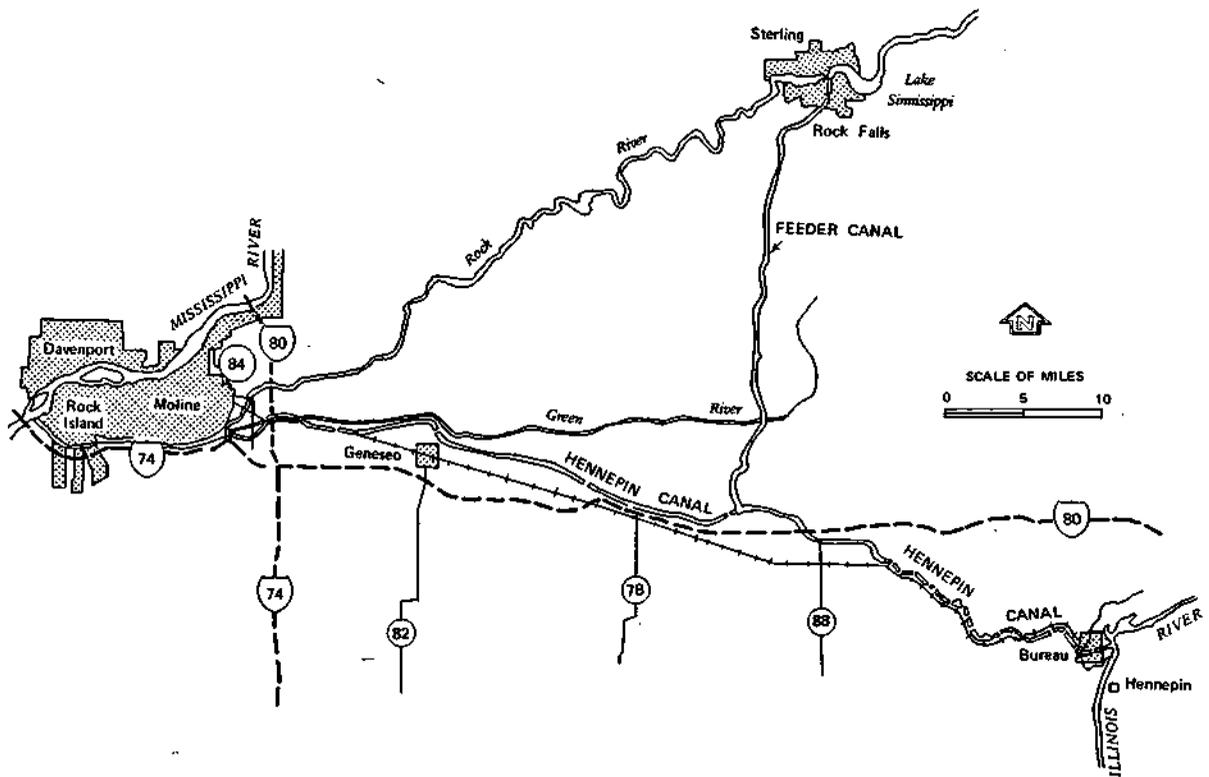1860 - The Chicago, Rock Island, and Pacific Railroad was constructed over the proposed canal route.

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Figure 1. Location of the Hennepin Canal

1866 - The first survey for a new canal route was made and a prelimi-
        nary design was proposed by a local citizens group.
1870 - COE made the first federal survey.
1881 - The Hennepin Canal Commission was established.
1882 - Another survey of the canal route was authorized.
1889 - The name of the canal was changed from the Hennepin Canal to
        the Illinois-Mississippi Canal.
1890 - Detailed plans were submitted to Congress by the COE for an
        estimated total cost of $6,925,960. Congress appropriated
        $500,000 for purchase of the right-of-way and for construction
        of the first five miles of the canal just above the mouth of
        the Rock River.
1907 - The canal was completed and opened for navigation. The final
        cost for the construction of the canal was $7,319,563.
1951 - Commercial navigation was terminated in the canal because of
        declining commercial use.
1970 - The COE gave the canal to the State of Illinois as a "gift."

FLOOD FLOW ANALYSES

This portion of the report analyzes the sources of flooding in the
vicinity of the Hennepin Canal and makes recommendations regarding the
proper canal levee elevations that will prevent future flood waters from
entering the canal. In the study area (from Lock 27 to the Rock River),
the major sources of flooding are the Rock and Green Rivers. The Rock
River primarily affects the Hennepin Canal through its backwater. The
Green River, however, runs almost parallel to the canal, and when it
overtops its banks there is always a possibility of floodwater entering
the canal. It should also be pointed out that in the study area the
Hennepin Canal is within the floodplains of both the Green and Rock
Rivers. Therefore, any extreme flooding in either of the rivers will in
some way impact the Hennepin Canal.

A detailed flood routing was done on the Green River from its conflu-
ence with the Rock River to Lock 27 where the Green River crosses the
Hennepin Canal. Flood elevations on the Rock River were obtained from

previous studies conducted for flood insurance purposes (Federal Emergency Management Agency, 1982).

Hydrologic Analyses

To determine the water surface elevations for floods of variable recurrence intervals, an analysis of the flood records of the streams affecting the area is required. As mentioned earlier, the major flooding sources are the Green and Rock Rivers. For the Green River the streamflow records at the Geneseo gage (USGS Gage No. 0544750) were utilized to determine the discharges for the 10-, 50-, and 100-year floods, which are expected to be equalled or exceeded once during any 10-, 50-, and 100-year period, respectively, on a long-term average. Their respective chances of being equalled or exceeded during any particular year are 10, 2 and 1 percent. It should always be emphasized that the recurrence intervals and their corresponding chances of being equalled or exceeded are based on long-term averages. It is conceivable to have two or more rare floods in any particular year or for rare floods not to occur for an extended period of time.

There are 46 years of streamflow data (1936 to 1981) for the Geneseo gage, which is about 8 miles upstream of the study area. The drainage area of the Green River upstream of the Geneseo gage is 1003 square miles, while its drainage area at its confluence with the Rock River is 1073 square miles. The Geneseo gage therefore represents 93 percent of the whole drainage basin of the Green River. The flood frequency analysis for the Green River was done according to the Guidelines for Determining Flood Frequency of the U.S. Water Resources Council. The flood frequency curve based on the 46 years of record is shown in figure 2, and the data used to develop the frequency curve are given in table 1. Also shown in figure 2 are the magnitudes of the floods of 1978, 1979, and 1981. The 1979 flood was the 2nd largest flood on record while the 1978 and 1981 floods were the 8th and 11th largest floods, respectively. Within the 4-year period from 1978 to 1981, the Green River had 3 of the largest 11 floods on record. It is therefore understandable that the flooding problems in the area have been acute in recent years.
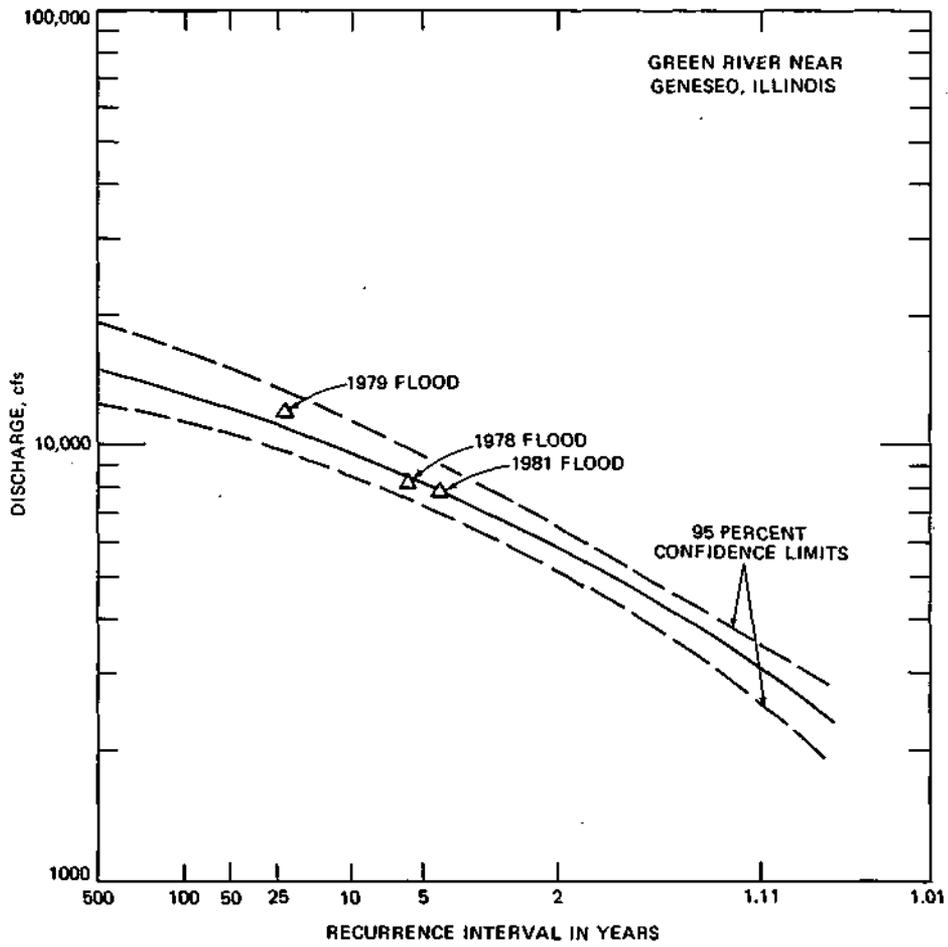
Figure 2. Flood frequency curve for the Green River near Geneseo

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Table 1. Ranked Streamflow Record for Flood Frequency Analysis

Rank	Discharge (cfs)	Water Year	Rank	Discharge (cfs)	Water Year
1	12,100	1974'	24	5,910	1965
2	12,000	1979*	25	5,890	1943
3	10,900	1973	26	5,590	1950
4	9,820	1970	27	5,300	1937
5	9,000	1971	28	5,240	1942
6	8,900	1955	29	5,240	1966
7	8,500	1960	30	5,230	1952
8	8,100	1978*	31	5,000	1936
9	8,050	1946	32	4,930	1945
10	8,000	1969	33	4,690	1961
11	7,870	1981*	34	4,500	1967
12	7,600	1972	35	4,350	1941
13	7,100	1959	36	3,970	1953
14	7,100	1962	37	3,790	1956
15	6,940	1951	38	3,750	1940
16	6,850	1938	39	3,600	1963
17	6,740	1976	40	3,320	1958
18	6,730	1944	41	2,630	1954
19	6,670	1948	42	2,500	1980
20	6,400	1947	43	2,380	1964
21	6,310	1949	44	2,300	1977
22	6,180	1939	45	1,690	1968
23	6,160	1975	46	1,340	1957 .

* High floods since 1978

The 10-, 50-, and 100-year flood discharges of the Green River near Geneseo, determined from the frequency curve shown in figure 2, are given in table 2.

Table 2. Flood Discharges for the Green River near Geneseo

Recurrence Interval (years)	Discharges (cfs)
10	9,600
50	12,200
100	13,100

As indicated previously, detailed hydrologic analyses of the Rock River were done by the C0E, Rock Island District, for a flood insurance study of the unincorporated areas of Rock Island County, Illinois (Federal Emergency Management Agency, 1982). Since the study was completed in 1982, the Rock River flood elevations were taken from that report without further analysis.

Hydraulic Analyses

The hydraulic analyses in this section of the report concern the flood water elevations along the Green and Rock Rivers. Initially a brief background on the hydraulics of open channel flow is presented, in which the basic equations and the assumptions involved in deriving the equations are discussed. The calculation techniques used in the HEC-2 computer program, which is used to calculate water surface profiles, are discussed next.

The water surface elevations in the Green and Rock Rivers for the 10-, 50-, and 100-year floods are presented, and the flood water elevations along the Hennepin Canal are then determined by investigating the flood elevations along both the Green and Rock Rivers. On the basis of the flood elevations, recommendations are made as to the proper levee elevations along the Hennepin Canal to prevent Green River flood water from entering the canal.

Hydraulics of Open Channel Flow

The basic equation used in hydraulic analysis is the one-dimensional energy balance equation. The energy equation for open channel flow (see figure 3 for an illustration of the terms) is given as follows (Chow, 1959):

$$z_1 + y_1 + \alpha_1 \frac{V^2_1}{2g} = z_2 + y_2 + \alpha_2 \frac{V^2_2}{2g} + h_L \tag{1}$$

where

    z = the elevation of the channel bottom above the datum
    y = the depth of water
    a = the energy correction coefficient for non-uniform velocity
        distribution across a cross section
    $V = Q/A$, the average velocity, where Q = total discharge and
        A = cross-sectional area
    g = gravitational acceleration
    h_L = energy loss between cross sections 1 and 2

The subscripts 1 and 2 refer to cross sections 1 and 2, respectively.

The energy loss, h_L, is calculated by:

$$h_L = \overline{S_f}(\Delta x) \tag{2}$$

$\overline{S_f}$, as shown in figure 3, is the average friction slope for the reach, and Ax is the distance between the two cross sections.

The friction slope, $\overline{S_f}$, at a cross section is computed from Manning's equation for uniform flow as follows:

$$\overline{S_f} = \left(\frac{Q}{\frac{C}{n} AR^{2/3}}\right)^2 \tag{3}$$

where

    C = 1.486 for English units and 1.0 for metric units
    R = hydraulic radius
    n = Manning's roughness coefficient
    Q and A are as defined before

The roughness coefficient, n, includes effects of cross-sectional size and shape, bed form and roughness, and river planform or horizontal alignment. Ideally, discharge, area, wetted perimeter, and energy slope are measured directly and n is computed from the data. Practically, the data are rarely sufficient to compute n, so hydraulic computations and engineering judgment are generally used to determine n values. Guides based on experience and measurement have been compiled to assist in the selection of Manning's n. Typically these guides include photographs and verbal descriptions of channels with n values of certain magnitudes. A
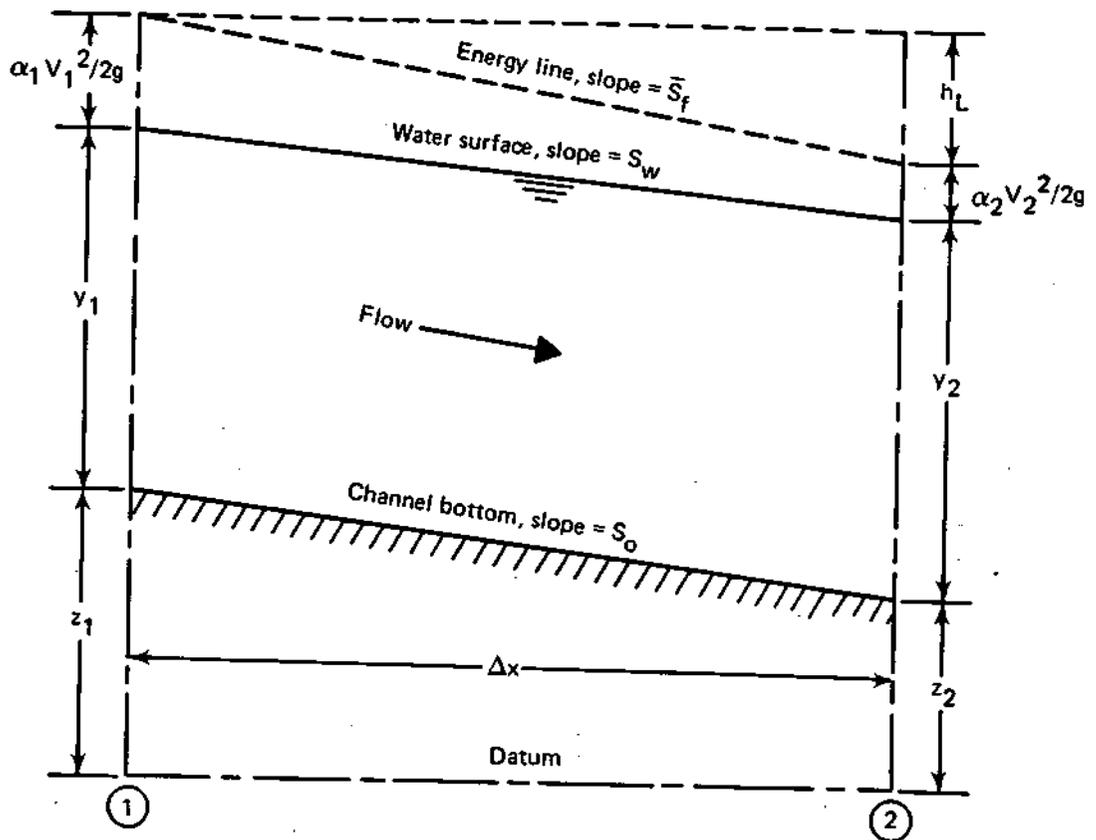
Figure 3. Schematics of open channel flow

classic text (Chow, 1959) contains such a guide. More recently the U.S.
Geological Survey has published a manual, <u>Roughness Characteristics of
Natural Streams</u> (Barnes, 1967), which includes similar information for
streams in all parts of the United States.

HEC-2 Water Surface Profile Calculation Technique

     The water surface elevations along the Green River for different
discharges were computed using the U.S. Army Corps of Engineers' Water
Surface Profile computer program (HEC-2) (Hydrologic Engineering Center,
1979). The HEC-2 program in its present form was developed by the U.S.
Army Corps of Engineers at the Hydrologic Engineering Center at Davis,
California (Thomas, 1975). The HEC-2 program calculates water surface
elevations along a stream channel for given discharges, cross-sectional
areas, and Manning's roughness coefficients. The flow along the river is
assumed to be steady; therefore the program does not deal with changes of
depth, discharge, and velocity in time. A flood wave, which changes
rapidly in time, cannot be simulated by HEC-2. The other basic assump-
tions are that the flow is one-dimensional and gradually varies in space
and that the channel slope is very mild. By assuming gradually varied
flow, it is possible to use uniform flow equations to compute energy
losses between successive sections (Chow, 1959; Feldman, 1981).

     The HEC-2 program computes water surface elevations along a stream
channel by using equations 1, 2, and 3. The solution technique is the
standard step method as outlined by Chow (1959). The standard step method
is a trial and error technique for determining the water surface elevation
at a cross section, provided that all the variables are known for the
preceding cross section. It starts the computation with known water
surface elevation, discharge, and cross-sectional area at the first cross
section, defined as the control point. The total energy at the first
cross section, represented by the left-hand side of equation 1, is then
calculated from the known values. The depth of water at the next cross
section, y_2 in equation 1, is given a trial value. From the given cross- *
sectional data and the assumed y_2, the variables A and R for the next
cross section are then computed.

     From the computed values of A and R and the given values of Q and n,
the friction slope at that cross section is computed by equation 3. The

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average friction slope for the reach, $\overline{S_f}$, can be calculated in several different ways, but the arithmetic mean of the slopes for the preceding cross section and the one computed for the present cross section usually provide an adequate approximation. The energy head loss due to friction is then computed with equation 2. Additional energy losses due to contraction and expansion of the channel are also added to the friction head loss.

The computed water surface elevation at the present cross section is then found by adding the difference in velocity head between the two cross sections and the total frictional and other head losses to the water surface elevation at the preceding cross section. From equation 1, the water surface elevation at the present cross section, $Z_2 + y_2$, is given by:

$$(z_2 + y_2) = (z_1 + y_1) + \left(\alpha_1 \frac{V^2_1}{2g} - \alpha_2 \frac{V^2_2}{2g} \right) - h_L \tag{4}$$

If this computed water surface elevation is different from the estimated value, another estimate is made and the same procedure repeated until the difference between the computed and estimated water surface elevations is reduced to an acceptable value. Once the water surface elevation at that cross section is computed, the computation proceeds to the next reach. The newly computed values will now serve as the known values on the left-hand side of equation 1 for the next series of calculations.

The direction of the water surface profile calculations depends on the nature of the flow. If the flow in the channel is subcritical, the control point is located at the downstream end of the study reach and the calculation proceeds in the upstream direction. On the other hand, if the flow is supercritical, the control point is on the upstream end and the computation proceeds in the downstream direction.

In most natural streams the flow is generally subcritical. Thus the water surface profile computation starts at a downstream control point, such as a dam, a constricted cross section, or any gaging station where the water surface elevation for a given discharge is known, and then proceeds in the upstream direction.

The data required to run the HEC-2 program include channel geometry such as cross-sectional profiles and distance between cross sections,

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bridge profiles, and hydraulic characteristics such as Manning's roughness
coefficients. A rating curve at the control point is also desirable.

The HEC-2 program has a number of options and capabilities to deal
with different kinds of flow problems encountered in water surface profile
computations. It has different methods to simulate flow under and over
bridges, it can map areas inundated by different frequency floods, and it
can simulate floodplain encroachments and channel improvements. Detailed
descriptions of these capabilities and options are found in several
publications (Feldman, 1981; HEC, 1979; Thomas, 1975).

Flood Elevations in the Green and Rock Rivers

The HEC-2 program was used to calculate the water surface elevations
in the Green River from its confluence with the Rock River to Lock 27 on
the Hennepin Canal, where the Green River crosses the canal. The stream
length of the Green River in the study reach is 6.3 miles. A topographic
map of the study area is shown in figure 4. A total of 34 cross sections
were used to run the HEC-2 program. Most of the cross-sectional data were
obtained from the Rock Island District of the Corps of Engineers. Addi-
tional cross sections were surveyed by the Water Survey staff for use in
this project. Values of Manning's roughness coefficient, n, were esti-
mated from field inspections and aerial photos. Manning's n for the main
channel ranged from 0.03 to 0.04, while for the floodplain it varied from
0.01 to 0.10.

To calculate the water surface elevations in the Green River it is
necessary to know the stage of the Rock River at the mouth of the Green '
River. The stages in the Rock River around the vicinity of the mouth of
the Green River for the 10-, 50-, and 100-year floods are shown in
figure 5.

For the 100-year flood calculations in the Green River, the Rock
River was assumed to be at the" 50-year flood stage, while for 50- and
10-year flood calculations the Rock River was assumed to be at the 10-year
flood stage. Since there is no reliable way of determining at which flood
stages the Rock River will be when the Green River discharge is at the
10-, 50-, or 100-year-flood magnitude, the rationale used in making the
above selections is the difference in the drainage areas between the Rock
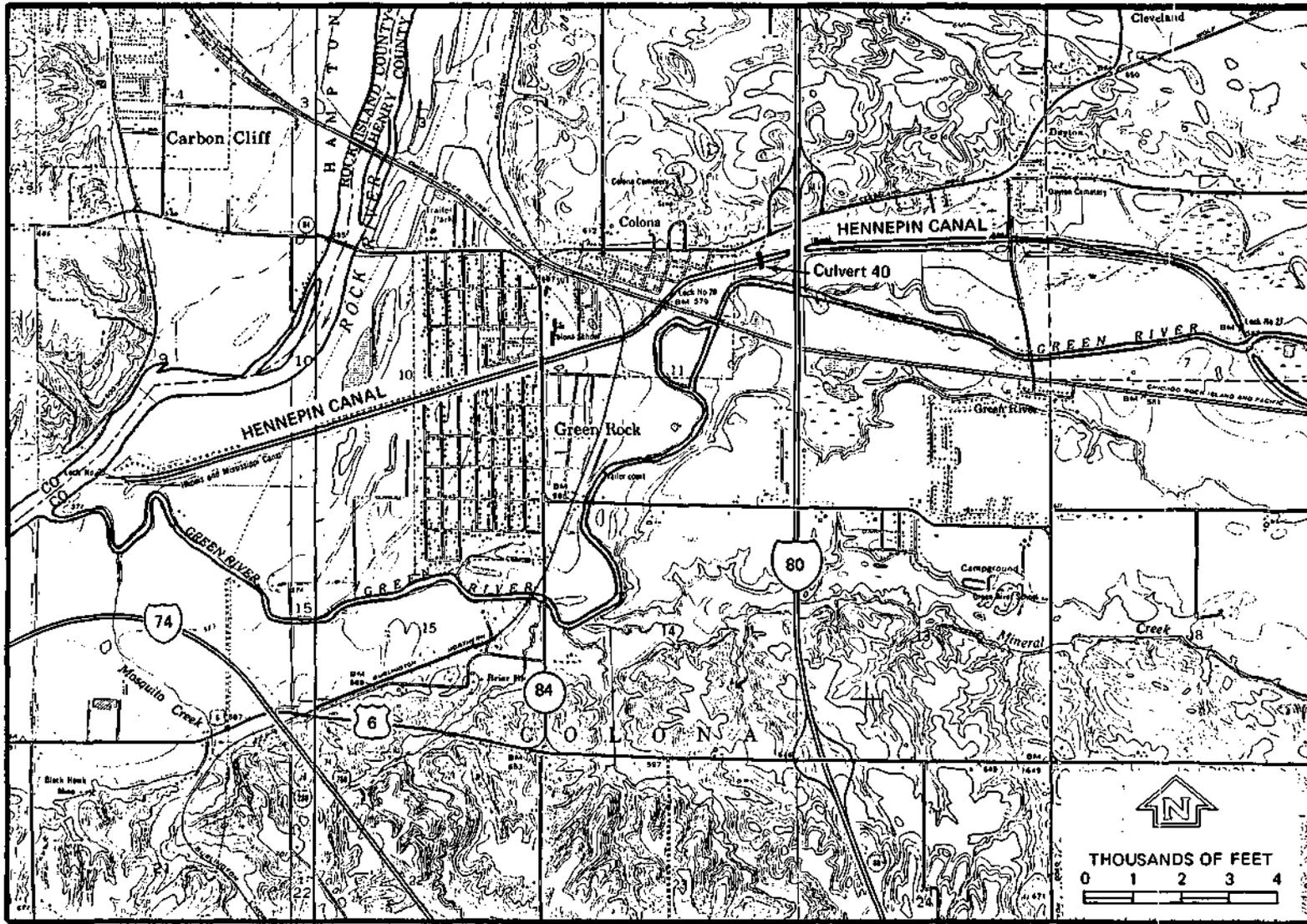and Green Rivers. Since the drainage area of the Rock River upstream of
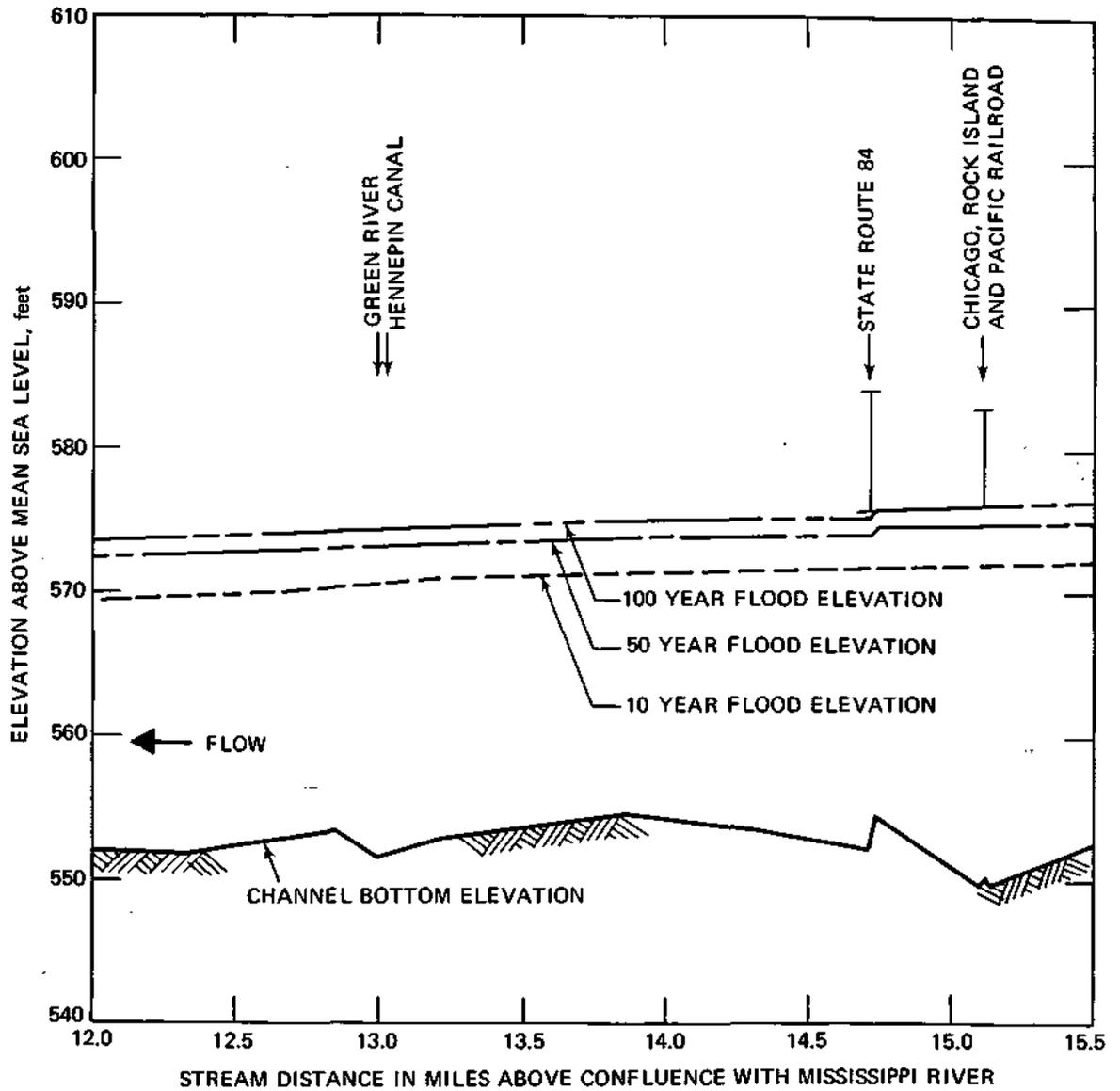
Figure 4. Location of study area

Figure 5. Flood elevations in the Rock River

the Green River mouth is about ten times greater than the drainage area of the Green River, it is reasonable not to expect floods of the same frequency to take place in both streams at the same time, except on extremely-rare occasions. Figure 6 illustrates the influence of 10-, 50-, and 100-year flood stages in the Rock River on the 100-year flood elevations in the Green River. As shown in the figure, the assumed stages in the Rock River affect the flood elevations in the lower 2-1/2 miles of the Green River, while the influence is almost unnoticeable further upstream.

The water surface elevations for the 10-, 50-, and 100-year floods along the Green River calculated under the aforementioned assumptions are shown in figure 7. Also shown in the figure are the channel bed and bank elevations.

As shown in figure 7, the Green River overtops its banks for the floods considered here. The areas flooded by the 100-year floods in the Green and Rock Rivers are shown in figure 8. Since most of the Hennepin Canal is within the Green River floodplain, it will be impacted whenever the Green River overtops its banks. If it is desired to keep the flood waters of the Green River from entering the canal, then the south canal levee has to be built up above the flood elevation in the Green River.

Flood Elevations along the Hennepin Canal

The water surface elevations along the Hennepin Canal during a 100-year flood in the Green River are shown in figure 9. Also shown in the figure are the present canal bottom and levee elevations, as well as a design levee elevation for the canal used by the COE in 1935 to build up the canal levees upstream of Lock 28. The design elevation of 585.7 ft was most probably used for the north levee, which is generally higher than the south levee.

There are breaks in the canal at two locations just downstream of I-80 and the Burlington Northern Railroad bridge, as shown by the low levee elevations in figure 9. At the present time, if the Green River overtops its banks, the flood waters can enter the canal through the canal breaks. Furthermore, if the water level rises in the canal from a storm runoff around the I-80 interchange, some of the water will flow into the Green River through the break west of I-80 and some will flood the area east of the Burlington Northern Railroad bridge through the break at that
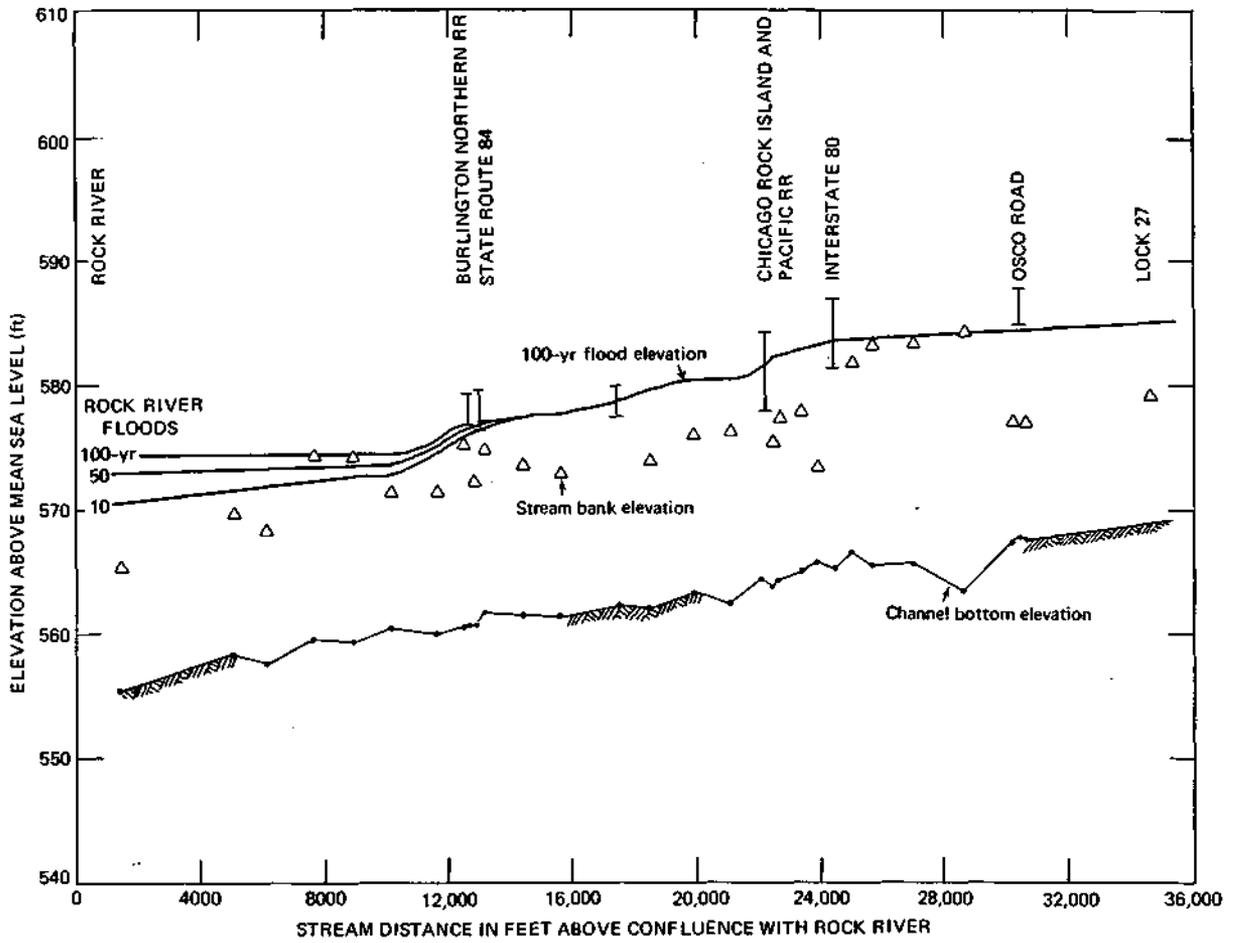
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Figure 6. Influence of Rock River flood stages on Green River flood elevations
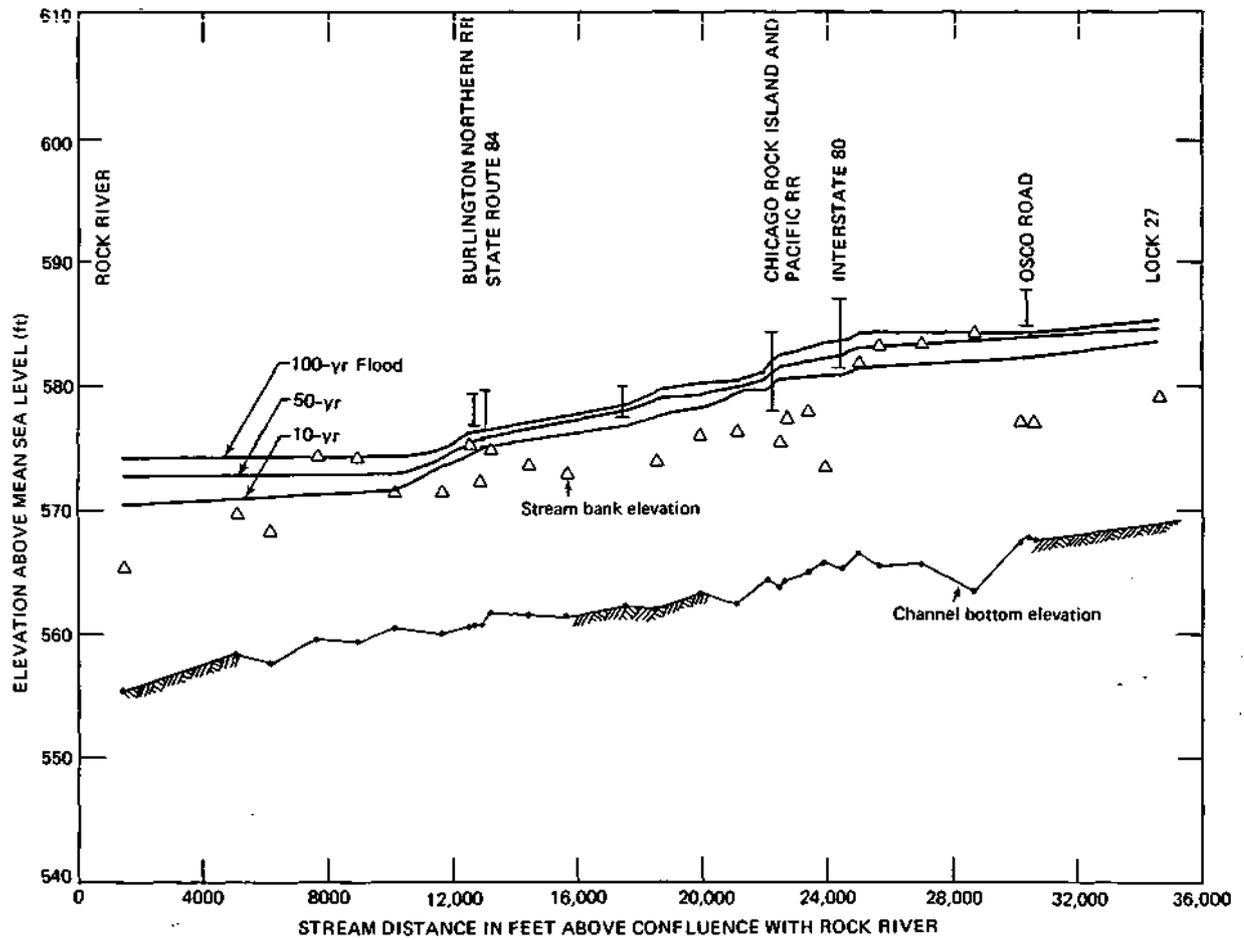
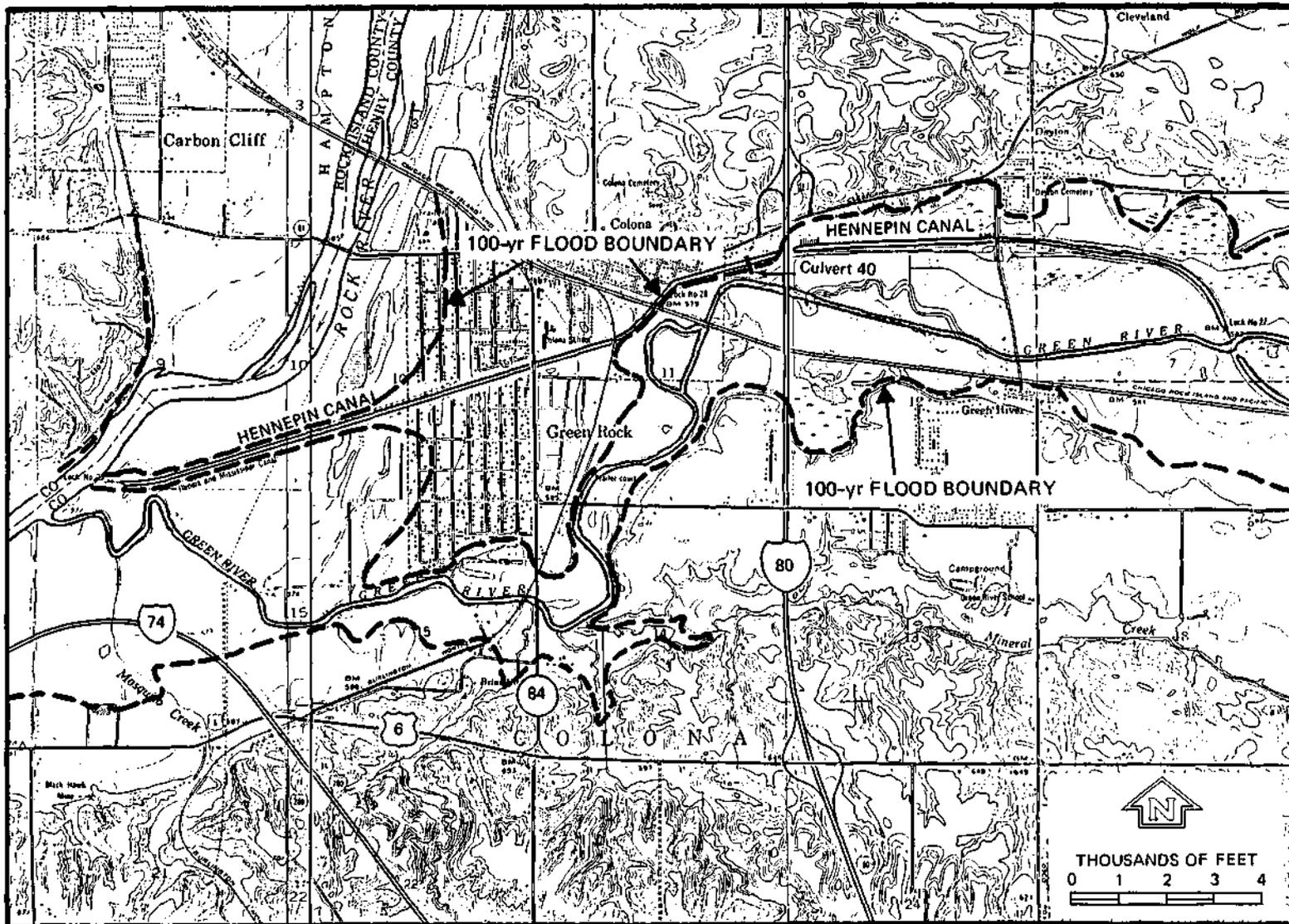*Figure 7. Flood elevations in the Green River*

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Figure 8. Areas flooded by the 100-year floods in the Green and Book Rivers
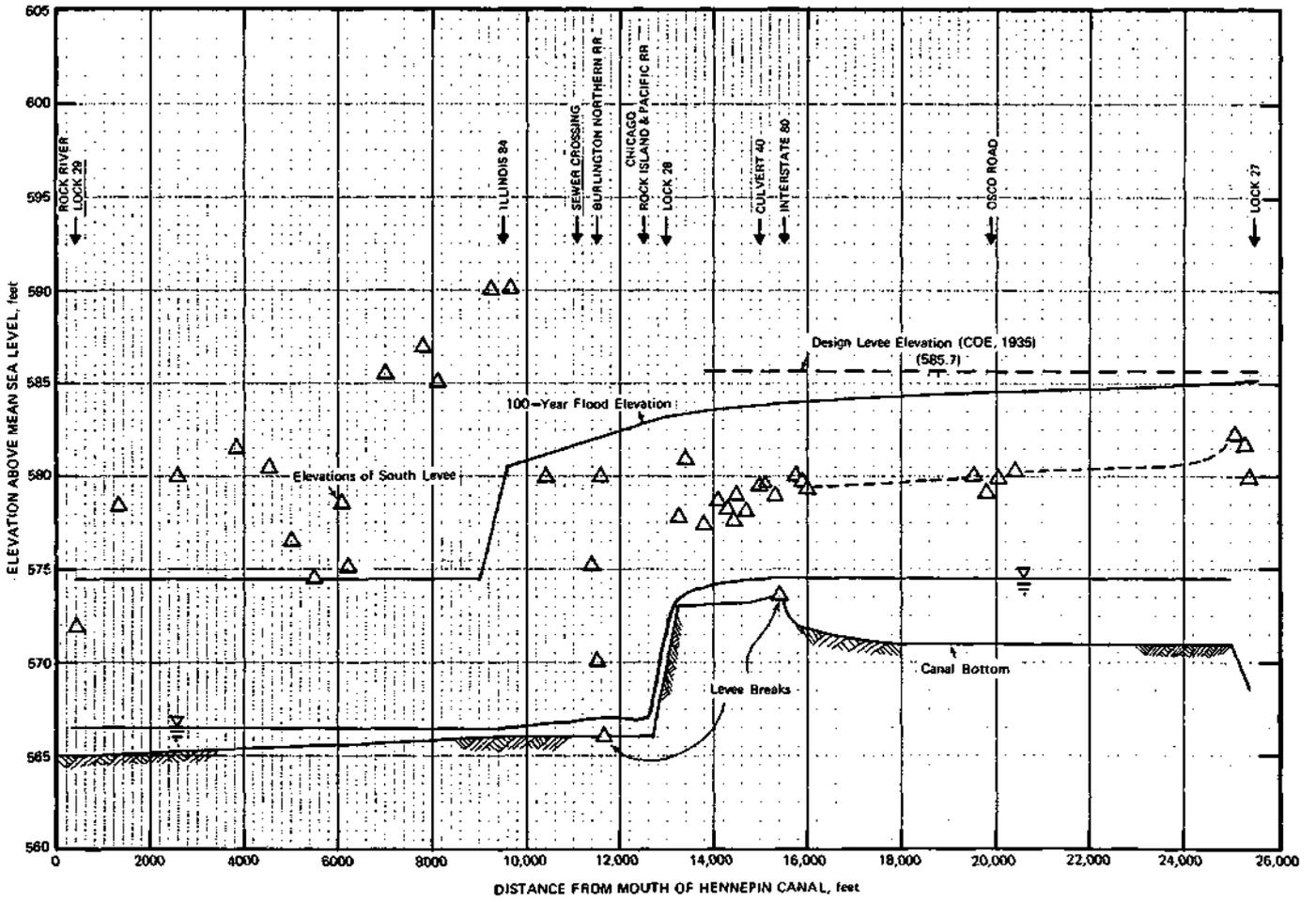
Figure 9. Flood elevations along the Hennepin Canal during a 100-year flood in the Green River

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site. Under the present levee conditions the Green River and Hennepin Canal are interconnected during high flows for all practical purposes.

Levee Elevations along the Hennepin Canal

     The existing levee elevations on the south side of the Hennepin Canal from Lock 27 to the Rock River are shown in figure 9. The locations of the two levee breaks, one just west of I-80 and one at the Burlington Northern Railroad, are also indicated in the figure. If it is desired to keep Green River flood water from entering the Hennepin Canal, the levee breaks at the two sites have to be repaired and the south levee elevations raised above the flood water levels in the Green River. For example, if it is decided to keep the 100-year flood in the Green River out of the canal, the south levee has to be raised 2 to 3 feet above the 100-year flood elevations along the Hennepin Canal shown in figure 9. However, the decision on the levee elevation should depend on the choice of the alter- natives concerning Culvert 40. If no water from the watershed around the I-80 interchange is allowed to enter the canal, it might not be necessary to raise the levee elevations above the Rock River backwater elevations. If drainage water from the I-80 interchange is allowed to enter the canal, it might be advisable to raise the south levee so the canal is isolated from the Green River. The different alternatives will be discussed in more detail later in this report.

CAPACITY ANALYSIS OF CULVERT 40

Location and Present Condition of Culvert 40

     Culvert 40 is located approximately 500 feet west of I-80 under the Hennepin Canal, as shown in figure 10. The original design of Culvert 40 is shown in figure 11. The culvert is an inverted siphon under the canal and is totally below ground level. Since this kind of culvert design is highly susceptible to sedimentation, a flushing mechanism was built on the north side of the canal, which allowed water from the canal to flow through the culvert and clean the sediment in the pipe. During the early periods of the canal, the water elevation was maintained at 577 ft MSL for navigation, so enough velocities may have been generated to clean the culvert.
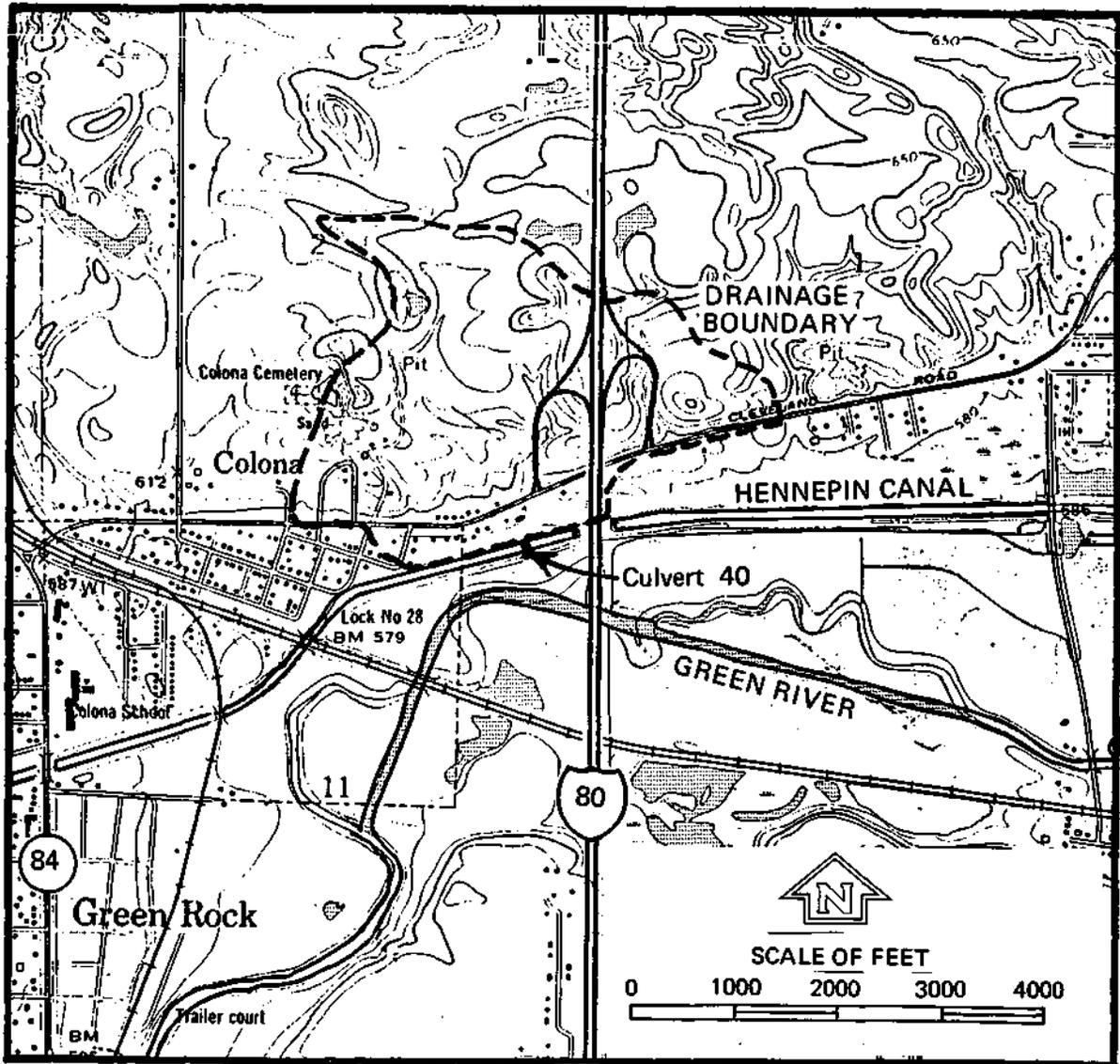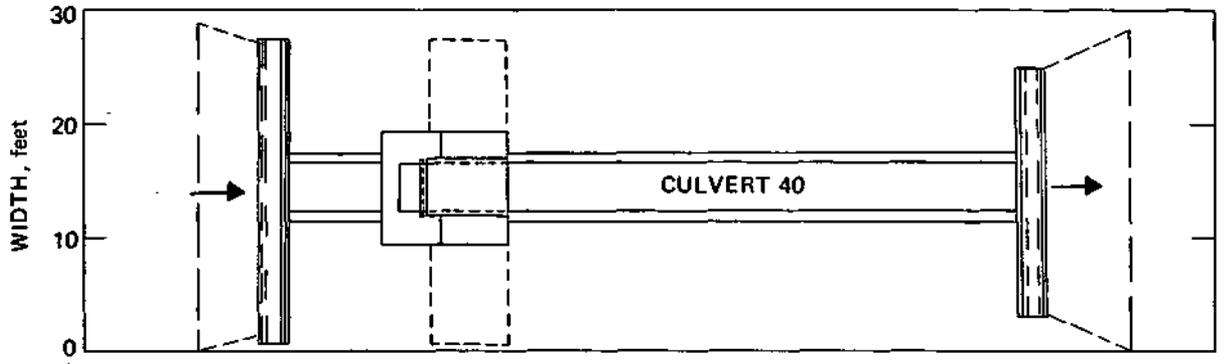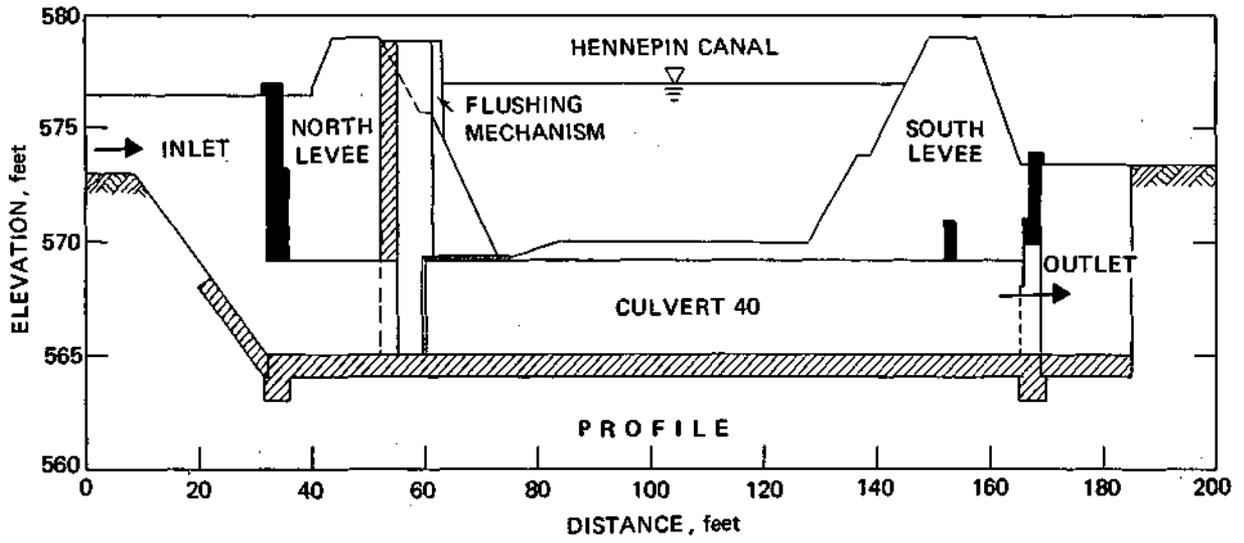
Figure 10. Location of Culvert 40 and its watershed

Figure   11.   Design   drawing   of   Culvert   40

At the present time, the culvert is totally silted up. The inlet and
outlet are also covered with sediment and totally invisible from the sur-
face, and the flushing mechanism is silted up and missing some parts. The
culvert will require major cleaning and repair to become operational again
and will require continuous maintenance to remain operative. Since the
culvert is only 400 feet from the Green River, it will experience sedimen-
tation whenever the Green River overtops its banks. The profiles of both
Culvert 40 and the Green River are shown in figure 12 to illustrate that
flood waters from the Green River will freely enter Culvert 40. As a
matter of fact, the invert of Culvert 40 is below the Green River's
channel bed. All factors considered, Culvert 40 is in very poor shape and
even if it is repaired and placed in operation it will continue to have
excessive sedimentation problems.

Surface Runoff Calculations for the Culvert 40 Watershed

The location of the Culvert 40 watershed is shown in figure 10. It
consists of the area north of the Hennepin Canal on both the west and east
side of I-80. Drainage channels and culverts along the I-80 interchange
and Cleveland Road drain the watershed towards Culvert 40. The total area
of the watershed is 200 acres.

The surface runoff to Culvert 40 from this watershed was calculated
using the Rational Equation (Rouse, 1950). Since the drainage area is
only 200 acres, it was felt that the Rational Equation was adequate and
that it was unnecessary to use the more elaborate runoff computation
methods.

The Rational Equation is given as

$$Q = C \, i \, A \tag{5}$$

where

Q = the rate of runoff in acre-inches per hour or cubic feet per
second (1 acre-inch per hour = 1.008 cubic feet per second)
C = the runoff coefficient, which is the ratio of peak runoff to
average rainfall
i = the average rainfall intensity in inches per hour
A = the drainage area in acres

The basic assumption in the Rational Method (based on the Rational
Equation) is that, for small drainage basins, the maximum rate of surface
runoff at a point of interest occurs when the entire drainage basin
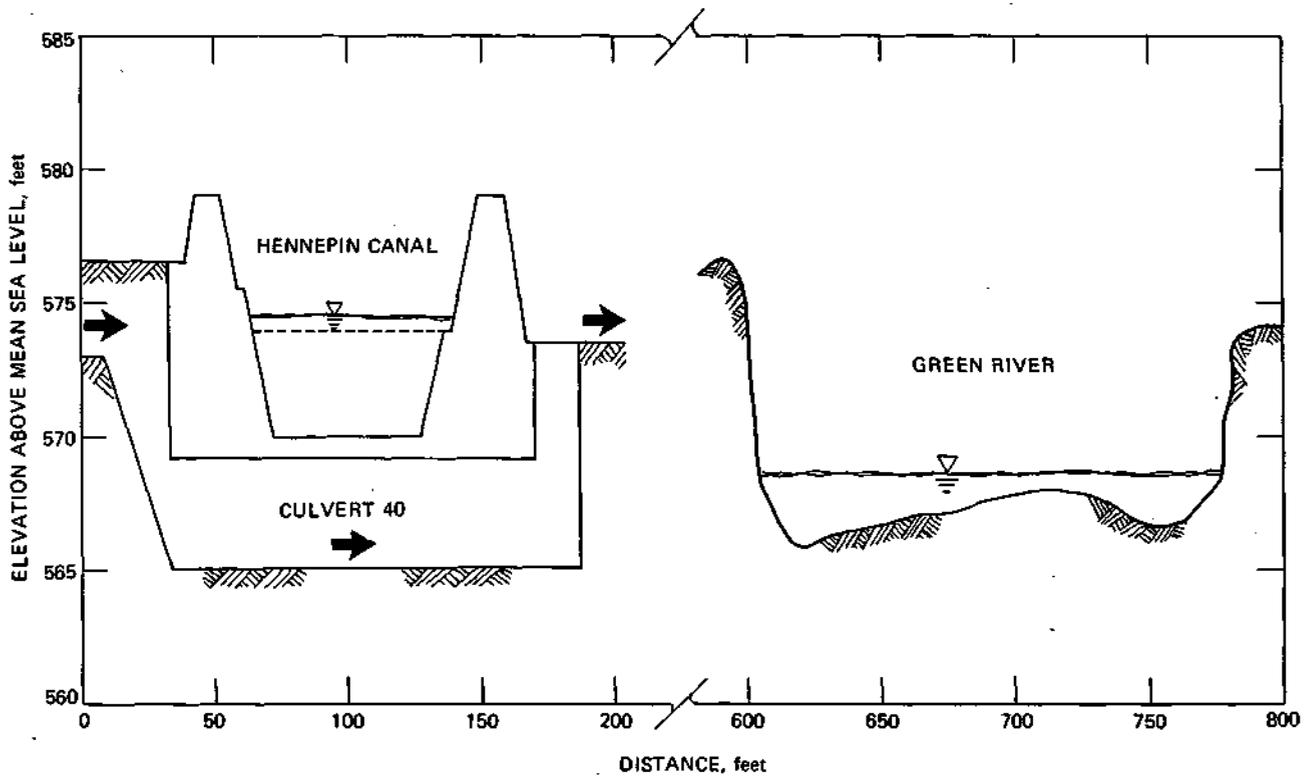upstream of that point is contributing to the runoff measured at that

Figure 12.   Relative elevations of Culvert 40 and the Green River

point. The maximum rate of runoff, usually known as the peak discharge, is then equal to a percentage, C, of the average rate of rainfall.

The drainage area of the basin, A, is generally determined from topographic maps or any other maps available. The runoff coefficient, C, is estimated on the basis of the surface cover, soil type, and topographic features of the drainage basin, especially the slope. The rainfall intensity, i, for different frequencies is determined from frequency analyses of rainfall records from nearby rain gages. Before the rainfall intensity is selected, however, the duration of the rainfall has to be selected. For the same frequency, the rainfall intensities are different for different durations.

In the Rational Method of peak discharge calculations, it is gener-ally assumed that the duration of the rainfall producing the peak dis-charge is equal to the time of concentration. The time of concentration, T_c, is defined as the time required for water to flow from the most remote part of the watershed to the point of interest. In certain cases the time of concentration might be defined differently if it is felt that the peak flow occurs due to runoff from a portion of the watershed close to the point of interest. In this case, assuming a shorter time of concentration might provide the appropriate peak discharges. One method of determining the time of concentration for non-urban areas is to use the following relationship between the time of concentration and a watershed factor based on the length and slope of the watershed (Rouse, 1950):

$$T_c = 0.0078 \ K^{0.770} \tag{6}$$

where

T_c = time of concentration in minutes
$K = L/\sqrt{s}$ = watershed factor
L = the maximum length of travel in feet
s = the slope of the watershed
 = H/L
H = the difference in elevation between the most remote point and the point of interest in feet

For the watershed draining to Culvert 40, the following parameters were measured or selected for calculating the peak discharge at the culvert following the discussion above:

Drainage area, A = 200 acres
Runoff coefficient, C = 0.45
Time of concentration, T_c = 20 minutes

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The 20-minute rainfall intensities for various frequencies are shown in table 3. The rainfall intensities were calculated from Illinois rainfall frequencies prepared by the Illinois State Water Survey (1970). The assumption was that the duration of the rainfall was equal to the time of concentration, which is 20 minutes.

Table 3. Rainfall Intensities for 20-Minute Durations
and Variable Recurrence Intervals

Recurrence interval (years)	Rainfall intensity, i (inches/hour)
2	2.40
5	3.15
10	3.75
25	−4.80

It is now possible to use the Rational Equation to calculate the different peak discharges generated by rainstorms of different frequencies for the Culvert 40 watershed. It should be pointed out that the frequencies of the runoff calculated by the Rational Equation do not necessarily equal those of the storms. This has been one of the major shortcomings of the Rational Method. However, the rainfall frequencies should provide a very good estimate of the runoff frequencies. Furthermore, if it is understood that the frequencies are for the rainfall and not for the runoff, the differences between the rainfall and runoff frequencies become unimportant.

The Rational Equation (equation 5) reduces to the following after the numerical values for C and A are substituted:

$$Q = (0.45 \times 200)\ i\ \ cfs$$
$$= (90)\ i\ \ cfs$$

The peak discharges generated by the rainfall intensities given in table 3 are shown in table 4.

Table 4. Peak Discharges Generated by 20 Minute Rainfall
of Different Frequencies

Rainfall recurrence interval (years)	Peak discharges (cfs)
2	216
5	284
10	338
25	432

Hydraulic Analysis of Culvert 40

After determination of the peak discharges given in table 4, the next
analysis is to determine if Culvert 40 has the capacity to carry these
discharges from the north side to the south side of the canal. It is
probably inappropriate to consider recurrence intervals greater than 25
years, since the failure to carry very rare discharges from the 50- and
100-year storms will not result in major economic losses or endanger the
lives of people in the surrounding areas. Therefore, Culvert 40 will be
evaluated only with regard to whether it can carry the runoff from the 2-,
5-, 10-, and 25-year storms.

In calculating the flow through Culvert 40, the limiting factor is
the available energy head from the entrance to the outlet of the pipe.
Therefore, the calculations are carried out by assuming reasonable energy
head differences between the entrance and the outlet. Because of the
close proximity of the Green River, the water elevation in the river will
have a strong influence on the flow through the culvert. Unless a new
control mechanism is established on the south side of the culvert, Green
River water will back up through the culvert to the north side of the
canal when the Green River overtops its banks.

The maximum available head across the canal occurs when the water
elevation on the north side of the canal is near the top of the north
levee and the Green River is within its banks. The elevation of the low
points on the north levee around Culvert 40 is about 580 ft MSL. The
elevation of the small ditch from the south side of the canal to the Green
River is about 575 ft MSL. Therefore the maximum head available from the
inlet to the outlet of Culvert 40 without overtopping the north levee is
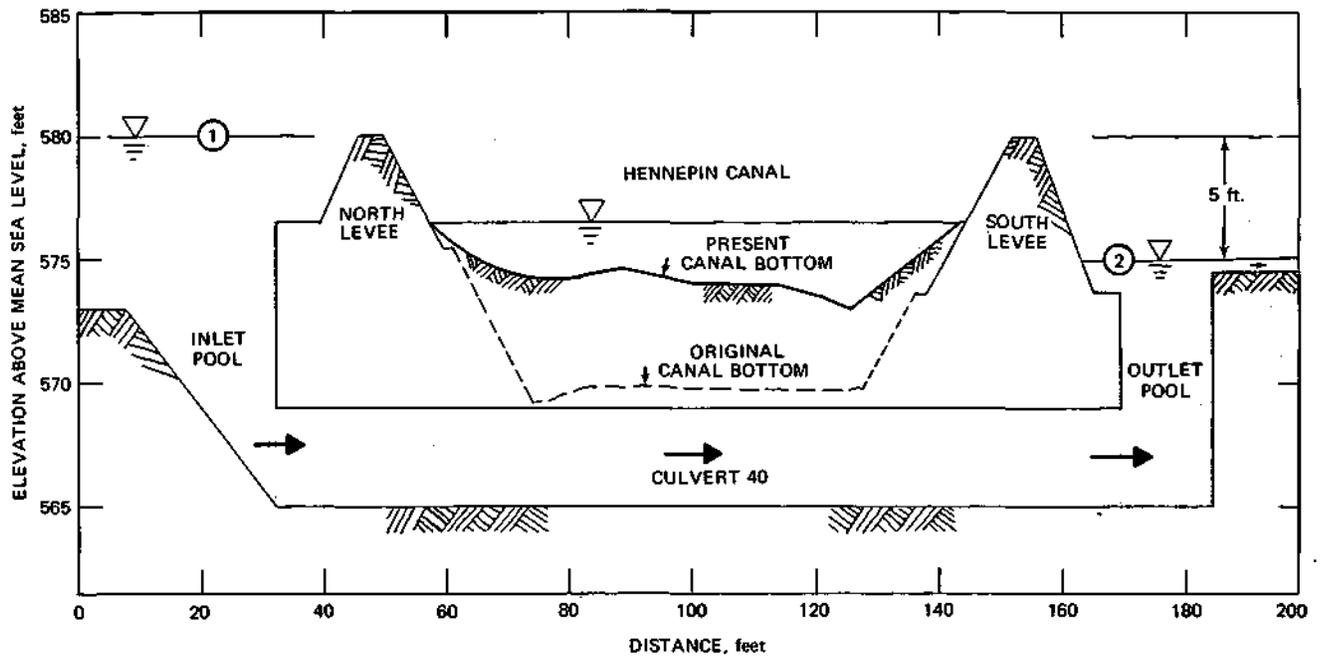5 ft (580-575), as shown in figure 13. The total head loss through

Figure 13.   Flow capacity of Culvert 40

Culvert 40 cannot be greater than 5 ft if the north levee is not to be overtopped.

The discharge through Culvert 40 can be calculated by using the one-dimensional energy equation (equation 1). Assuming the energy correction coefficient a = 1, equation 1 can be rewritten as follows:

$$\left(z + y + \frac{V^2}{2g} \right)_1 = \left(z + y + \frac{V^2}{2g} \right)_2 + h_L \tag{7}$$

All the terms are as defined previously. The energy head loss, 1 culvert flow is comprised of three components:

1) h_e = energy loss at the entrance of the pipe

2) h_f = energy loss due to friction in the pipe

3) h_o = energy loss at the outlet

Therefore, the total energy loss, h_L, is calculated as follows:

$$h_L = h_e + h_f + h_o \tag{8}$$

The energy losses at the entrance and outlet are generally expressed in terms of kinetic energy head as follows:

$$h_e = K_e \frac{V^2}{2g} \tag{9}$$

$$h_o = K_o \frac{V^2}{2g} \tag{10}$$

where V is the average velocity in the pipe and K_e and K_o are dimensionless energy head loss coefficients. K_e and K_o are determined from experimental results. K_e for square cornered entrances set flush with a wall is generally taken as 0.5. If the pipe is projecting out of the wall it will be higher. The outlet energy coefficient, K_o, is taken as 1.0 for most cases, since all the kinetic energy of the water from the pipe is dissipated around the pipe outlet.

The energy loss due to friction, hf, is determined by using either the Manning formula or the Darcy-Weisbach equation (Chow, 1959) for energy loss due to friction, which is given as:

$$h_f = f \frac{L}{D} \frac{V^2}{2g} \tag{11}$$

where

    f = the coefficient of friction
    L = the length of the pipe
    D = the diameter of the pipe

All other terms are as defined previously. The coefficient of friction, f, depends on the shape and roughness of the pipe and on the Reynolds number R. The Reynolds number is defined as follows:

$$R = VD/\nu \tag{12}$$

where ν is the kinematic viscosity of the water. The Reynolds number indicates the relative strength of the viscous and inertial forces. When the flow is highly turbulent the Reynolds number is large, indicating that viscous forces are not dominant in the flow. In this case the friction factor is almost independent of the Reynolds number. On the other hand, when the Reynolds number is small the flow is not turbulent (i.e., it is laminar flow), indicating that the viscous forces are dominant in the flow field. In this case the friction factor is inversely proportional to the Reynolds number.

In the analysis for Culvert 40, the flow through the culvert will be assumed to be fully turbulent, and thus the friction factor will depend only on the type and roughness of the pipe. Since the pipe for Culvert 40 is made of cast iron, the friction factor would be 0.019 if it were new. However, since the pipe is very old and has not been maintained properly, the friction factor will be higher than 0.019. The range of Manning's n values for dirty or tuberculated cast iron pipes is given as 0.015 to 0.035 (King and Brater, 1963). This range in n corresponds to a range of 0.026 to 0.14 in the friction factor, f, for a 4-foot diameter pipe. Since there is no accurate method of determining the friction factor for old pipes without field measurements, a value of 0.05 is selected on the basis of engineering judgment.

The total energy loss from the entrance to the outlet of Culvert 40 can then be calculated using the following equation:

$$h_L = 0.5 \frac{V^2}{2g} + 0.05 \frac{L}{D} \frac{V^2}{2g} + 1.0 \frac{V^2}{2g} \tag{13}$$

For Culvert 40, the length of the pipe, L, is 137 ft and the diameter, D, is 4 ft. Equation 13 then reduces to:

$$h_L = 3.21 \frac{V^2}{2g} \tag{14}$$

For a 5-ft head loss, the velocity through Culvert 40 can be calculated from equation 14 as follows:

$$V = \left[\left(\frac{5}{3.21}\right) \times 2g\right]^{1/2}$$

$$= 10.0 \text{ ft/sec}$$

The discharge through the culvert is then calculated from the relationship:

$$Q = VA \tag{15}$$

$$= 10 \text{ ft/sec} \times 2.57 \text{ ft}^2$$

$$= 126 \text{ cfs}$$

The maximum flow through Culvert 40 without overtopping the north levee is, therefore, 126 cfs. This flow is much less than 216 cfs, which is the overland flow generated by the 2-year storm. This means that if Culvert 40 is the only structure carrying water from the north side to the south side of the canal, the north canal levee will most probably be overtopped every year.

It should also be noted that the Green River was assumed not to overtop its banks. When the Green River overtops its banks, the available energy head from the inlet to the outlet of Culvert 40 will be less than 5 feet, resulting in discharges less than 126 cfs through the culvert. If the Green River overtops its banks by 5 feet there will be no flow through the culvert. Furthermore, when the Green River overtops its banks there will be a reverse flow through the culvert from the south side to the north side depending on the storm runoff from the I-80 interchange area. In the above analysis it is also assumed that Culvert 40 is cleaned and maintained properly. Excessive sedimentation will take place in the pipe during moderate storms which do not generate enough velocity in the pipe to clean the sediment. Accumulation of sediment and debris in the pipe and at the inlet will reduce the flow carrying capacity of the culvert significantly.

It can thus be concluded that Culvert 40 by itself is totally inade-quate to carry the overland flow from the 200 acres of drainage basin around the I-80 interchange near Colona across the canal. Assuming the maximum available head is 5 feet without overtopping the north levee, the diameter of the pipe at' Culvert 40 has to be at least 5 feet to carry the 216 cfs peak discharge from a 2-year storm. Again, the Green River is assumed to be within its banks during the peak discharge through

Culvert 40. If the 5- or 10-year storms are considered, the diameter
of the pipe that will carry the peak discharges will be still larger.

ALTERNATIVE SOLUTIONS

As discussed in the previous section, Culvert 40 by itself cannot
handle the runoff from the drainage area around the I-80 interchange near
Colona. In this section, several alternative solutions will be considered
and their advantages and disadvantages discussed.

Alternative 1. Do Nothing
If this alternative is chosen, the situation will continue as it is
now. The runoff from the I-80 interchange will continue to freely enter
the canal, with part of it moving downstream and part of it overflowing to
the Green River through the break in the south levee, as shown in
figure 14. At the present time the levee break in the south canal has
been fixed; however, it will break again during a major storm. This
alternative will also allow flood water from the Green River to enter the
canal.

There will be flooding in areas adjacent to the canal either during
Green River floods or during severe storms around the I-80 interchange.
The sedimentation problem in the canal west of I-80 will continue and
eventually cause the culverts at the Colona sewer crossing and at Route 84
to silt up. ' If this alternative is chosen, there is no benefit in raising
the levee elevations.

Alternative 2. Reactivate Culvert 40 with an Overflow Weir into the Canal
This alternative will involve cleaning and repairing Culvert 40 and
reconstructing the north levee with an overflow weir east of the culvert,
as shown in figure 15. The north levee from the I-80 embankment to
Culvert 40 will have to be rebuilt with proper material so it will not be
washed out when the water level on the north side is high.

With this alternative, Culvert 40 will carry the water across the
canal during low flows, and the combination of the overflow weir and
Culvert 40 will handle the runoff from heavy storms. As discussed in the
section dealing with the capacity of Culvert 40, the maximum discharge the
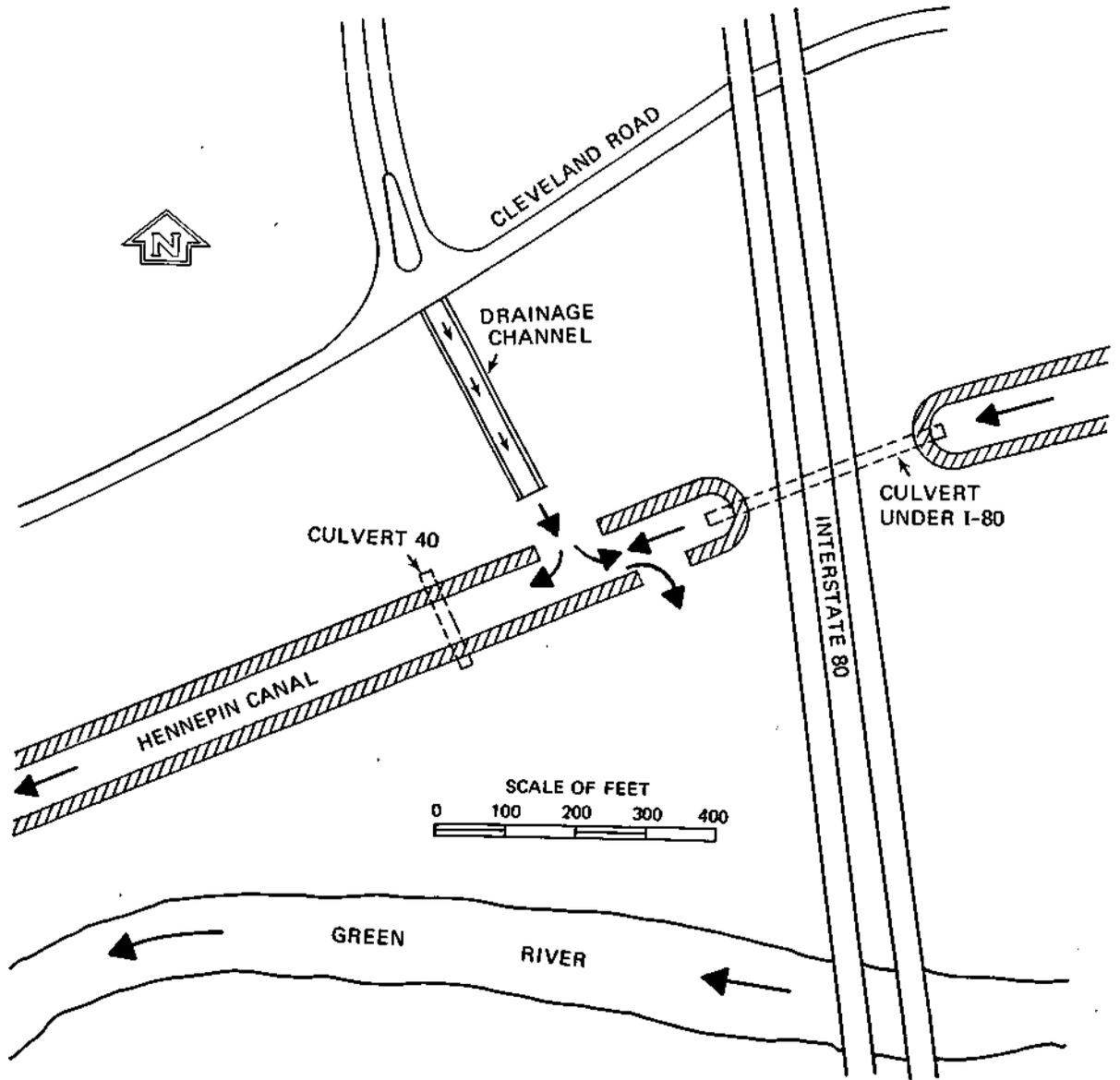
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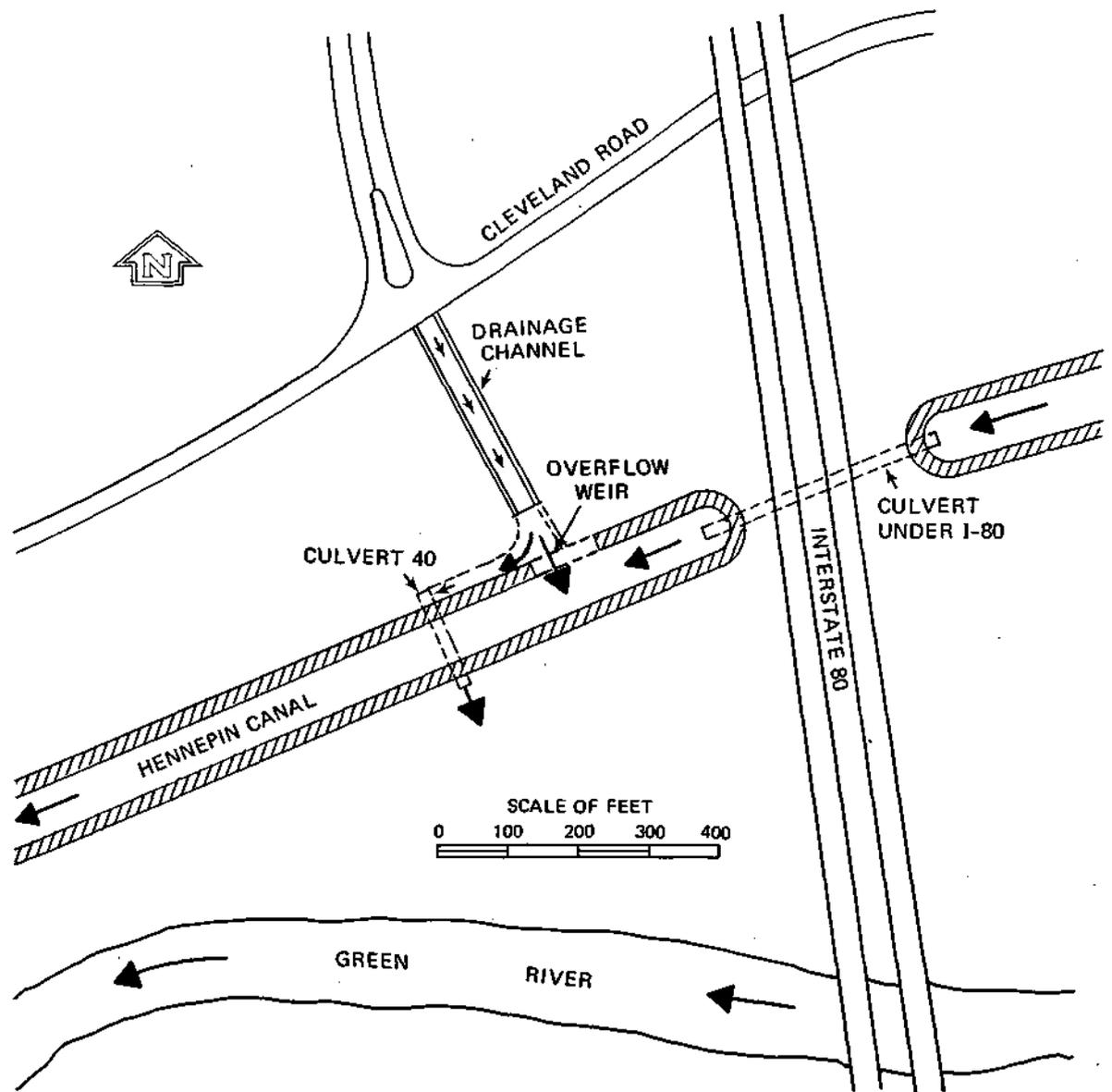*Figure   14.   Alternative   1   —   No   change*

Figure 15. Alternative 2 — Reactivation of Culvert 40 with an overflow weir into the canal

culvert can carry is 126 cfs if the Green River stays within its banks and the culvert is free of sediments. When these conditions are not met, the capacity of Culvert 40 is greatly reduced.

Assuming that the culvert is clean and the Green River is within its banks, the excess water entering the canal will be the runoff from the different storms minus the 126 cfs carried by Culvert 40. The division of water between the culvert and the canal for the 2-, 5-, 10-, and 25-year storms is given in table 5.

Table 5. Division of Storm Runoff Between Culvert 40 and Hennepin Canal

Rainfall recurrence interval (years)	Peak discharge (cfs)	Flow through Culvert 40 (cfs)	Flow into Hennepin Canal (cfs)
2	216	126	90
5	284	126	158
10	338	126	212
25	432	126	306

Once the water enters the canal, the next consideration is whether the culverts at the sewer crossing at Colona and at the State Route 84 bridge can carry the water downstream without creating high backwater elevation that can overtop the canal levees. There are also problems with continuing sedimentation and water quality in the canal that have to be considered when the storm runoff is allowed to enter the canal.

Capacity of Culvert at Sewer Crossing at Colona

The embankment in the canal and the location of the culvert at the Colona sewer crossing are shown in figure 16. During storm events around the I-80 interchange, the flow that should be handled by the culvert will be the same as the flow in the canal and the discharges given in table 5 for the 2-, 5-, 10-, and 25-year storms.

The maximum flow capacity of the culvert at the Colona sewer crossing without overtopping the fill in the canal is 147 cfs. This will be adequate to handle the runoff from storms with 4-year recurrence intervals or less. However, if the fill in the canal is allowed to be overtopped, and there is no obvious reason why it should not, then the total flow past the sewer crossing will consist of flow through the culvert and weir flow

CROSS SECTION VIEW

ELEVATION ABOVE MEAN SEA LEVEL, feet

590 / 580 / 570 / 560

Sewer Pipe

Top of Culvert

4 ft

DISTANCE, feet
0   20   40   60   80   100   120   140

PROFILE VIEW

ELEVATION ABOVE MEAN SEA LEVEL, feet

590 / 580 / 570 / 560

Over flow

Pipe flow

DISTANCE, feet
0   50   100   150   200   250   300   350
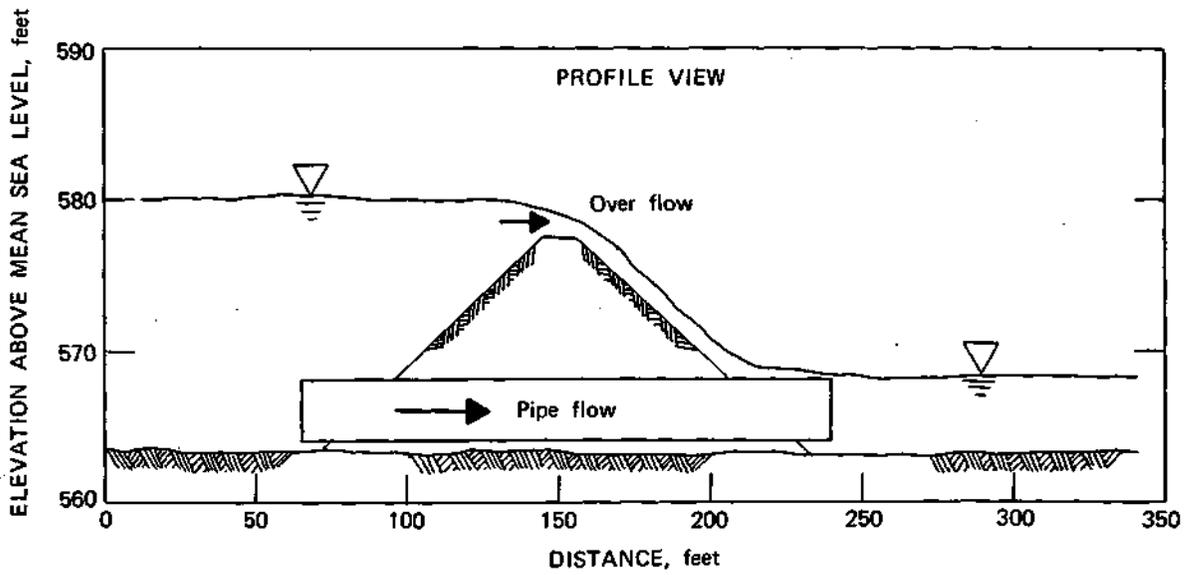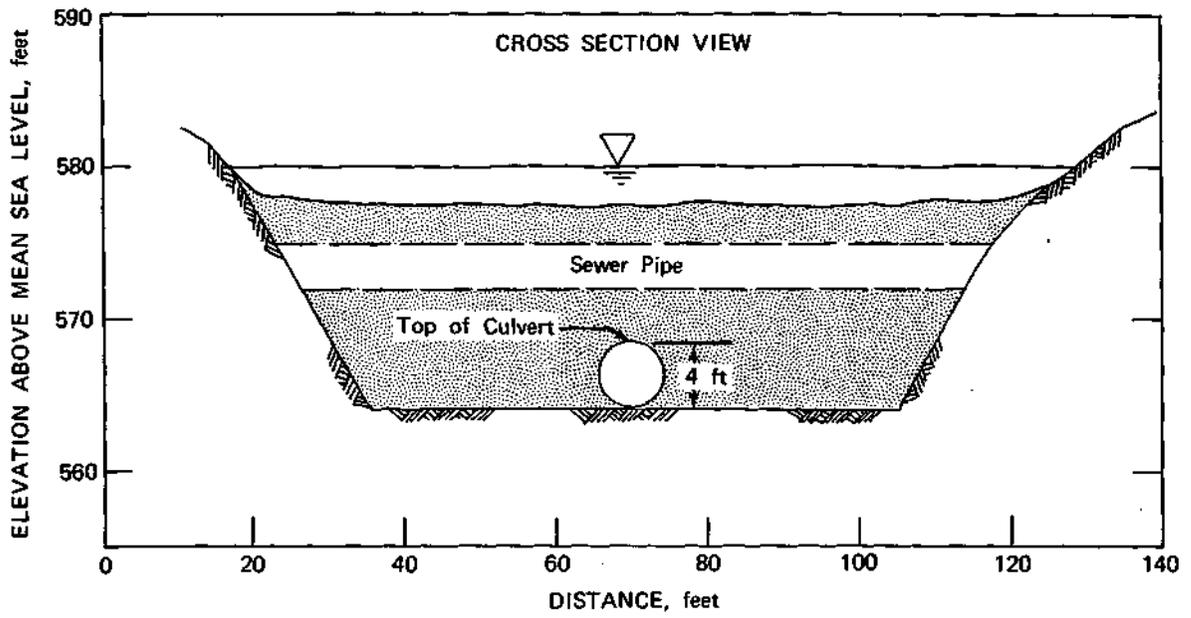
Figure   16.   Culvert   and   canal   fill   at   the   Colona   sewer   crossing

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over the sewer crossing. Assuming the storm runoff in the canal is backed
up by the sewer crossing to an elevation of 580 ft MSL, which will still
be within the canal levees, the combined flows through the culvert and
over the sewer crossing are calculated as follows.

For the culvert flow,

$$Q_p = V_p A_p$$

where the subscript p denotes pipe and Q_p, V_p, and A_p are the discharge
through the culvert, velocity in the culvert, and area of the culvert,
respectively. Assuming the culvert is a corrugated pipe 4 ft in diameter
and 175 ft long, the head loss through the culvert can be calculated as
follows:

$$h_L = 0.9 \frac{V^2}{2g} + 0.065 \left(\frac{175}{4}\right) \frac{V^2}{2g} + 1.0 \frac{V^2}{2g}$$

$$= 4.74 \frac{V^2}{2g}$$

The head loss coefficients K_e and K_o were selected as 0.9 and 1.0,
respectively. The friction coefficient, f, is assumed to be 0.065.

Assuming that the maximum available head is 11.5 ft (580-568.5), and
that it is equal to the total head loss, h_L, the velocity in the pipe will
be 12.5 fps. The culvert flow is then calculated by equation 15:

$$Q_p = 12.5 \times \frac{\pi(4)^2}{4} = 157 \text{ cfs}$$

The flow over a weir is given by the following equation:

$$Q_w = C L H^{3/2} \tag{16}$$

where Q_w is the flow over the weir, C is the discharge coefficient, and H
is the energy head of the weir. Assuming C = 3.6, the length of the weir
L = 100 ft, and the energy head H = 2.5 ft (580-577.5), the discharge over
the weir will be

$$Q_w = 3.6 \times 100 \times 2.5^{3/2} \text{ cfs}$$

$$= 1,423 \text{ cfs}$$

This discharge is much above what will be expected to flow in the Hennepin
Canal. Therefore, the sewer crossing at Colona will be able to handle
most of the discharges from the I-80 interchange if it is allowed to be
overtopped.

Capacity of Culvert under Illinois Route 84

The embankment in the canal and the location of the culvert at
Route 84 are shown in figure 17. The flow in the canal that has to be
handled by the culvert is the same as the flow in the canal during storm
events around the I-80 interchange. The discharges for the 2-, 5-, 10-,
and 25-year storms were given in table 5.

It is essential to first evaluate the maximum capacity of the culvert
under the most favorable conditions. The maximum available energy head at
the culvert will occur when the water in the canal is backed up to an
elevation of 580 ft MSL (which is the elevation of the levees in the
area), and the tailwater elevation (the water elevation at the downstream
end of the culvert) is at normal water elevation in the canal. Assuming
that the water elevation in the canal downstream of Route 84 is 567 ft
MSL, the maximum available energy head is 13 ft (580-567).

Assuming that the maximum available head is equal to the total head
loss, h_L, the velocity in the culvert is calculated from the head loss
equation as follows:

$$h_L = 13 \text{ ft} = 0.9 \frac{v^2}{2g} + 0.065 \left(\frac{L}{D}\right) \frac{v^2}{2g} + 1.0 \frac{v^2}{2g}$$

Since the culvert is a corrugated pipe 4 ft in diameter and 225 ft
long, the head loss equation reduces to:

$$13 = 4.57 \frac{v^2}{2g}$$

The maximum velocity in the culvert is then calculated from the above
equation as 13.5 fps. The maximum discharge through the culvert is then

$$Q = V A$$
$$= 13.5 \frac{\pi D^2}{4}$$
$$= 170 \text{ cfs}$$

This maximum discharge is approximately the same as the flow in the
canal during a 5-year storm in the I-80 interchange. For storms with
higher recurrence intervals, such as 10 and 25 years, the culvert will not
be able to handle the flow. Furthermore, if the Rock River stage at the
mouth of the Hennepin Canal is above 567 ft MSL, the available energy head
will be reduced and as a result the flow through the culvert will be less
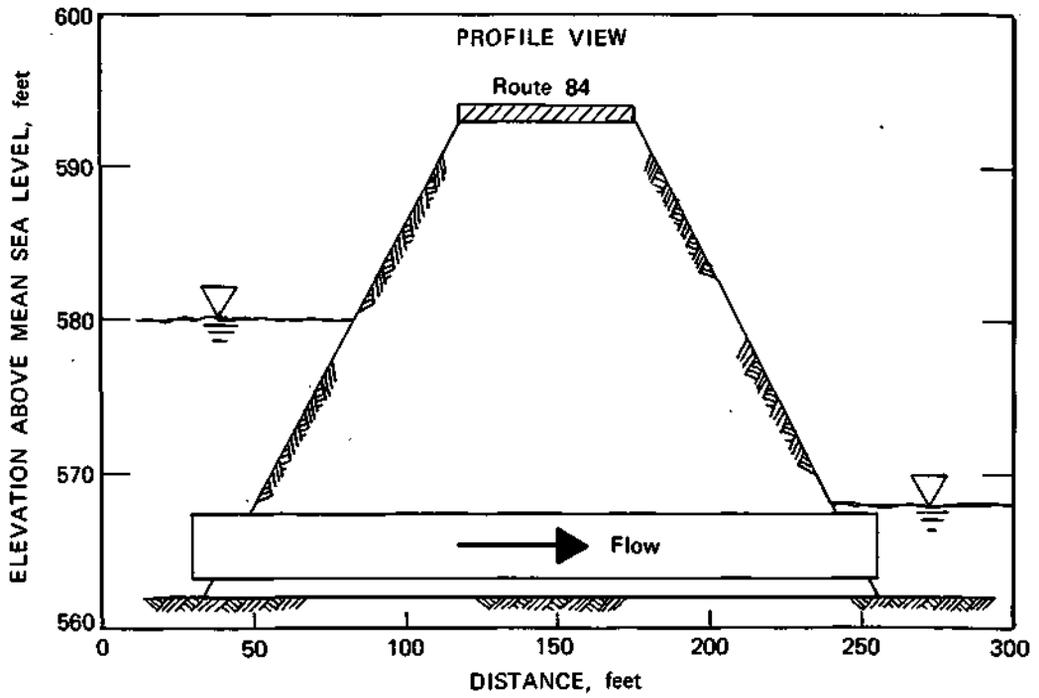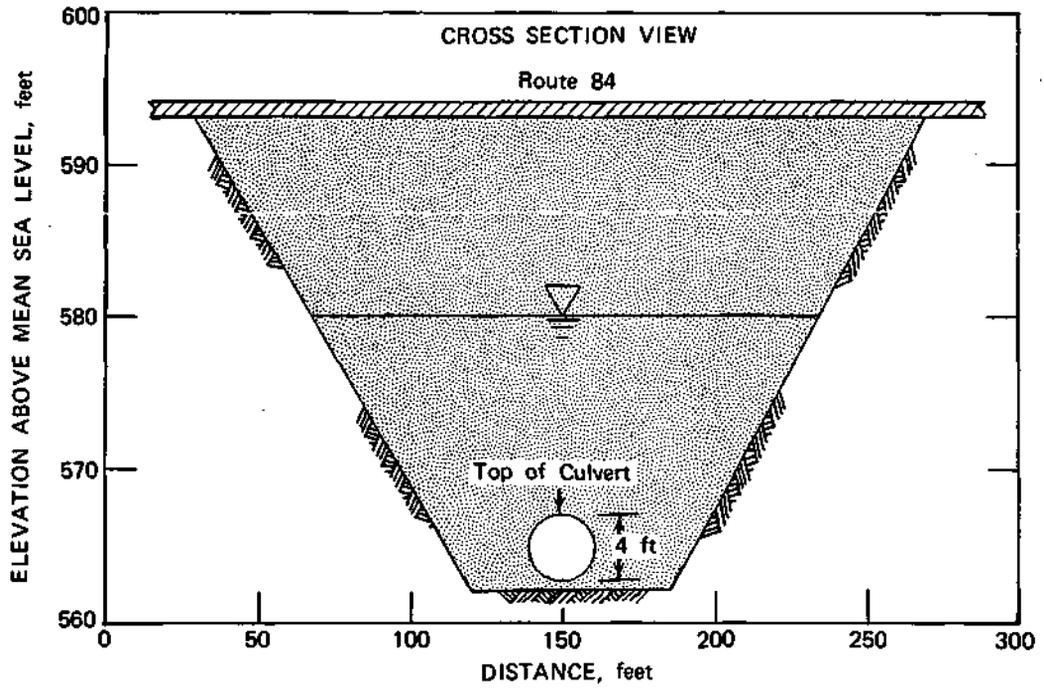than the 170 cfs maximum capacity.

Figure   17.   Culvert   and   canal   fill   at   Illinois   Route   84

In general, it can be concluded that there will be flooding from the Hennepin Canal during storm events of recurrence intervals greater than 5 years because of the backwater from the Route 84 culvert. A combination of high water in the Rock River and moderate storms with less than a 5-year recurrence interval might also result in flooding around the Hennepin Canal.

If Alternative 2 is selected and flood protection is provided for storm events of recurrence intervals greater than 5 years, either the capacity of the culvert at Route 84 has to be expanded or the Hennepin Canal levees have to be raised above 580 ft MSL all the way from Route 84 to Lock 27.

Another problem with this alternative is the sedimentation in the canal and at Culvert 40. The sedimentation in the canal will continue at a moderate rate, and if the canal is not dredged occasionally it will create problems with the culverts at the Colona sewer crossing and at Route 84. The sedimentation problem at Culvert 40 will be almost continuous and cleaning may be required several times a year. Unless a good maintenance program is planned for Culvert 40, it will be useless to reactivate the culvert at all.

Alternative 3. Install a New Culvert across the Canal East of Culvert 40

This alternative will require installing a new culvert from the drainage channel to the south side of the canal, as shown in figure 18. If the culvert is installed properly and the levees around the new culvert are riprapped, the runoff from the I-80 interchange will be routed directly to the Green River. There is no need to build a channel from the outlet of the culvert to the Green River, as the flow will carve its own channel.

The transition from the drainage channel to the new culvert has to be watertight so the culvert will not be undermined by seepage and eventually get washed out, as happened to the previous culvert at the same site. The size of the culvert depends on the design storm. If the culvert is to handle the runoff from the 10- and 25-year storms, the diameter of the pipe has to be 6 feet. For storms less than the 10-year storm, a 5-ft culvert will be adequate.
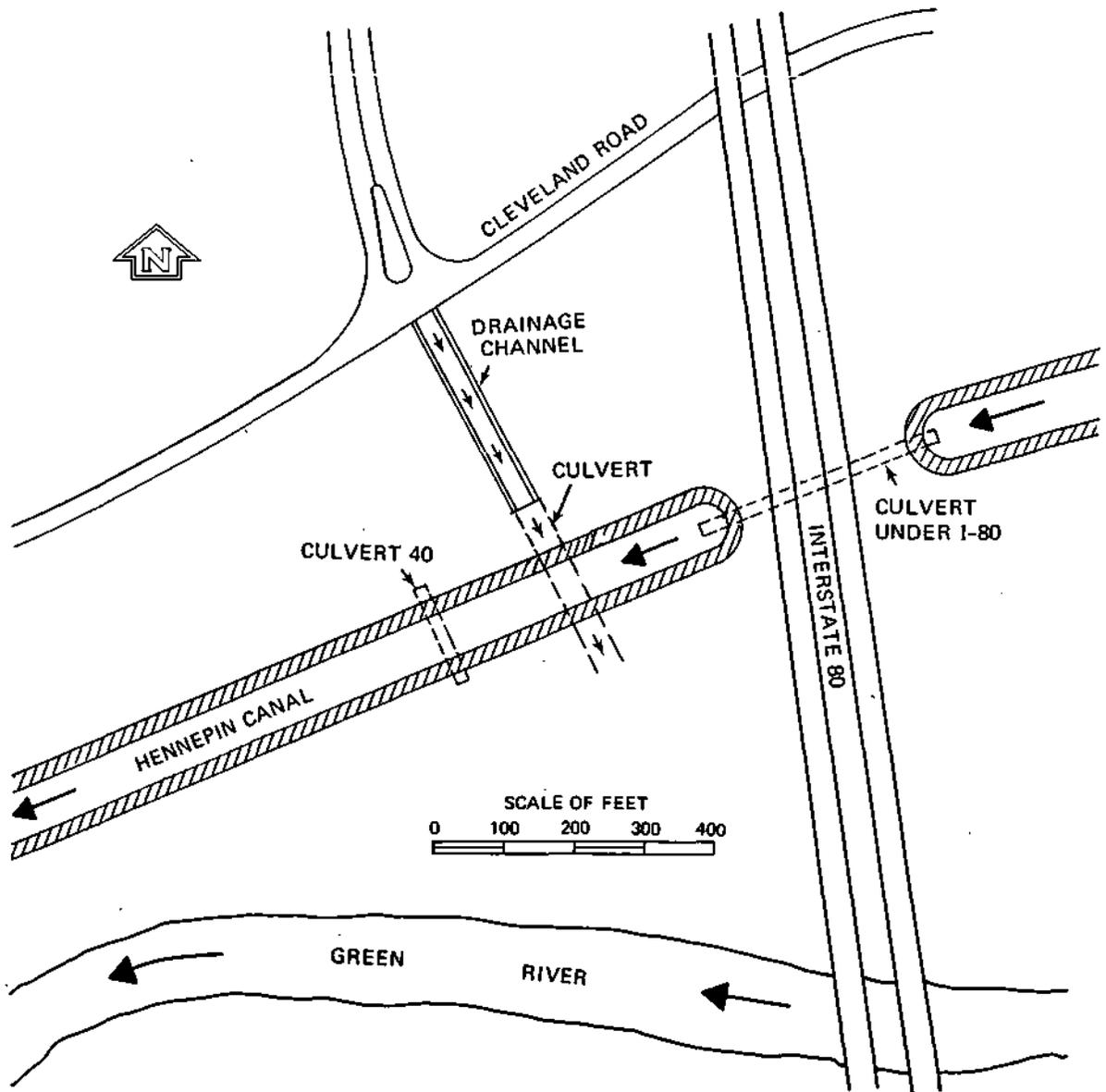
Figure 18. Alternative 3 — Installation of a new culvert across the canal east of Culvert 40

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The placement of the culvert at this site will isolate about 300 feet
of the canal between the culvert and I-80. The canal will have to be
dredged in the area of the new culvert to allow water to flow underneath
the culvert. Once the culvert is installed, the south canal levee can
either be raised to prevent Green River water from entering it or just
reinforced and maintained to prevent levee breaks. Since the runoff from
the I-80 interchange will be diverted to the Green River, there is no way
adjacent areas can be flooded by the Hennepin Canal. The only source of
flooding will be the Green and Rock Rivers. Once the existing levee
breaks are repaired, the present levee elevations are adequate to prevent
Rock River backwater from flooding some areas through the Hennepin Canal.

Alternative 4. Build a North-South Levee Upstream of Culvert 40
     This alternative will require building a north-south levee from the
end of the drainage channel to the south canal levee and extending the
culvert under I-80 to go past the new levee, as shown in figure 19. This
will allow all of the drainage water from the I-80 interchange to go
directly into the Green River. Water will come into the canal only
through the culvert under I-80. This will also solve the sedimentation
problem in the canal and provide a manageable flow through the canal.

     In conjunction with this alternative, the south canal levee can
either be raised to prevent Green River water from entering it or just
reinforced and maintained to prevent levee breaks. After the runoff from
the I-80 interchange is diverted into the Green River and out of the
canal, there is no way adjacent areas can be flooded by the Hennepin
Canal. The only source of flooding will be the Green and Rock Rivers.
Once the existing levee break at the Burlington Railroad bridge is
repaired, the present levee elevations are adequate to contain Rock River
backwater within the canal.

     The major disadvantage of this alternative is that some 300 feet of
the canal adjacent to I-80 will be lost. However, with proper management
the area can be planted with trees to blend it with the surroundings. The
loss of the canal can be reduced if the north-south levee is built closer
to I-80, as shown in figure 20. This, however, will require building a
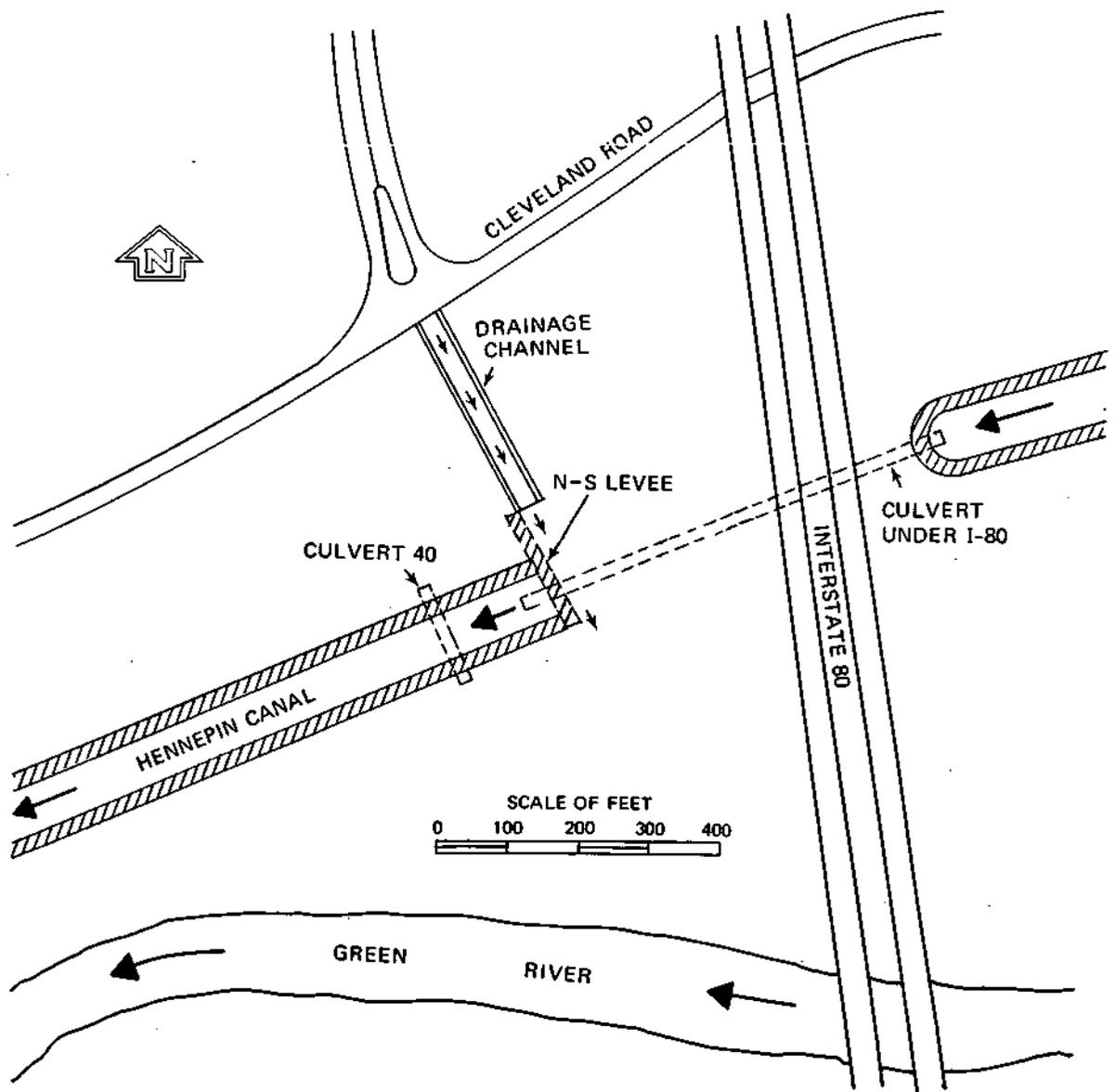concrete channel just north of the canal to direct the flow east and

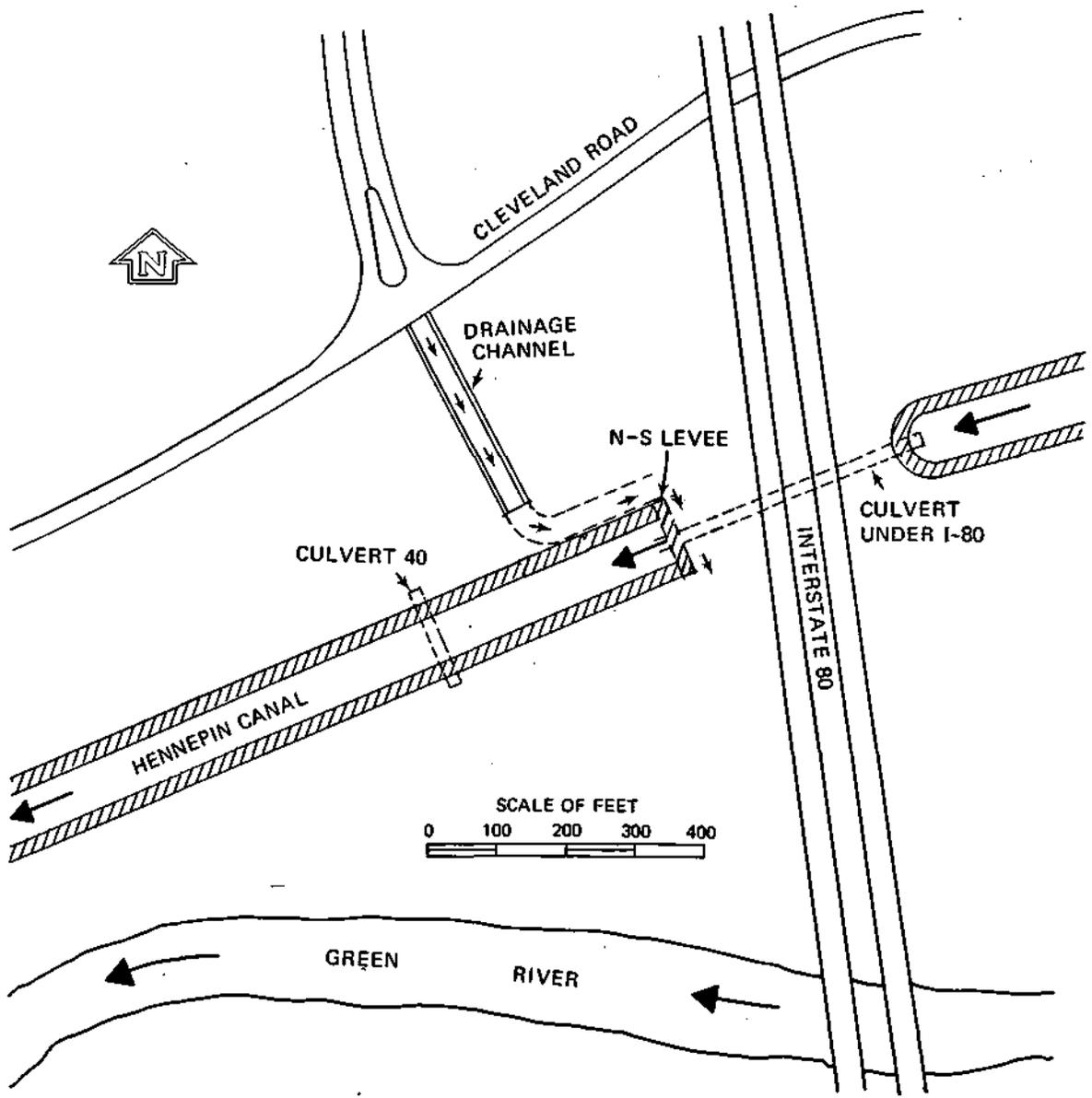*Figure 19. Alternative 4 — Construction of a north-south levee upstream of Culvert 40*

Figure 20. Alternative 4a — Construction of a north-south- levee close to I-80

reinforcing the north levee so that it will not be washed out. This will
be a more expensive project but will save about 200 feet of the canal.

Alternative 5. Allow All of the Runoff to Enter the Canal

This alternative will require fixing up the levee just south of the
drainage channel and extending the drainage channel into the canal, as
shown in figure 21. The south levee has to be riprapped properly so it
will not be washed away by the runoff from the I-80 interchange.

Once the runoff is allowed to enter the canal, the same problems
discussed in Alternative 2 will have to be dealt with, except that in this
case the problems will be more severe since all of the runoff will be in
the canal. Unless the capacity of the culvert under State Route 84 is
increased, this alternative will have serious problems during storm
events. There will be backwater from the culvert at Highway 84 during any
major storms, and the levees will be overtopped occasionally. The sedi-
mentation problem in the canal will continue and eventually cause the
culverts at the Colona sewer crossing and at Route 84 to silt up. This
will cause more severe problems than the present problem.

If this alternative is chosen, it will be advisable to reinforce the
canal levees rather than raise their elevations. If the levees are
properly riprapped at the weak areas, the occasional levee breaks which
now occur will be prevented and the damage to the levees during flooding
will be reduced.

RECOMMENDATIONS

The following recommendations for the proper handling of the runoff
from the I-80 interchange area are made on the basis of a thorough discus-
sion of the different alternatives outlined in the preceding section with
DOC and DOT personnel at a meeting on October 18, 1983. A combination of
Alternative 3 and Alternative 4, with special consideration for uninter-
rupted travel along the tow path (north levee) for maintenance vehicles
and hikers, was selected as the best alternative. The major components of
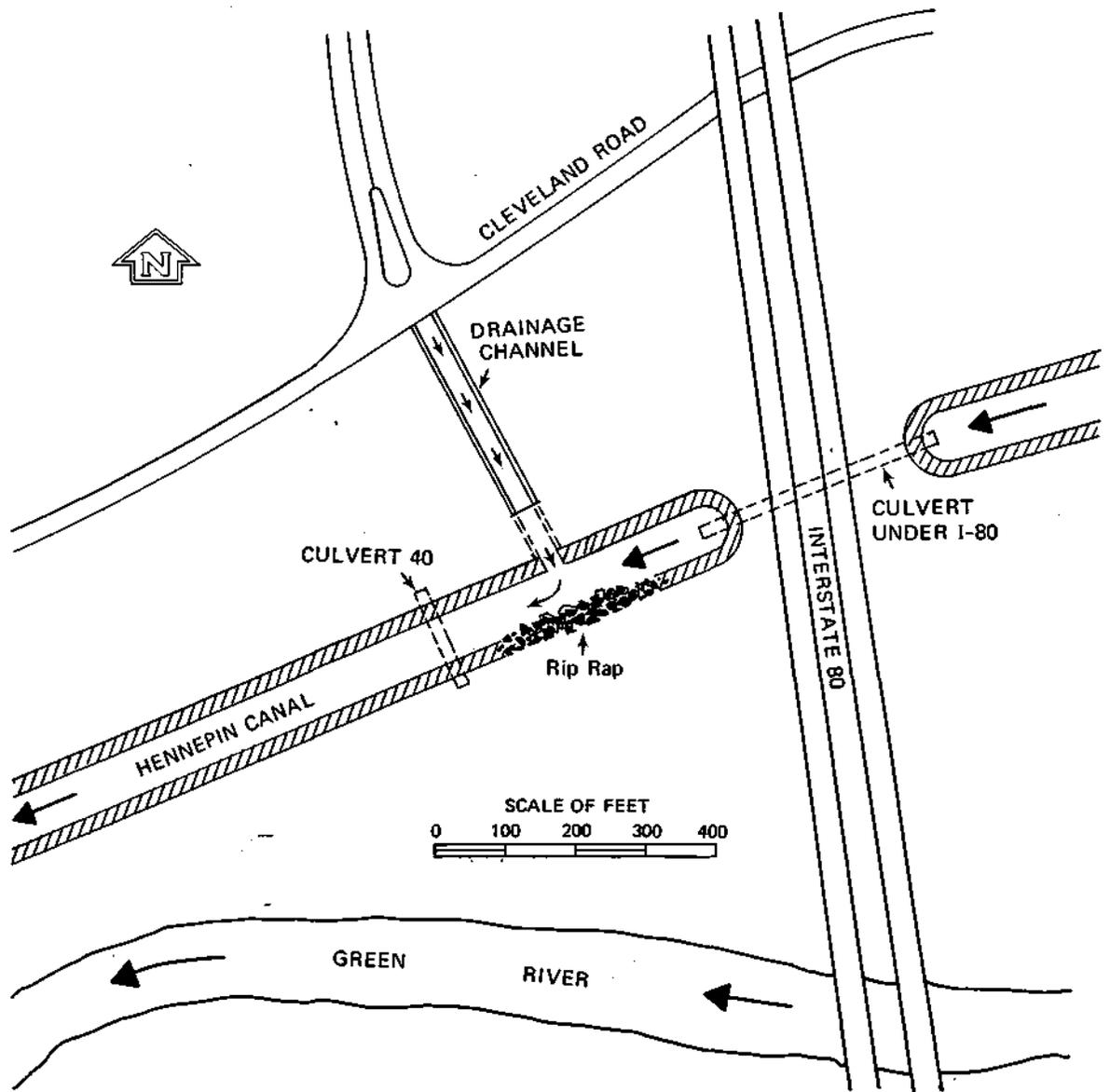the recommendation, shown in figure 22,include:

48

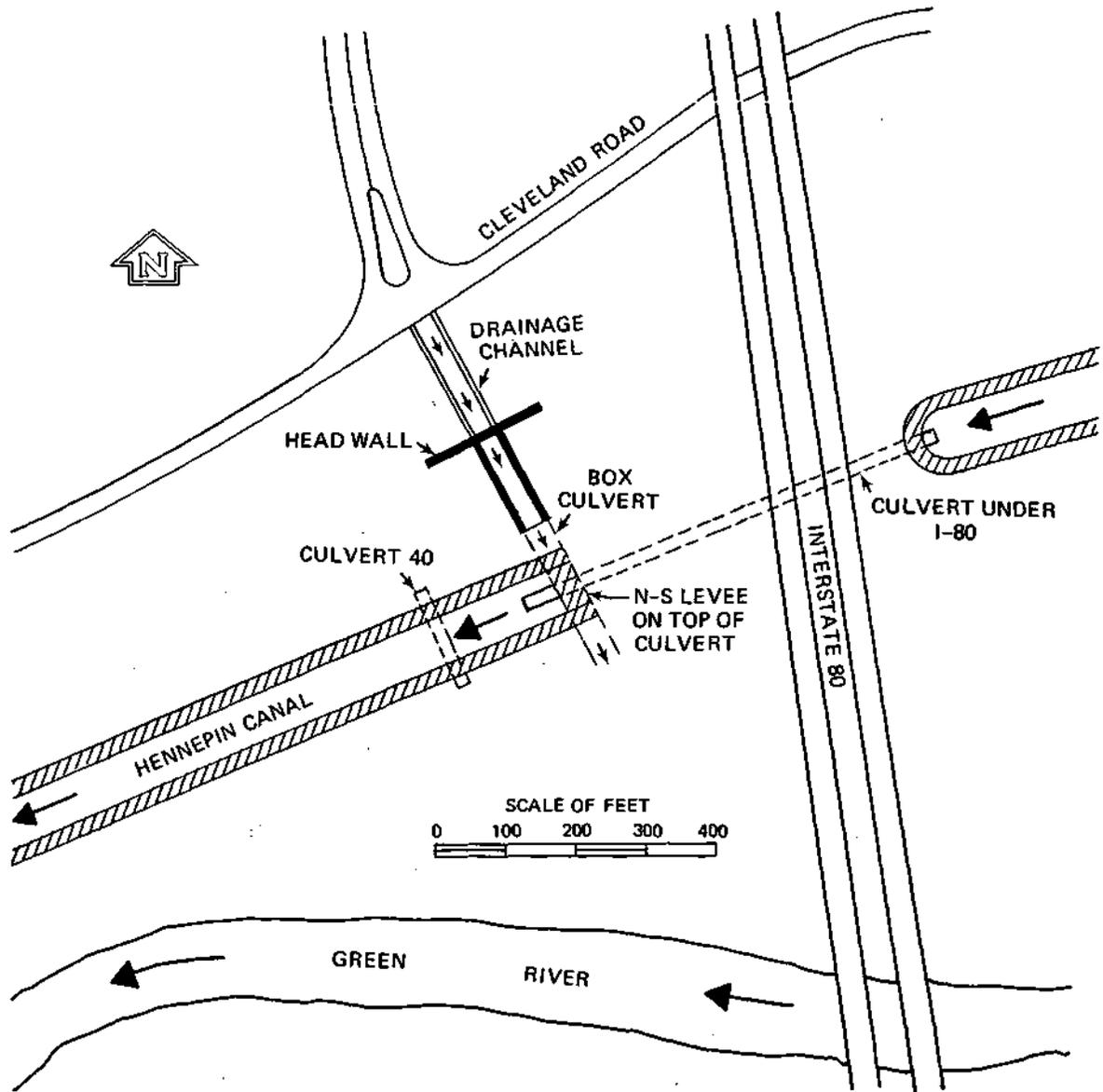*Figure 21. Alternative 5 — Allowing all of the runoff to enter the canal*

Figure 22. Main components of the recommendation for proper handling of the runoff from the I-80 interchange

50

1) Install a culvert from the drainage channel across the canal to the south side of the south canal levee.

2) Extend the culvert under I-80 for about 300 ft so that canal water can flow past the new culvert.

3) Fill in the canal east of the new culvert in such a way that it blends with the surrounding topography. Seed the area for erosion control and aesthetics.

4) Dredge the excessive sediment in the canal west of the new culvert.

5) Build a north-south levee over the new culvert with proper riprap so it will not be washed out when the levee is overtopped.

6) Repair and reinforce the south levee from the new culvert to Lock 28.

Figure 23 has been prepared as a guideline for detailed design specifications for the construction of the new culvert. As shown in the figure, a box culvert 4 feet high and 8 feet wide has been selected. The box culvert was selected because of the limitations imposed on the culvert height by the extension of the 54-inch-diameter culvert under I-80 and the elevation of the tow path on the north levee. The new culvert will be 1.0 foot above the top of the extension culvert from under I-80. A 1.5-foot graded riprap will be placed on top of the new culvert and on the fill material which will form a north-south levee at the culvert location. This arrangement will provide an uninterrupted pathway along the north levee and will also provide access to the south levee.

One important aspect of the culvert construction is the transition from the trapezoidal drainage channel to the box culvert. It is very important that the transition be watertight so the culvert will not be undermined by leakage at the transition.

Other recommendations involve existing and impending problems within the study area. One of the existing problems is the canal levee break at the Burlington Northern Railroad bridge. In all likelihood the levee break there was caused by Green River flood waters. The best evidence of the cause of levee failure is the movement of the levee material that is washed into the canal, which indicates the direction of flow during the levee failure. The levee break has to be repaired to prevent the Rock River backwater from flooding the adjacent private property. However, the

PLAN VIEW

TO ELEV. 585 ft

HEAD WALL

EXTENSION OF CULVERT UNDER I-80

BOX CULVERT 8' x 4'

DRAINAGE CHANNEL

22.5 ft

8 ft

CANAL LEVEES

TO ELEV. 585 ft

PROFILE VIEW

ELEVATION IN FEET ABOVE MEAN SEA LEVEL

590

585

HEAD WALL

GRADED RIP-RAP

BOX CULVERT 8' x 4'

580

4'

CANAL LEVEES

575

DRAINAGE CHANNEL

WATER TIGHT TRANSITION

RIP-RAP

54"

EXTENSION OF CULVERT UNDER I-80

570

0   150   300   450   600

DISTANCE FROM CLEVELAND ROAD, feet
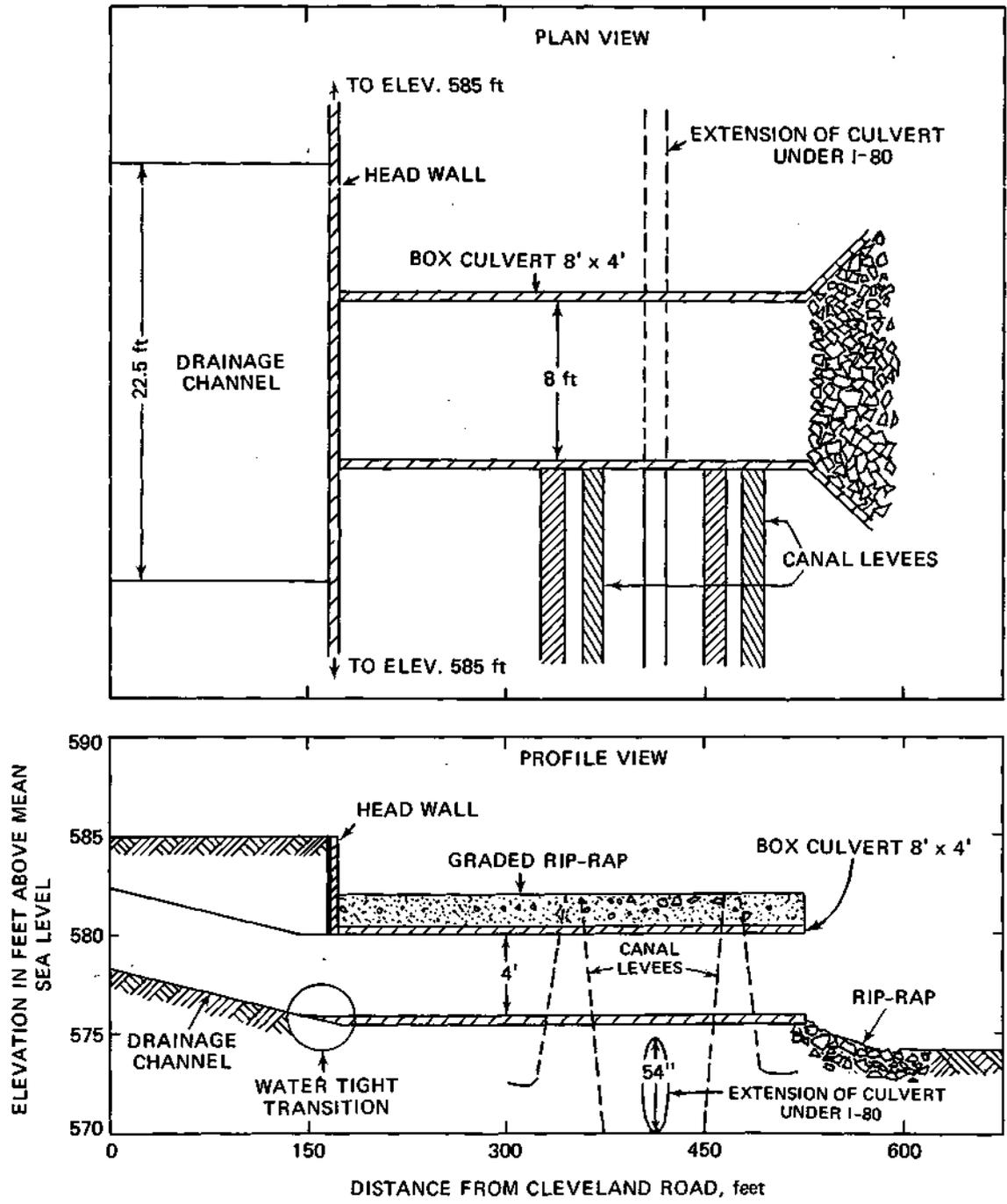
Figure   23.   Design   drawing   for   the   new   culvert   west   of   I-80

repair of the levee has to take the cause of the break into consideration. If the levee break at this location is fixed up to prevent Green River flood water from entering the canal, there will be a levee break at some other location unless the whole south levee is built up and reinforced.

The best way to repair the levee is to raise the levee elevation above the Rock River backwater (which is 574 feet MSL for a 100-year flood) but to leave it low enough for the Green River to overtop it when it is at high flood stages. The south levee will need to be riprapped on both sides so that it will not be washed out when overtopped.

Another potential problem area is Culvert 39, which might clog up with debris and sediment unless it is continuously monitored and cleaned. Of special concern is the runoff from the sand quarry just north of Cleveland Road. If excessive sediment manages to get to Culvert 39, it will definitely cause a major problem. It is therefore recommended that the area be routinely monitored and that the culvert inlet and outlet be cleaned from time to time.

INVESTIGATION OF DRAINAGE FROM THE PRIVATE PROPERTY JUST EAST OF I-80

The private property just east of I-80 between the Green River and the Hennepin Canal is completely surrounded by levees on all sides, as shown in figure 24. The only drainage from that property is through a 4-ft-diameter drainpipe through the levee at the end of the drainage ditch within the levee system. The drainage from the pipe is carried to the Green River by a drainage ditch which runs in between the west levee and the embankment of I-80. The drainpipe has a set of three flap gates at the pipe outlet, the purpose of which is to close the drainpipe when the Green River stage is up to the level of the drainpipe opening. This prevents Green River water from entering the farm through the drainpipe. However, if the Green River stage is up and remains up for an extended period of time, drainage water from the farm is impounded within the levees surrounding the farm for the same amount of time. The water elevation in the drainage ditch leading to the Green River is the same as that in the Green River during flood stages.

Since the flooding in water year 1979 was severe, it was decided to investigate how long the drainage pipe from the private property was closed because of high water in the Green River. The highest flooding in 1979 took place from March 16 to April 11, a period of about 27 days. The daily average discharges from March 16 to April 11 are shown in table 6. The Green River flood elevation just upstream of I-80, which is the location of the drainage ditch leading to the drainpipe, was calculated using the HEC-2 program. The Green River water surface elevation during that 1979 flooding period is shown in figure 25, along with the elevation of the drainpipe. As can be seen in the figure, the Green River stage was higher than the drainpipe's opening. Therefore, during this period of flooding, the flap gates on the drainpipe were completely closed for at least 21 days. Any rainfall and drainage water from the farm within the levees was therefore impounded on the farm for at least 21 days. Furthermore, the drainpipe was partially closed even for a longer period of time after the major flood, preventing the farm from draining very quickly.

Another period of severe flooding in water year 1979 was from August 18, 1979, to September 1, 1979. The discharges from August 17,
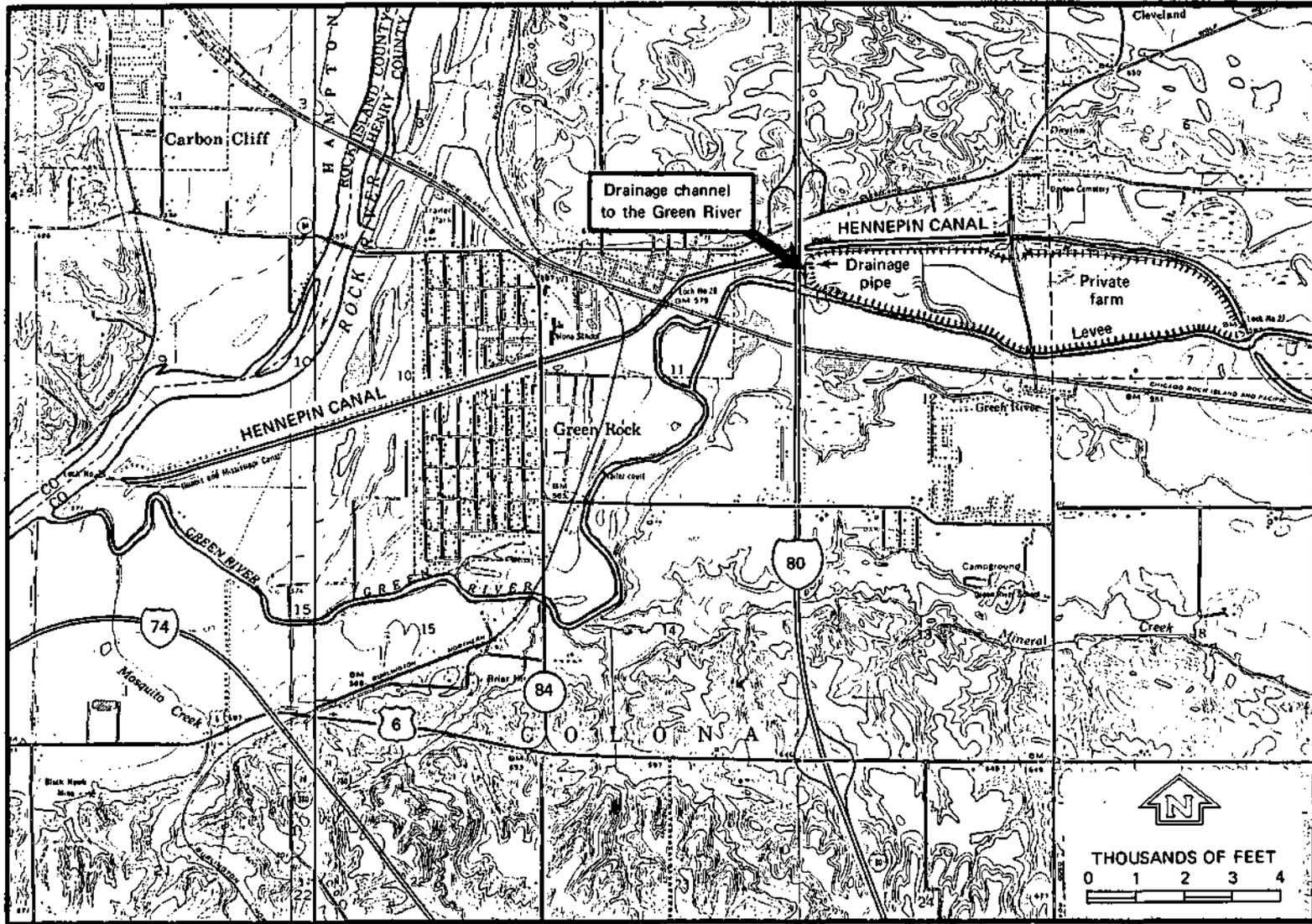
Figure 24.   Location of private property just east of I-80

Table 6. Average Daily Discharges for Green River
near Geneseo from March 16 to April 11, 1979

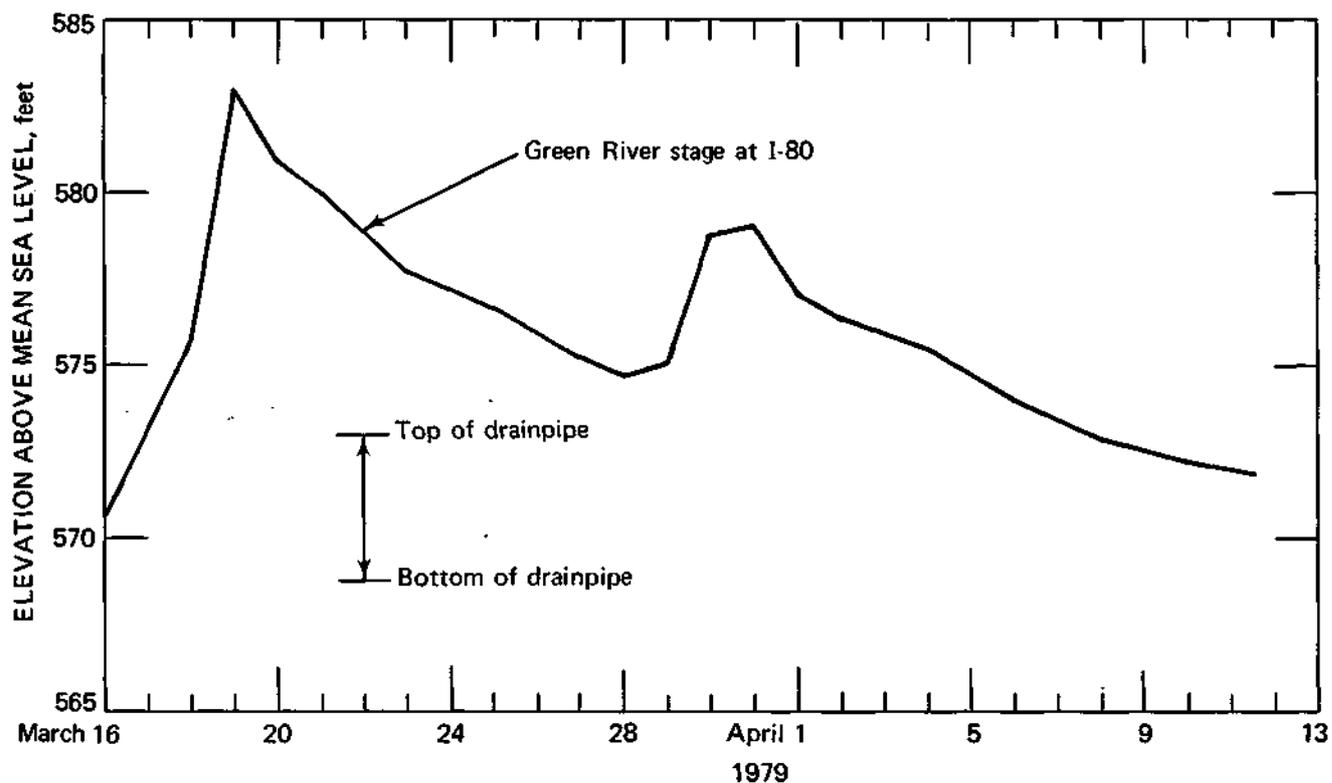| Date	Discharge (cfs)	Date	Discharge (cfs)
March 16	1,200	March 30	6,900
17	2,100	31	7,150
18	4,000	April 1	5,020
19	10,100	2	4,400
20	9,150	3	4,140
21	8,080	4	3,700
22	6,550	5	3,260
23	5,700	6	2,720
24	5,280	7	2,370
25	4,630	8	2,130
26	3,930	9	1,960
27	3,490	10	1,770
28	3,170	11	1,690
29	3,470		

Figure 25. Green River stages at I-80 from March 16 - April 11, 1979

1979, to September 5, 1979, are shown in table 7. The Green River flood elevation just upstream of I-80 from August 17 to September 5, 1979, is shown in figure 26. Also shown in figure 26 is the elevation of the. drainpipe's opening. As shown in the figure, the drainpipe opening was completely under water from August 18 to August 26 for a period of 8 days. During this period no water was draining out of the farm area. In addition, the drainpipe was not completely open for at least 10 days from August 26 to September 5.

In general, the drainage system from the farm enclosed by the levee system east of I-80 is totally inadequate during high waters in the Green River. When there is an extended period of time during which the Green River is high, the farm will be under water unless the standing water within the levee system is pumped out.

Table 7. Average Daily Discharges for Green River
near Geneseo from August 17 to September 5, 1979

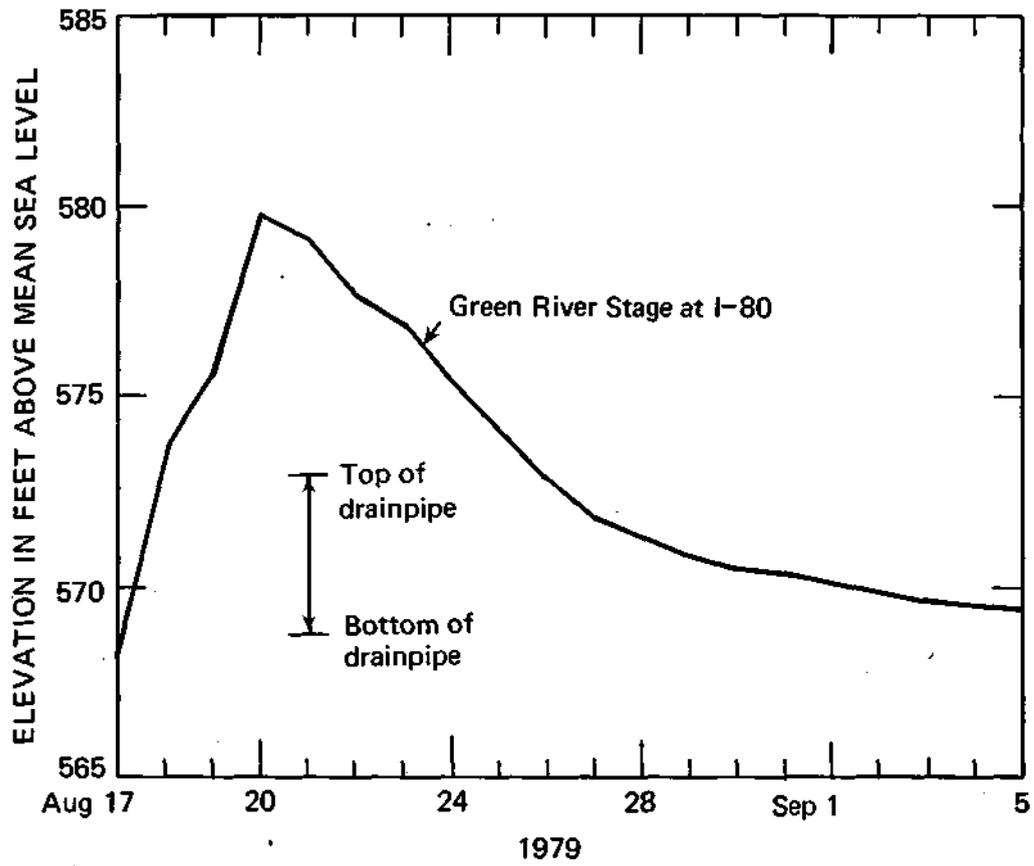| Date	Discharge (cfs)	Date	Discharge (cfs)
August 17	210	August 27	1,680
18	2,690	28	1,380
19	3,910	29	1,140
20	7,850	30	997
21	7,350	31	928
22	5,720	September 1	800
23	4,940	2	705
24	3,780	3	637
25	2,870	4	571
26	2,160	5	531

Figure 26. Green River stages at I-80 from August 17 - September 5, 1979

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streams. U.S. Geological Survey Water Supply Paper 1849, Washington,
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