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Hydrologic and Hydraulic Analyses of the Illinois and Michigan Canal at Lockport, Illinois

by Misganaw Demissie and Abiola A. Akanbi
Office of Sediment and Wetland Studies

Prepared for the
City of Lockport,
Lockport Township Park District,
Illinois Department of Conservation,
I & M Canal National Heritage Corridor Commission,
and the Lockport Area Development Commission

December 1994

Illinois State Water Survey
Hydrology Division
Champaign, Illinois

A Division of the Illinois Department of Energy and Natural Resources

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INTRODUCTION

The Illinois and Michigan (I&M) Canal was constructed by the state of Illinois in the mid-1800s to link Lake Michigan to the Illinois River and eventually to the Mississippi River for navigational purposes. Construction of the canal started in 1836 and was completed in 1848. The route of the canal generally followed that of the DesPlaines River in its eastern portion and the Illinois River in its western portion, as shown in figure 1. The canal extended from the South Branch of the Chicago River in Chicago to the Illinois River at LaSalle-Peru, for a total length of 96.4 miles (mi). The original canal cross section was designed with a 36-foot (ft) bottom width, a 60-ft water-line width, and a 6-ft depth. The canal had 15 locks to regulate water levels and four aqueducts to pass over streams and rivers. It also had four feeders to supply water to the canal and maintain adequate depth for navigation. For many decades the canal was an important commercial thoroughfare until navigation in the canal eventually declined and finally was terminated in 1933 after the opening of the Chicago Sanitary & Ship Canal and the construction of a series of locks and dams on the Illinois River in the early 1900s, which permitted the opening of the Illinois Waterway (Demissie and Stephanatos, 1986; Illinois Department of Conservation, 1948; Illinois Division of Waterways, 1951; Howe, 1956).

In 1984, the federal government designated the I&M Canal as a National Heritage Corridor because of its historical significance and strong local interest in preserving, rehabilitating, and restoring it. The state now manages the canal as a historical and recreational corridor. Local groups and the state want to maintain some flowing water in the canal for recreational purposes and to improve the aesthetics of the canal. However, the canal has not been maintained for a very long time, and in some places it has been completely filled in with sediment. Most of the major hydraulic controls of the original canal are not operational. Furthermore, most of the small streams in the region drain either into or through the canal, causing flooding problems in some areas.

It is obvious that the remnants of the old canal are not the same as the canal that was designed and built for navigation. Significant changes have occurred in both the canal and

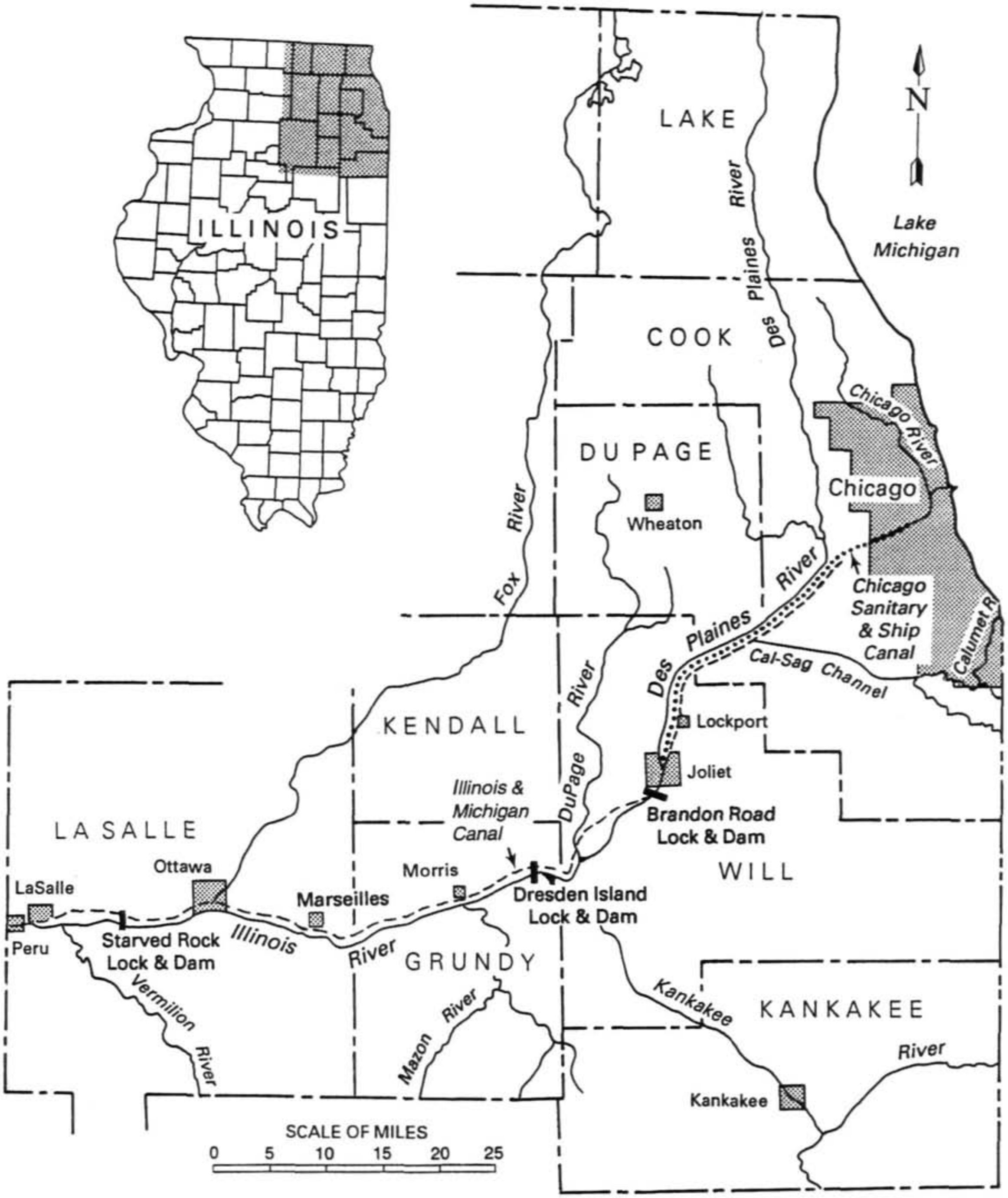


Figure 1. Location of the Illinois and Michigan Canal

the surrounding areas. Therefore it is very important to evaluate the canal's present conditions and to conduct feasibility studies of any hydraulic works to rehabilitate the canal before initiating any major rehabilitation work.

Objectives of the Study

This study was designed as a two-phase project. The main objective of phase I of the project was to conduct a preliminary feasibility study to investigate the hydraulics and hydrology of the canal and the streams that drain into it, and to evaluate the potential for rehabilitating the canal in the Lockport area. Major areas of concern include water supply to maintain sufficient depth, flooding adjacent to the canal, sedimentation within the canal, and the rehabilitation potential and problems.

Following the completion of phase I, phase II was initiated to conduct a detailed hydrologic analysis of the drainage areas of all the tributaries that flow into the canal from downstream of the Cal-Sag Channel to Lock 2. This phase also included a hydraulic analysis of the segment of the canal from the Texaco Refining and Marketing, Inc. (TRMI) plant to Lock 2. The following 11 tasks were developed for phase II of the project:

1. Determine flood hydrographs for various storm frequencies (10-, 50-, 100-, and 500-year) for all tributary streams entering the I&M Canal from Lock 2 to downstream of the Cal-Sag Channel.
2. Route flood flows for each event through the tributary channels and the I&M Canal to determine flood elevations along the canal.
3. On the basis of the results of task 2 for each event, determine flooded areas during storms of different frequencies under existing conditions.
4. Evaluate flooding impacts of the proposed weir at Lock 1.
5. Evaluate impact of modifying existing Deep Run overflow weir at the TRMI plant and the bypass at Lock 1 on flooding along the I&M Canal.
6. Observe field conditions for a period of 16 months, including the spring of 1991 and the spring of 1992.
7. On the basis of available data and field inspections, assess the stability of the canal levees for different flow conditions and potential seepage through the levees.
8. Estimate future sedimentation rates within the canal and evaluate potential maintenance dredging requirements or sediment control measures.
9. Conduct further discussions with TRMI on water and sediment quality issues, and evaluate additional information mat might become available.

10. Discuss permit requirements with state and federal agencies for proposed modifications to the canal, and incorporate the results into the report.
11. Prepare a draft report based on the study, submit it to the Illinois Department of Conservation (IDOC) and to me City of Lockport for review and comments, and then prepare a final report incorporating the comments.

Scope of the Study

The reach of the canal that is of primary interest, shown in figure 1, extends from south of Lockport to the TRMI plant northwest of Lockport. An aerial photograph of the canal from Joliet to Lockport is shown in figure 2. A control structure at the refinery, known as the Texaco Dam, was first agreed upon as the northern limit of the study area. Because it was assumed that a regulated flow enters the canal at the control structure, the influence of tributary streams upstream on the hydraulics of the canal was not going to be investigated. However, it was later determined that water flow control and modifications at and upstream of the Texaco Dam are very important tasks in the future rehabilitation of the canal. Therefore, the study reach was extended up to and including the mouth of Long Run upstream of the TRMI property.

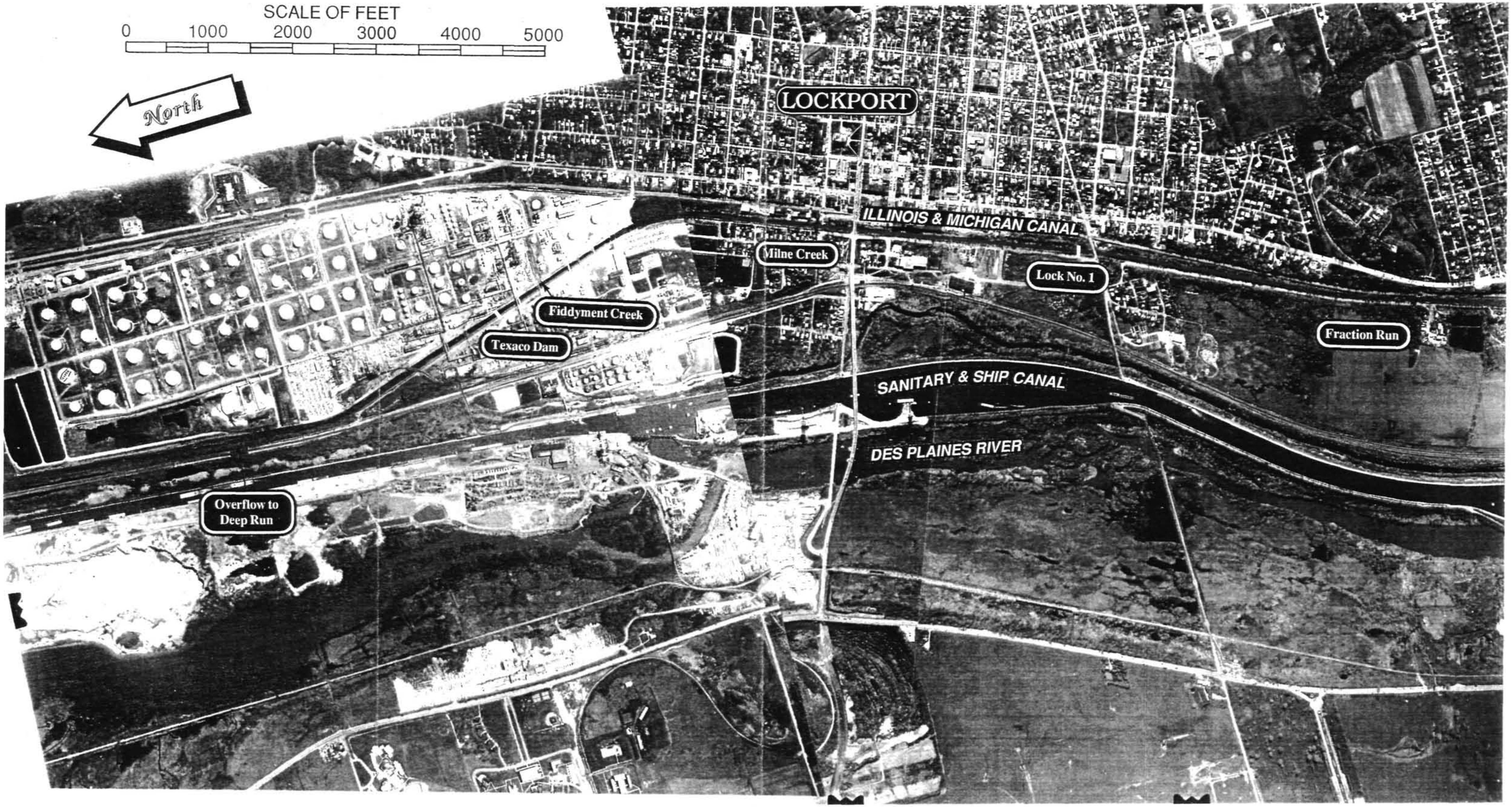
The scope of the second phase was also expanded to include the drainage areas of the seven tributaries draining into the canal from downstream of the Cal-Sag Channel to Lock 2. Thus the total amount of inflow to the canal could be determined by the hydrologic analysis. The hydraulic analysis and sedimentation study cover only the canal segment from the side-overflow weir diversion structure within the TRMI plant to Lock 2.

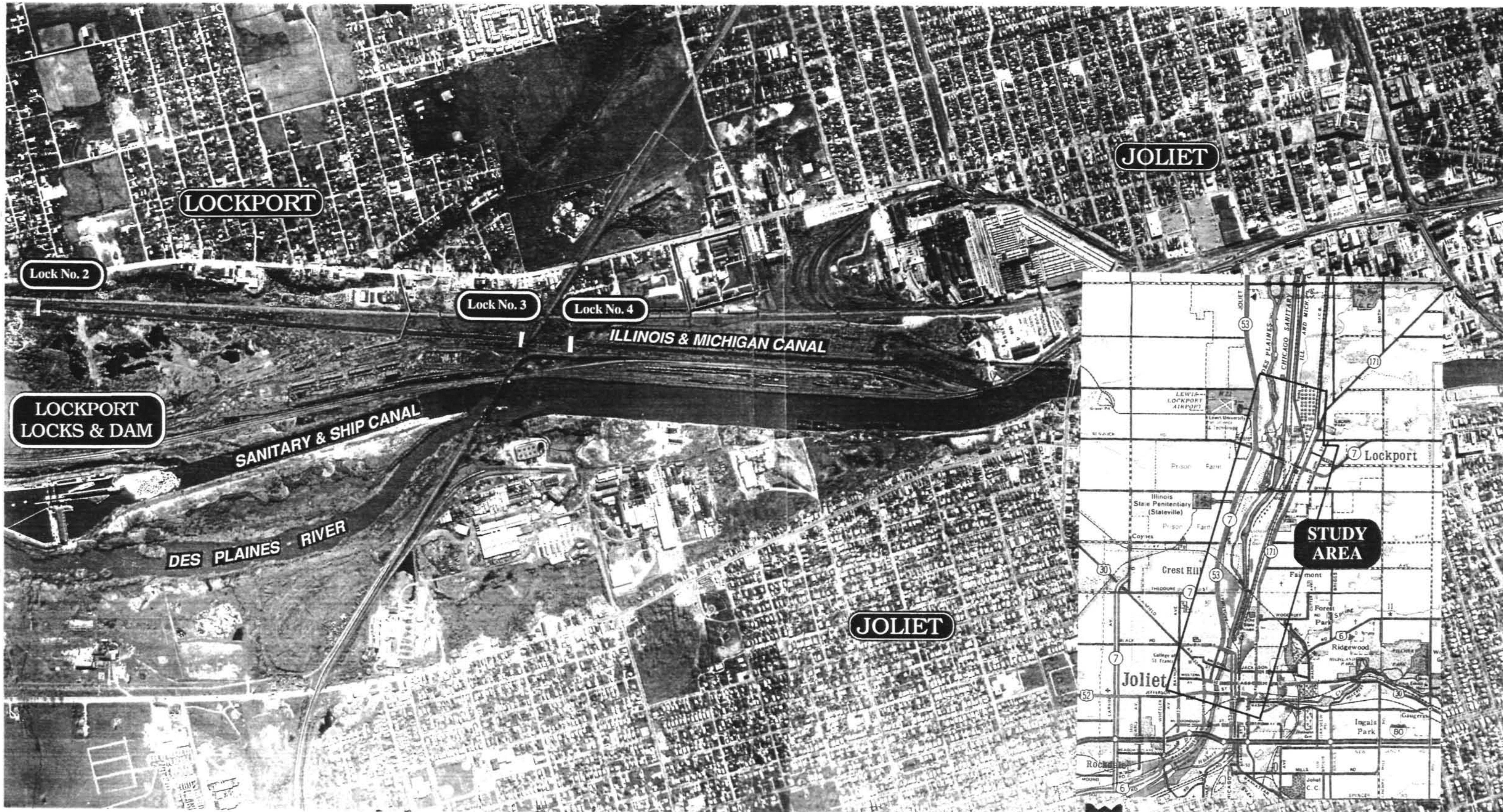
Acknowledgments

This report was based on work supported in part by funds provided by the Lockport Area Development Commission, Ann G. Hintze, Executive Director; the City of Lockport, Gordon McCluskey, City Administrator; the Illinois Department of Conservation, Kim Gibson, Don Kochevar, and Greg Kiaer, Project Managers; the I&M Canal National Heritage Corridor Commission, Bob Stewart, chair; and the Lockport Township Park District. George Cracraft and Bonnie Packley at Texaco Refining and Marketing, Inc. (TRMI) in Lockport were very cooperative in providing unlimited access to the plant and some of the elevation and water quality data used in this report. The valuable review comments of the I&M Canal Hydrology Study Committee are also gratefully acknowledged.

Water Survey assistant hydrologist Laura Keefer assisted in the installation of crest gages, the survey of gage elevations and culvert inverts, and in stream discharge measurements. University of Illinois Department of Civil Engineering undergraduate Kalpesh

Figure 2. Areal photo foldout of the Illinois and Michigan Canal from Joliet to the Texaco Refinery in Lockport (reproduced from Demissie and Xia, 1990)


SCALE OF FEET
0
1000
2000
3000
4000
5000
North
LOCKPORT
ILLINOIS & MICHIGAN CANAL
Milne Creek
Lock No. 1
Fiddyment Creek
Texaco Dam
Fraction Run
SANITARY & SHIP CANAL
DES PLAINES RIVER
Overflow to Deep Run



LOCKPORT
JOLIET
Lock No. 2
Lock No. 3
Lock No. 4
ILLINOIS & MICHIGAN CANAL
LOCKPORT LOCKS & DAM
SANITARY & SHIP CANAL
DES PLAINES RIVER
JOLIET
STUDY AREA
Lockport
Crest Hill
Joliet
Illinois State Penitentiary (Stateville)
Ridgewood
Ingals Park
Rockdale

Patel helped prepare data and generate results for the Hydrologic Engineering Center's HEC-1 and HEC-2 modeling results. Former Water Survey staff member William P. Fitzpatrick also assisted in the installation of the gages. Becky Howard typed the report, which Eva Kingston edited, and Linda Hascall and David Cox prepared some of the illustrations.

DRAINAGE

Historically the drainage of tributary streams into the I&M Canal has been a source of major problems, generally associated with flooding. The canal was originally designed for navigation. Tributary drainage into the canal has resulted in overtopping or breaks in the levees, which have led to flooding of adjacent lands and properties. In many cases the presence of the canal provided flood protection for adjacent areas during periods of frequent floods. However, when major floods occur, floodwaters overtop or breach the levees, flooding the surrounding areas, and the canal is then blamed for the flooding. Since management of the canal also includes maintaining its natural drainage without increasing the flooding problems to adjacent lands and properties, it is important to examine the expected drainage into the I&M Canal in the Lockport area and also to examine the impact of any improvements to the canal on flooding.

Figure 3 shows the tributary streams that drain into the I&M Canal in the Lockport area and their drainage areas, and table 1 provides the names of the streams and the sizes of their drainage areas. Seven streams with a total drainage area of 45 square miles (sq mi) drain directly into the I&M Canal in the study area. Long Run, the largest stream, has a drainage area of 25.5 sq mi. School Gully, the smallest stream, has a drainage area of 1.0 sq mi. School Gully and Convent Creek drain the area northeast of the city of Lemont and north of Long Run. Milne Creek enters the canal at the Gaylord Building, about one-half mile upstream of Lock 1. Big Run and Fiddyment Creek drain the area south of Long Run and north of Milne Creek. Both streams enter the I&M Canal within the boundaries of the TRMI plant. Fraction Run drains the area south of Milne Creek and joins the I&M Canal about 1,500 ft upstream of Lock 2.

Table 1. Characteristics of Streams that Drain into the I&M Canal in the Lockport Area

Name of stream	*Drainage area (sq mi)*	*Stream length (mi)*	*Average stream slope (ft/mi)*
Convent Creek	3.0	3.0	1A
School Gully	1.0	1.1	15.3
Long Run	25.5	11.9	11.5
Big Run	2.2	4.4	43.1
Fiddyment Creek	4.8	5.2	34.0
Milne Creek	2.3	3.6	44.2
Fraction Run	6.2	7.4	26.2

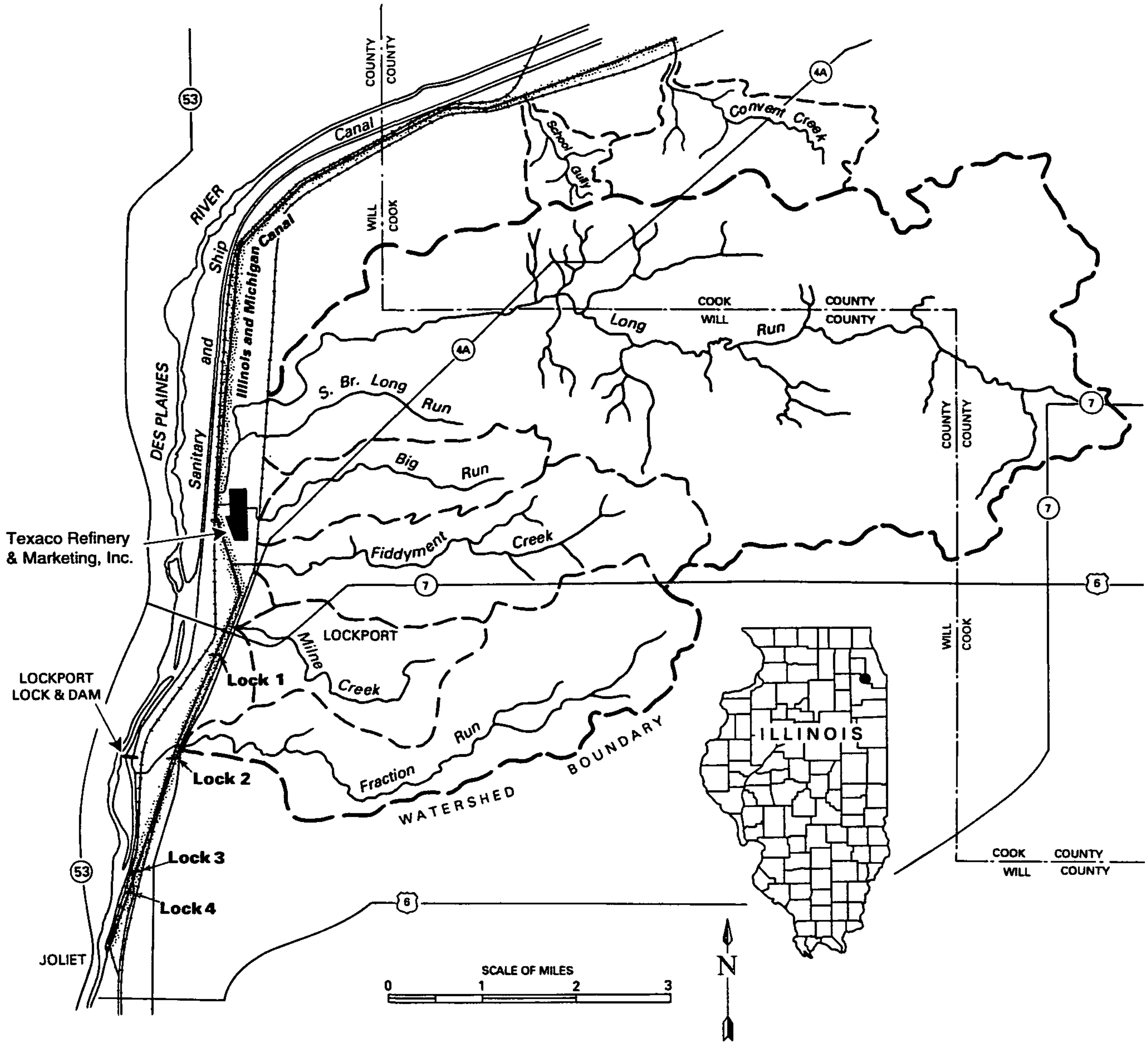


Figure 3. Watersheds of tributary streams draining into the Illinois and Michigan Canal within the study area

One of the major sources of the problems associated with the tributary streams is their steep gradients. Since the streams drain the bluff area of the DesPlaines floodplain, they have very high gradients, ranging from 7.39 to 44.2 feet per mile (ft/mi) before they enter the canal. Figure 4 shows the profiles of the streams, and table 1 gives their average gradients. Streams with such steep slopes generally have higher peak flows and tend to carry more sediment than streams with moderate or low slopes. During major storms, most of these streams deliver their floodwaters and sediment to the I&M Canal very quickly, causing potential flooding problems. The I&M Canal was not designed to contain the floodwaters from all the tributary streams at all times, and it cannot do so without major modifications. Therefore, to reduce flooding in the I&M Canal in the Lockport area, it is important to have controlled overflow structures in the canal so mat floodwaters overflow into Deep Run (as is presently done at the upstream end of the TRMI plant and at the culverts upstream of Lock 1) and eventually into the Chicago Sanitary & Ship Canal. Deep Run, the only drainage channel between the I&M Canal and the Sanitary & Ship Canal, begins at the end of Long Run Creek and runs almost parallel to the Sanitary & Ship Canal on the east side and joins it just downstream of the Lockport locks and dam.

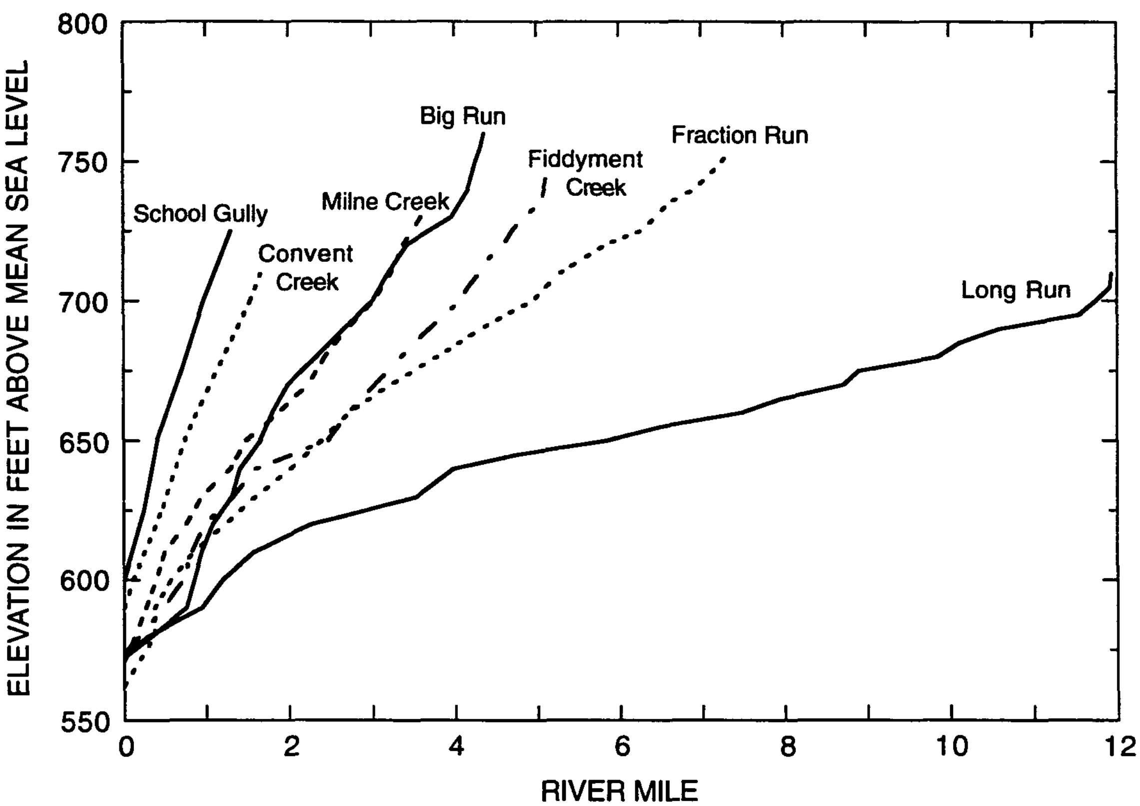


Figure 4. Profiles of tributary streams in the study area

PHYSICAL CONDITION OF THE CANAL

The main objective of this project was to evaluate the potential for rehabilitating the I&M Canal in the Lockport area on the basis of the canal's physical condition and the hydrology and hydraulics of the canal and the tributary streams. The I&M Canal in the study area has been abandoned as a navigation canal and has not been properly maintained, either as a navigation channel or as a drainage channel. The City of Lockport has cleaned the canal twice in the 1970s to improve its flood-carrying capacity. Because of the lack of adequate maintenance, the canal is not in very good condition. To assess the existing physical conditions, a detailed survey of the canal was conducted by Baird & Company in 1989 under contract with the IDOC (Baird & Company, 1989). The survey included 51 cross-sectional profiles of the canal and the intersections of main tributary streams from Joliet to the TRMI plant north of Lockport. The results of the survey are contained in appendix B of the phase I report by Demissie and Xia (1990). The same report also assesses changes in the canal over the years by comparing the 1989 cross-sectional profiles with those surveyed in 1949 by the Illinois Division of Waterways, Department of Public Works and Buildings. The phase I study concluded that major physical changes have not occurred in the canal cross sections since 1949 except in the upper and lower ends of the study reach.

Sedimentation

One of the major problems with the I&M Canal has been the accumulation of sediment in the canal prism when soil eroded from the watersheds of the tributary streams washes into the canal. Since the slopes of the tributary streams are very steep, the flow velocities in the stream channels during storm events are high enough to carry all the sediment downstream into the canal. Once the flow enters the canal, flow velocities are reduced significantly because the gradient of the canal between locks is very small. As the flow velocities are reduced, most of the sediment settles out in the canal. In 1951, the Illinois Division of Waterways reported a sediment accumulation of up to 8 ft in segments of the canal (Illinois Division of Waterways, 1951). Sediment accumulation has continued since then, with some areas experiencing more than others, especially the segment downstream of Lock 4 in the Joliet area and the segment upstream of the Texaco Dam. The main factor responsible for the high sedimentation rates in these two areas is the reduction in velocity due to the backwater effects of the Des Plaines River for the lower segment and the Texaco Dam for the upper segment.

Sediment removal operations have been conducted in the I&M Canal on several occasions. With the help of the U.S. Army Corps of Engineers in 1974, the City of Lockport removed some sediment between 2nd and 10th Streets. The City, under contract to Dobczak, an independent contractor, again removed many truckloads of sediment at the mouth of Milne Creek and Fraction Run Creek in June and July of 1977. In 1987, several loads of sediment were removed from the mouth of Milne Creek where it empties into the I&M Canal (Hintze, 1990). Figure 5 shows the canal bottom profiles from the canal mouth at Joliet to the TRMI plant **at** Lockport and the change in the canal bottom over the years, reflecting the accumulation of sediment. The three profiles are based on the 1989 survey (solid line), a 1951 report (Illinois Division of Waterways, 1951), and bedrock that can be assumed to be the original bed. As discussed earlier, sediment has accumulated over the whole stretch of the canal; but the areas with extreme sediment accumulation are located at the upstream and downstream ends of the study area. One localized area has accumulated significant sediment and is just downstream of Lock 2. Most of that sediment is probably brought into the canal by Fraction Run, which enters the canal 1,500 ft upstream of the lock. In the Lockport area, sediment removal at different times distorts the real situation. Because there is no accurate record of how much sediment was removed and from where, it will be difficult to estimate the canal profile for conditions without dredging. In any case, there is a problem of sediment accumulation in the canal, which should be expected for a canal draining a significant area with a steep gradient.

Canal Bottom and Levees

Because of minimal or infrequent maintenance along the canal, the condition of the canal and the levees is not very good. Major problems include overgrown trees and brush; accumulations of sediment, trash and junk; and weak levees. Any segment of rehabilitated canal will require clearing, cleaning, and checking of the levees in terms of their ability to hold water. In the Lockport area, the canal and levees appear to be in fair condition. "In the late 1970s the city rebuilt the levee with clay on the west bank from 2nd to 8th Streets." (McCluskey, 1992). In any case, a certain amount of clearing and cleaning of the canal will be required to improve its condition.

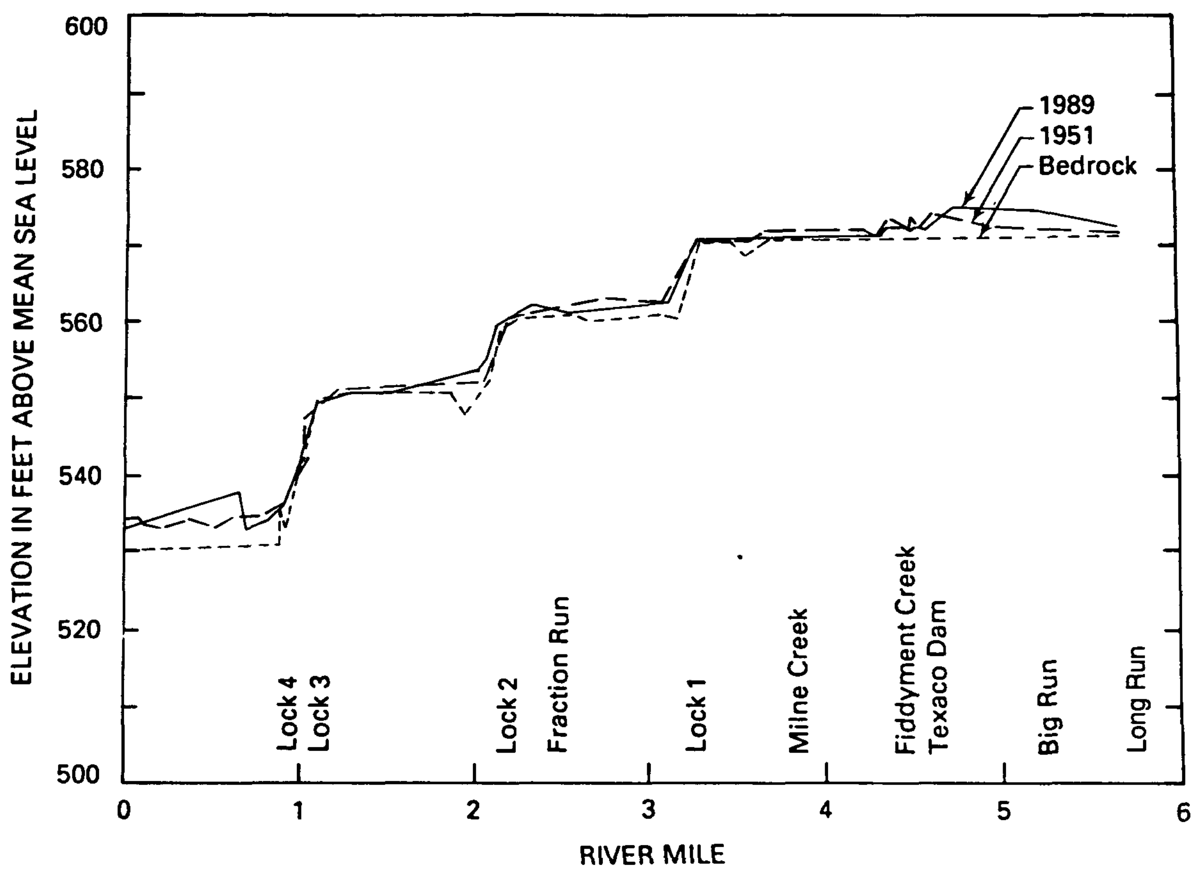


Figure 5. Comparison of bed profiles of the Illinois and Michigan Canal from Joliet to Lockport in 1951, 1989, and bedrock

Control Structures

The main water flow control structures within the study area are four locks from Joliet to Lockport, Texaco Dam, the side-overflow weir upstream of the Texaco Dam, and three 4-ft culverts between 9th Street and Lock 1. The locks have all been out of use and have deteriorated, and thus they no longer control the flow of water in the canal except for their constricted cross sections that might control water surface elevations during flood flows.

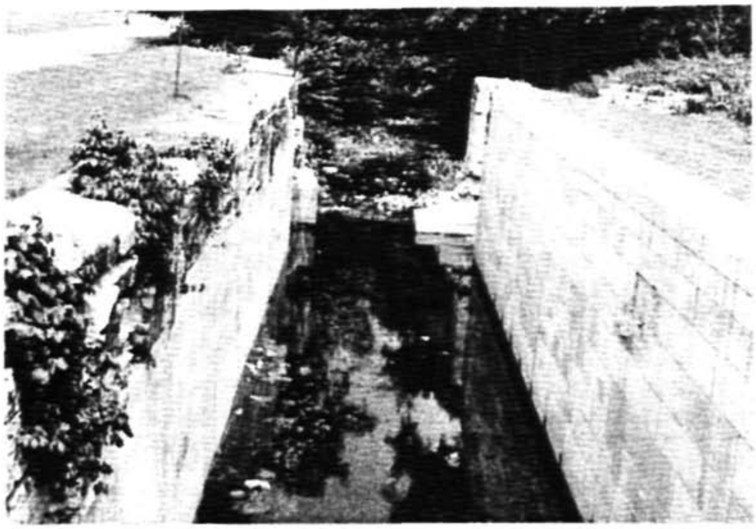
Figure 6 shows the present condition of Lock 1 and the condition of the canal upstream and downstream of Lock 1. Lock 1 has been partially stabilized, but the renovation work needs to be completed along with the addition of a water-level control weir or new gates at the lock to control the water level upstream of the lock.

Lock 2 is located about one mile downstream of Lock 1, and figure 7 shows the present condition of the canal here. Trees and brush have overgrown the lock structure. Further upstream the canal is unmaintained, has overgrown trees and brush, and has been breached in the past. Immediately downstream, the canal has accumulated significant amounts of sediment and will require major channel cleaning.

The Texaco Dam, shown in figure 8, was built by TRMI to control the flow of water in the canal. Each of the dam's eight gates were formerly controlled separately but have not been operational for several years. The dam is presently used to hold back water for use in firefighting in the TRMI plant.

The side-overflow weir upstream of the Texaco Dam, shown in figure 9, is a stack of two steel beams placed along a break in the levee to allow overflow of floodwater from the canal to Deep Run Creek to reduce flooding in the TRMI plant area. The condition of the canal in the vicinity of the overflow weir is shown (top and bottom pictures in figure 9). Overgrown vegetation has made the canal more of a marsh-wetland environment than a navigation canal.

Figure 10 shows the three 4-ft culverts upstream of Lock 1 and downstream of 9th Street. These culverts were installed by the City of Lockport on the west levee of the canal to divert floodwaters from the I&M Canal to Deep Run. They function only during major floods when water elevation in the canal exceeds 575.7 feet above mean sea level (ft-msl).



Looking Upstream from Lock 1

Looking at Lock 1 from Downstream

Looking Downstream from Lock 1

Figure 6. Lock 1 and its surroundings

Looking Upstream
from Lock 2

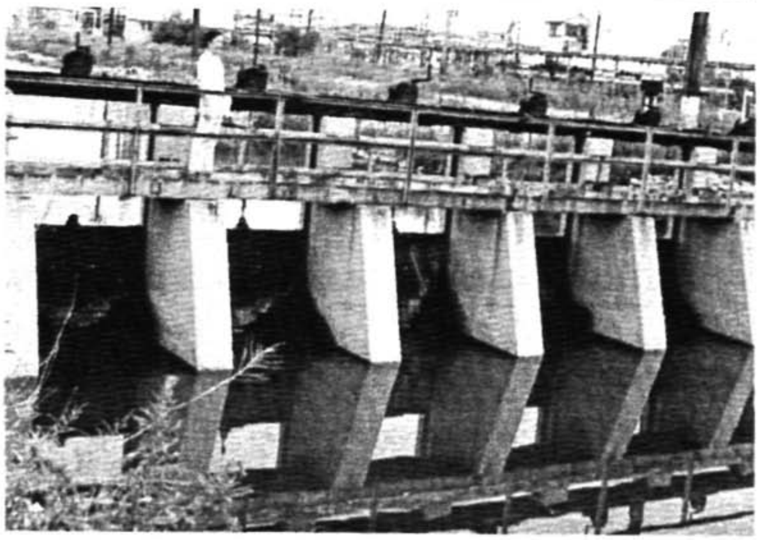
Looking at Lock 2
from Upstream

Looking at Lock 2
from Downstream

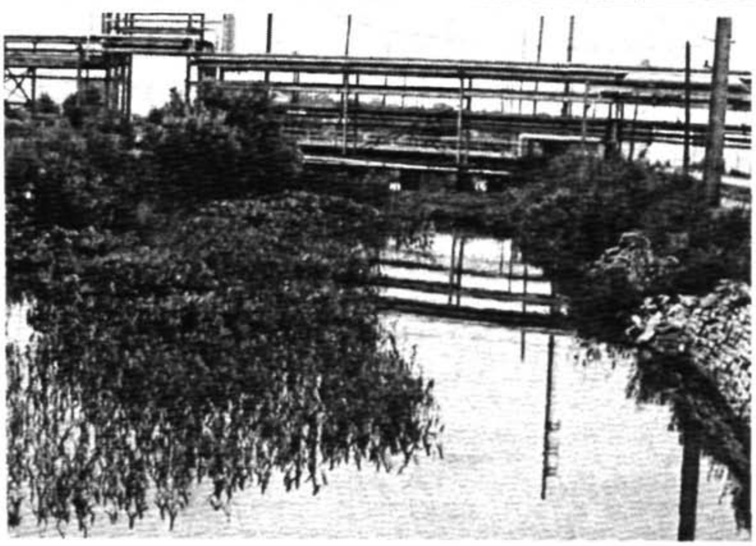
Looking Downstream
from Lock 2

Figure 7. Lock 2 and its surroundings

Looking Upstream from
the Texaco Dam



Texaco Dam



Looking Downstream from
the Texaco Dam

Figure 8. The Texaco Dam and its surroundings

Looking Upstream from
the Overflow Weir

Looking at the
Overflow Weir

Looking Downstream
from the Overflow Weir

Figure 9. The overflow weir at the TRMI plant and its surroundings

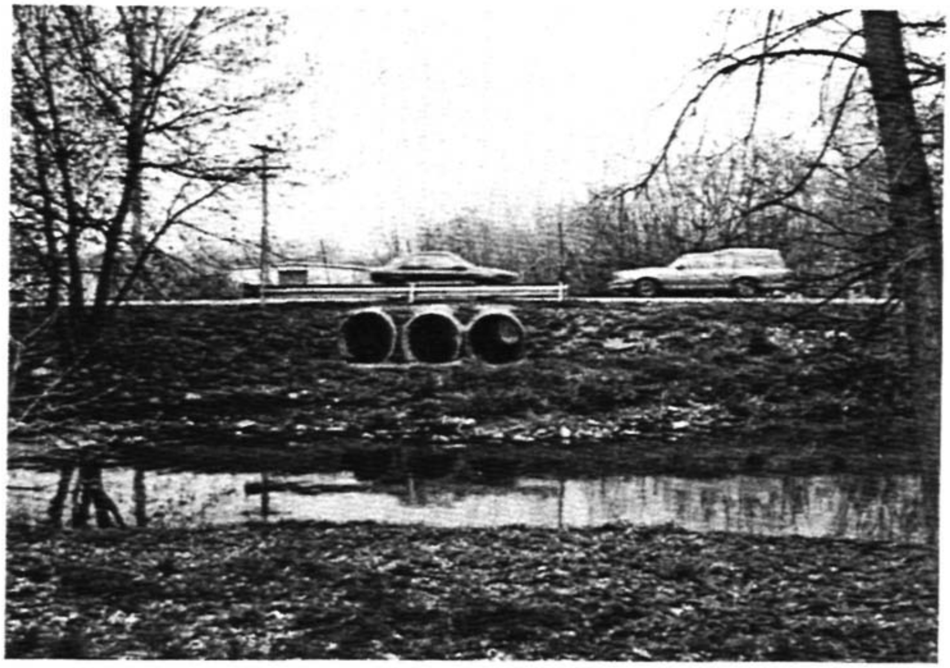


Figure 10. Culverts under Canal Street that are located between 9th Street Bridge and Lock 1

FLOOD ANALYSIS

The main objective of this phase of the project was to conduct a detailed hydrologic and hydraulic study to investigate flooding conditions along the I&M Canal so that any changes in the canal will not adversely impact existing conditions. The hydrologic analysis involved the analysis of storm events and the computation of flood hydrographs. When streamflow records of storm events were available, flood hydrographs were generated as a tabulated or plotted set of stage or discharge data at different times during each storm event. However, when streamflow records were not available, the hydrologic analysis used some mathematical models to generate flood hydrographs from records of precipitation, soil moisture, and watershed hydrologic characteristics that are either measured or estimated. Once flood hydrographs were generated, hydraulic models were used to compute flood elevations, velocities, and the areal extent of flooding. Hydrologic and hydraulic analyses for the I&M Canal and its drainage areas are presented in the following sections.

Determination of Synthetic Storm Events

The study area has no long-term precipitation gaging station within its boundaries. The closest precipitation station is at the Joliet-Brandon Road dam, south of the study area. Figure 11 shows the locations of the precipitation stations in the northeastern section of Illinois. Rainfall values for storm events with 10-, 50-, and 100-year return periods, based on the Joliet-Brandon Road dam data, were obtained from Illinois State Water Survey Bulletin 70 (Huff and Angel, 1989). Rainfall values for storm events with 500-year return periods were obtained by simple extrapolation from Bulletin 70. Figure 12 shows the rainfall frequency-duration curves for the study site, and table 2 presents the values. Storm durations of 2, 6, 12, and 24 hours were selected for hydrologic analysis.

Gage Installation and Data Collection

To observe field conditions during high water levels, two crest gages and one staff gage were installed along the I&M Canal. A crest gage measures the highest water level during flood events. A staff gage measures the water level in the canal during field inspections.

On April 24, 1991, a crest gage was installed on the east entrance wall of Lock 1, and a staff gage was mounted to a platform on the west bank downstream of the Texaco Dam. A second crest gage was mounted on the west wall of Lock 2 on August 8, 1991. The crest gage

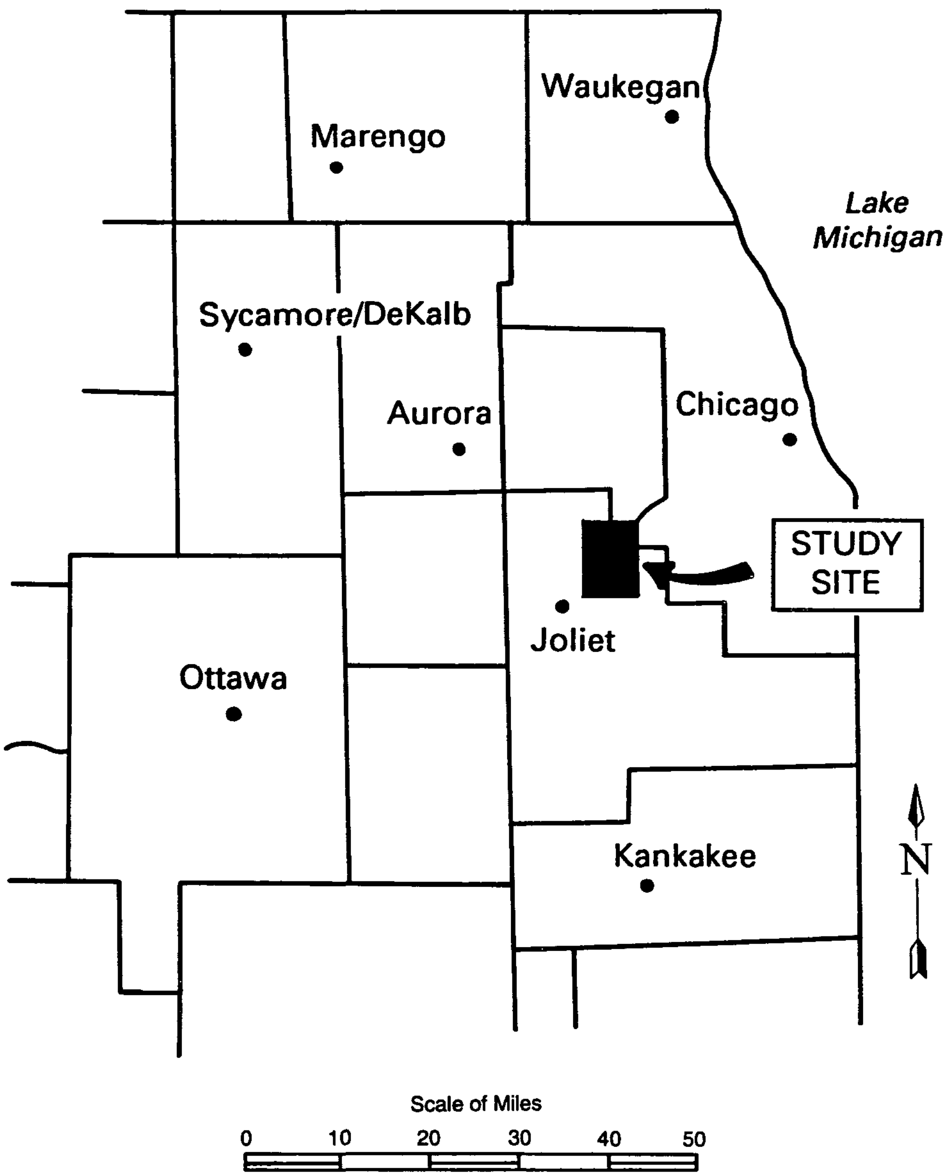


Figure 11. Location of precipitation reporting stations in northeastern Illinois

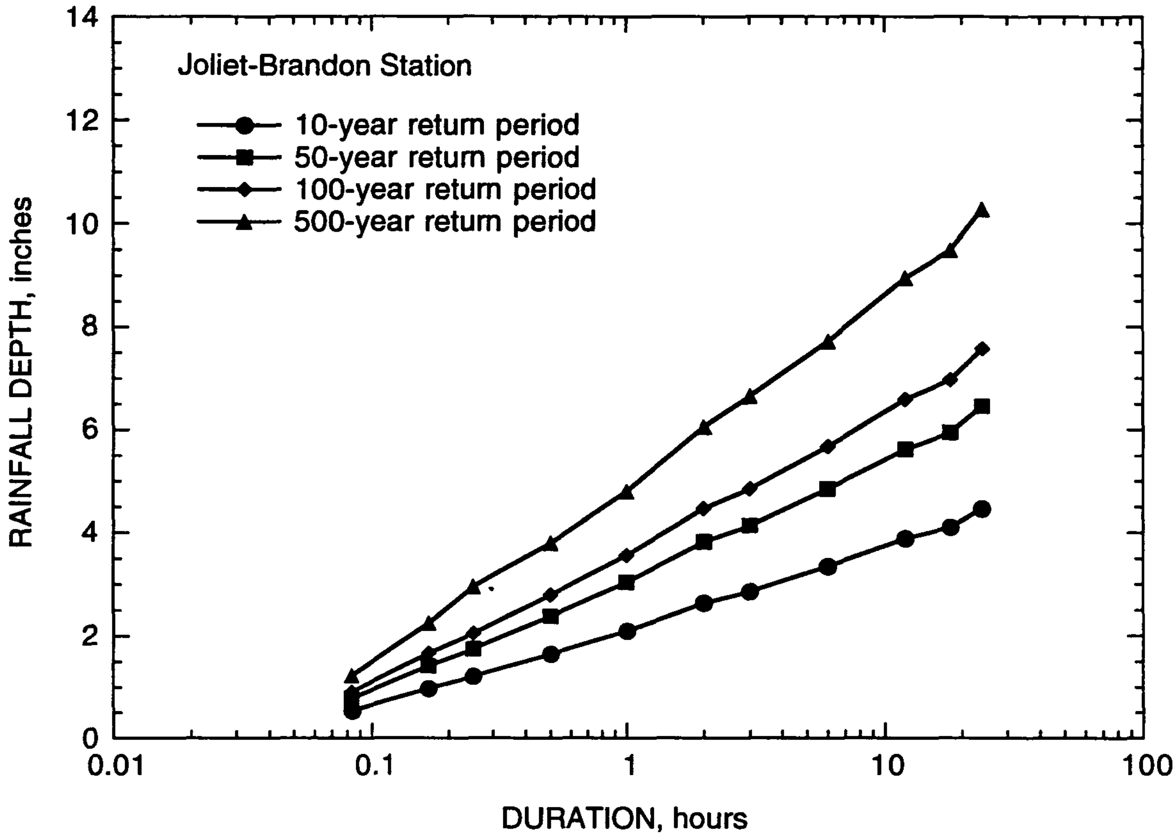


Figure 12. Rainfall duration-frequency curves
for the Joliet-Brandon Road dam precipitation recording station

Table 2. Rainfall Depths (inches) for Different Durations and Frequencies

Duration	Return period 10-year	50-year	100-year	500-year
5 min	0.54	0.78	0.91	1.21
10 min	0.98	1.42	1.67	2.25
15 min	1.21	1.75	2.05	2.95
30 min	1.65	2.39	2.80	3.80
1 hr	2.10	3.04	3.56	4.80
2 hr	2.64	3.82	4.47	6.06
3 hr	2.86	4.14	4.85	6.65
6 hr	3.35	4.85	5.68	7.72
12 hr	3.89	5.62	6.59	8.95
18 hr	4.11	5.95	6.97	9.49
24 hr	4.47	6.46	7.58	10.28

on Lock 1 was vandalized a few weeks after it was installed and was never found. On October 1, 1991, a new crest gage was installed beside a foot bridge within the TRMI plant in the vicinity of the entrance gate for safety reasons. Figure 13 shows photographs of the crest gages and the staff gage, and figure 14 shows the site locations of the gages. Gage elevations have been surveyed, and all elevations were referenced to benchmarks established by Baird & Company Land Surveyors.

Water-level marks on the crest gages were measured after any storm event mat resulted in lowering of the gates on the locks on the Chicago Sanitary & Ship Canal at Lockport. Water elevations at the staff gage, the overflow weir, and the Texaco Dam were also measured men. Table 3 provides water-level data collected to date.

Daily precipitation data from January 1991 to March 1993 have been plotted (figure 15). Precipitation totals for 1990 and 1991 are 46.55 and 35.8 inches, respectively. The 40-year average annual precipitation is 35.33 inches. Although 1991 is considered a drier year man 1990, the annual precipitation in 1991 was nevertheless close to the long-term average.

Two flooding events of significant magnitude were recorded during this project. On September 9, 1992, a 4.5-inch rainfall was recorded by a raingage within the TRMI plant. This storm event caused overbank flooding along the segment of the canal between Division Street and Lock 2. Flood crests of 578.08 ft and 560.98 ft were recorded by crest gages at the TRMI foot bridge and at Lock 2, respectively. Crest elevations were measured from water-level marks on the crest gage rods during a field trip on September 14, 1992.

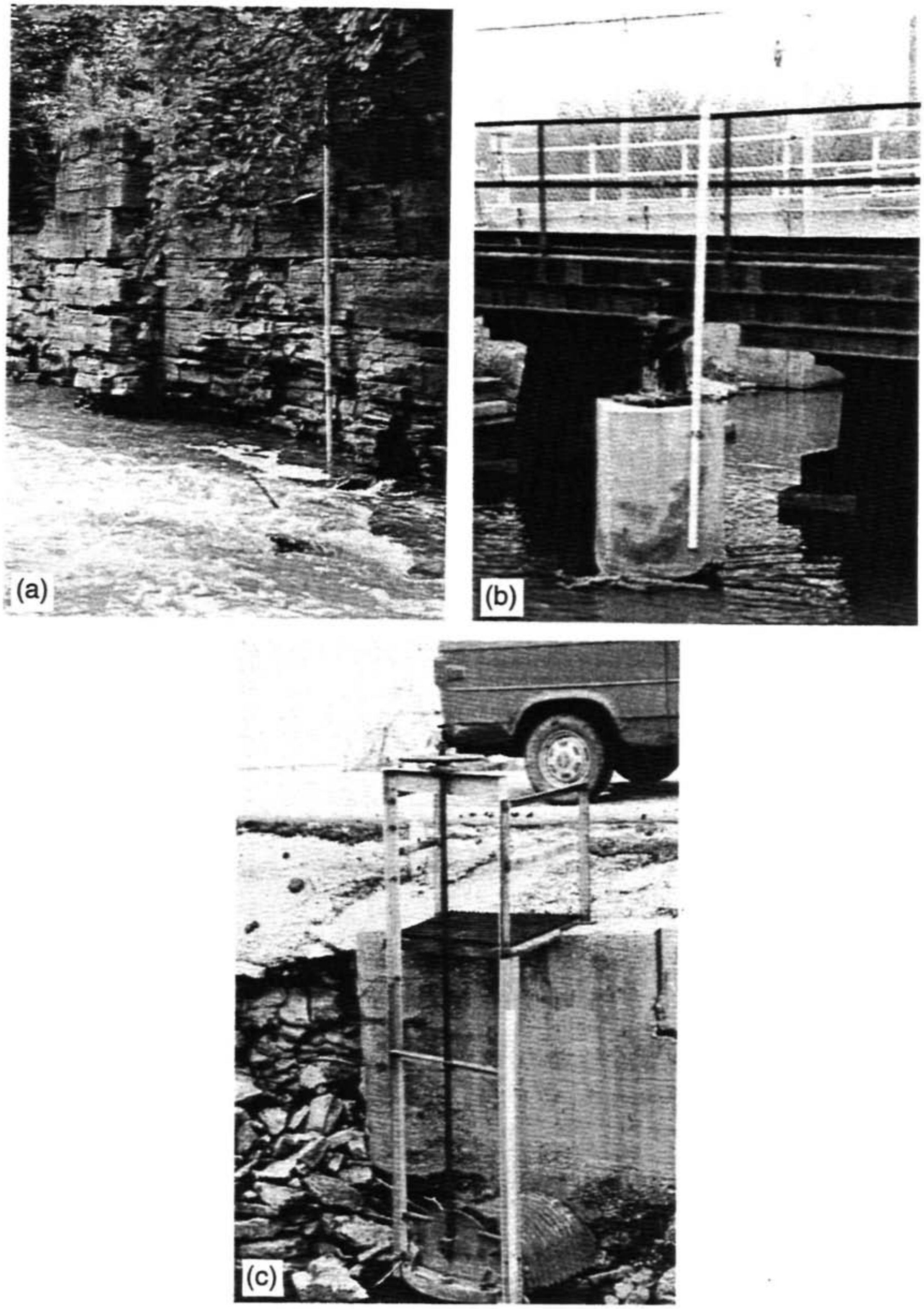
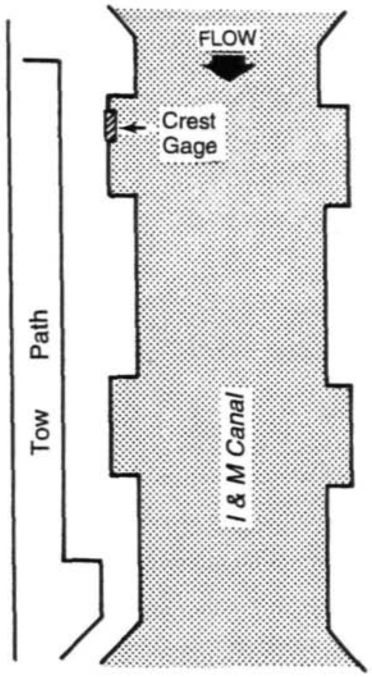
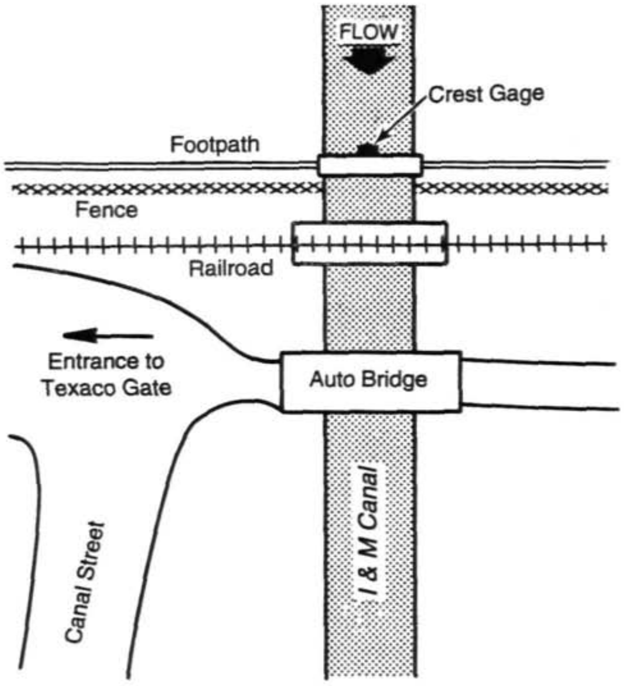


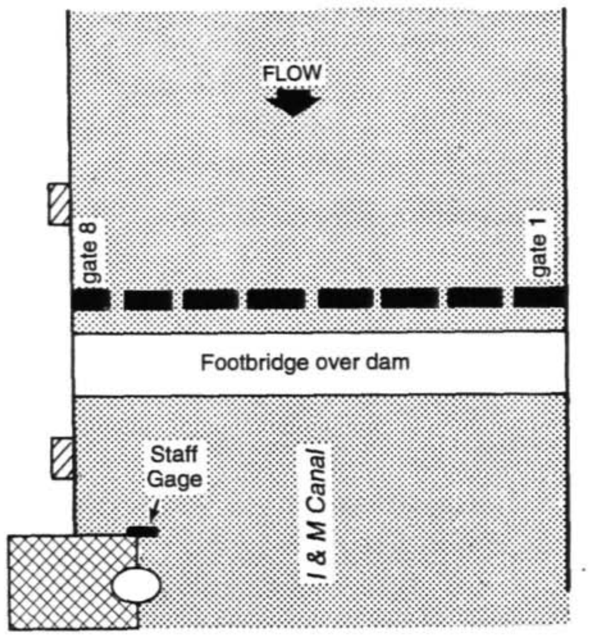
Figure 13. Crest gage at Lock 2 (a), crest gage at the foot bridge in the TRMI plant (b), and staff gage near the Texaco Dam (c)



a. Position of the crest gage on the west upstream wall of Lock 2



b. Location of the crest gage on the foot bridge inside the Texaco facility



c. Location of the staff gage on the west bank of canal, downstream of the Texaco Plant

Figure 14. Site locations of gages as shown in figure 13

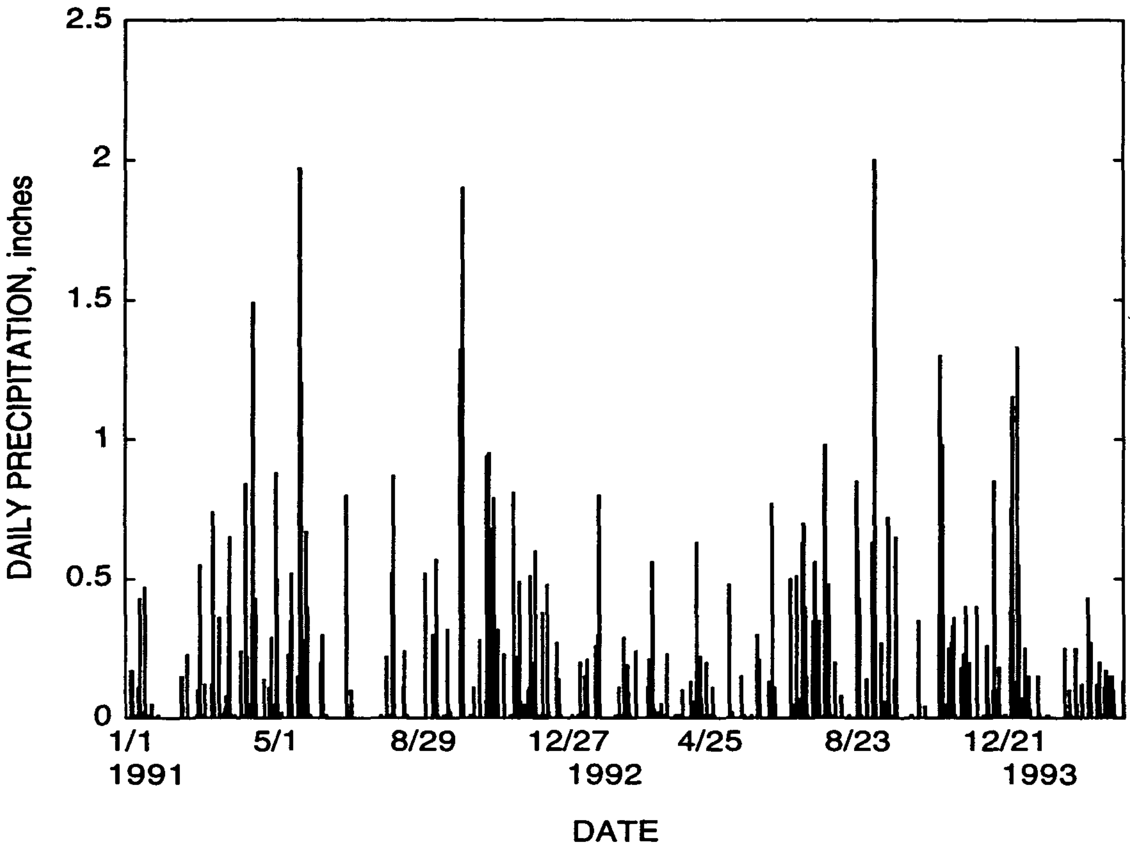


Figure 15. Daily precipitation at Lockport-Brandon Road Dam

Table 3. Measured Water-Level Elevations (feet)

Date	*Texaco Dam staff gage*	*TRMIplant crest gage*	*Lock 2 crest gage*	*Deep Run overflow weir*
04/24/91	573.75			
08/28/91		*		578.23
10/01/91	572.77	556.18		
03/24/92	573.25	574.29	557.08	
04/07/92	573.10	*	556.43	
04/24/92	573.30	*	556.76	578.19
07/17/92	573.50	574.13	556.93	
09/14/92	573.70	578.08	560.98	
01/05/93	576.35	577.55	560.92	579.79
06/11/93	574.25	**	559.34	

Notes:
Elevation of zero mark on staff gage: 572.35
Elevation of TRMI plant crest gage, top of stick: 583.63
Elevation of Lock 2 crest gage, top of stick: 568.38
Top of Deep Run overflow weir: 577.79
Average bed elevation at Lock 2: 555.40
** The water level was lower than the bottom of the gage.
*** Gage out of service.

The second flood event occurred on January 5, 1993. Figure 16 depicts the condition along several segments of the canal on that date. The flood elevations at the TRMI plant and Lock 2 gages were 577.55 and 560.92 ft, respectively. The water-level reading on the TRMI plant staff gage was 576.35 ft around 3:00 p.m. On the downstream side of the Texaco Dam, this water level is about 2 ft below the top of the dam.

Hydrologic Analysis

The hydrologic analysis requires investigating streamflow records when available and generating flood discharges for different return periods. However, there is only one water discharge record station in the study area, which is located on Long Run. This U.S. Geological Survey (USGS) station is installed on the State Street bridge in Cook County about two miles south of Lemont at river mile 5.4. This gaging station measures runoff from 20.9 sq mi of Long Run watershed. Although it cannot be used to analyze runoff from the whole watershed draining into the I&M Canal, it will be very useful in developing and calibrating hydrologic models for the whole area so mat model results represent real values.

Figure 16a. The Illinois and Michigan Canal at Lock 1 on January 5, 1993

Figure 16b. The Illinois and Michigan Canal at Lock 2 on January 5, 1993

Figure 16c. The Illinois and Michigan Canal at the Texaco Dam on January 5, 1993

Figure 16d. The Illinois and Michigan Canal downstream of Lock 2 on January 5, 1993

Figure 16e. The Illinois and Michigan Canal at the Deep Run Junction on January 5, 1993

Figure 16f. The overland flow condition from the Illinois and Michigan Canal looking north from the Deep Run foot bridge on January 5, 1993

The lack of detailed streamflow records required that a hydrologic model be developed to generate flood hydrographs for each tributary stream. The HEC-1 flood hydrograph computer program (Hydrologic Engineering Center, 1990a) was used to compute the streamflow hydrographs for each of the seven watersheds draining into the I&M Canal. Streamflow records at the USGS station on Long Run were used in the model to calibrate the model parameters.

The application of the HEC-1 hydrologic model to simulate streamflow hydrographs involved the computation of precipitation losses, runoff, and baseflow contributions. The model uses several options to determine each of these hydrologic components. The Soil Conservation Service (SCS) Runoff Curve Number method and the Clark unit hydrograph method were selected to compute the infiltration loss and the total runoff, respectively. Baseflow was computed with an exponential function. Model parameters can be determined or estimated from hydrologic records and field observations or by using the calibration option in the model.

Calibration with Historical Events. Calibration of the hydrologic parameters for the HEC-1 model was based on existing streamflow records of the upper sub-watershed of Long Run. It is the only watershed with a streamflow recording station in the study area and thus provides a good basis for estimating the model parameters. With a drainage area of 25.5 sq mi, the Long Run watershed is the largest watershed and a major contributor of flow to the I&M Canal. The streamflow recording station is located at the downstream side of the bridge on State Street, south of the city of Lemont. The sub-watershed of Long Run has its drainage outlet at this location, with a drainage area of 20.9 sq mi and a reach length of 8.3 mi.

Model calibration used available daily discharge records from the USGS station and the corresponding daily precipitation records from the Joliet-Brandon Road precipitation station. Calibration involved a comparison of observed hydrographs and hydrographs computed by the model. Four rainfall events occurring during different months between 1976 and 1988 were selected for calibration purposes. Table 4 presents the results of the calibration and the percentage error for runoff volume, peak discharge, and time to peak. Figure 17 presents the observed and computed streamflows of the April 1979, June 1981, July 1976, and November 1988 events. Parameter values were optimized by calibrating each parameter while other parameter values remained constant. The percentage errors for runoff volume are all below 1

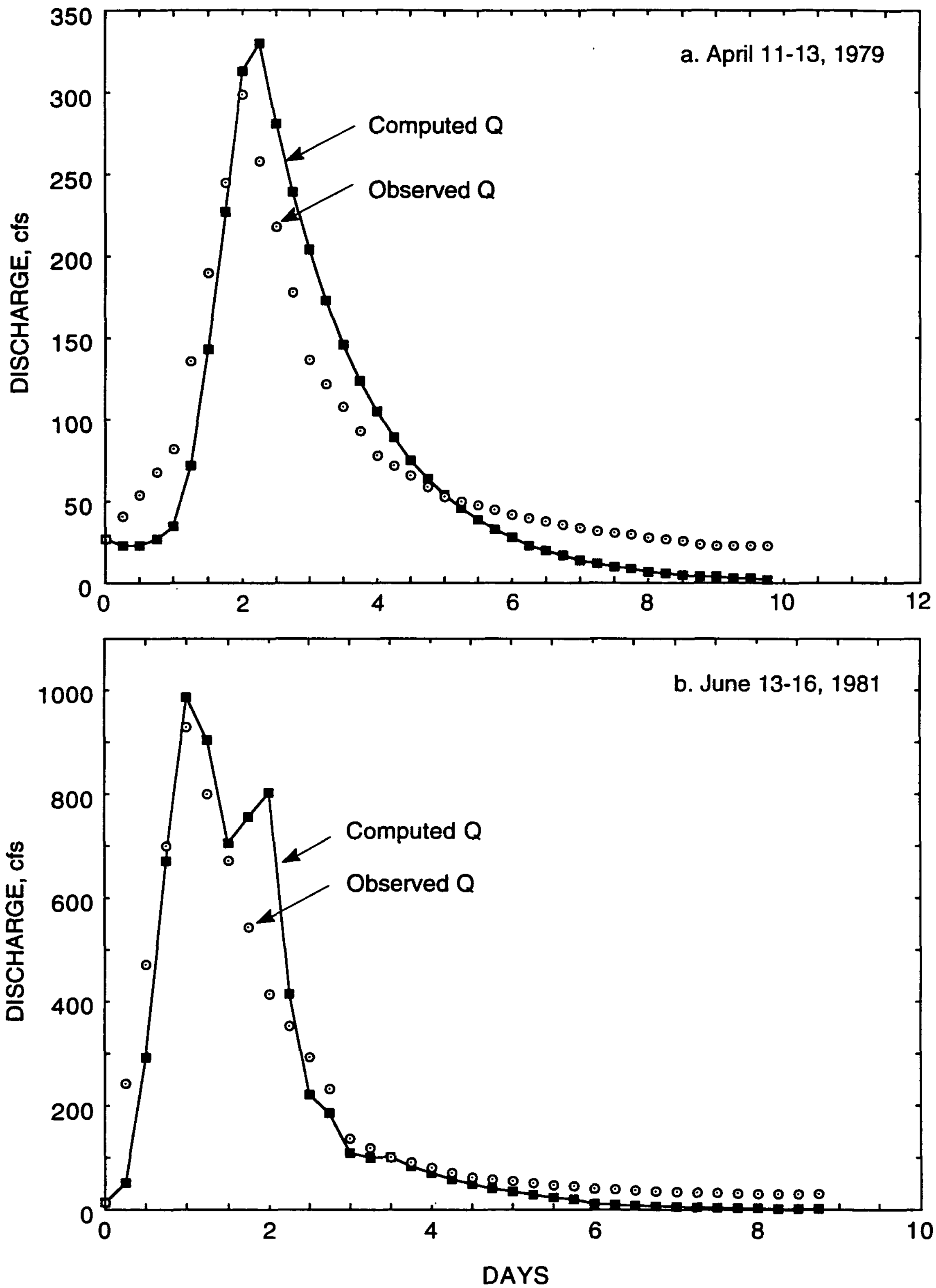


Figure 17. Observed and computed hydrographs for Long Run for daily rainfall of (a) April 11-13, 1979 and (b) June 13-16, 1981

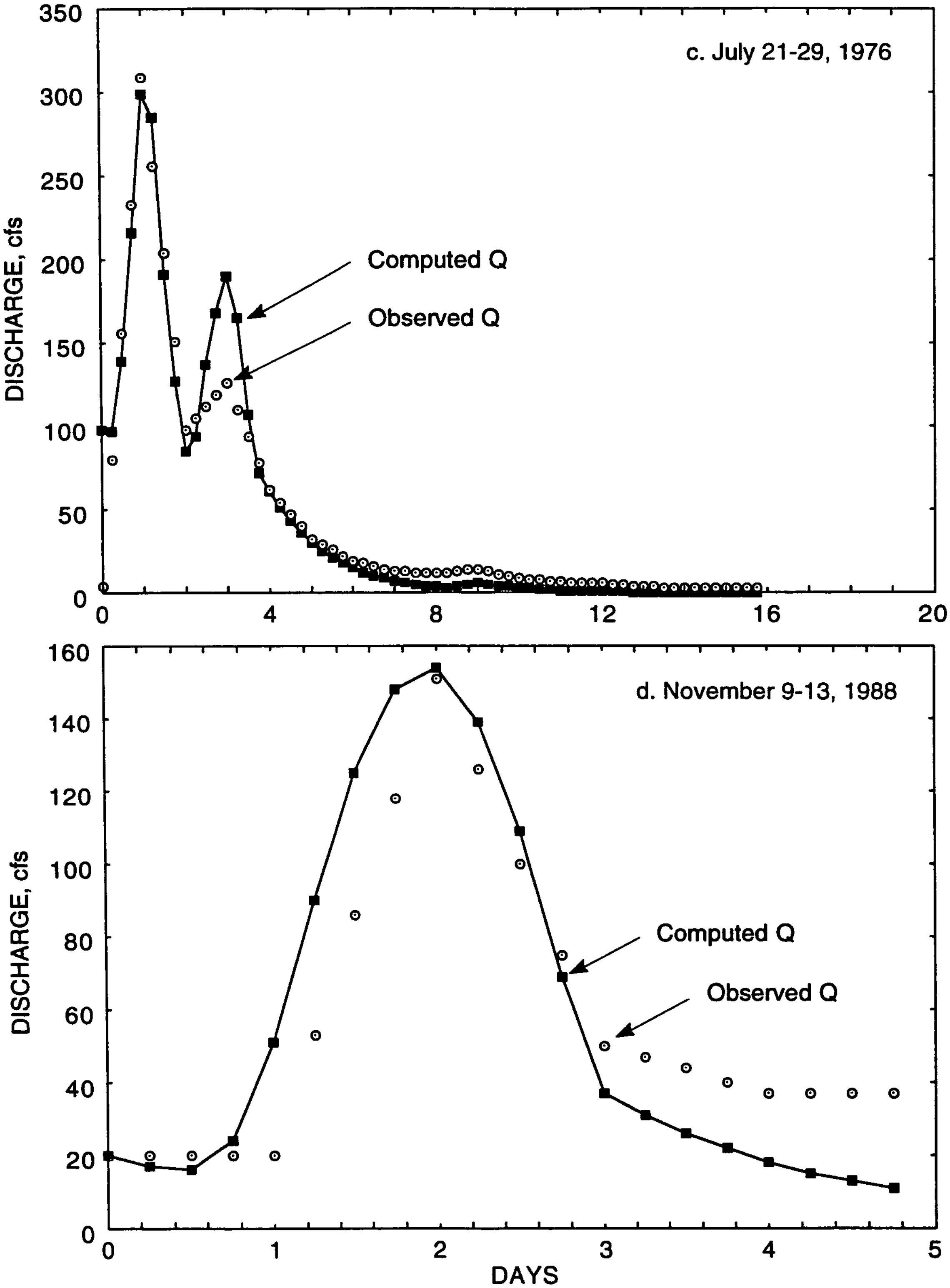


Figure 17. Observed and computed hydrographs for Long Run for daily rainfall of (c) July 21-29, 1976 and (d) November 9-13, 1988

percent. The errors for the time to peak are minimal only for the July 1976 and June 1981 events. In general, the computed mean daily discharge fits closely with the observed values.

The curve number (CN) in table 4 represents the rate of infiltration of rainwater into the ground. The parameter STRTL is the initial abstraction or the amount of rainwater lost due to ground surface storage. These parameters vary with the antecedent moisture condition of the soil. For example, rain falling on frozen ground in the winter quickly runs off **the** ground surface with very little infiltration, but dry conditions in the summer will favor higher initial abstraction and infiltration. The seasonal variability of CN and STRTL are closely reflected in the calibration of these parameters for the historical storms shown in table 4.

Table 4. Calibration Results and Percentage Errors for Long Run

	Optimized parameters				*Percent error*		
Rainfall event	*TC (hours)*	*R (hours)*	*STRTL (inches)*	*CN*	*Runoff volume*	Q_p	t_p
04/79	6.18	35.87	0.13	81.25	-0.05	33.31	3.57
06/81	6.18	3.0	0.55	43.67	0.82	12.46	0.00
07/76	6.68	13.03	0.04	80.98	-0.07	-3.21	0.00
11/88	6.33	13.29	0.41	66.71	-0.16	43.11	12.50
Average	6.34	16.30	0.28	68.15			

Note: TC = time of concentration, R = storage factor, STRTL = initial loss, CN = curve number, Q_p = peak discharge, and t_p = time to peak discharge.

The values of the time of concentration (TC) and the storage factor (R) shown in table 4 are for the segment of the Long Run drainage basin upstream of the USGS gaging station at Lemont. Parameter values were estimated for the ungaged tributary watersheds using the procedure established by Graf et al. (1982). They showed that the sum of the parameters TC and R is related to the stream length (L) and the main channel slope (S) by the following expression:

$$(TC + R)_e = 35.2\, L^{0.39}\, S^{-0.78}$$

where $(TC + R)_e$ is the estimate of the sum of TC and R. Graf et al. (1982) also obtained regional values of the ratio R/(TC + R), which can be used with values of $(TC + R)_e$ to compute estimated values of the time of concentration and the storage coefficient for ungaged basins. For the study area, the ratio R/(TC + R) is obtained as 0.6. The Graf et al. procedure was used to estimate TC and R for the upper Long Run basin as 7.07 and 10.64, respectively. This result is in reasonable agreement with the averages of the calibrated parameters, which are also given in table 4. Table 5 shows the estimated values of time of concentration and storage coefficient for the tributary stream watersheds in the study area. These values are used in subsequent analysis.

Table 5. Estimated Values of TC and R for Ungaged Drainage Basins Draining into the I&M Canal

River Basin	*TC (hours)*	*R (hours)*
Convent	4.54	6.81
School	1.74	2.61
Long Run	5.51	8.26
Big Run	1.33	2.00
Fiddyment	1.71	2.57
Milne	1.21	1.81
Fraction	2.41	3.61

Verification with Recent Storms. The parameter calibration with historical storms presented in the previous section is useful in the selection of parameters for the ungaged tributary watersheds in the study area. However, since CN and STRTL depend partly on the antecedent moisture condition, their values vary with each storm event. The model parameters were verified with the two storm events, which occurred during the execution of the project and for which peak stages were recorded.

The September 9, 1992, storm produced a recorded rainfall depth of 4.5 inches at the TRMI plant. The measured crest elevations as a result of this storm can be used to verify the HEC models once the time distribution of the storm event is known. The raingage on the TRMI plant does not measure hourly or sub-hourly rainfall. However, the corresponding rainfall distribution at a nearby station can be used to reconstruct the temporal rainfall distribution for the TRMI gage. The hourly stations closest to the study site are the Crete and Gebhard precipitation recording stations, and the distribution of the recorded hourly rainfall depth for the two stations on September 9, 1992 is similar. Therefore me data for Crete were chosen to synthesize the hourly rainfall depth for the study site.

For the second flooding event, which occurred on January 5, 1993, the hourly precipitation record for Gebhard (1.80 inches) was used to generate the hourly rainfall depth. The Gebhard recorded precipitation is very close to the 1.76 inches recorded at the TRMI plant. Crete, on the other hand, recorded a mean daily precipitation of 1.90 inches. Table 6 shows the hourly rainfall depths generated for the September 1992 and January 1993 storm events.

The HEC models were used to compute flood stages that correspond to the recorded crest elevations for the storm events. Water surface profiles were computed for the September 1992 and January 1993 flooding events (figures 18a and 18b, respectively). As shown in the figures, very close fittings of the recorded stages were obtained for the two events. The STRTL and CN values used for the storm events are:

Flooding event	*STRTL (inches)*	*CN*
09/09/92	1.85	52
01/05/93	0.22	90

These values appear to be reasonable since in the winter, infiltration and initial abstraction are small due to the freezing of the soil. Also, dry conditions in the summer require higher initial abstraction and infiltration rates. The STRTL and CN values corresponding to the September 1992 event were used in the analysis described in subsequent sections of this report.

Model Application. Using the values of TC and R in table 5 and CN and STRTL used to generate the water elevations for the September 1992 flooding event, flood hydrographs were computed for different durations and frequencies of rainfall events for all the tributary watersheds. Using the Muskingum-Cunge routing option in the HEC-1 model, flood hydrographs in the I&M Canal were then computed at cross sections immediately upstream of the Deep Run diversion, Lock 1, and Lock 2. These discharge hydrographs are for the 2-, 6-, 12-, and 24-hour storm durations for each frequency of rainfall event. The 24-hour duration events produced the highest peakflows and runoff volumes, so the computed discharge hydrographs corresponding to the 24-hour storm duration rainfall events were used for the hydraulic analysis in the next section. Figure 19 shows the 10-year, 24-hour hydrographs for the seven tributary streams and figure 20 for the three locations along the canal. Appendix A provides the computed hydrographs for other frequencies. The computed hydrograph for Long

Table 6. Hourly Rainfall Depths Generated for Recent Storm Events

Date	*Time*	*09/09/92 flood (indies)*	*Date*	*Time*	*01/05/93flood (inches)*
09/09/92	9:00 a.m.	0.63			
	10:00	0.13	01/03/93	11:00 p.m.	0.1
	11:00	0.0	01/04/93	12:00 a.m.	0.29
	12:00 p.m.	0.0		1:00	0.1
	1:00	0.12		2:00	0.1
	2:00	0.25		3:00	0.19
	3:00	0.12		4:00	0.19
	4:00	0.12		5:00	0.1
	5:00	0.0		6:00	0.1
	6:00	2.12		7:00	0.0
	7:00	0.75		8:00	0.0
	8:00	0.13		9:00	0.0
	9:00	0.13		10:00	0.0
		Total 4.50		11:00	0.0
				12:00 p.m.	0.0
				1:00	0.1
				2:00	0.0
				3:00	0.19
				4:00	0.0
				5:00	0.1
				6:00	0.1
				7:00	0.0
				8:00	0.0
				9:00	0.1
					Total 1.76

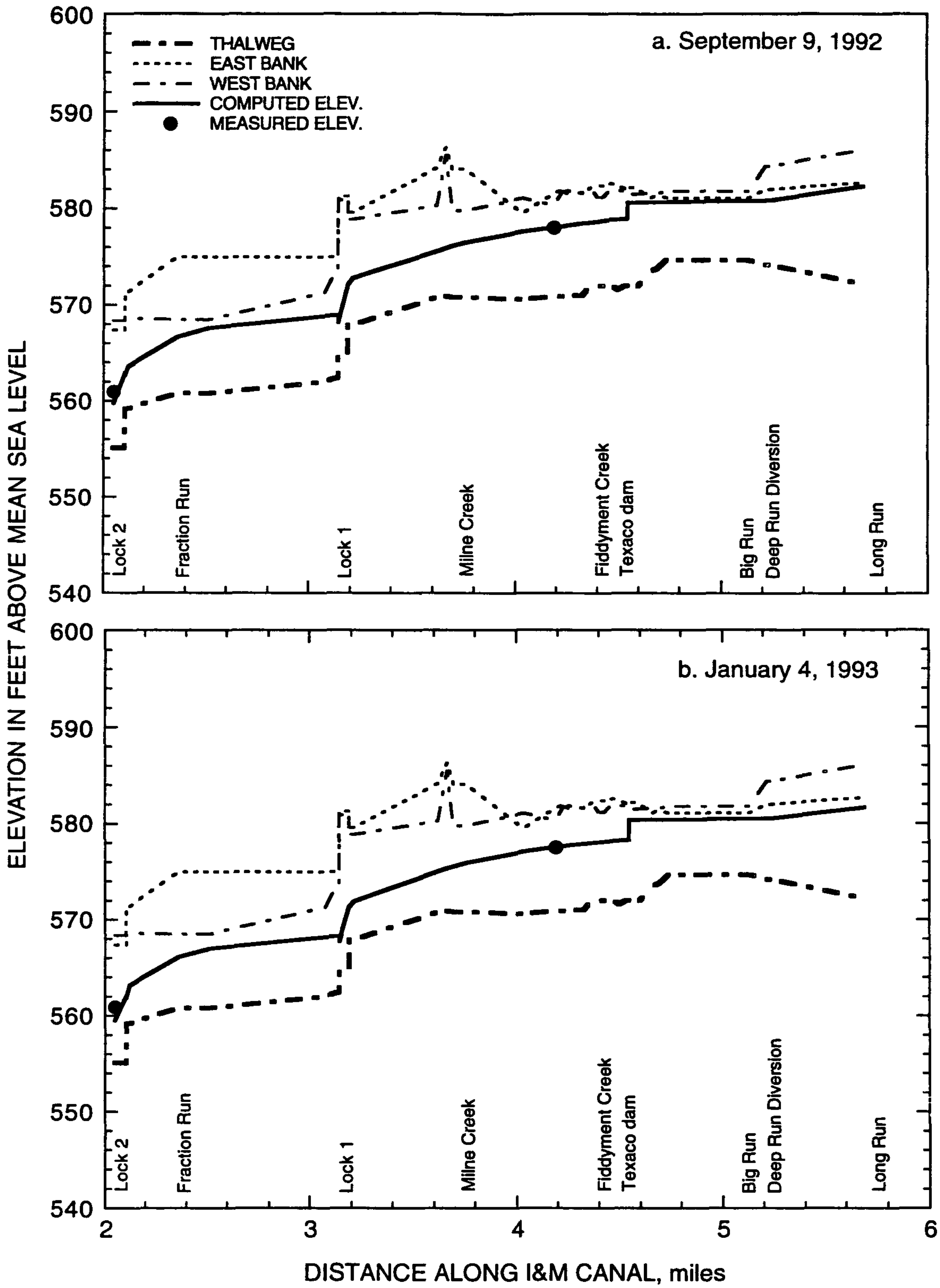


Figure 18. Measured and computed water surface elevations along the Illinois and Michigan Canal for (a) September 9, 1992 storm and (b) January 4, 1993 storm

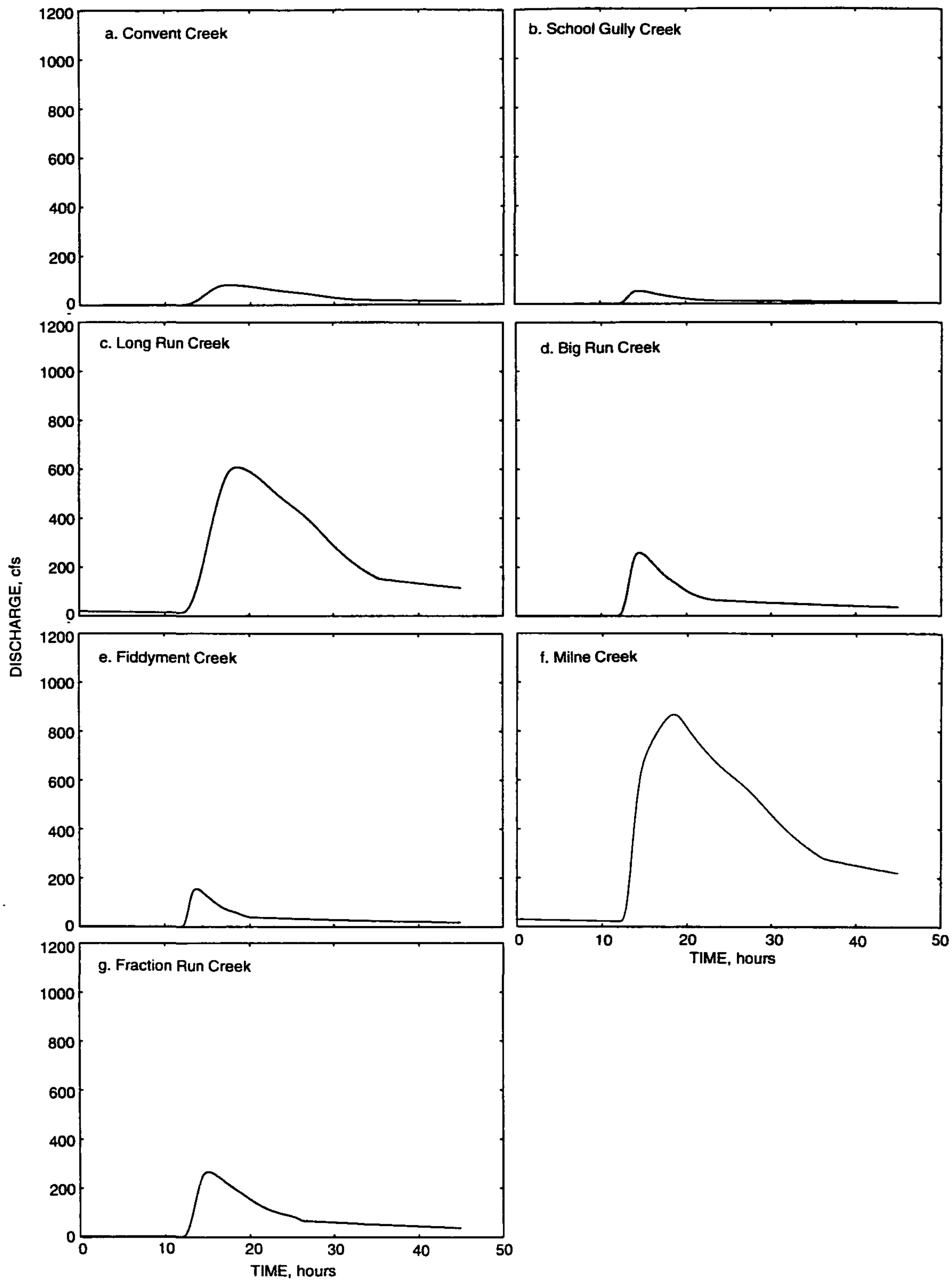


Figure 19. 10-year, 24-hour flood hydrographs for the tributary streams

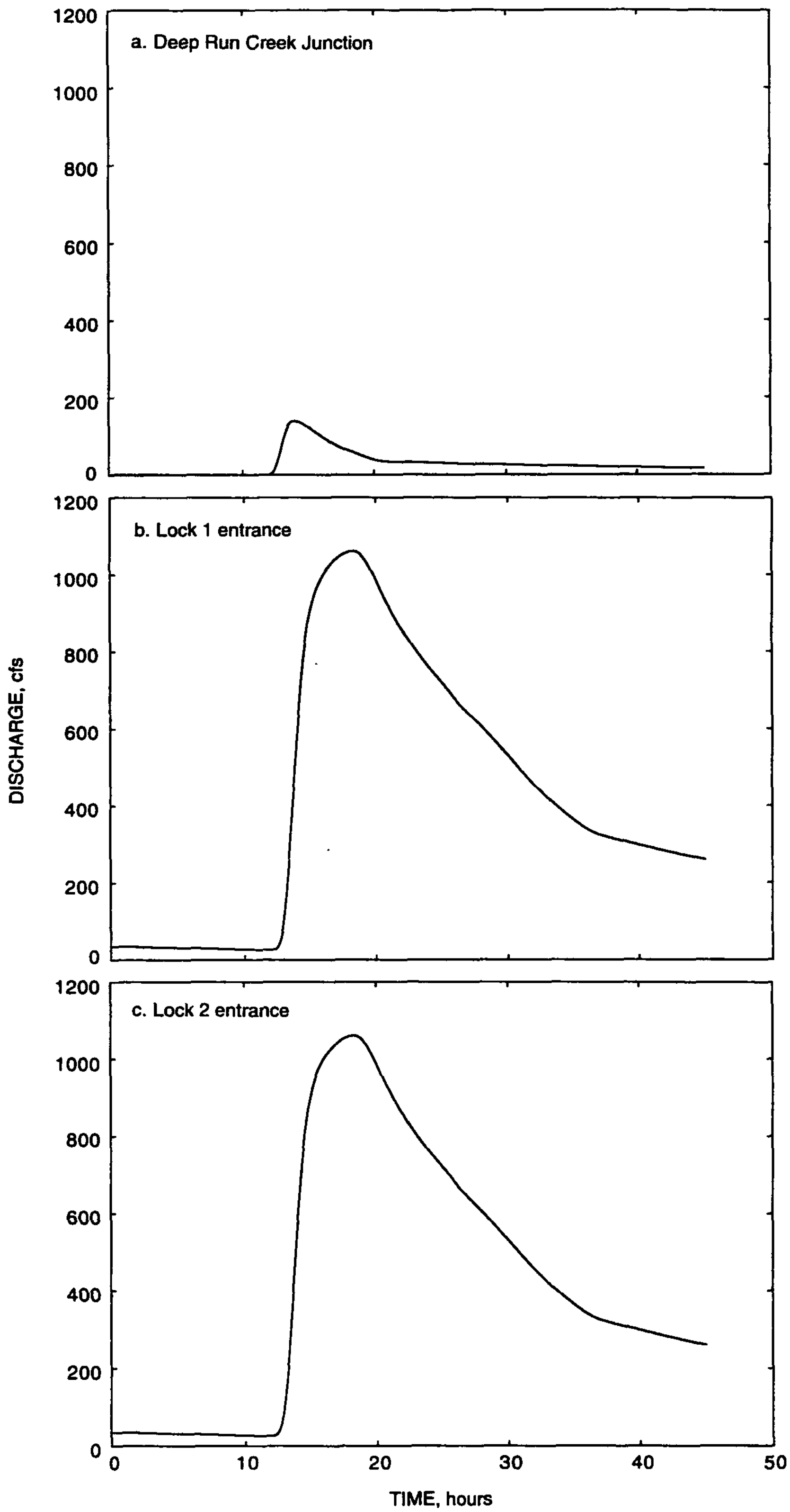


Figure 20. 10-year, 24-hour flood hydrographs for selected cross sections along the canal

Run depicts peak discharges several times larger than the hydrographs for the other watersheds since Long Run's drainage area (25.5 sq mi) is more than four times larger than, for instance, Fraction Run's, which is the second largest watershed (table 1). Appendix B provides a sample HEC-1 data file for the hydrologic simulations.

Hydraulic Analysis

The hydraulic analysis involves flood routing each storm event through the I&M Canal to determine the flood elevations along the canal. The HEC-2 flood-routing computer program (Hydrologic Engineering Center, 1990b) was used to calculate the water surface elevations in the 12.4-mi reach of the I&M Canal from the TRMI plant at Lockport to Lock 2. A total of 26 cross sections were used in the HEC-2 model. These cross sections were reported by Demissie and Xia (1990) and were originally obtained from a survey report by Baird & Company Land Surveyors (1989). Manning's roughness coefficients of 0.045 (main channel) and 0.055 (floodplains) were used. These values were obtained from a previous hydrologic study report on the I&M Canal (Demissie and Stephanatos, 1986).

Using the results of me HEC-1 simulations, me highest peakflows for the storm frequencies were obtained from the computed peak discharges. Table 7 lists these maximum peak discharges for the routed flow at the tributary junctions and the three cross sections along the canal. Water surface profiles were generated with the HEC-2 model using these discharges. Appendix B provides a sample HEC-2 input file for the hydraulic simulations.

Table 8 compares the peak discharges for the tributary streams obtained using the HEC-1 model for different frequency storms and flood peaks obtained from the frequency analysis of streamgage data in Illinois (Curtis, 1977). The analyses compared discharge values generated by the HEC-1 model to the approximate flood peak values generally obtained from regional equations. Peakflow values generated with the HEC-1 program using rainfall events underpredict or overpredict the regional values depending on whether me storm frequency is less man or greater than a 100-year storm. It is, therefore, noted that the frequencies of

Table 7. Peak Discharges along the I&M Canal
for Rainfall Events of 24-Hour Duration

Sections along I&M Canal	*Peak discharges (cfs)*			
	10-year storm	*50-year storm*	*100-year storm*	*500-year storm*
Long Run Junction	625	1,790	2,632	5,004
Deep Run Junction	279	911	1,353	2,793
Big Run Junction	335	1,066	1,573	3,191
Fiddyment Junction	472	1,429	2,090	4,139
Milne Junction	530	1,582	2,306	4,538
Fraction Run Junction	723	2,112	3,089	5,689
Lock 1	530	1,564	2,277	4,134
Lock 2	723	2,112	3,089	5,689

Table 8. Comparison of Peak Discharges (cfs) Generated by HEC-1 with those Computed
from USGS Regional Regression Equations

River basin	*Source*	*10-year storm*	*50-year storm*	*100-year storm*	*500-year storm*
Convent	HEC-1 model	82	234	241	640
	USGS regression equation	211	319	365	470
School	HEC-1 model	53	160	237	453
	USGS regression equation	130	200	230	299
Long Run	HEC-1 model	609	1,709	2,487	4,653
	USGS regression equation	1,357	2,047	2,343	3,026
Big Run	HEC-1 model	140	426	633	1,219
	USGS regression equation	398	619	715	942
Fiddyment	HEC-1 model	259	777	1,148	2,201
	USGS regression equation	644	995	1,147	1,505
Milne	HEC-1 model	156	478	712	1,372
	USGS regression equation	417	649	750	987
Fraction Run	HEC-1 model	266	782	1,151	2,189
	USGS regression equation	689	1,059	1,219	1,592

the storm events do not correspond to the frequencies of the flood events that they generate. There is also a limitation in applying regional equations for small watersheds such as those draining into the I&M Canal.

To investigate flooding in the areas adjacent to the canal, from the TRMI plant to Lock 2, water surface profiles were computed for this section of the canal (figure 21). The profiles include 10-, 50-, 100-, and 500-year storm events. The figures also show bank elevations so that areas where overtopping and flooding occur can be identified. During HEC-2 modeling, levees are overtopped for almost the entire stretch of the canal for storm frequencies greater than SO years and 6-hour duration. Figure 22 shows the computed water surface profile for the 50-year, 6-hour storm prior to bank overtopping. During simulation of larger storms, bank overtopping starts on the west bank of the canal between Fraction Run and Lock 1 and also upstream of the Texaco Dam.

Because Lock 1 acts as a control structure by constricting the flow, its impact on flooding was investigated by generating water surface profiles for the existing condition and a scenario for which the lock is replaced with the natural cross section in its vicinity. Figures 23a, b, and c show the computed water surface profiles for the 50-, 100- and 500-year storms, respectively. Removal of the lock will result in flood elevation reduction, with the impact being more pronounced in the vicinity of the lock. The influence extends upstream to the Milne Creek junction. The maximum drops in water surface elevations for the 50-, 100- and 500-year storms are 4.92, 3.98, and 3.35 ft, respectively.

The impact of the bypass upstream of Lock 1 on flooding in the I&M Canal was also examined by scrutinizing the flood-reducing capability of the existing three 4-foot-diameter culverts located between Lock 1 and Milne Creek. Comparison of the hydraulic simulation of the existing condition and a situation without culverts indicates that for storm frequencies of less than 10 years and 2-hour duration, the flood elevation does not reach the inverts of the culverts. Above this frequency, the maximum reduction in flood elevations due to the existing culvert bypass are, respectively, about 0.04, 0.31, 0.36, and 0.34 ft for the 10-, 50-, 100-, and 500-year storms.

Since the existing culvert bypass does not significantly influence the flood heights, other options were evaluated. One of the options is to place additional culverts at the lower elevations to divert more floodwaters. Three 4-ft diameter culverts were assumed to be placed about 30 feet downstream of the existing culverts for simulation purposes. The inverts

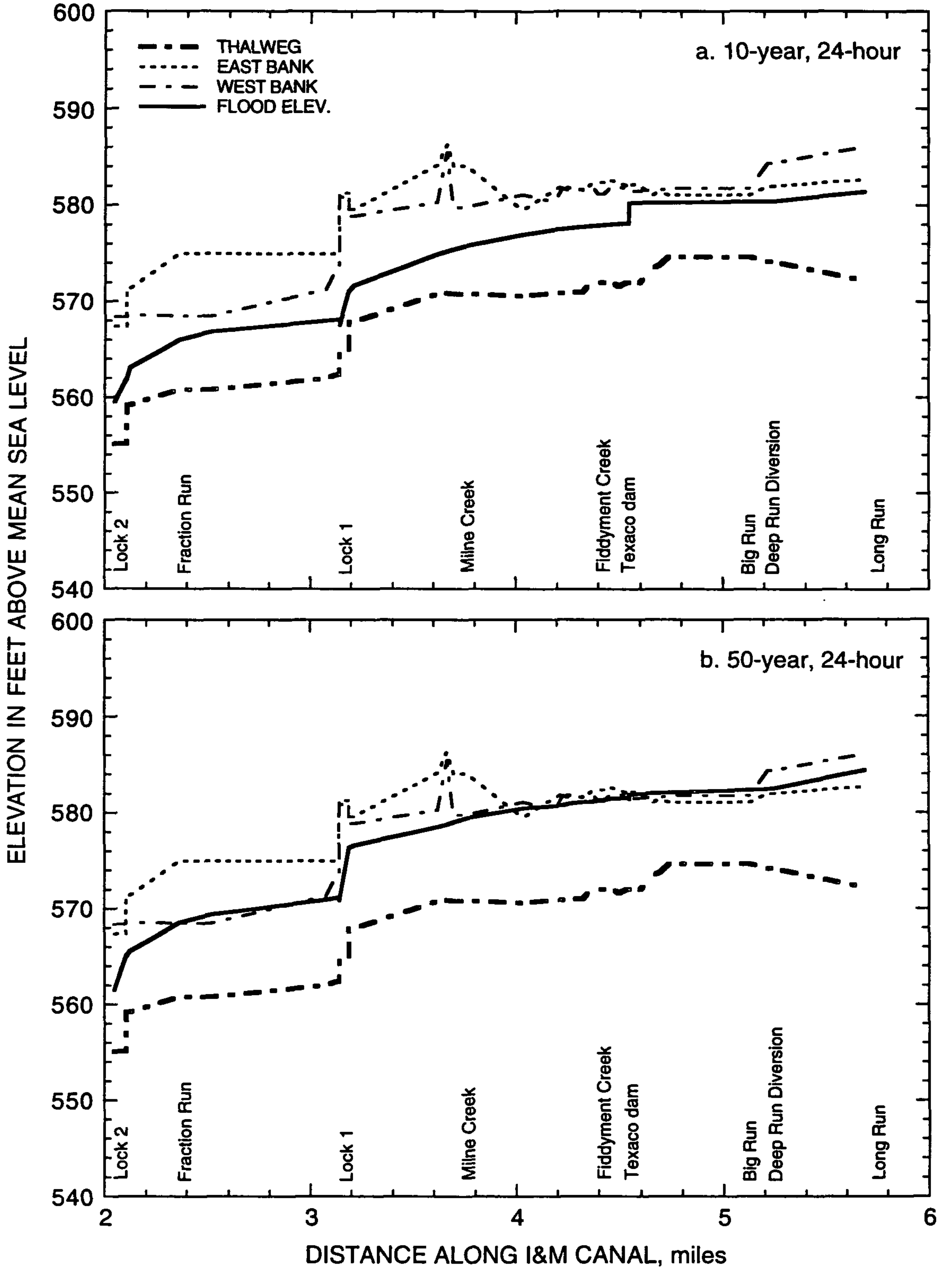


Figure 21. Water surface profiles along the Illinois and Michigan Canal for (a) 10-year, 24-hour storm and (b) 50-year, 24-hour storm

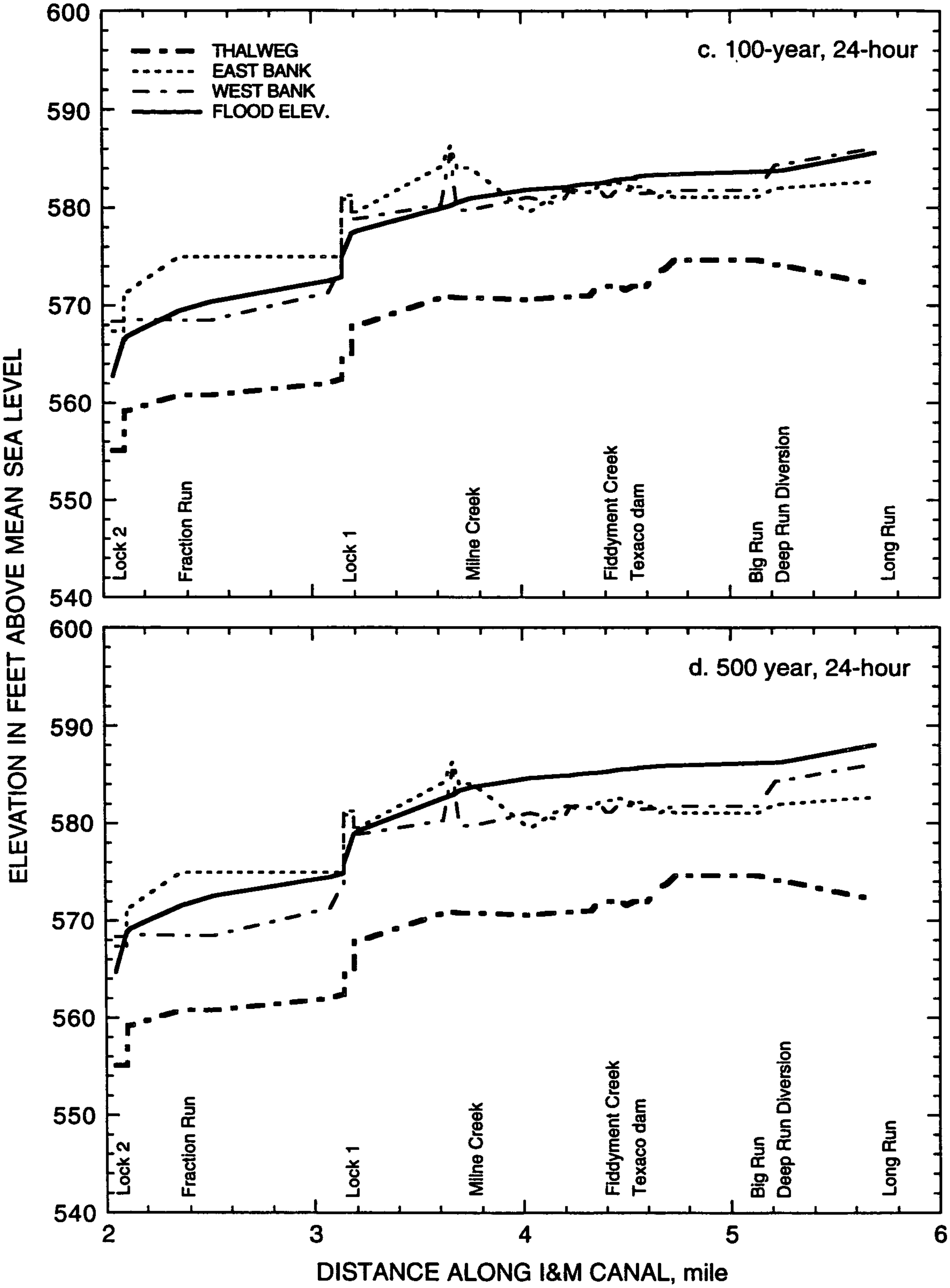


Figure 21. Water surface profiles along the Illinois and Michigan Canal for (c) 100-year, 24-hour storm and (d) 500-year, 24-hour storm

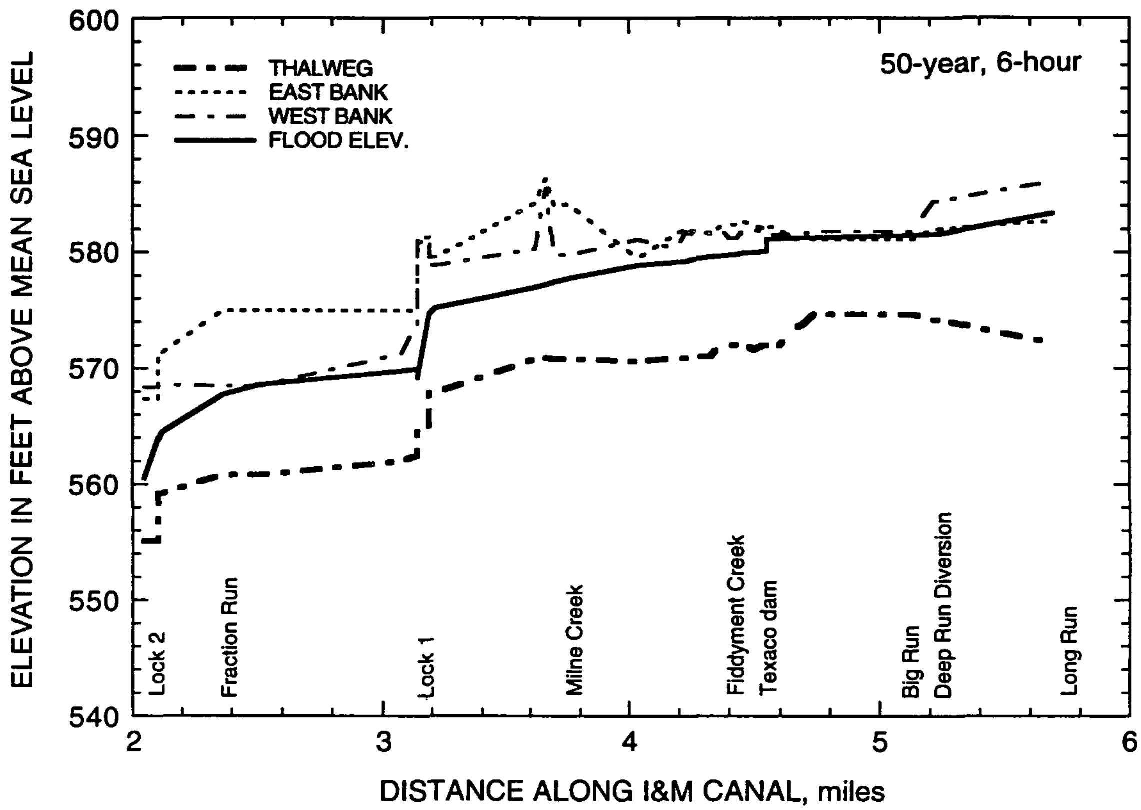


Figure 22. Computed water surface profile just prior to bank overtopping along the Illinois and Michigan Canal (50-year, 6-hour storm)

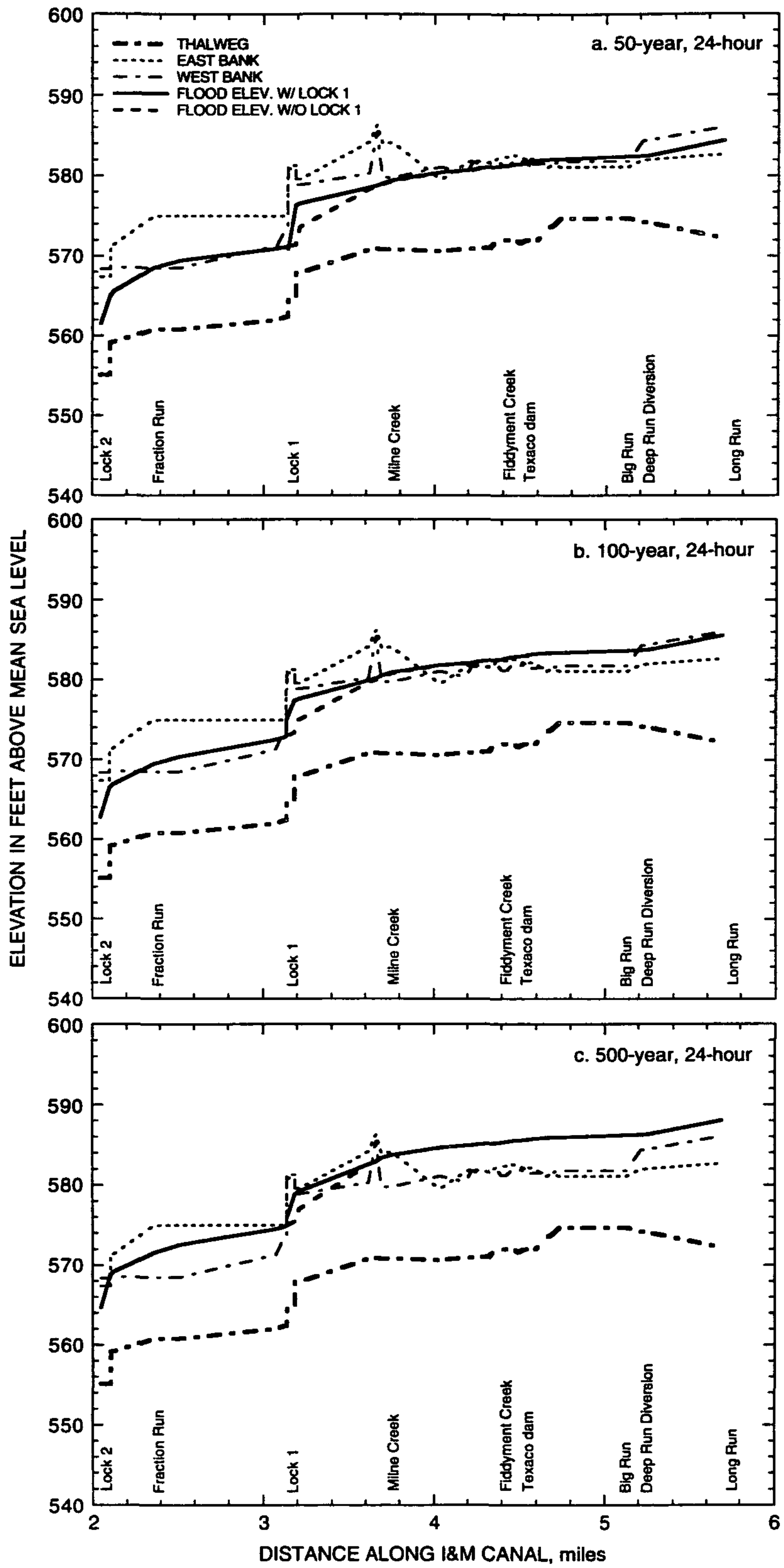


Figure 23. Water surface elevations showing existing conditions and impact of removal of Lock 1 for (a) 50-year, 24-hour storm, (b) 100-year, 24-hour storm, and (c) 500-year, 24-hour storm

of the additional three culverts were placed at least 2 ft below the invert elevations of the existing culverts. The additional reduction in flood elevations due to the assumed culverts is 3.83, 0.87, 0.86, and 0.85 ft for the 10-, 50-, 100-, and 500-year storms, respectively.

REHABILITATION POTENTIAL AND PROBLEMS

For purposes of evaluating the I&M Canal's rehabilitation potential and problems, it was subdivided into four segments from Joliet to Lockport. Segment 1 extends from the mouth of Long Run Creek to Lock 1, segment 2 from Lock 1 to Lock 2, segment 3 from Lock 2 to Locks 3 and 4 (Lock 4 is adjacent to Lock 3), and segment 4 from Lock 4 to the junction of the canal with the DesPlaines River. Figure 24 shows the canal profile and the relative locations of the four segments. The major factors considered in subdividing the study area into four segments were the condition of the canal within each segment and the possibilities and problems of rehabilitation for the different segments. Other man the reach within the TRMI plant, the canal in segment 1 is in fair condition. The portion of the canal within the TRMI plant is in worse condition than the rest of the canal in segment 1 and will require more effort to rehabilitate because of accumulation of sediment, whose quality is unknown. Downstream of the TRMI plant to Lock 1, the canal can be rehabilitated fairly easily once the Lock 1 rehabilitation is completed.

The canal in segment 2 (between Locks 1 and 2) is also in fair condition, but it will require significant clearing of trees and brush along the levees, and possibly some levee repairs. The major requirement for this segment is rehabilitation of Lock 2, which is in very bad condition. Because segments 3 and 4 are outside the Lockport area, they were not seriously investigated in terms of their rehabilitation potential. An important conclusion based on the survey data and field reconnaissance is that the rehabilitation of segments 1 and 2 will not be influenced or impacted by the conditions of segments 3 and 4.

Water Supply

One of the major considerations in rehabilitating the I&M Canal is the availability of water to maintain a desirable depth in the canal. The design source of water for this segment of the I&M Canal was initially Lake Michigan and later the Calumet Feeder Canal, which received its water from a reservoir built on the Little Calumet River (Howe, 1956). When the city of Chicago completed the Calumet-Sag Channel in 1920, the I&M Canal south of the Calumet-Sag Channel was isolated, and thus navigation terminated in this segment (Illinois Division of Waterways, 1951). Since men, the source of water to this segment of the canal has been the tributary streams that drain the area. Even though it is well known that the tributary

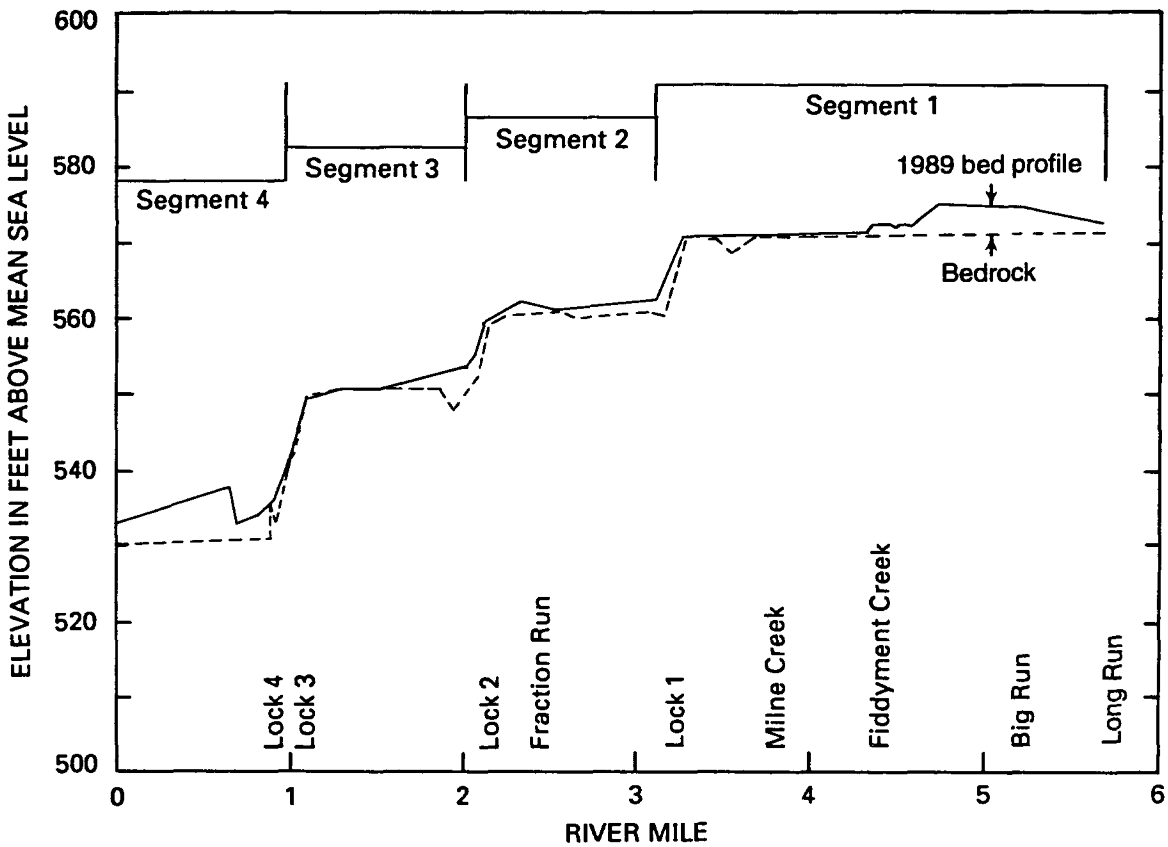


Figure 24. Segments of the Illinois and Michigan Canal in the study area

streams provide excess water far beyond the carrying capacity of the I&M Canal during storm events, it is not known how much sustainable water they provide during periods of low flow.

The majority of the tributary streams are very small and have no flow in their channels most of the year. However, Long Run, with a drainage area of 25.5 sq mi, might be capable of providing sufficient water to the I&M Canal most of the time. At times mere is very little or no flow in Long Run, but it is still the most logical source of water for the I&M Canal. Therefore, analyzing the Long Run flow conditions and the impact of the control structures in the TRMI plant can determine me percentage of time when mere might be little or no flow in the canal.

Because of the lack of long-term streamflow records, synthetic values of streamflow for different flow durations were generated by analyzing available streamflow records. Knapp (1990) has developed regional flow duration relationships applicable to the study area. Figure 25 shows the flow duration curve obtained for Long Run, and table 9 also presents relevant data for Long Run.

Table 9. Flow Duration for Long Run
(drainage area = 25.5 sq mi)

Percent exceedance	*Flow (cfs)*
99	0.0
98	0.0
95	0.0
90	0.0
85	0.02
75	0.55
60	2.75
50	5.42
40	9.25
25	18.94
15	33.38
10	48.35
5	83.20
2	151.33
1	218.74

Based on figure 25 and table 9, it can be concluded mat near-zero flow conditions occur at about 85 percent exceedance, which corresponds to 55 days in a year, but the zero-flow days do not necessarily occur consecutively. In any case, mere is a need to supply additional water to the canal during low- or no-flow periods, which could be accomplished by storing water upstream of the Texaco Dam.

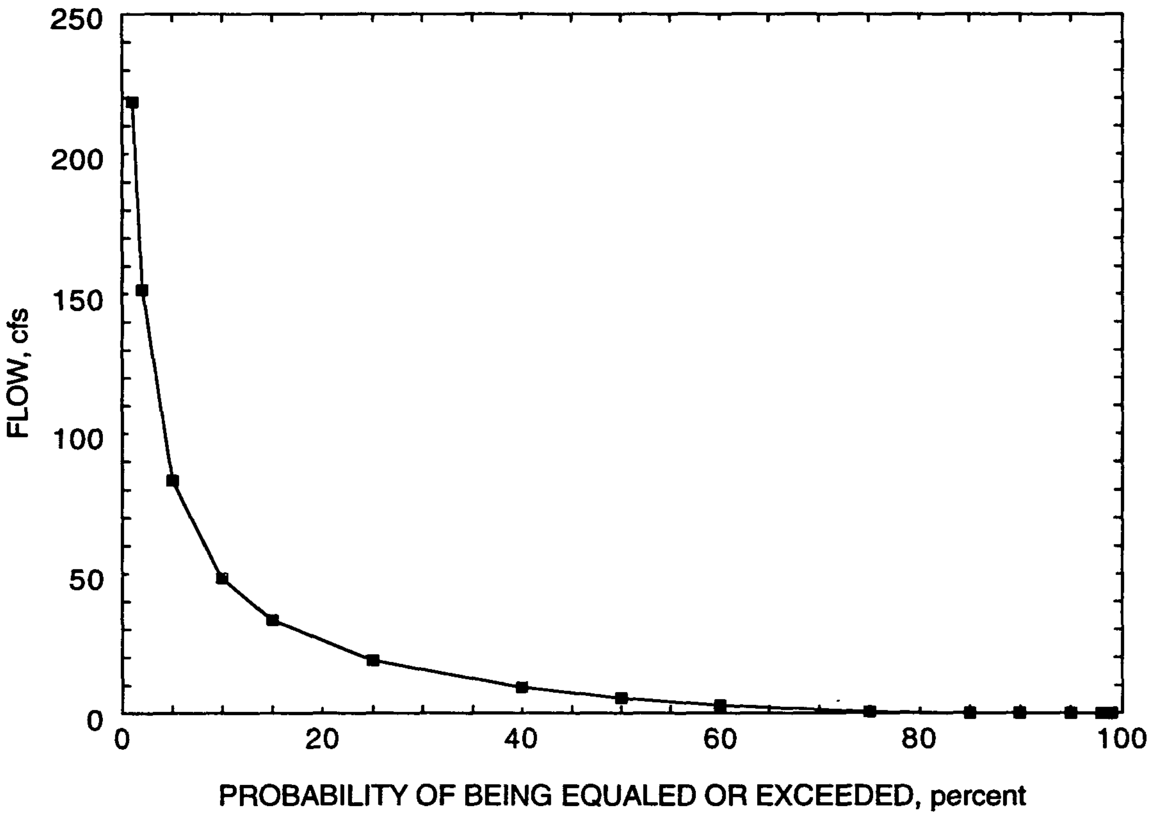


Figure 25. Flow duration curve for Long Run

In addition to natural runoff, discharges from area sanitary districts might be significant. "The Bonnie Brace/Forest Manor Sanitary District provides 1 mgd into the Fiddyment Creek, and the Lockport Heights Sanitary District provides 300,000 g/d into the south branch of the Big Run Creek" (McCluskey, 1992). The Uno-Ven Company discharges cooling water into the Metropolitan Sanitary District Canal, and the potential for using 1 mgd of this water was discussed at the I&M Canal Hydrologic Study Committee meeting. Because of the anticipated expense for repiping at Uno-Ven and potential water quality problems, however, this option was not investigated further.

Control Structures

The Texaco Dam and the side-overflow weir upstream of the dam usually control the storage and flow of water in this segment of the canal. The Texaco Dam controls the elevation of water upstream of the dam (figure 26). Depending on the water elevation and the amount of water flowing in from tributary streams and from upstream, water either flows downstream through the dam or is diverted into Deep Run through the side weir (figure 27). The relative elevations of the dam and side weir determine how much water is stored upstream of the dam and how much water flows into the canal downstream of the dam, so the flow of water in the canal downstream of the Texaco Dam depends on how these two structures are modified and managed.

Since the gates at the Texaco Dam are not operational, it will be difficult to manipulate me dam to control water levels. If it is feasible to make them operational, men it will be possible to control the amount of water flowing downstream into the canal by raising and lowering the gates. During flood conditions the gates could be raised higher to divert more flow into Deep Run, and during low-flow conditions they could be slowly lowered to supply water to the canal. The possibility of rehabilitating the Texaco Dam so that the gates could again be operational should be negotiated with TRMI.

The other major controlling structure mat influences flow in the canal is the side weir upstream of the Texaco Dam. By changing the elevation of the weir, it is possible to control the distribution of water between the I&M Canal and Deep Run. By raising the height of the weir, it is possible to keep most of me flow in the canal during low-flow periods. During periods of floods, however, the weir could be lowered to divert most of the floodwater into Deep Run. TRMI personnel were very cooperative and willing to assist in mat

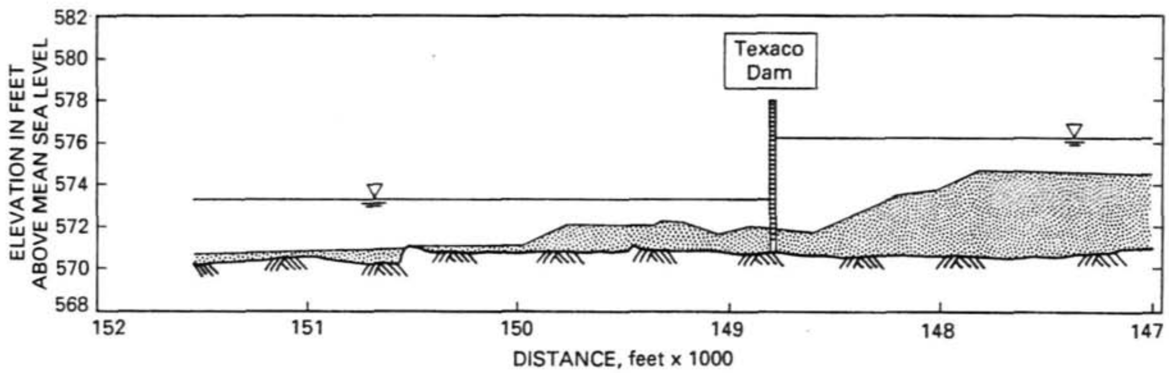


Figure 26. Water-level control at the Texaco Dam

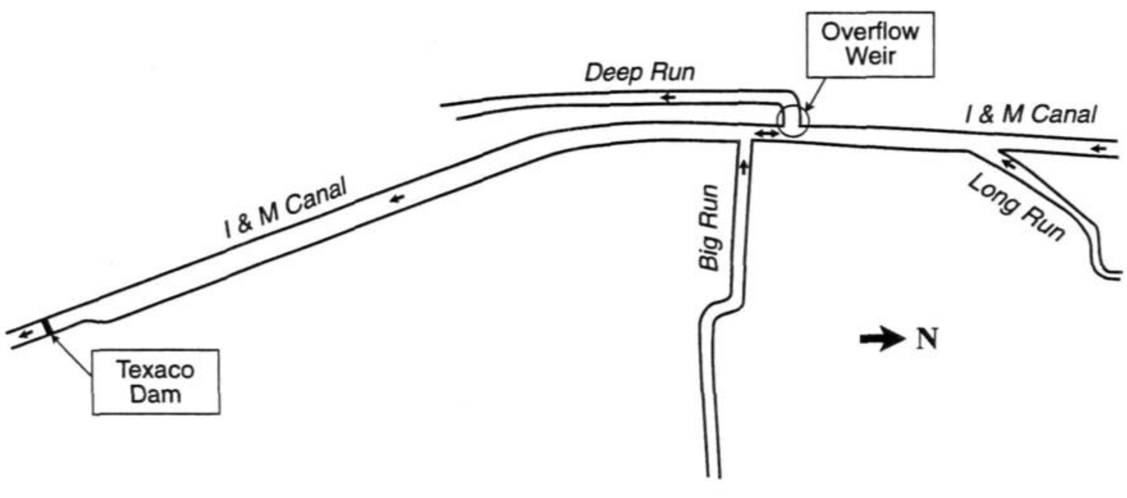


Figure 27. Flow conditions and controls in the Illinois and Michigan Canal within the TRMI plant boundary

arrangement. Therefore, it seems possible to raise the elevation of the weir during low-flow periods and lower it during flood events so that more floodwater flows into Deep Run and not into the canal. The problem will be how to control the timing of lowering and raising the height of the weir and who would be responsible for this operation. The solution has to be negotiated among interested parties.

Proposed Modification at Lock 1

The main objective of this study was to evaluate the feasibility of modifying the existing canal structures to maintain adequate water depth for recreation and aesthetics. Sufficient water depth cannot presendy be maintained in the canal because the locks are not operational. Therefore, mere is a need either to restore the locks to their original operational condition or to install water-level control structures. Although for historical preservation purposes it would be nice to restore the locks to their working conditions, mis expensive project is not the only method to maintain the desired water level.

In phase I of this study, a water depth of 3 ft was proposed as convenient for recreation and aesthetics. This will require water levels to be maintained at an elevation of 574 ft-msl, which could be accomplished by installing a 9.5-ft weir at the upstream end of Lock 1 and designed so mat it can either be removed or lowered to the channel bottom during periods of flooding.

Water-level calculations were performed to illustrate water-level conditions under different control options. Three typical flow conditions, corresponding to 40, 50, and 60 percent exceedance probabilities, were selected. Table 10 shows the magnitudes of the flow in each tributary stream corresponding to these flow durations. These flows were used in the HEC-2 program to calculate water surface profiles for different control options.

Table 10. Low Flow at Different Exceedances for the Tributary Streams

Tributary stream	*Flow, cfs*		
	40%	50%	60%
Convent	1.089	0.636	0.322
School Gully	0.363	0.212	0.107
Long Run	9.255	5.402	2.739
Big Run	0.798	0.466	0.236
Fiddyment	1.742	1.017	0.516
Milne Creek	0.835	0.487	0.247
Fraction Run	2.250	1.314	0.666

Figure 28 shows water surface profiles in the I&M Canal under existing conditions for two side weir elevations at 40, 50, and 60 percent exceedance. The side weir elevations were 578 and 579 ft-msl. As shown in the figures, the water depths are normally less than a foot in most locations except for the portion of the canal upstream of the Texaco Dam. Thus there is a need for water-level control along the canal downstream of the Texaco Dam. The next best location to control the water level in the canal is at Lock 1.

Figure 29 shows the influence of installing a weir at Lock 1 for 60 percent exceedance. As shown in the figure, the weir maintains a water level near the 574 ft-msl elevation for the portion of the canal from Lock 1 to the Texaco Dam. The water level downstream of Lock 1 is essentially unchanged. Figure 30 shows six cross sections of the canal with the water elevation at 574 ft-msl from Lock 1 to the Gaylord Building and indicates that the gravel path on the west bank of the canal will not be flooded with the assumed 9.5-ft weir. The gravel path is shown to be at least 1.2 ft above the water surface elevation of 574 ft-msl.

During extended periods of low or no flow, periodic flushing of the partially stagnant water behind the 9.5-ft weir may be necessary to reduce odor and control breeding of mosquitoes and other insects. It is necessary to know the volume of water impounded by the weir in order to estimate the time it will take to refill the canal behind the weir to the proposed 3-ft depth after flushing. Figure 31 plots the volume of water behind the weir versus different weir height settings. When the weir is fully raised to the 9.5-ft height, it will have impounded 1,279 x 10^3 cubic feet of water. The Texaco Dam must be raised to at least 580.25 ft-msl in order to have sufficient water available to replenish the impoundment behind the proposed 9.5-ft weir.

To compute the time required to fill the reservoir behind the proposed weir, flow rates of 3, 6, and 10 cubic feet per second (cfs) were selected, corresponding to the variation of low flows at 40, 50, and 60 percent exceedances. This should reflect the conditions during extended periods of low flows. Figure 32 plots the time required to fill the reservoir against the weir height settings. At 60 percent exceedance (i.e., 3 cfs), it will take about five days after flushing to restore the 3-ft water depth in the canal section upstream of the proposed weir.

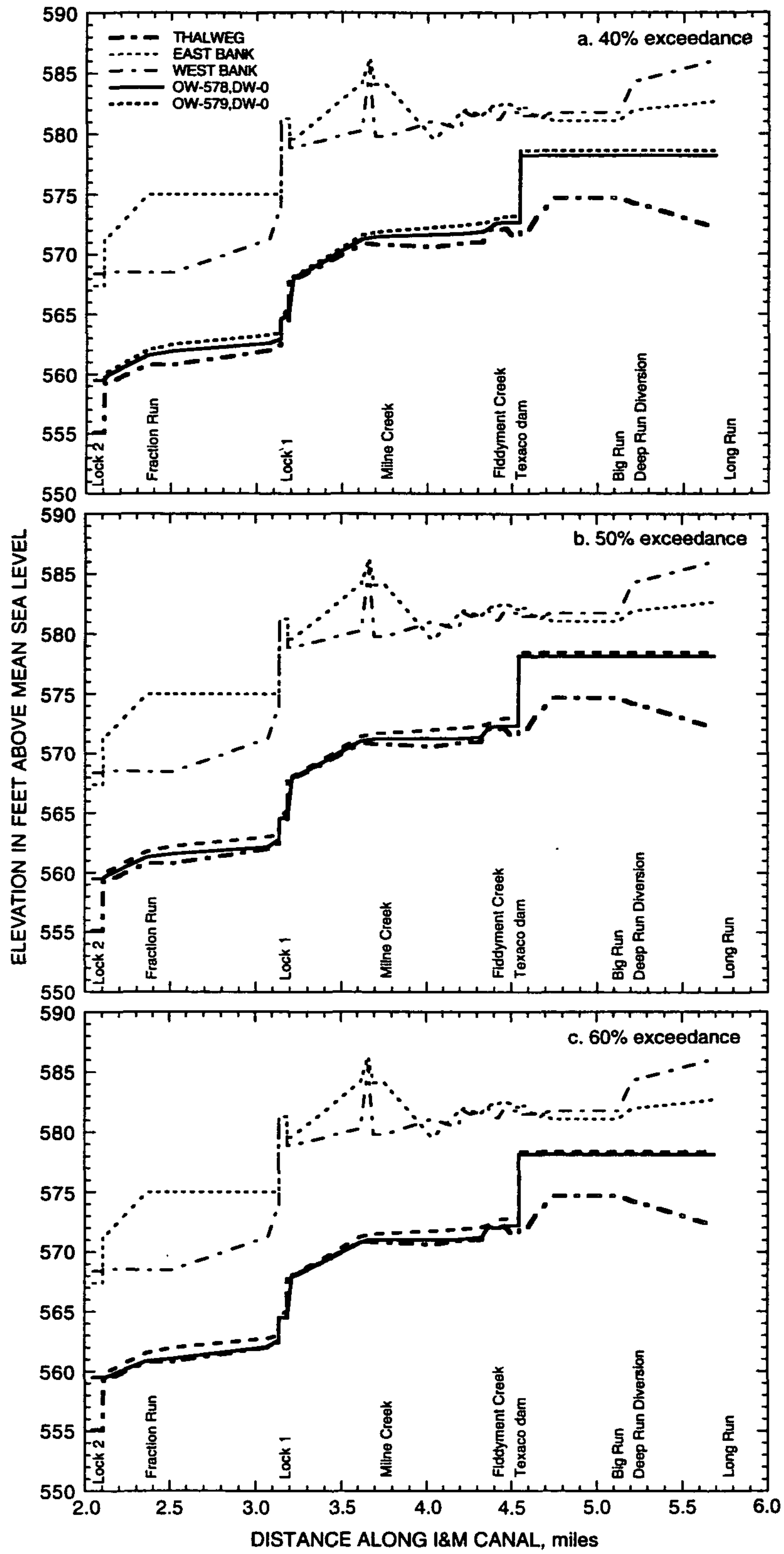


Figure 28. Water surface profiles for existing conditions at Lock 1 and for different overflow weir elevations at (a) 40 percent exceedence, (b) 50 percent exceedence, and (c) 60 percent exceedence

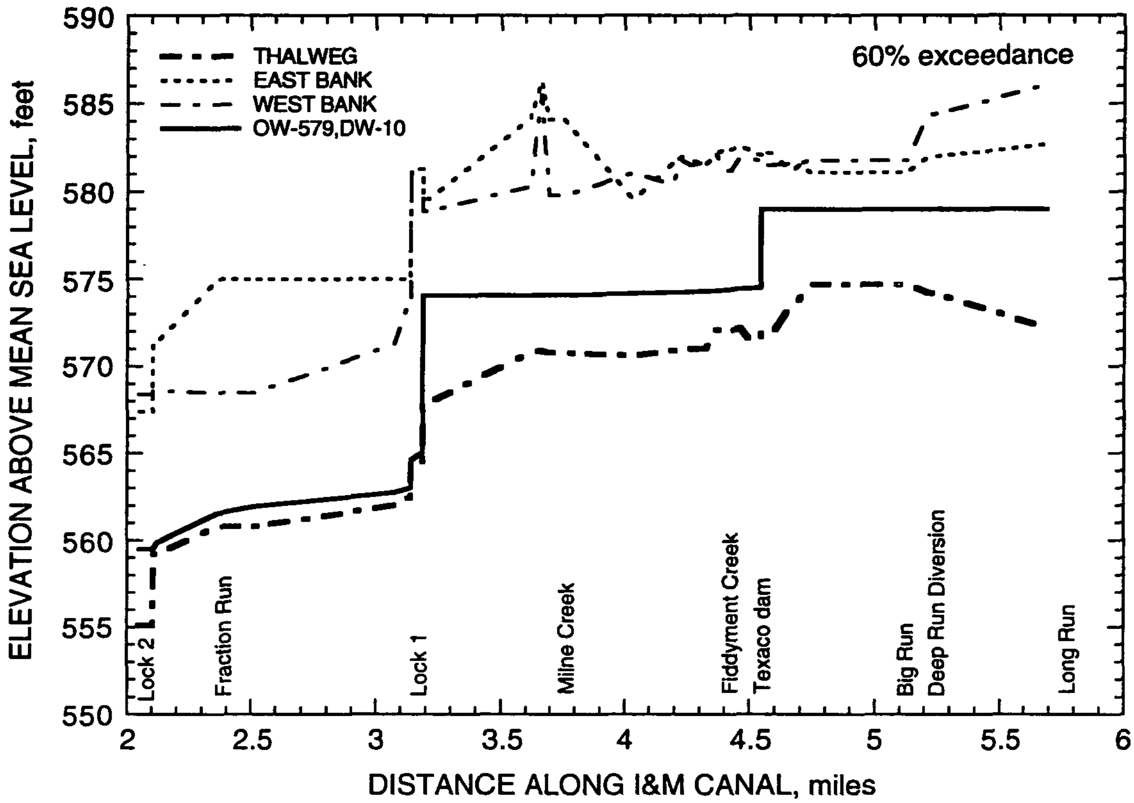


Figure 29. Water surface profile for proposed 9.5-foot weir at Lock 1
(60 percent exceedance)

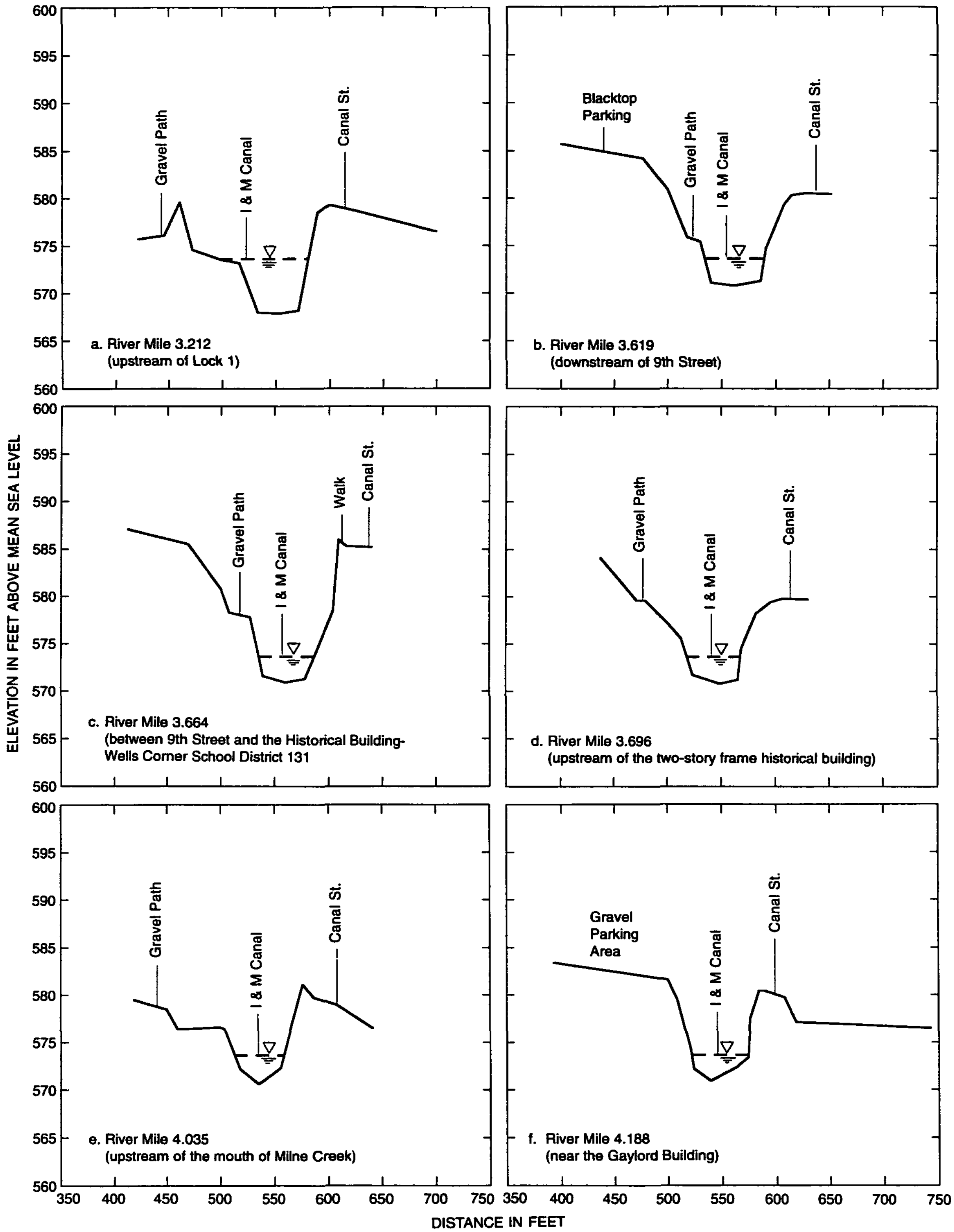


Figure 30. Cross sections of the Illinois and Michigan Canal

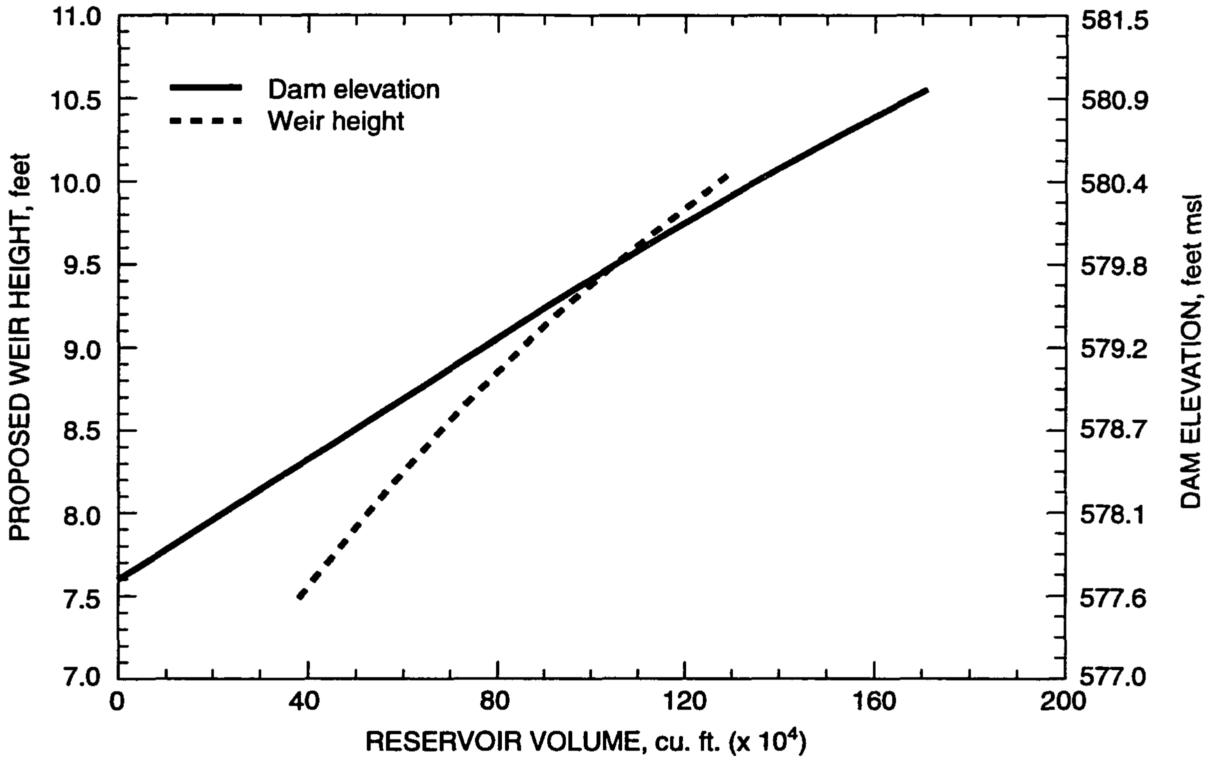


Figure 31. Relationship between the proposed weir height-Texaco Dam elevation and upstream reservoir volumes

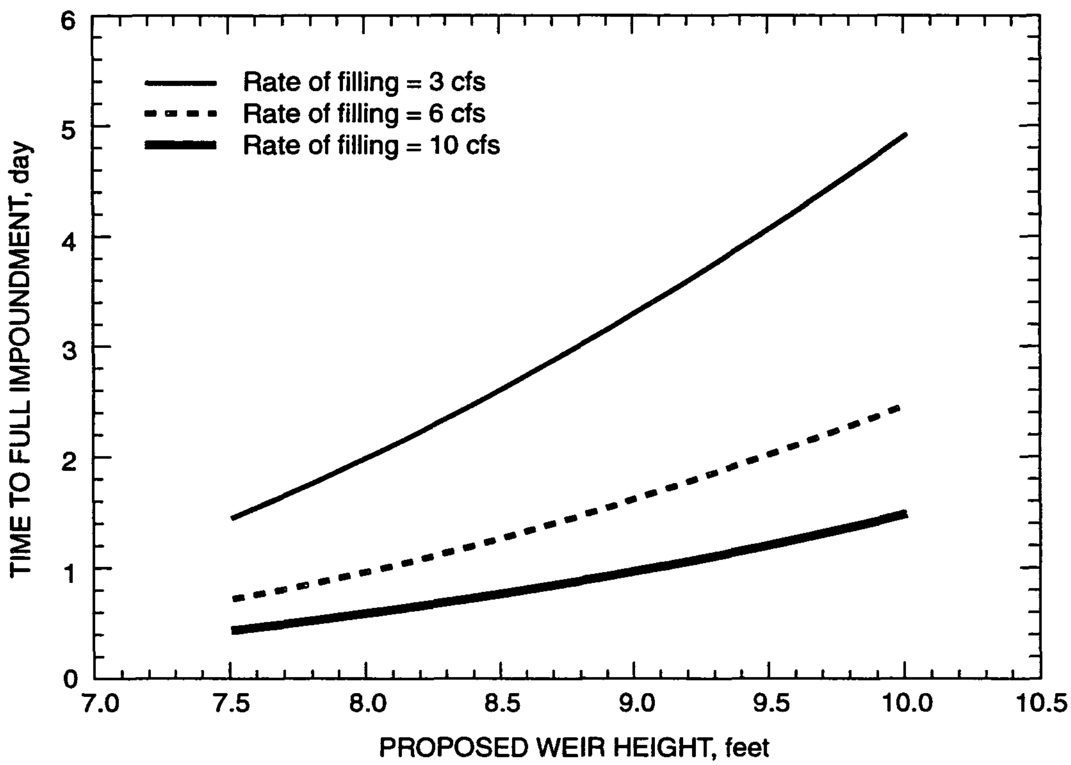


Figure 32. Variation of time to complete filling of the reservoir upstream of the proposed weir and the height of the weir

The impact of the proposed weir on flooding along the canal was evaluated by comparing computed water surface profiles with and without the proposed 9.5-ft weir at the entrance to Lock 1. Figures 33a and b depict the water surface profiles for 10- and 50-year storms, respectively, with the greatest impact of the proposed weir observed for the 10-year storm. The impact of the weir on flooding was observed to diminish as the frequency of the storm event increased. The influence of the weir becomes insignificant for storm frequencies higher than 50 years.

Water Quality Concerns

Water quality concerns in the I&M canal in the Lockport area are related primarily to conditions within the TRMI plant. Since the TRMI plant is located at the upstream end of the study area (figure 3), contamination of soils and water within the plant are likely to influence water quality in the I&M canal as the water flows through the plant.

Water quality has been previously monitored in the I&M Canal. In 1984, water samples were taken under the supervision of the U.S. Environmental Protection Agency (USEPA) and analyzed for priority pollutants and volatile organics (TRMI, 1984). Water samples were taken in the canal section within the TRMI plant at four different locations, and sampling test results indicated concentrations below detection limits for the tested pollutants

In 1987, TRMI submitted a statement of work and a sampling plan to the USEPA that included both water and sediment sampling in the I&M canal at a location between the Texaco Dam and the side overflow weir (figure 27). The plan specified a background water sample to be taken in the I&M Canal, and the chain of custody indicated laboratory tests for extractables, volatile organics, and metals.

Table 11 provides results of the inorganic analysis for the canal water sample and the background sample (TRMI, 1987). Column three of the table shows the USEPA recommended concentration limits for some of the elements, and concentrations of aluminum, barium, calcium, iron, manganese, and potassium exceeded background concentrations. Only lead and sodium concentrations are below the background. No comparison could be made for copper and zinc because the background sample was not tested for these two metals. However, the zinc concentration was below the general use/secondary contact standard, and the concentration of copper was approximately equal to the recommended limit.

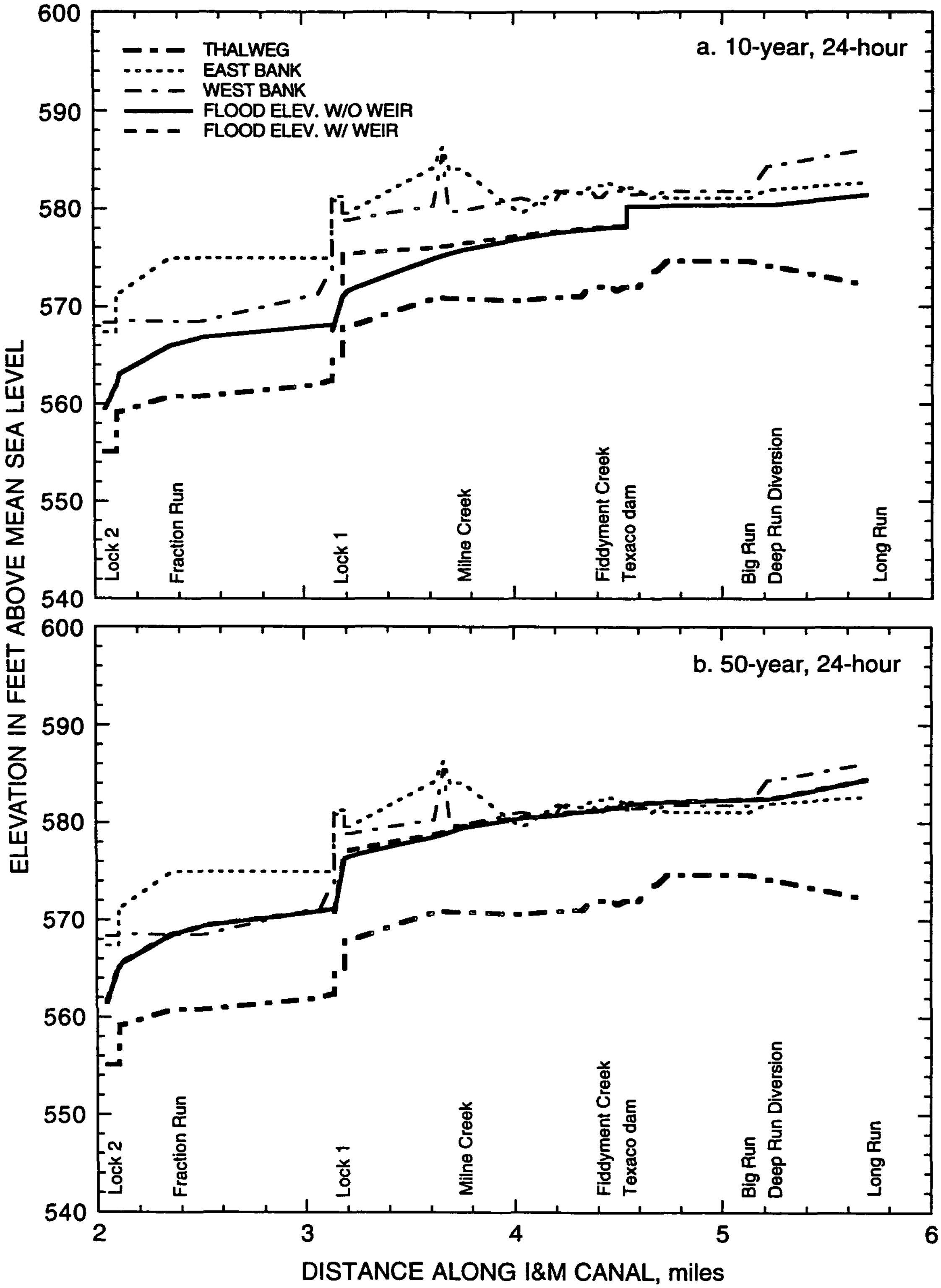


Figure 33. Water surface elevations showing the impact of proposed weir at Lock 1 on flooding for (a) 10-year, 24-hour storm and (b) 50-year, 24-hour storm

Table 11. Metal Analysis Sampling Results, 1987
(in milligrams per liter)

Element	*I&M Canal*	*Background*	*Standard*
Aluminum	1.35	0.155	-
Barium	1.29	0.034	1.0[a]
Calcium	83.6	67.4	-
Copper	0.007	-	0.0069[b]
Iron	2.45	0.74	1.0[b]
Lead	0.005	0.006	0.005[b]
Magnesium	49.4	41.0	-
Manganese	0.146	0.059	0.15[b]
Potassium	4.09	3.3	-
Sodium	27.4	50.3	-
Zinc	0.342	-	5.0[b]

a = Public water supply
b = General use/secondary contact

Three of the seven metals with concentrations exceeding background levels have published standards. However, magnesium is the only metal among the three with a concentration below the general use/secondary contact level. The barium concentration was about 30 percent above the recommended limit; and calcium, manganese, and potassium concentrations were no more than 25 percent higher than the background levels. By taking into consideration the margins of error in laboratory test results, the concentrations of the latter three metals cannot be used as reliable indicators of the level of pollution caused by past activities within the TRMI facility. The laboratory report (TRMI, 1987) indicates that the quality control for calcium, iron, magnesium, and manganese are unacceptable and suggests viewing the concentration levels of these elements with caution. Since the concentration of iron is unreliable, and without any published limits for aluminum, none of the concentrations of the seven metals that exceeded the background can be used to assess the actual level of water pollution in the I&M Canal. Therefore additional water and sediment sampling will be needed to adequately assess the impact of the TRMI plant on water quality in the canal.

TRMI has also provided the results of the laboratory analysis of a 1990 sediment sample taken at a location upstream of the plant's north fence line. The sediment sampling data for this unspecified location outside the plant fence line showed low-level concentrations of organic compounds. However, without any sediment sampling data for the section of the canal within the TRMI plant, no sediment quality assessment can be made for the I&M Canal downstream of the plant, which is the main area of interest.

SUMMARY

A detailed hydrologic and hydraulic investigation of the I&M Canal in the Lockport area was conducted to evaluate the feasibility of restoring and rehabilitating the canal in this area. The investigation included a detailed survey of canal cross sections, two-year monitoring of peak stages in the canal, development of hydrologic models for tributary watersheds, and routing of floods through the canal using a hydraulic model. The model was also used to evaluate effects of different changes in the control structures along the canal.

Based on the survey data, it can be concluded that no major changes have occurred in the physical dimension of the canal cross sections since 1949. However, there are significant sediment accumulations in the upper and lower sections of the study area. In the mid-section of the canal, sediment was removed twice by the City of Lockport, which implies significant sediment input into the canal. There is also significant tree and brush growth on the levees and sometimes in the canal itself.

Major control structures along the canal are not operational and in bad shape, except for Lock 1, where some restoration had been started but not completed. The locks will require major work to make them operational, but water-level control structures could be installed without complete restoration of the locks. The control structures in the TRMI plant are operated to store water for firefighting purposes only. The operation needs to be modified to accommodate the need for restoration of the canal downstream of the plant.

The issue of flooding was investigated extensively. Flood discharges were computed for all the tributary streams draining into the canal using the HEC-1 hydrologic model. Computed flood discharges were then used in the HEC-2 hydraulic model to determine flood elevation along the canal. The HEC-2 model was used to estimate flood elevations for storm events with 10-, 50-, 100-, and 500-year return periods under existing conditions. Different potential changes in the canal or control structures were then incorporated in the model to evaluate the impact of those changes on flood elevations.

Three options were evaluated. These included:

1. Removal of Lock 1,
2. Installation of additional side culverts upstream of Lock 1 to divert more flood flows, and
3. Installation of a water-level control structure at Lock 1.

The removal of Lock 1 will significantly reduce flood elevations in the immediate vicinity of the lock but will not have much impact further upstream. Flood elevations will be reduced by 4.9, 4.0, and 3.4 ft immediately upstream of the lock for 50-, 100-, and 500-year storm events, respectively. Note that the influence of Lock 1 on flood elevation is only limited to the immediate vicinity of the lock (about 2,500 ft upstream) because of the steep gradient of the canal upstream of Lock 1.

The installation of additional side culverts upstream of Lock 1 was considered because the existing culverts are at higher elevations to be effective during the most frequent flood events. Therefore, it was felt that culverts installed at lower elevations might be more effective in reducing flood elevations downstream. The maximum influence of such installation was estimated to be 3.8 ft for the 10-year storm event and about 0.9 ft for storms above the 50-year return period.

The impact of a water-level control weir at Lock 1 on flood elevations is confined to the immediate vicinity of the lock and during more frequent floods. During major floods with 50-year return periods and greater, the weir's impact is insignificant. Note that the design for the weir should include mechanisms to remove or open the weir during flood events. If that is always done, then the weir will have no impact on flood elevations. If it is left in place during flood events, it will affect flood elevation immediately upstream of the lock.

The water quality data available are not sufficient to make very conclusive statements on the impact of the TRMI plant and other discharges on water quality in the I&M Canal. Moreover, without additional sediment quality analysis within the plan, no sediment quality assessment can be made. However, the existing data from TRMI does not show major water-quality problems.

Recommendations

Rehabilitation and restoration of the I&M Canal in the Lockport area is feasible but will require a significant amount of effort and money because of the canal's present condition. Most of the water-control structures in the canal have deteriorated significantly because of lack of maintenance and repair over the years. A major effort will be required to clear and maintain the canal because it and its levees are so overgrown with trees, weeds, and brush.

Because rehabilitation of the I&M Canal will be a major and long-term effort, a multi-stage plan is recommended. The first stage should concentrate on the segment of the canal from Lock 1 to the mouth of Long Run Creek. Rehabilitation of the other segments of the canal, both upstream and downstream of this segment, could wait until the rehabilitation of this segment is completed. The main reasons for choosing this segment are: 1) it is the most visible, accessible segment in the Lockport area, 2) rehabilitation of Lock 1 was initiated but has not been completed, and 3) it is in better condition than other segments of the canal in the area.

The rehabilitation of this segment of the canal, from Lock 1 to the TRMI plant, will require implementation of three main tasks:

1. Rehabilitating Lock 1 and installing a gate or a water-level control weir.
2. Clearing and cleaning the canal within this segment.
3. Making arrangements with TRMI to modify the two control structures within the TRMI plant so mat adequate water is available in the I&M Canal during periods of low flow.

Brief discussions of the three main tasks are presented below:

1. Rehabilitating Lock 1 and installing a weir with variable height at the lock.

As part of me restoration effort of the canal, rehabilitation of Lock 1 was initiated but not completed. The rehabilitation of the lock needs to be completed and a gate or a weir structure installed at the upstream end of the lock to maintain a desirable depth of water in the canal under variable flow conditions.

During the last meeting of the I&M Canal Hydrology Study Committee, the issue of water-level control structures at the locks was extensively discussed. One of the major suggestions was the possibility of restoring the old gates at the locks for historical and aesthetic reasons. Plans of the old gates can be obtained from old drawings or from information from similar lock gates along the Chesapeake and Ohio Canal. The major issue will be the cost. If the resources are available, there are no reasons the old gates could not be restored. For the first phase of the project, there is only a need to install the upstream set of lock gates.

The weir should either be movable or tiltable so that its height can be controlled for different flow conditions. During flood periods, the weir could be designed either to be removed or to be tilted to the bottom of the channel, so mat its impact on flooding would be controlled. During periods of low flow, weir height would be set at a desirable level. If the weir is not movable or tiltable, so that it can either be raised above the water surface or tilted to the canal bottom during flood events, it will definitely increase flood elevations upstream of Lock 1 for some flow conditions.

The canal within this reach (Lock 1 to the TRMI plant) can be managed to hold up to 6 ft of water with 2 ft freeboard without major modifications, but it is not currently necessary to maintain this depth. Initially the canal should be designed to hold 3 ft of water by controlling the opening of the gate or by setting the height of the weir at the lock to 574 ft-msl. The weir should be designed such that the height could be either increased or decreased in the future without much structural modification to the lock or weir.

Figure 34 shows the relative elevations of water in the canal with respect to the canal bottom and the top of the levees for the proposed water elevation (574 ft-msl). As shown in the figure, the water elevation will be well below the top of the levees. This is further illustrated in figure 35, which indicates the proposed water surface elevations for selected cross sections between Lock 1 and the Texaco Dam. However, it should be mentioned that even though the elevation of the levees is well above the proposed water elevation, some segments of the levees might require repair after clearing overgrown brush and trees along the canal.

2. Clearing and cleaning the canal.

The canal has accumulated sediment in certain segments and is overgrown with weeds and brush. Tree experts from the IDOC and city officials from Lockport should inspect me canal from Lock 1 to the access road outside of the TRMI plant and should remove all brush and cut undesirable trees and weeds from within the canal and on the levees. Further field investigation of the levees after tree and sediment removal will be required to ascertain their suitability to hold water 3 ft deep. Survey data and preliminary field inspections indicate mat this segment of the canal should not require major levee rehabilitation.

3. Arranging with TRMI to modify the two control structures within the TRMI plant so that adequate water is available in the I&M Canal during periods of low flow.

The amount of water flowing in the canal downstream of the TRMI plant during non-flood periods is largely controlled by the two structures within the plant. Because the tributary streams mat enter the canal downstream of the control structures and upstream of Lock 1 (Fiddyment Creek and Milne Creek) do not contribute significant flow to the canal during most of the year when mere is no runoff from storms or snowmelt, they cannot be relied upon to

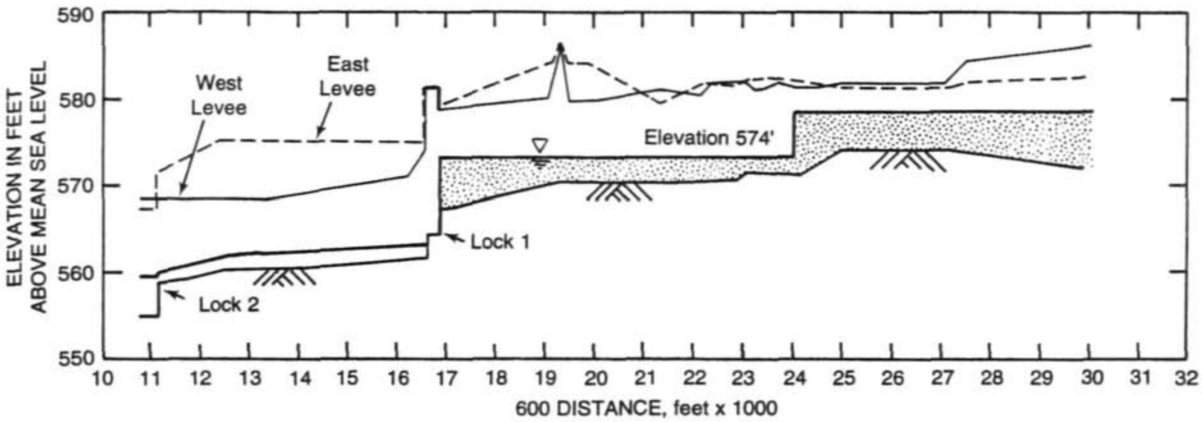


Figure 34. Proposed water elevations from Lock 1 to the Texaco Dam

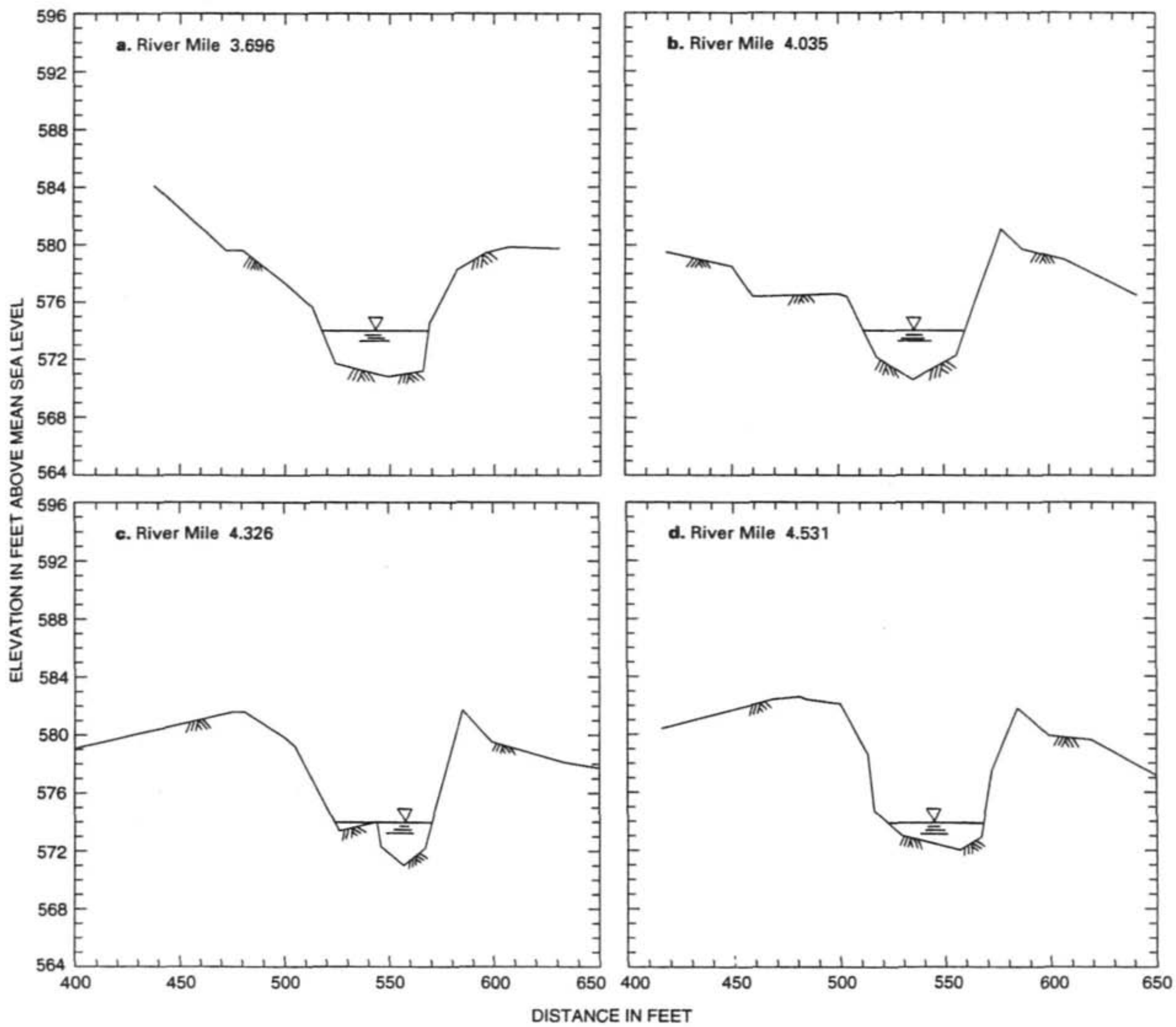


Figure 35. Proposed water elevations at selected cross sections between Lock 1 and the Texaco Dam

supply water to the canal throughout the year. The most reliable supply of water available to the canal is from Long Run, which enters the canal north of the TRMI plant boundary.

The two structures within the TRMI plant boundary control the flow of water downstream into the canal. The control gates of the Texaco Dam have not been operated for a long time, and a side weir on the west levee of the canal allows the overflow of water into Deep Run. Modifications and control of these two structures are necessary to control the amount of water in the canal. Since both structures are located within the TRMI plant, arrangements have to be made with TRMI to carry out the modifications. Throughout this investigation, the TRMI personnel have been very cooperative.

If such arrangements cannot be made, other reliable sources of water need to be investigated. The tributary streams entering the canal downstream of the control structures cannot be relied upon to maintain adequate water depth in the canal. Consequently, other sources of water to investigate include ground water and the Chicago Sanitary & Ship Canal.

Modification of the canal control structures within the TRMI plant is the best alternative, and the possibilities with TRMI should be explored before investigating these other sources of water. Therefore, it is recommended that the IDOC and the Lockport Area Development Commission need to initiate discussion with TRMI concerning the control structures within the TRMI plant.

A major concern about the supply of water from the TRMI plant area to the I&M Canal downstream is the quality of water. Concerns have been expressed that the sediment within the TRMI plant area might be polluted and be a potential source of pollution in the canal. Based on water quality data available from TRMI, major water-quality problems were not found. However, very little data exist on sediment quality. If significant disturbance and resuspension of the sediment do not occur, the water-quality concerns from the sediment might be minimal.

Once the rehabilitation of this segment of the canal is either completed or near completion, phase II of the canal rehabilitation in the Lockport area can be initiated. This phase will involve rehabilitating Lock 2 and the segment of the canal between Lock 2 and Lock 1. The rehabilitation of Lock 2 will be similar to that of Lock 1, including the addition of a water-level-control weir at the downstream end. Phase II should be easier and cheaper than phase I because the problems of water supply would have been resolved in phase I and phase II would benefit from experience gained in rehabilitation of Lock 1 and the first segment.

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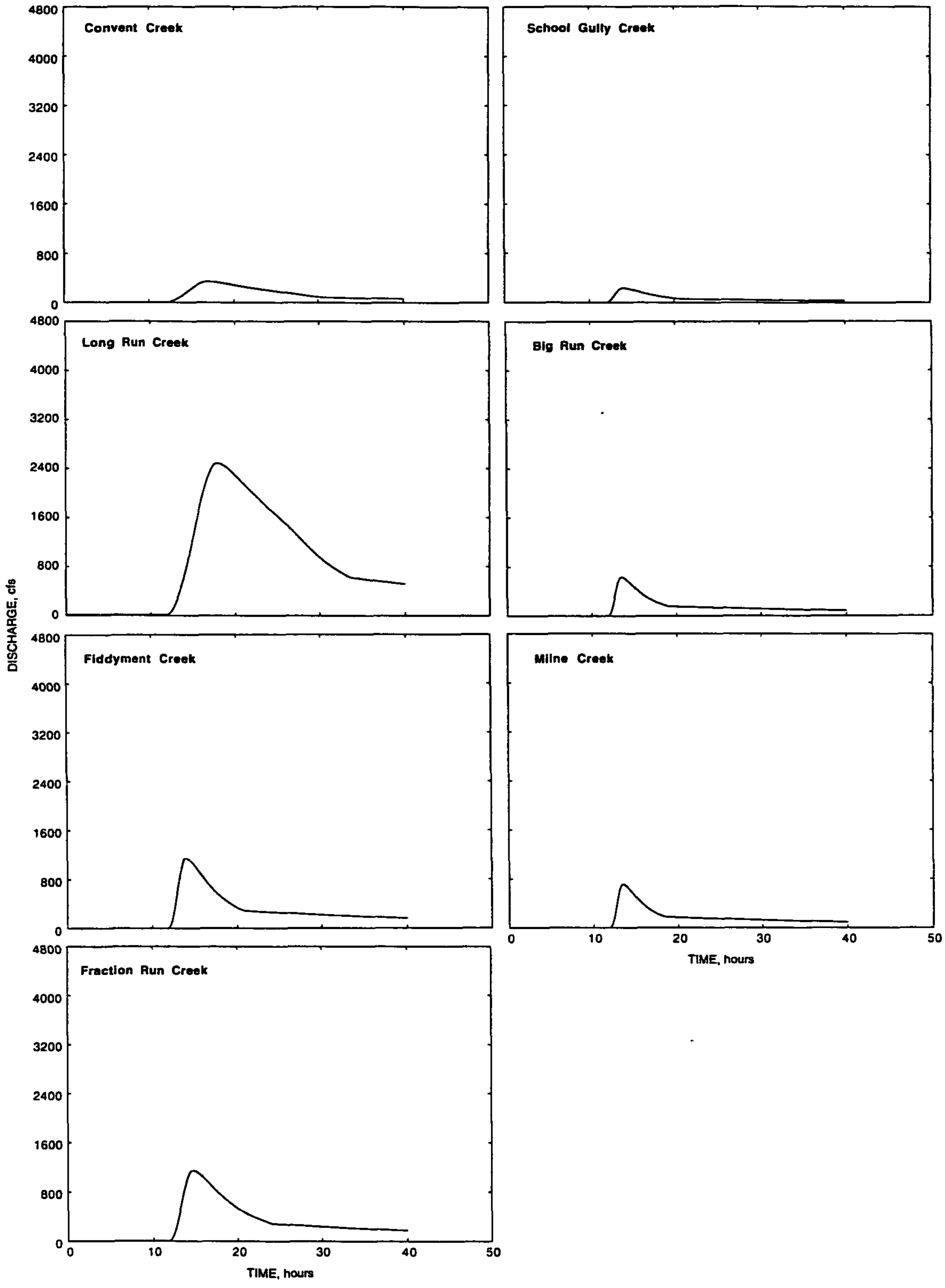
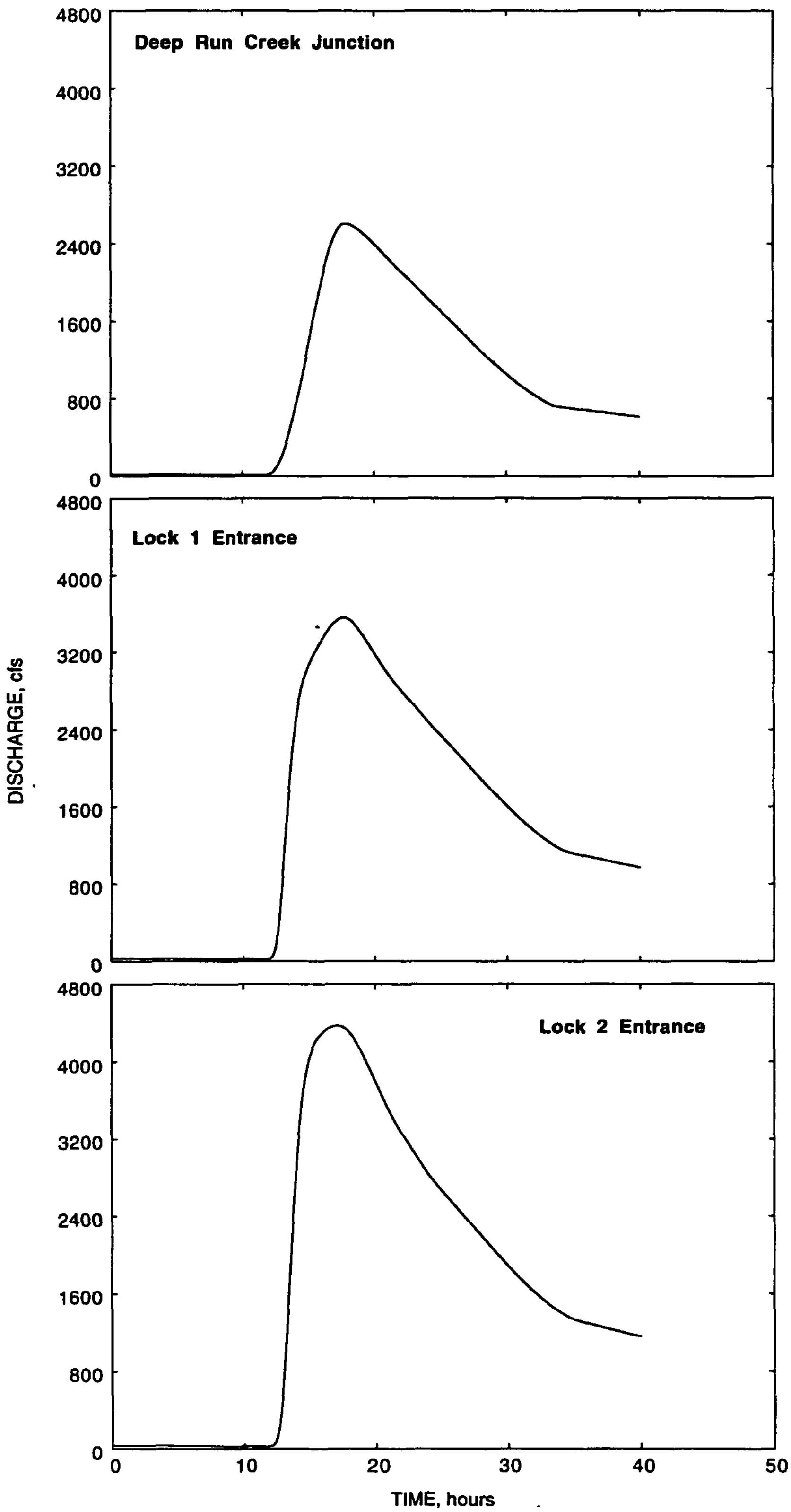
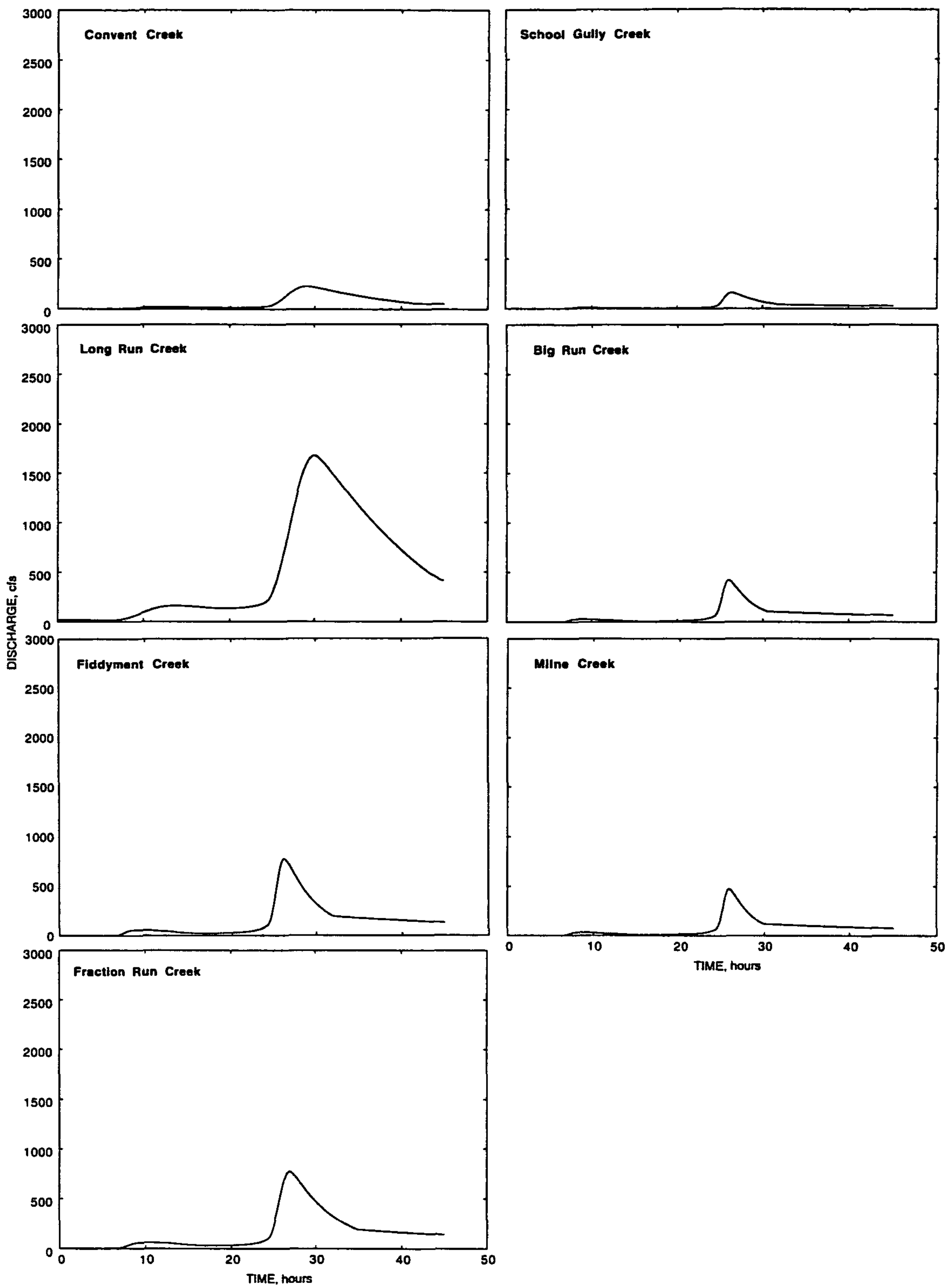
APPENDIX A. COMPUTED FLOOD HYDROGRAPHS FOR DIFFERENT FREQUENCY STORMS



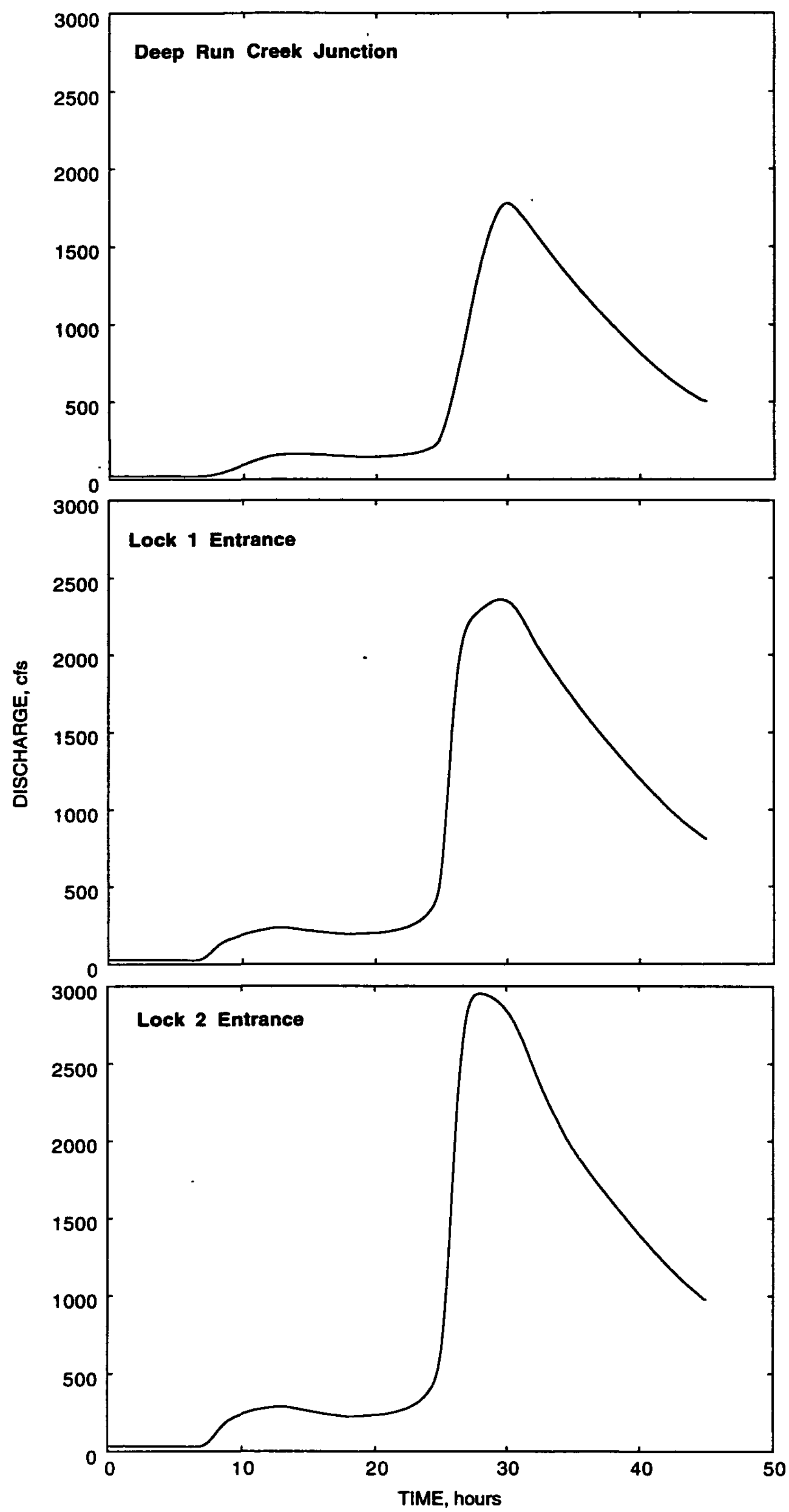
50-year, 24-hour storm hydrographs for the tributary streams



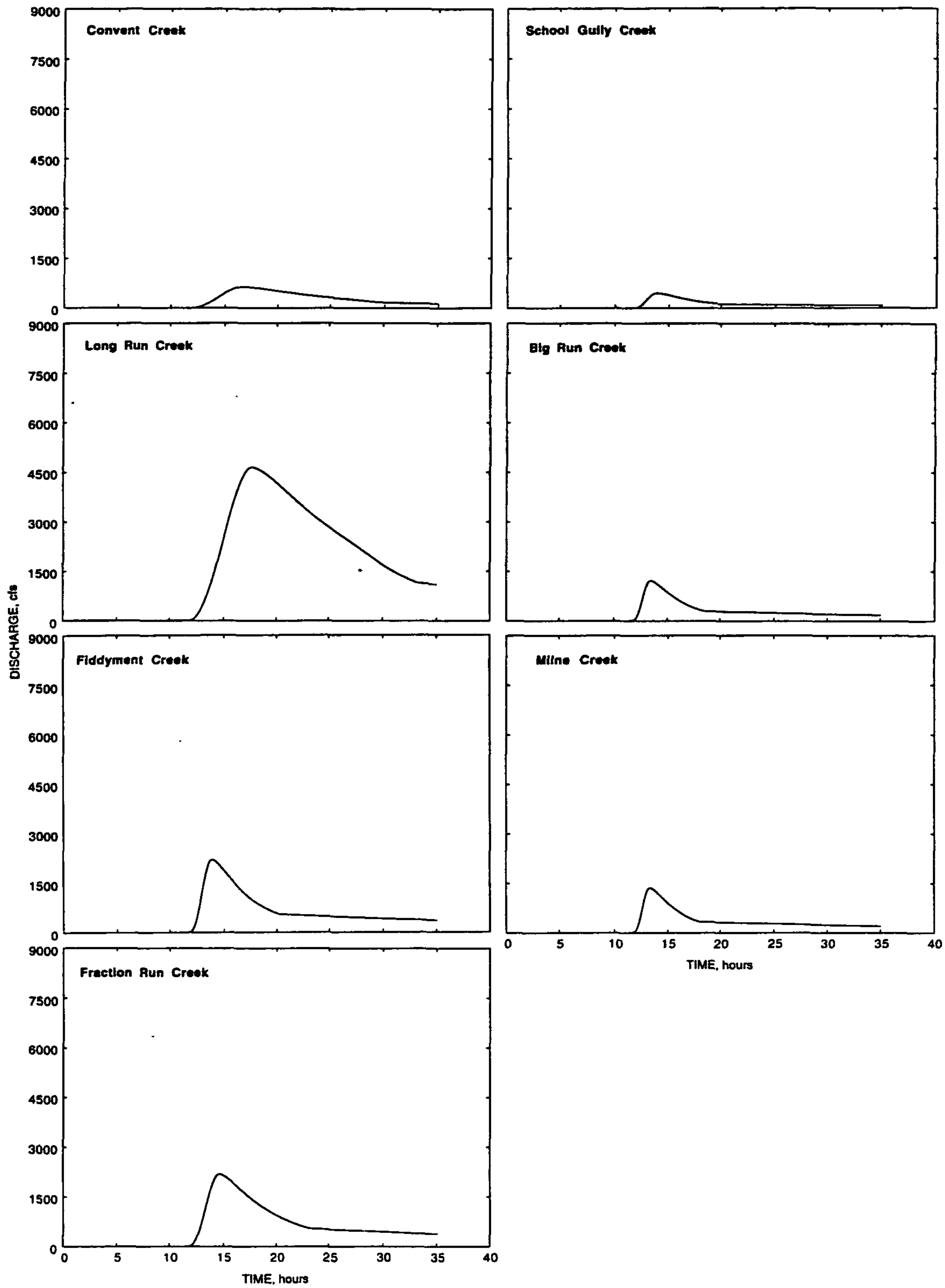
50-year, 24-hour flood hydrographs for selected cross sections along the Illinois and Michigan Canal



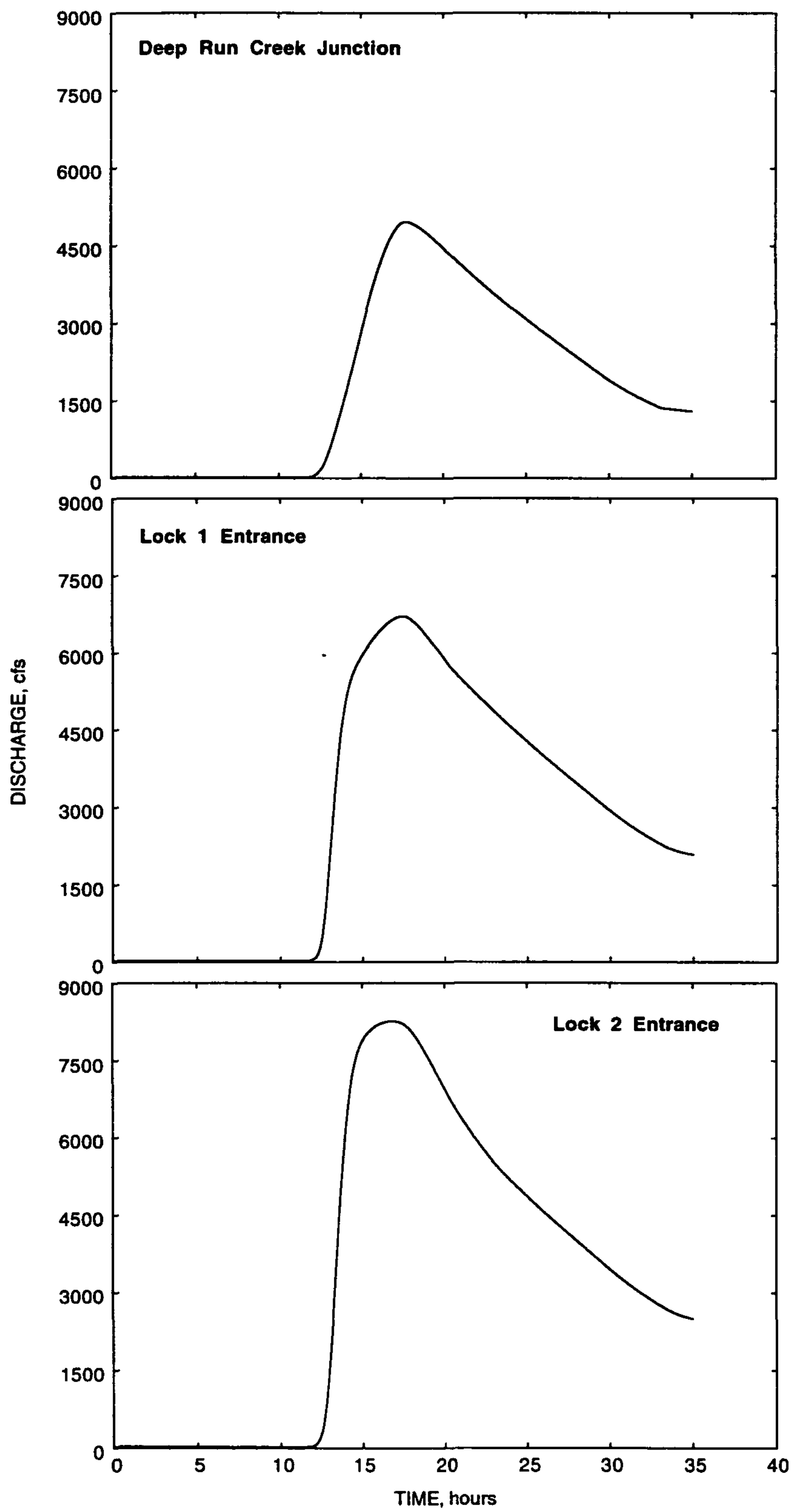
100-year, 24-hour flood hydrographs for the tributary streams



100-year, 24-hour flood hydrographs for selected cross sections along the canal



500-year, 24-hour storm hydrographs for the tributary streams



500-year, 24-hour storm hydrographs for selected cross sections along the canal

APPENDIX B. SAMPLE HEC-1 AND HEC-2 DATA FILES

HEC-1 DATA FILE FOR I & M CANAL AND THE WATERSHEDS OF TRIBUTARIES DRAINING INTO THE CANAL

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ID      LONG RUN RIVER BASIN
ID      10-YEAR 24-HR SYNTHETIC STORM
ID      ILLINOIS STATE HATER SURVEY MAY  1993
ID      HYDROLOGIC & HYDRAULIC STUDY OF THE I&M CANAL AT LOCKPORT
*DIAGRAM
IT        9  15APR91     100     300
IN       15  15APR91     100
IO        5        0
*
KK COV02
KM  LOCAL RUNOFF FROM SUBBASIN COVENANT02
KO           1        0
BA           3
PB        4.47
PI        0.01     0.01    0.01     0.01      0.01   0.01   0.01   0.01   0.01   0.01
PI        0.01     0.01    0.01     0.01      0.01   0.01   0.01   0.01   0.01   0.01
PI        0.01     0.01    0.01     0.01      0.01   0.02   0.02   0.02   0.02   0.02
PI        0.02     0.02    0.02     0.02      0.03   0.03   0.03   0.03   0.04   0.04
PI        0.05     0.06    0.08     0.09      0.12   0.16   0.29   1.23   0.48   0.21
PI        0.13      0.1    0.08     0.07      0.05   0.04   0.04   0.04   0.03   0.03
PI        0.03     0.02    0.02     0.02      0.02   0.02   0.02   0.02   0.02   0.02
PI        0.02     0.01    0.01     0.01      0.01   0.01   0.01   0.01   0.01   0.01
PI        0.01     0.01    0.01     0.01      0.01   0.01   0.01   0.01   0.01   0.01
PI        0.01     0.01    0.01     0.01      0.01   0.01
BF         2.4    -0.25    1.03
LS           0       52
UC        4.54     6.81
*
KK02TO04
KM ROUTE COVENANT02 HYDROGRAPH FROM 02 TO 04
RD
RC       0.055    0.045   0.055   3379.2  0.0000546
RX         431      506     515      540        560    572    578    614
RY       582.7      581   577.1    572.2      573.8  577.1  581.2  586.1
*
KK SCH04
KM  LOCAL RUNOFF FROM SUBBASIN SCHOOL04
KO           1        0
BA           1
BF         0.8    -0.25    1.03
LS           0       52
UC        1.74     2.61
*
KK SCH04
KM  COMBINE LOCAL SCHOOL04 WITH HYDROGRAPH FROM COVENANT02
KO           1        0
HC           2
*
KK04TO06
KM ROUTE COMBINED SCHOOL04 HYDROGRAPH FROM 04 TO 06
RD
RC       0.055    0.045   0.055  30465.6  0.0000546    800
RX         431      506     515      540        560    572    578    614
RY       582.7      581   577.1    572.2      573.8  577.1  581.2  586.1
*
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KK06TO08
KM ROUTE HYDROGRAPH FROM 06 TO 08
RD
RC        0.055   0.045   0.055  2386.6 0.0000546     800
RX          431     506     515     540       560     572     578     614
RY        582.7     581   577.1   572.2     573.8   577.1   581.2   586.1
*
KK LRN10
KM  LOCAL RUNOFF FROM SUBBASIN LRUN10
KO            1       0
BA         25.5
BF           20   -0.25    1.03
LS            0      52
UC         5.51    8.26
*
KK LRN10
KM  COMBINE LOCAL LRUN10 WITH HYDROGRAPH FROM 08
KO            1       0
HC            2
*
KK10TO12
KM ROUTE COMBINED LRUN10 HYDROGRAPH FROM 10 TO 12
RD
RC        0.055   0.045   0.055   116.1 0.0000546     800
RX          422     500     523     540       555     568     571     666
RY          582   580.8   576.9   574.2     574.9     577   580.1   584.4

KK12T014
KM ROUTE COMBINED LRUN10 HYDROGRAPH FROM 12 TO 14
KO            1       0
RD
RC        0.055   0.045   0.055   176.9 0.0000546     800
RX          422     500     523     540       555     568     571     666
RY          582   580.8   576.9   574.2     574.9     577   580.1   584.4
*
*KM DIVERT FLOW TO DEEP RUN14
*
KK14T016
KM ROUTE HYDROGRAPH FROM 14 TO 16
KO            1       0
RD
RC        0.055   0.045   0.055     293 0.0000546     800
RX          428     500     520     565       580     602     616     714
RY        582.9   580.6   574.7   575.4     575.6   577.2   581.8   577.1
*
KK BIG16
KM  LOCAL RUNOFF FROM SUBBASIN BIGR16
KO            1       0
BA          2.2
BF          1.7   -0.25    1.03
LS            0      52
UC         1.33       2
*
KK BIG16
KM  COMBINE LOCAL BIGR16 WITH HYDROGRAPH FROM 14
KO            1       0
HC            2
*
```

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KK16TO18
KM ROUTE BIGR16 HYDROGRAPH FROM 16 TO 18
RD
RC        0.055   0.045   0.055  2064.5  0.001811     800
RX          428     500     520     565       580     602     616     714
RY        582.9   580.6   574.7   575.4     575.6   577.2   581.8   577.1
*
KK18TO20
KM ROUTE HYDROGRAPH FROM 18 TO 20
RD
RC        0.055   0.045   0.055   227.1  0.001811     800
RX          402     500     520     560       600     607     614     675
RY        582.3   581.3   574.2   573.8     574.3   577.3   581.6   578.9
*
KK20TO22
KM ROUTE HYDROGRAPH FROM 20 TO 22
RD
RC        0.055   0.045   0.055   174.2  0.001811     800
RX          413     500     510     555       600     606     612     634
RY        581.5   581.1   573.8   573.5     573.6   577.4   580.7   580.3
*
KK22TO24
KM ROUTE HYDROGRAPH FROM 22 TO 24
RD
RC        0.055   0.045   0.055   406.6  0.001811     800
RX          412     505     516     526       541     570     581     599
RY        583.1   581.1   573.6   571.7     573.3   573.8   581.1   581.7
*
KK24TO26
KM ROUTE HYDROGRAPH FROM 24 TO 26 TEXDAM
RD
RC        0.055   0.045   0.055   194.3  0.000516     800
RX          500     516     529     557       567     572     584     588
RY        582.1   574.7     573     572     572.9   577.5   581.8   588.6
*
KK32TO34
KM ROUTE HYDROGRAPH FROM 32 TO 34
RD
RC        0.055   0.045   0.055    78.2  0.000516     800
RX          468     500     516     529       557     567     584     619
RY        582.4   582.1   574.7     573       572   572.9   581.8   579.6
*
KK34TO36
KM ROUTE HYDROGRAPH FROM 34 TO 36
RD
RC        0.055   0.045   0.055   190.1  0.000516     800
RX          401     500     514     554       569     573     583     656
RY        583.1   582.4   574.5   571.6     572.8   578.9   581.9   578.8
*
KK36TO38
KM ROUTE HYDROGRAPH FROM 36 TO 38
RD
RC        0.055   0.045   0.055   174.2  0.000516     800
RX          397     500     521     547       574     576     587     660
RY          583   582.6   572.9   572.2     572.8     579     582   578.5
*
KK38TO40
KM ROUTE HYDROGRAPH FROM 38 TO FIDDYMT40
RD
RC        0.055   0.045   0.055   190.1  0.000516     800
RX          405     510     528     557       570     577     588     641
RY        581.8   581.7   574.2   572.5       572   572.5   581.2   578.2
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KK FI040
KM  LOCAL RUNOFF FROM SUBBASIN FIDDYMT40
KO           1      0
BA         4.8
BF         3.8  -0.25    1.03
LS           0     52
UC        1.71   2.57
*
KK FID40
KM  COMBINE LOCAL FIDDYMT40 WITH HYDROGRAPH FROM 38
KO           1      0
HC           2
*
KK40TO42
KM ROUTE COMBINED FIDDYMT40 HYDROGRAPH FROM 40 TO 42
RD
RC       0.055  0.045  0.055  142.6  0.000868    800
RX         405    510    528    557       570    577    588    641
RY       581.8  581.7  574.2  572.5       572  572.5  581.2  578.2
*
KK42TO44
KM ROUTE HYDROGRAPH FROM 42 TO 44
RD
RC       0.055  0.045  0.055  174.2  0.000868    800
RX         430    510    529    538       575    590    630    692
RY       580.9  580.7  573.1  572.4     572.2  582.1  579.8  577.3
*
KK44TO46
KM ROUTE HYDROGRAPH FROM 44 TO 46
RD
RC       0.055  0.045  0.055  211.2  0.000868    800
RX         481    526    544    546       557    567    585    663
RY       581.6  573.4    574  572.3       571  572.2  581.7  577.4
*
KK46TO48
KM ROUTE HYDROGRAPH FROM 46 TO 48
RD
RC       0.055  0.045  0.055  337.9  0.000868    800
RX         394    500    516    544       561    579    584    651
RY       579.3  579.2  573.1    571     572.4    581  581.5  577.8
*
KK48TO50
KM ROUTE HYDROGRAPH FROM 48 TO 50
RD
RC       0.055  0.045  0.055  179.6  0.000868    800
RX         414    500    523    537       550    574    683    700
RY         582  581.4  572.4  570.9     572.2  581.8  577.2    582
*
KK50TO52
KM ROUTE HYDROGRAPH FROM 50 TO 52
RD
RC       0.055  0.045  0.055  211.2  0.000868    800
RX         393    500    524    539       574    576    584    608
RY       583.4  581.6  572.2  570.9     573.4  577.5  580.4  579.7
*
KK52TO54
KM ROUTE HYDROGRAPH FROM 52 TO 54
RD
RC       0.055  0.045  0.055  807.8  0.000868    800
RX         419    460    504    518       535    556    577    607
RY       579.5  576.4  576.4  572.2     570.6  572.3  581.1    579
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KK54TO56
KM ROUTE HYDROGRAPH FROM 54 TO MILNE56
RD
RC        0.055   0.045   0.055  1462.6  0.000868    800
RX          438     480     513     524       549    566    582    631
RY        584.1   579.6   575.6   571.7     570.8  571.2  578.2  579.7
*
KK MIL56
KM  LOCAL RUNOFF FROM SUBBASZN MILNE56
KO            1       0
BA          2.3
BF          1.8   -0.25    1.03
LS            0      52
UC         1.21    1.81
*
KK MIL56
KM  COMBINE LOCAL MILNE56 WITH HYDROGRAPH FROM 54
KO            1       0
HC            2
*
KK56TO58
KM ROUTE COMBINED MILNE56 HYDROGRAPH FROM 56 TO 58
RD
RC        0.055   0.045   0.055   327.3  0.002878    800
RX          438     480     513     524       549    566    582    631
RY        584.1   579.6   575.6   571.7     570.8  571.2  578.2  579.7
*
KK58TO60
KM ROUTE HYDROGRAPH FROM 58 TO 60
RD
RC        0.055   0.045   0.055     169  0.002878    800
RX          469     500     527     539       597    604    609    640
RY        585.5   580.8   577.8   571.6     571.3  578.5    586  585.2
*
KK60TO62
KM ROUTE HYDROGRAPH FROM 60 TO 62
RD
RC        0.055   0.045   0.055   232.3  0.002878    800
RX          477     500     530     540       586    591    615    652
RY        584.2   580.9   575.4   571.1     571.3  574.7  580.3  580.4
*
KK62TO66
KM ROUTE HYDROGRAPH FROM 62 TO LOC #1 ENTRANCE 66
KO            1       0
RD
RC        0.055   0.045   0.055  2272.8  0.002878    800
RX          460     472     516     533       572    589    600    700
RY        579.6   574.6   573.2     568     568.2  578.5  579.3  576.5
*
KK66TO70
KM ROUTE FROM ENTRANCE 66 TO LOCK #1 EXIT 70
RD
RC        0.055   0.045   0.055     250  0.002878    800
RX          420     491     526     527       544    545    567    656
RY        574.1   575.7   581.3   564.5     564.5  580.9  597.4  572.7
*
KK70TO74
KM ROUTE HYDROGRAPH FROM LOCK #1 EXIT 70 TO 74
RD
RC        0.055   0.045   0.055   354.8  0.003405    800
RX          489     500     519     537       557    574    603    688
RY        575.3     575   563.3     562     563.3  571.3  571.2  572.6
*
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KK74TO76
KM ROUTE HYDROGRAPH FROM 74 TO 76
RD
RC        0.055   0.045   0.055  2951.5  0.003405    800
RX          453     474     504     510       532    549     553     582
RY          575   568.1   566.6   563.2     560.8  563.2   568.5   567.3
*
KK76TO78
KM ROUTE HYDROGRAPH FROM 76 TO 78
RD
RC        0.055   0.045   0.055   813.1  0.003405    800
RX          450     486     512     520       555    568     586     607
RY        571.4     565   563.9     561     559.2  559.9   568.6   567.6
*
KK FRA78
KM  LOCAL RUNOFF FROM SUBBASIN FRACTION78
KO            1       0
BA          6.2
BF          4.9   -0.25    1.03
LS            0      52
UC         2.41    3.61
*
KK FRA78
KM  COMBINE LOCAL FRACTION78 WITH HYDROGRAPH FROM 76
KO            1       0
HC            2
*
KK78TO80
KM ROUTE COMBINED FRACTION78 HYDROGRAPH FROM 78 TO 80
RD
RC        0.055   0.045   0.055  1272.5  0.000766    800
RX          450     486     512     520       555    568     586     607
RY        571.4     565   563.9     561     559.2  559.9   568.6   567.6
*
KK80TO82
KM ROUTE HYDROGRAPH FROM 80 TO LOCK #2 ENTRANCE 82
KO            1       0
RD
RC        0.055   0.045   0.055   143.5  0.000766    800
RX          458     543     555     557       577    579     586     601
RY        570.6   567.9   567.4   555.7     555.1  568.4     568   567.3
*
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HEC-2 DATA FILE FOR I & M CANAL AT LOCKPORT

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SF SPLIT FLOW DIVERSION TO DEEP RUN CREEK
TW BREAK IN RIGHT BANK LEVEE BETWEEN SECTIONS 5.230 AND 5.236
WS           4   5.23  5.236      -1    3.08
WC           0  581.3      1 577.98      29  577.98      30   581.3
TC CULVERT IN RIGHT BANK LEVEE AT SECTION 3.62
CS          44  3.619  3.619      -1
CR           0  570.8      0 575.75    1.76  576.25     4.6  576.75    8.13  577.25
CR       12.16 577.75  16.22 578.25   20.26  578.75   24.33  579.25   28.01  579.75
CR       31.48 580.25 230.82 580.75  287.63  581.25  334.93  581.75  376.34  582.25
CR      413.62 582.75 447.81 583.25  479.57  583.75  509.35  584.25  537.48  584.75
CR      564.22 585.25 589.74 585.75   614.2  586.25  637.73  586.75  660.41  587.25
CR      682.35 587.75  703.6 588.25  724.22  588.75  744.28  589.25  763.81  589.75
CR      782.85 590.25 801.44 590.75  819.61  591.25  837.38  591.75  854.79  592.25
CR      871.85 592.75 888.57 593.25     905  593.75  921.12  594.25  936.97  594.75
CR      952.56 595.25  967.9 595.75  982.99  596.25  997.86  596.75
EE
*
C
C      7         PSE        OCT. 1991
C  0.000 BEGINNING OF I & M CANAL DIVERSION HEC-2 RUN
C  0.000 CROSS WITH AT & SF RR
C  1.031 CROSS WITH E.J. & E. RR
C  2.376 MOUTH OF FRACTION RUN
C  3.620 CROSS WITH ILL RT. 7
C  3.696 CLOSE TO MOUTH OF A SMALL CREEK
C  4.366 MOUTH OF FIDDYMENT CREEK
*
T1       I & M canal
T2       I & M canal
T3       I & M canal
J1           0      0      0      0       0       0    1062   559.5       0
J2          -1     -1
J3          40      1     43      8       4      26
*
* SECTION 6B  LOCK #2 DOWNSTREAM END
NC       0.055  0.055  0.045    0.3     0.5
X1       2.048     21    440    671       0       0       0
GR       571.2    440  571.1    444   570.6     453   570.6     458   566.2     469
GR       566.5    500  564.6    510   562.4     520   558.5     523   562.3     530
GR       567.9    543  567.4    555   555.7     557   555.1     577   568.4     579
GR         568    586  567.3    601   559.1     625   558.3     641   563.7     648
GR       561.2    671
*
* SECTION 6B  LOCK #2 ENTRANCE
NC       0.055  0.055  0.045    0.3     0.5
X1       2.104     21    440    671     300     300     300
GR       571.2    440  571.1    444   570.6     453   570.6     458   566.2     469
GR       566.5    500  564.6    510   562.4     520   558.5     523   562.3     530
GR       567.9    543  567.4    555   555.7     557   555.1     577   568.4     579
GR         568    586  567.3    601   559.1     625   558.3     641   563.7     648
GR       561.2    671
*
* SECTION  6C UPSTREAM OF LOCK # 2 ENTRANCE
NC       0.055  0.055  0.045    0.3     0.5
X1       2.105     17    432    671       2       2       2
GR       571.9    432  571.9    437   571.4     445   571.4     450   568.9     460
GR         565    486  564.7    500   563.9     512     561     520   559.9     544
GR       559.2    555  559.9    568   568.6     586   567.5     604   567.6     607
GR       569.1    629  560.1    671
*
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* SECTION 6C
NC       0.045  0.045  0.035      0       0
X1       2.121     17    432    671   83.31   83.31   83.31
GR       571.9    432  571.9    437   571.4     445   571.4     450  568.9     460
GR         565    486  564.7    500   563.9     512     561     520  559.9     544
GR       559.2    555  559.9    568   568.6     586   567.5     604  567.6     607
GR       569.1    629  560.1    671
*
* SECTION 8A  MOUTH OF FRACTION RUN
NC       0.045  0.045  0.035      0       0
X1       2.362     16    434    647 1272.8  1272.8  1272.8
                                          9       9       9
X2        1063
GR       574.3    434  574.7    440   574.6     448     575     453  568.1     474
GR         567    500  566.6    504   563.2     510   560.8     532  563.2     549
GR       568.5    553  567.8    557   567.3     582   567.3     582  565.7     605
GR       563.6    647
*
* SECTION 8A
NC       0.045  0.045  0.035      0       0
X1       2.516     16    434    647  815.01  815.01  815.01
X2         869
GR       574.3    434  574.7    440   574.6     448     575     453  568.1     474
GR         567    500  566.6    504   563.2     510   560.8     532  563.2     549
GR       568.5    553  567.8    557   567.3     582   567.3     582  565.7     605
GR       563.6    647
*
* SECTION 9A
NC       0.045  0.045  0.035      0       0
X1       3.075      9    447    688 2952.7  2952.7  2952.7
                                          4       4       4
GR       575.1    447  575.3    489     575     500   563.3     519    562     537
GR       563.3    557  571.3    574   571.2     603   572.6     688
*
* SECTION 9B
NC       0.045  0.045  0.035      0       0
X1       3.128     13    455    696  278.12  278.12  278.12
GR       573.8    455  571.5    500     570     513   563.3     525  562.4     544
GR       563.3    562  573.6    588   571.1     588   572.5     604    575     616
GR         575    631  575.1    647   572.5     696
*
* SECTION 9B DOWNSTREAM OF LOCK #1 EXIT
NC       0.055  0.055  0.045    0.3     0.5
X1        3.14     13    455    696   63.71   63.71   63.71
GR       573.8    455  571.5    500     570     513   563.3     525  562.4     544
GR       563.3    562  573.6    588   571.1     588   572.5     604    575     616
GR         575    631  575.1    647   572.5     696
*
* SECTION 9C LOCK #1 DOWNSTREAM END
NC       0.055  0.055  0.045    0.3     0.5
X1       3.141     17    420    656       2       2       2
GR       574.1    420  575.2    440   574.7     445   574.7     484  575.7     491
GR       577.9    500  580.6    523   581.3     527   564.5     527  564.5     544
GR       580.9    545  580.5    555   579.4     567   574.5     578  574.7     582
GR       574.7    605  572.7    656
*
* SECTION 9C LOCK #1
NC       0.055  0.055  0.045    0.3     0.5
X1       3.187     17    420    656     248     248     248
GR       574.1    420  575.2    440   574.7     445   574.7     484  575.7     491
GR       577.9    500  580.6    523   581.3     527   564.5     527  564.5     544
GR       580.9    545  580.5    555   579.4     567   574.5     578  574.7     582
GR       574.7    605  572.7    656
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*
* SECTION 9C LOCK #1 UPSTREAM END
NC      0.055  0.055  0.045    0.3     0.5
X1      3.188     17    420    656       2       2       2
GR      574.1    420  575.2    440   574.7     445   574.7     484  575.7     491
6R      577.9    500  580.6    523   581.3     527   564.5     527  564.5     544
GR      580.9    545  580.5    555   579.4     567   574.5     578  574.7     582
GR      574.7    605  572.7    656
*
* SECTION 9C UPSTREAM OF LOCK #1 ENTRANCE
NC      0.055  0.055  0.045    0.3     0.5
X1      3.189     17    420    656       2       2       2
GR      574.1    420  575.2    440   574.7     445   574.7     484  575.7     491
GR      577.9    500  580.6    523   581.3     527   564.5     527  564.5     544
GR      580.9    545  580.5    555   579.4     567   574.5     578  574.7     582
GR      574.7    605  572.7    656
*
* SECTION 9D
NC      0.055  0.055  0.045      0       0
X1      3.212     14    422    700  128.25  128.25  128.25
GR      575.7    422  576.1    446   579.6     460   574.6     472  573.5     500
GR      573.2    516    568    533   567.9     554   568.2     572  578.5     589
GR      578.9    594  579.3    600   578.7     624   576.5     700
*
* SECTION 10A
NC      0.055  0.055  0.045      0       0
X1      3.619     13    400    652  2149.3  2149.3  2149.3
GR      585.7    400  584.2    477   580.9     500   575.9     518  575.4     530
GR      571.1    540  570.8    562   571.3     586   574.7     591  579.3     608
GR      580.3    615  580.5    628   580.4     652
*
* SECTION 10B
NC      0.055  0.055  0.045      0       0
X1      3.664     12    413    640  235.26  235.26  235.26
GR      587.1    413  585.5    469   580.8     500   578.3     508  577.8     527
GR      571.6    539  570.9    561   571.3     579   578.5     604    586     609
GR      585.3    616  585.2    640
*
* SECTION 10C
NC      0.055  0.055  0.045      0       0
X1      3.696     13    438    631  169.29  169.29  169.29
GR      584.1    438  579.6    472   579.6     480   577.3     500  575.6     513
GR      571.7    524  570.8    549   571.2     566   574.5     569  578.2     582
GR      579.4    596  579.8    607   579.7     631
*
* SECTION  10C  MILNE CREEK
NC      0.055  0.055  0.045      0       0
X1      3.758     13    438    631  324.67  324.67  324.67
X2        870
GR      584.1    438  579.6    472   579.6     480   577.3     500  575.6     513
GR      571.7    524  570.8    549   571.2     566   574.5     569  578.2     582
GR      579.4    596  579.8    607   579.7     631
*
* SECTION 12A
NC      0.055  0.055  0.045      0       0
X1      4.035     12    419    642 1465.2  1465.2  1465.2
                                         8       8       8
X2        811
GR      579.5    419  578.5    450   576.4     460   576.6     500  576.4     504
GR      572.2    518  570.6    535   572.3     556   581.1     577  579.7     587
GR        579    607  576.5    642
*
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* SECTION 13A
NC   0.055   0.055   0.045       0       0
X1   4.188      14     393     743  805.06  805.06  805.06
GR   583.4     393   581.6     500   579.5     509   574.3     521   572.2     524
GR   570.9     539   572.3     563   573.4     574   577.5     576   580.4     584
GR   580.4     589   579.7     608   577.1     619   576.5     743
*
* SECTION 13B
NC   0.055   0.055   0.045       0       0
X1   4.228       7     414     683  213.94  213.94  213.94
GR     582     414   581.4     500   572.4     523   570.9     537   572.2     550
GR   581.8     574   577.2     683
*
* SECTION 13C
NC   0.055   0.055   0.045       0       0
X1   4.262      13     394     670  179.91  179.91  179.91
GR   579.3     394   581.7     463   581.8     468   579.2     500   573.1     516
GR   572.3     529     571     544   572.4     561     581     579   581.5     584
GR   579.2     593   577.8     651   577.9     670
*
* SECTION 14A
NC   0.055   0.055   0.045       0       0
X1   4.326      15     401     741  334.16  334.16  334.16
GR   579.1     401   581.6     476   581.6     481   579.8     500   579.2     505
GR   573.4     526     574     544   572.3     546     571     557   572.2     567
GR   581.7     585   579.5     599   578.1     633   577.4     663     575     741
*
* SECTION 14B
NC   0.055   0.055   0.045       0       0
X1   4.366      12     430     692  215.27  215.27  215.27
GR   580.9     430   582.2     483   582.2     488   581.7     500   580.7     510
GR   573.1     529   572.4     538   572.1     559   572.2     575   582.1     590
GR   579.8     630   577.3     692
*
* SECTION 14C
NC   0.055   0.055   0.045       0       0
X1   4.399      13     405     641  175.16  175.16  175.16
GR   581.8     405   582.7     484   582.8     489   582.4     500   581.7     510
GR   574.2     528   572.5     557     572     570   572.5     577   581.2     588
GR   579.1     602   578.6     614   578.2     641
*
* SECTION 14C  FIDDYMENT CREEK
NC   0.055   0.055   0.045       0       0
X1   4.426      13     405     641   140.9   140.9   140.9
X2     814
GR   581.8     405   582.7     484   582.8     489   582.4     500   581.7     510
GR   574.2     528   572.5     557     572     570   572.5     577   581.2     588
GR   579.1     602   578.6     614   578.2     641
*
* SECTION 17A
NC   0.055   0.055   0.045       0       0
X1   4.462      12     397     725  187.67  187.67  187.67
X2     672
GR     583     397   582.6     500   578.2     514   572.9     521   572.2     547
GR   572.8     574     579     576     582     587   579.5     601   578.6     636
GR   578.5     660   575.9     725
*
* SECTION 17B
NC   0.055   0.055   0.045       0       0
X1   4.495      11     401     656  177.98  177.98  177.98
GR   583.1     401   582.4     500   578.7     512   574.5     514   571.6     554
GR   572.8     569   578.9     573   581.9     583   579.4     596   579.4     622
GR   578.8     656
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*
* SECTION 17C
NC        0.055  0.055  0.045      0       0
X1        4.531     16    416    663  189.39  189.39  189.39
GR        580.4    416  582.4    468   582.6     481   582.4     484  582.2     495
GR        582.1    500  578.6    513   574.7     516     573     529    572     557
GR        572.9    567  577.5    572   581.8     584   579.9     599  579.6     619
GR        576.1    663
*
* SECTION 17C  DOWNSTREAM TEXACO DAM
NC        0.055  0.055  0.045    0.3     0.5
X1        4.544     16    416    663   71.39   71.39   71.39
GR        580.4    416  582.4    468   582.6     481   582.4     484  582.2     495
GR        582.1    500  578.6    513   574.7     516     573     529    572     557
GR        572.9    567  577.5    572   581.8     584   579.9     599  579.6     619
GR        576.1    663
*
* SECTION 17D  TEXACO WEIR DAM SECTION
NC        0.055  0.055  0.045    0.3     0.5
X1        4.545     19    407    638       4       4       4
GR        583.1    407  583.1    412     583     460     583     465  582.3     471
GR        582.2    484  581.7    500  580.34     516  578.62   518.3 578.6    536.3
                                                                       2
 578.16   536.5 578.16  548.8 578.62     549  578.62     567  580.34    570
GR        581.1    581  581.5    583   581.7     599   579.1     638
*
* SECTION 17D  UPSTREAM OF TEXACO DAM
NC        0.055  0.055  0.045    0.3     0.5
X1        4.546     19    407    638       8       8       8
GR        583.1    407  583.1    412     583     460     583     465  582.3     471
GR        582.2    484  581.7    500   581.1     505   579.3     513  573.6     516
GR        571.7    526  573.3    541   573.4     562   573.8     570  577.5     575
GR        581.1    581  581.5    583   581.7     599   579.1     638
*
* SECTION 17D
NC        0.055  0.055  0.045      0       0
X1        4.583     19    407    638  188.66  188.66  188.66
GR        583.1    407  583.1    412     583     460     583     465  582.3     471
GR        582.2    484  581.7    500   581.1     505   579.3     513  573.6     516
GR        571.7    526  573.3    541   573.4     562   573.8     570  577.5     575
GR        581.1    581  581.5    583   581.7     599   579.1     638
*
* SECTION 18A
NC        0.055  0.055  0.045      0       0
X1         4.66     13    413    710  406.73  406.73  406.73
GR        581.5    413  580.6    495   581.1     500   577.4     507  573.8     510
GR        573.5    555  573.6    600   577.4     606   580.7     612  581.6     614
GR        580.7    615  580.3    634   577.1     710
*
* SECTION 18B
NC        0.055  0.055  0.045      0       0
X1        4.693     16    402    718  174.31  174.31  174.31
GR        582.3    402  581.5    492   581.3     500   579.1     506  577.3     509
GR        574.2    520  573.8    560   574.3     600   577.3     607  580.2     612
GR        581.6    614  580.7    615   580.8     619   579.8     650  578.9     675
GR          577    718
*
* SECTION 18C
NC        0.055  0.055  0.045      0       0
X1        4.736     13    428    714  230.44  230.44  230.44
X2          674
GR        582.9    428  581.2    475   581.1     495   580.6     500  577.4     508
GR        574.7    520  575.4    565   575.6     580   577.2     602  581.3     611
GR        581.8    616  578.6    653   577.1     714
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*
* SECTION 18C  BIG RUN CREEK
NC       0.055  0.055  0.045      0       0
X1       5.127     13    428    714 2061.65 2061.65 2061.65
X2         618
6R       582.9    428  581.2    475   581.1     495   580.6     500  577.4     508
6R       574.7    520  575.4    565   575.6     580   577.2     602  581.3     611
GR       581.8    616  578.6    653   577.1     714
*
* SECTION 19A
NC       0.055  0.055  0.045      0       0
X1       5.216     13    422    666  470.11  470.11  470.11
GR         582    422  581.5    444   580.8     500   580.2     504  577.6     511
GR       576.9    523  574.2    540   574.9     555     577     568  580.1     571
GR       580.6    580  581.3    597   584.4     666
*
* SECTION 19A DEEP RUN CREEK DIVERSION  DOWNSTREAM
NC       0.055  0.055  0.045      0       0
X1        5.23     13    422    666   76.17   76.17   76.17
GR         582    422  581.5    444   580.8     500   580.2     504  577.6     511
GR       576.9    523  574.2    540   574.9     555     577     568  580.1     571
GR       580.6    580  581.3    597   584.4     666
*
* SECTION 19A DEEP RUN CREEK DIVERSION  UPSTREAM
NC       0.055  0.055  0.045      0       0
X1       5.236     13    422    666      30      30      30
GR         582    422  581.5    444   580.8     500   580.2     504  577.6     511
GR       576.9    523  574.2    540   574.9     555     577     568  580.1     571
GR       580.6    580  581.3    597   584.4     666
*
* SECTION 20A
NC       0.055  0.055  0.045      0       0
X1        5.69     13    418    614 2395.77 2395.77 2395.77
X2         625
GR       587.2    418  582.7    431     581     500     581     506  577.1     515
GR       574.8    525  572.2    540   573.8     560   577.1     572  581.2     578
GR         581    579  580.7    585   586.1     614
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