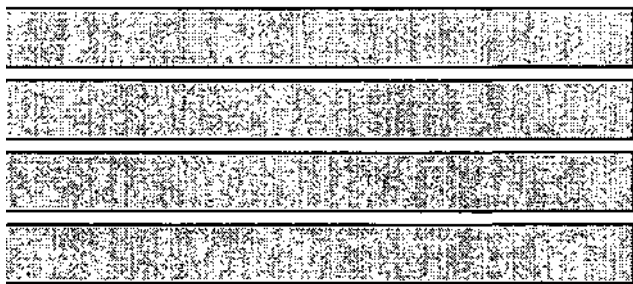


Contract Report 651

Hydraulic Analyses for LaGrange Pool of the Illinois River: A Component of the Restoration of Large River Ecosystems Project

by
Renjie Xia and Misganaw Demissie

August 1999



Illinois State Water Survey
Watershed Science Section
Champaign, Illinois

A Division of the Illinois Department of Natural Resources

**Hydraulic Analyses for LaGrange Pool
of the Illinois River:
A Component of the Restoration
of Large River Ecosystems Project**

Prepared by

Renjie Xia and Misganaw Demissie

Watershed Science Section

Illinois State Water Survey
2204 Griffith Drive
Champaign, IL 61820-7495

August 1999

This report was printed on recycled and recyclable papers.

Contents

	<i>Page</i>
Introduction.....	1
Acknowledgments.....	2
UNET Model.....	3
Formulation.....	3
Continuity and Momentum Equations.....	3
Implicit Finite Difference Scheme.....	4
Input Data.....	4
Channel Cross Sections.....	4
Manning's Roughness Coefficient.....	5
Boundary Conditions.....	5
Lateral Inflow.....	5
Initial Flow Condition.....	5
UNET Model for LaGrange Pool.....	7
Channel Cross-Sectional Data.....	12
Manning's Roughness Coefficient.....	15
Stage Data.....	15
Flow Discharge Data.....	16
Stage and Flow Discharge Data for Different Flood Conditions.....	24
Model Calibration and Verification.....	24
Results.....	27
10-year and 100-year Flood Routings.....	27
Effects of Changes in Floodplain Conveyance.....	27
Effects of Changes in Roughness.....	41
Conclusions.....	53
References.....	55

List of Tables

	<i>Page</i>
1 River Miles and Drainage Areas of Tributary Streams in LaGrange Pool.....	12
2 Data on Drainage and Levee District within LaGrange Pool.....	15
3 Manning's Roughness Coefficients Used in LaGrange Pool.....	16
4 Statistics for Illinois River Gaging Station at Kingston Mines.....	16
5 Information for USGS Streamgaging Stations.....	23
6 Effects of Changes in Roughness on River Stage.....	41
7 Effects of Changes in Roughness on Flow Discharge.....	42

List of Figures

	<i>Page</i>
1 Illinois River Waterway profile.....	8
2 Plan view of LaGrange Pool	9
3 Schematic of LaGrange Pool.....	11
4 Selected cross sections in LaGrange Pool.....	13
5 Observed water surface elevation at Peoria Lock and Dam and at LaGrange Lock and Dam.....	17
6 Observed water surface elevation at Kingston Mines, Havana, and Beardstown.....	18
7 Variation of annual total discharge at Kingston Mines.....	20
8 Variations of annual total discharge in four major tributaries.....	21
9 Water surface elevation hydrographs for upstream and downstream boundaries.....	25
10 Computed water surface elevation hydrographs.....	28
11 Computed flow discharge hydrographs.....	30
12 Effects of levee removal on water stage for 10-year flood.....	33
13 Effects of levee removal on water stage for 100-year flood.....	35
14 Effects of levee removal on flow discharge for 10-year flood.....	37
15 Effects of levee removal on flow discharge for 100-year flood.....	39
16 Effects of roughness changes on water stage for 10-year flood.....	43
17 Effects of roughness changes on water stage for 100-year flood.....	45
18 Effects of roughness changes on flow discharge for 10-year flood.....	47
19 Effects of roughness changes on flow discharge for 100-year flood.....	49

Hydraulic Analyses for LaGrange Pool of the Illinois River: A Component of the Restoration of Large River Ecosystems Project

Renjie Xia and Misganaw Demissie

Introduction

The Illinois River is one of the major tributaries of the Upper Mississippi River and one of the major inland waterways in the United States. One reach along the Illinois River that provides great possibilities for the restoration of large river ecosystems is LaGrange Pool located between LaGrange Lock and Dam at river mile (RM) 80.2 and Peoria Lock and Dam at RM 157.7. River miles on the Illinois River are the distance measured from Grafton, Illinois, the confluence of the Illinois River with the Mississippi River. The segment of the Illinois River from LaGrange to Peoria is the least regulated and contains diverse physical environments that provide opportunities for experimentation with different restoration techniques (Braden et al., 1996). In addition, the availability of scientific data on geomorphology, water quality, and biota in LaGrange Pool was an important factor in selecting this pool for the study of restoration efforts.

Detailed hydraulic modeling is required to provide information to assess the ecological and economic consequences of floodplain management options. Since the flow in most rivers and most of the time is unsteady, unsteady flow models should be used for this purpose. River flow is generally three-dimensional in character, which presents a varying degree of complexities and problems in nature. In practice, to circumvent these problems and to be able to find satisfactory solutions, certain simplifying assumptions are introduced into flow equations; thus, a three-dimensional flow problem can be reduced to either a two- or a one-dimensional problem (Ahmed, 1987). The assumption of one-dimensional flow provides us only with information of gross quantities, such as average stage, average velocity, and total discharge at each river cross section. However, this type of information is sufficient for this stage of the study.

Several one-dimensional unsteady flow models are available to simulate the flow of water in a river channel. These models include DWOPER (Dynamic Wave OPERational Model) developed by Fread (1978) and used by the National Weather Service (NWS); FEQ (Full Equations) Unsteady Flow Model developed by Franz (1990) and used by the Division of Water Resources of the U.S. Geological Survey (USGS); and UNET (One-Dimensional Unsteady Flow through a Full Network of Open Channels) developed by Barkau (1993) and maintained and distributed by the Hydrologic Engineering Center (HEC) of the U.S. Army Corps of Engineers (USACOE). An updated 1995 UNET hydraulic model (Barkau, 1995) was used for this study.

The UNET model, containing partial differential equations for the conservation of mass and momentum, simulates one-dimensional unsteady flow through a full network of open channels, including single, dendritic, or fully looped systems. The UNET model can simulate water-level fluctuations due to changes in inflow from upstream or tributaries, channel and floodplain geometry, boundary conditions, or river roughness. In addition, the UNET model enables users to simulate levee failures and storage interactions, gated spillways and weir overflow structures, bridge and culvert hydraulics, and pumped diversions.

The UNET model requires: (a) river cross sections, (b) Manning's roughness coefficients, (c) boundary conditions of flow and stage, and (d) observed flow and stage data at selected locations for model calibration. The model requires two separate files for the cross-sectional geometry and boundary conditions of flow and stage. Boundary conditions for the UNET model can be input from any existing database of the HEC Data Storage System or HEC-DSS (Barkau, 1995). The advantages of HEC-DSS are that it eliminates the tabular input of hydrographs and creates an input file that can be easily adapted for many scenarios (Barkau, 1995). The UNET model outputs include a time-series of flow and stage at prescribed locations and plots of water surface elevation profiles. All model input, output, and calculations are performed in U.S. foot-pound units.

This report evaluates the application of the one-dimensional UNET model for LaGrange Pool to examine 10-year and 100-year floods and different management scenarios (changes in floodplain conveyance and river roughness).

Acknowledgments

This research project is supported in part by the U.S. Environmental Protection Agency and the National Science Foundation Water and Watershed Program (Grant DEB-9613562). The views expressed in this report are those of the authors and do not necessarily reflect the views of the sponsors or the Illinois State Water Survey. John B. Braden, Project Manager, and Richard E. Sparks, David C. White, and Daniel Schneider, Co-P.I.s for this project, provided valuable inputs and review comments. Their assistance is greatly appreciated. Becky Howard formatted the camera-ready report, which was edited by Eva Kingston.

UNET Model

Formulation

Continuity and Momentum Equations

The UNET model is a one-dimensional unsteady flow model that consists of one continuity equation and one momentum equation, both derived by Liggett and Cunge (1975). The continuity equation provides the mass balance of water for a segment of the river channel. For an elementary control volume with the distance x measured along the channel, the flow and total flow area at the midpoint of the control volume are denoted as Q and A , respectively. The conservation of mass for the control volume states that the net rate of flow into and out of the volume should equal the rate of change of storage inside the volume. Respective inflow and outflow to the control volume can be written as:

$$Q - \frac{\partial Q}{\partial x} \frac{\Delta x}{2} \quad (1)$$

and

$$Q + \frac{\partial Q}{\partial x} \frac{\Delta x}{2} \quad (2)$$

Assuming that Δx is small, the change in mass in the control volume is equal to

$$\rho \frac{\partial A}{\partial t} \Delta x = \rho \left[\left(Q - \frac{\partial Q}{\partial x} \frac{\Delta x}{2} \right) - \left(Q + \frac{\partial Q}{\partial x} \frac{\Delta x}{2} \right) + Q_t \right] \quad (3)$$

where ρ is the fluid density, A is the cross-sectional area, Q is the flow, x is the distance along the channel, t is the time, and Q_t is the lateral flow entering the control volume. Equation (3) can be further simplified by dividing by $\rho \Delta x$; thus, the final form of the continuity equation in the UNET model is:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} - q_t = 0 \quad (4)$$

where q_t is the lateral inflow per unit length, i.e., $Q_t / \Delta x$.

The momentum equation is a balance of all the forces and accelerations acting on the water in a segment of the river channel. The conservation of momentum for a control volume states that the net rate of momentum entering the volume plus the sum of all external forces acting on the volume should equal the rate of accumulation of momentum. This is a vector equation applied in the x -direction. The momentum flux is the fluid mass times the velocity vector in the direction of flow. Forces considered in the UNET model are: (a) pressure, (b)

gravity, and (c) boundary drag or friction. Using Newton's second law, the final form of the momentum equation is expressed as:

$$\frac{\partial Q}{\partial x} + \frac{\partial(VQ)}{\partial x} + gA\left(\frac{\partial h}{\partial x} + S_f\right) - q_l v_l = 0 \quad (5)$$

where V is the velocity, h is the water depth, S_f is the frictional slope, g is the gravitational acceleration, and v_l is the lateral inflow velocity. The friction slope, S_f , in Equation (5) is estimated using the Manning equation:

$$S_f = \frac{Q|Q|n^2}{2.208R^{4/3}A^2} \quad (6)$$

where R is the hydraulic radius and n is the Manning's roughness coefficient. Equation (6) uses $Q|Q|$ to maintain a proper direction for the flow.

Implicit Finite Difference Scheme

The UNET model solves Equations (4) and (5) using the implicit finite difference scheme. A four-point implicit scheme, also known as the box scheme, is considered to be the most successful for solving unsteady flow equations. The simultaneous solution is an important aspect of this scheme because it allows information from the entire reach to influence the solution at any one point. Consequently, the time step can be significantly larger than with explicit numerical schemes. The UNET user's manual (Barkau, 1995) presents a detailed discussion of the general implicit four-point form, including function value, time derivative, and space derivative; the finite difference form of the continuity and momentum equations; and the stability analysis.

Input Data

Channel Cross Sections

Cross sections used in the UNET model are usually taken at gaging stations, at locations where changes in cross-sectional geometry significantly affect the flow, at the confluences of tributaries, and around hydraulic structures (such as bridge piers, culverts, weirs, and locks and dams). After each of these selected cross sections is surveyed, pairs of elevation and distance values are used to create a cross-sectional geometry file for the UNET model. The first pair of elevation and distance values for a certain cross section is taken from a predetermined point on the left bank of the river looking downstream.

Manning's Roughness Coefficient

Values of the Manning's roughness coefficient, n , for the channel area and for the left and right overbank areas of each cross section should be specified in the cross-sectional geometry file to describe the flow resistance. The value of n is highly variable and depends on a number of factors, such as bed forms, vegetation, channel irregularity, channel alignment, silting and scouring, obstruction, channel size and shape, stage and discharge, seasonal change, and suspended material and bed load (Chow, 1959). In general, roughness is given as a function of either discharge or stage and is determined by comparing observed and computed stages.

Boundary Conditions

For a river reach with N computational nodes, which bound $N-1$ finite difference cells, $2N-2$ finite difference equations can be developed. Because there are $2N$ unknowns (Q and h or V and A for each node), two additional equations are needed. The UNET model applies the boundary conditions at the upstream and downstream as the two additional equations. A stage hydrograph or a discharge hydrograph can be specified for the upstream boundary condition. A stage hydrograph, a discharge hydrograph, or a rating curve indicating a known relation between stage and discharge can be specified for the downstream boundary condition. Specifications for any combination of these boundary conditions are satisfactory. For a river system with tributaries for which cross-sectional data exist, the UNET model requires a discharge (or stage) hydrograph at the upstream boundary for each tributary. A boundary condition file describing all boundary conditions is input into the UNET model from the HEC-DSS database.

Lateral Inflow

As tributaries flow into the main river, the pattern of the river system becomes dendritic and the main stem is divided into several sub-reaches between the tributaries. For example, a main stem with two tributaries has three reaches: from the upstream end of the river to the mouth of tributary 1, between the two tributaries, and from the mouth of tributary 2 to the downstream end of the river. The UNET model considers all tributaries as lateral inflow, q_l . A tributary with available cross-sectional geometry data is treated as a separate upstream reach, and discharge and stage along the tributary reach can be simulated when the condition at the upstream end of the tributary is specified. A tributary without available cross-sectional geometry data is input into the UNET model as point inflow or as inflow uniformly distributed along the banks of the main river.

Initial Flow Condition

The initial flow condition for each river reach can be specified using the time-series input file. If no initial condition is specified, the UNET model will assume steady-state subcritical flow as the initial condition.

UNET Model for LaGrange Pool

There are eight locks and dams that control the water level along the Illinois Waterway. Five of the lock and dam structures are on the Illinois River, and one is on the Mississippi River and controls the lower 80 miles of the Illinois River (Figure 1). The locks and dams divide the Illinois River into two distinctly different segments: the upper Illinois River and the lower Illinois River. The upper Illinois River has a steep gradient with an average slope of 1.2 feet per mile, and the lower Illinois River has a very gentle gradient with an average slope of only 0.17 feet per mile (Demissie et al., 1988). LaGrange Pool is located within the lower Illinois River and extends from RM 80.2 to Peoria Lock and Dam at RM 157.7. LaGrange Lock and Dam is on the right bank of the Illinois River about 8 miles downstream from Beardstown, Illinois. The dam controls the pool level, and the lock provides passage for river traffic from one pool to the next. Table 1 provides relevant information on tributary streams that drain into LaGrange Pool.

In the UNET model, the Illinois River system is represented as dendritic, consisting of a series of sub-reaches because of tributaries. The major tributaries draining into LaGrange Pool are the Mackinaw River (drainage area 1,136 square miles or sq. mi.), Spoon River (1,855 sq. mi.), Sangamon River (5,419 sq. mi.), and LaMoine River (1,350 sq. mi.). These tributaries drain 11,094 sq. mi. of primarily agricultural land into LaGrange Pool and contribute significant flow and a large amount of sediment to the Illinois River valley. The Mackinaw River, a left-bank tributary, enters the pool about 5 miles below Peoria. The Spoon and LaMoine Rivers, right-bank tributaries, enter the pool at Havana and 3 miles above LaGrange Lock and Dam, respectively. The largest of the four tributaries, the Sangamon River, accounts for 49 percent of the drainage area between Peoria and LaGrange. This left-bank tributary enters the pool near Beardstown at RM 88.9. Figure 2 shows a plan view of the LaGrange Pool area using 1994 land cover data.

The Sangamon River is the only tributary for which cross-sectional geometry data are available. Thus, the Illinois River within LaGrange Pool consists of three river reaches shown in the schematic for LaGrange Pool (Figure 3). Reach 1 (A to B) is the segment of the Illinois River from Peoria Lock and Dam to the section just upstream of the Sangamon River junction. Reach 2 (D to B) is the segment of the Sangamon River from the gage at Oakford to its confluence with the Illinois River. Reach 3 (B to C) is the segment of the Illinois River from the Sangamon River junction to the LaGrange Lock and Dam.

It was assumed that the contributions of lateral inflow from tributaries of the Sangamon River were uniformly distributed along the entire length of the Sangamon River. Other tributaries of the Illinois River, including the Mackinaw River, Copperas Creek, Quiver Creek, Spoon River, Otter Creek, Sugar Creek, and LaMoine River, were considered as point lateral inflows at their respective confluences with the Illinois River.

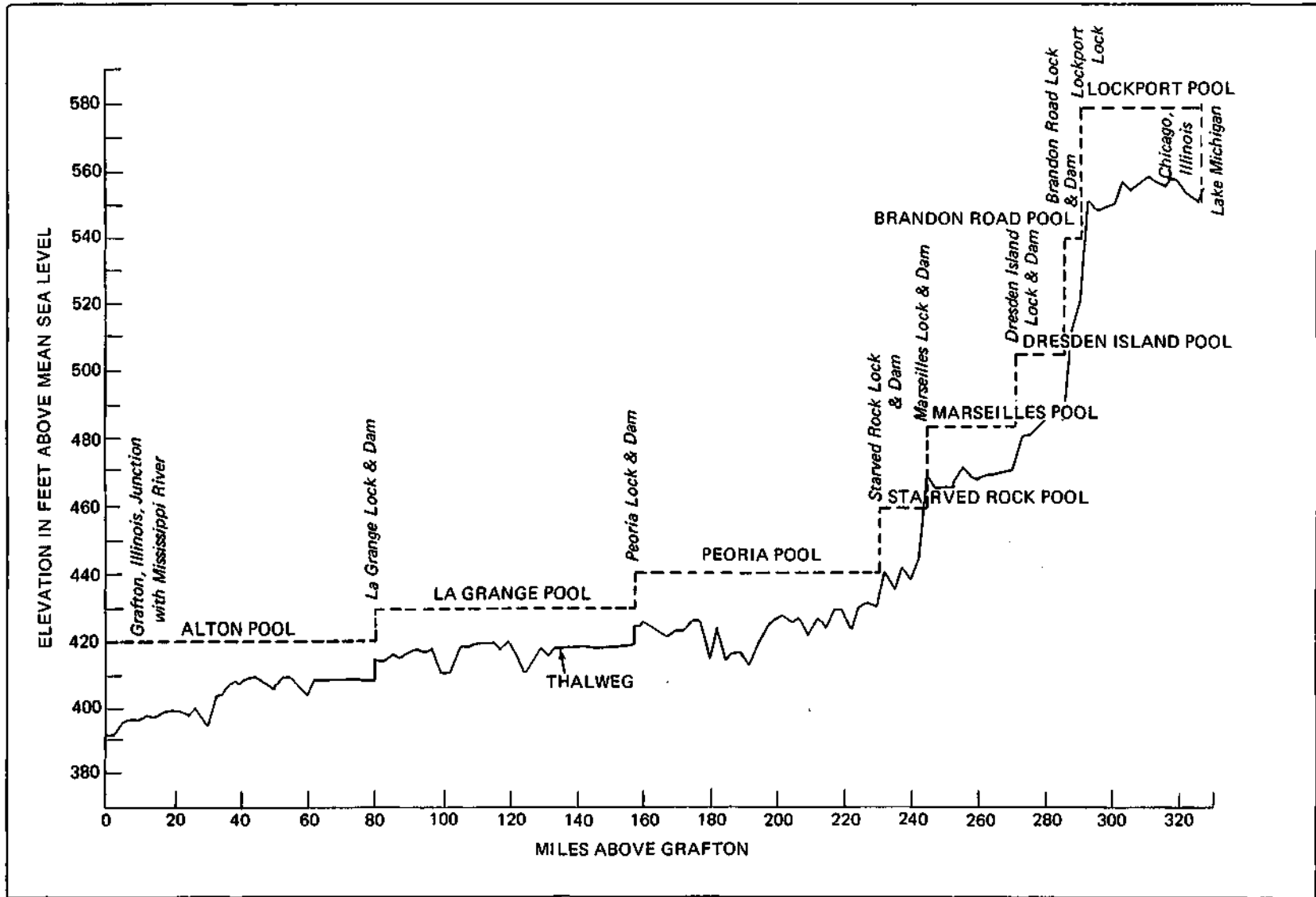


Figure 1. Illinois River Waterway profile

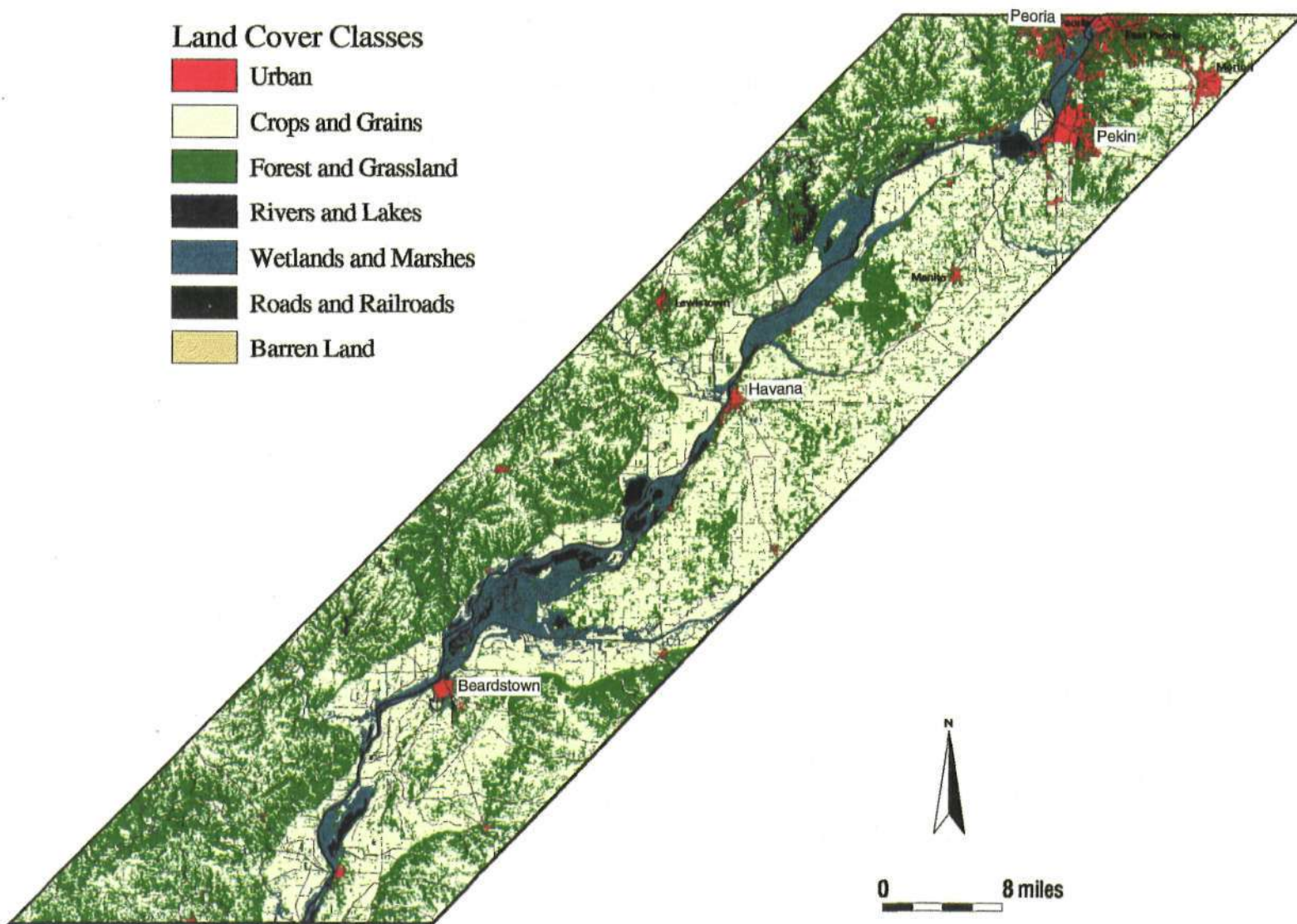


Figure 2. Plan view of LaGrange Pool

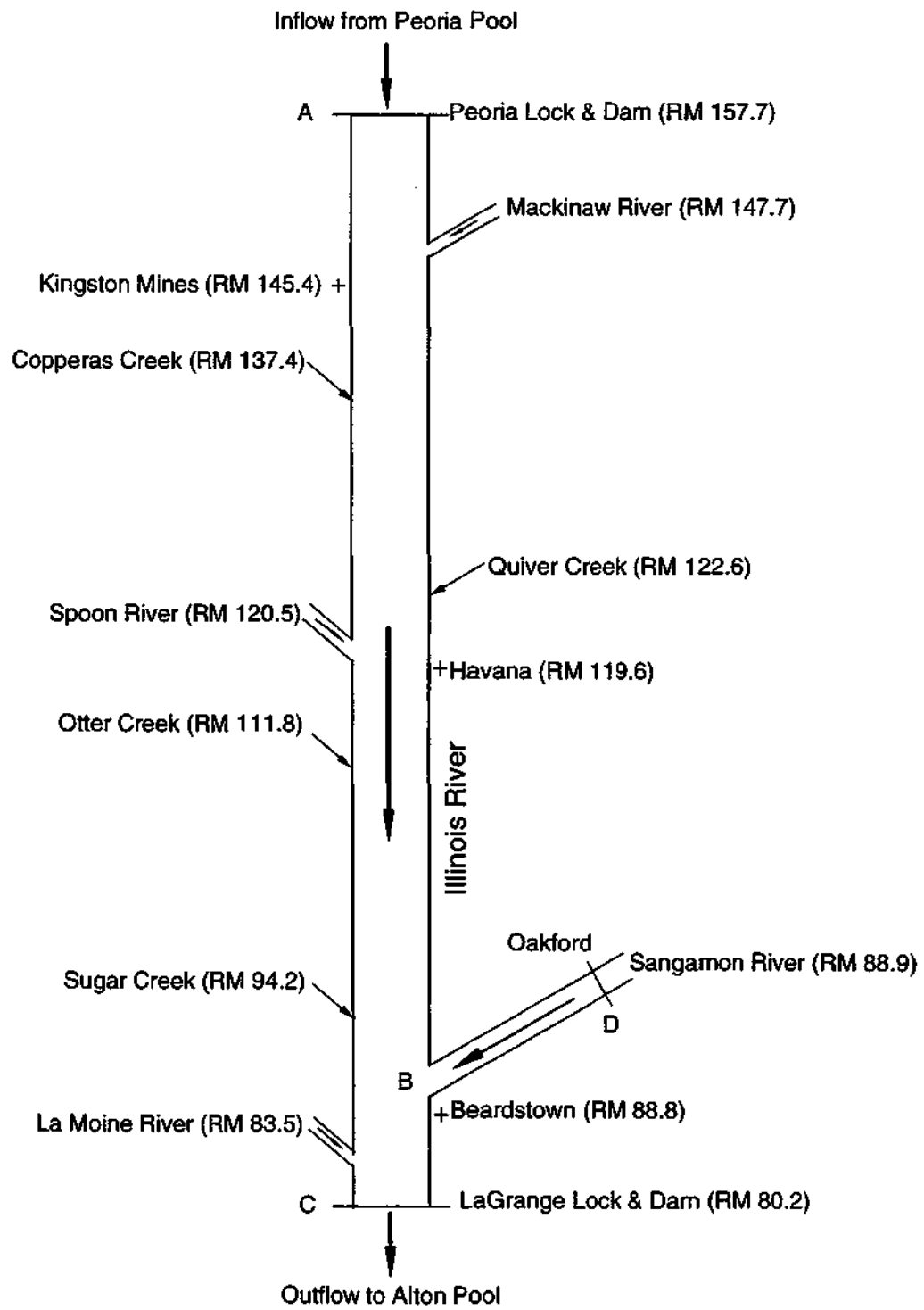


Figure 3. Schematic of LaGrange Pool

Table 1. River Miles and Drainage Areas of Tributary Streams in LaGrange Pool

<i>Main stem stations and tributary streams</i>	<i>River miles above mouth of Illinois River</i>	<i>Drainage area (sq. mi.)</i>
<u>LaGrange Lock & Dam</u>	80.2	25,648.0
LaMoine River	83.5	1,350.0
Crane Creek	84.9	40.2
<u>Illinois River at Beardstown</u>	88.8	24,227.0
Sangamon River	88.9	5,419.0
Lost Creek	89.0	16.5
Sugar Creek	94.2	162.0
Elm Creek	102.7	9.2
Wilson Creek	108.3	13.3
Otter Creek	111.8	126.0
<u>Illinois River at Havana</u>	119.6	18,299.0
Spoon River	120.5	1,855.0
Quiver Creek	122.6	261.0
Big Sister Creek	126.3	28.4
Buckheart Creek	128.2	20.6
Duck Creek	131.7	20.5
Copperas Creek	137.4	127.0
<u>Illinois River at Kingston Mines</u>	145.4	15,819.0
Mackinaw River	147.7	1,136.0
LaMarsh Creek	149.7	40.2
Lost Creek	151.0	23.0
<u>Illinois River at Pekin</u>	152.9	14,585.0
Lick Creek	156.4	19.2
<u>Peoria Lock & Dam</u>	157.7	14,554.0

Channel Cross-Sectional Data

The USACOE included 168 surveyed cross sections for LaGrange Pool in the Illinois River and 25 surveyed cross sections for the Sangamon River. The Illinois River follows a winding course in a wide floodplain bordered by bluffs. Within this wide floodplain, numerous lakes, ponds, sloughs, and connecting channels provide a great deal of valley storage. Valley width ranges from 2 miles near Pekin to more than 5 miles near Beardstown. When the levees are considered, the average width of cross sections in LaGrange Pool from RM 157.7 to RM 80.2 is 8,829 feet (ft), according to USACOE data. Maximum and minimum widths of cross sections in this pool are 36,100 ft at RM 100.2 and 1,200 ft at RM 152.8, respectively. The average width of cross sections for the Sangamon River is 4,910 ft. Figure 4 shows cross sections at selected locations on the Illinois River. Figures 4(a) and 4(b) show the large difference in the cross-sectional widths within LaGrange Pool. Figures 4(c) and 4(d) show the shapes of cross sections at Havana and Beardstown with levees on one side or both sides.

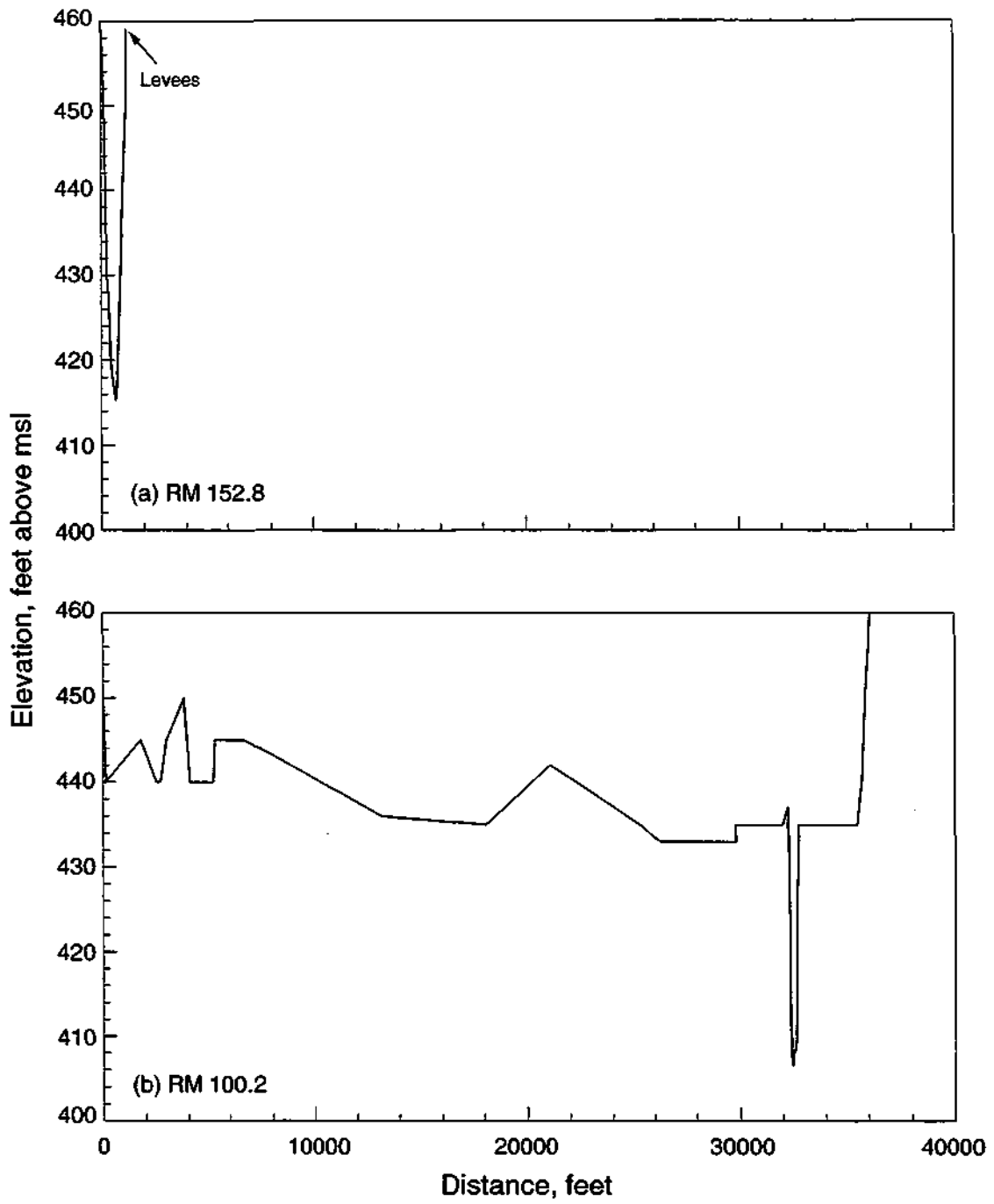


Figure 4. Selected cross sections in LaGrange Pool

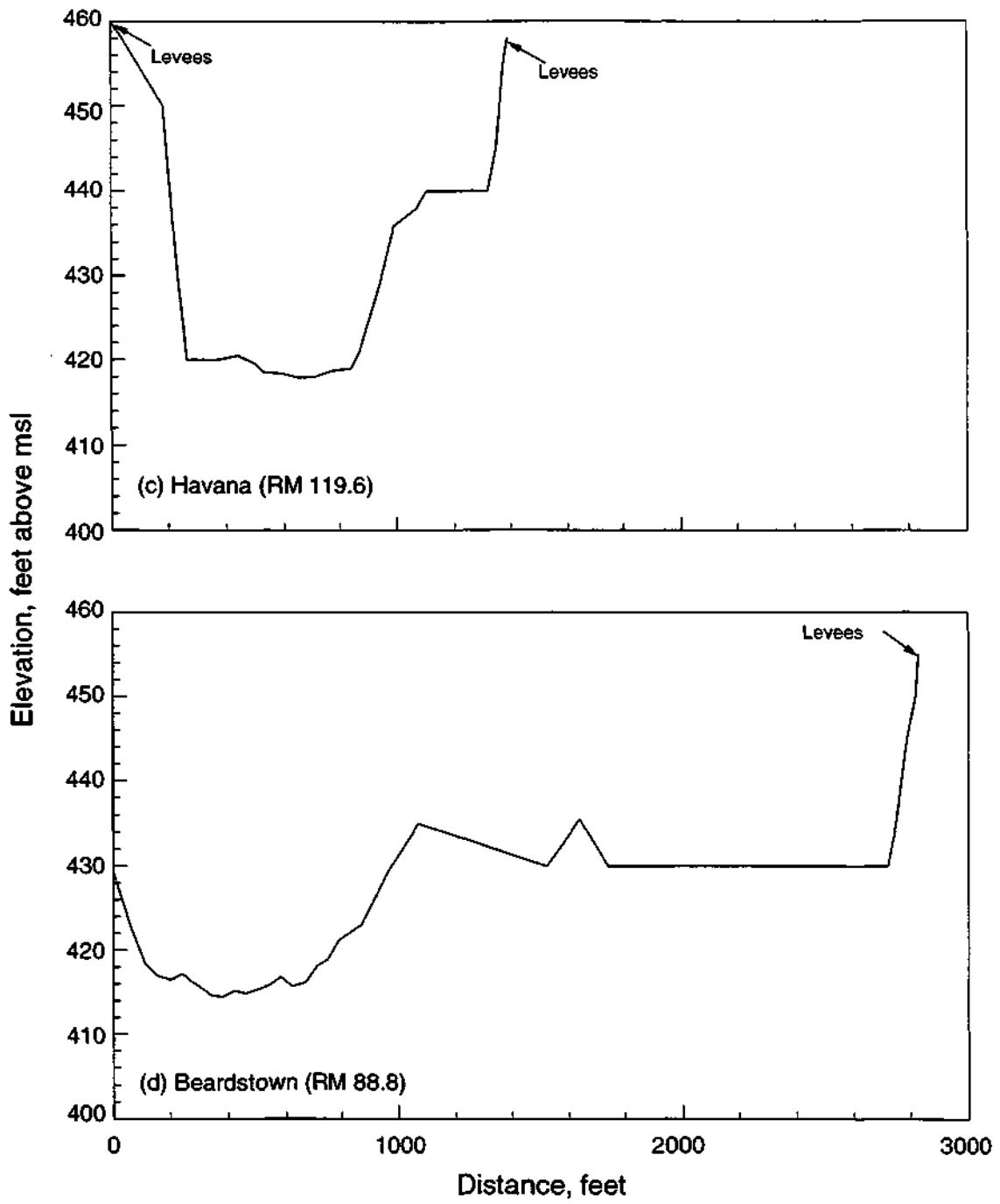


Figure 4. (concluded)

Agricultural levees constructed or rehabilitated by the USACOE protect much of the floodplain below Peoria. These levees were designed to provide protection against a 100-year flood (USACOE, 1986). There are also numerous private levees along the tributaries. Within LaGrange Pool, there are ten drainage and levees districts in Reach 1, and three in Reach 3. The crown elevation of levees in Reach 1 and Reach 3 ranges from 450 to 458 feet above mean sea level (ft-msl). Table 2 provides data on existing drainage and levees districts in Reach 1 and Reach 3. These data were combined in a cross-sectional geometry file for the UNET model.

Manning's Roughness Coefficient

The USACOE estimated Manning's roughness coefficient values for cross sections in the Illinois River and Sangamon River based on field reconnaissance surveys. The Manning's roughness coefficient value is 0.02 within the channel area for all cross sections and varies between 0.03 and 0.1 for the left and right overbank areas, respectively. Table 3 shows the Manning's roughness coefficient values used in the UNET model. As the model is calibrated, these Manning's roughness coefficient values are determined so that stage hydrographs generated by the model at selected gage locations match historical records. The Manning's roughness coefficient has a significant impact on the stage and flow discharge hydrographs.

Stage Data

Daily mean stage records from 1936 to 1998 measured by USACOE are available at Peoria Lock and Dam and at LaGrange Lock and Dam. The following years (1973,1974,1979,

Table 2. Data on Drainage and Levee District within LaGrange Pool

<i>Levee district</i>	<i>River mile</i>		<i>Protected area (acres)</i>	<i>Elevation (ft-msl)</i>	
	<i>Upstream</i>	<i>Downstream</i>		<i>Interior</i>	<i>Crown</i>
Reach 1					
Pekin & La Marsh	155.0	149.7	3,010	438.0	458.0
Spring Lake	147.2	134.2	13,120	430.0	455.0
Banner Special	146.0	138.0	4,561	440.0	455.6
East Liverpool	132.0	129.0	2,885	435.0	455.0
Liverpool	129.0	127.0	2,885	430.0	455.0
Thompson Lake	127.0	119.4	5,498	430.0	453.7
Lacey, Langellier, W. Mantaza, & Kerton valley	119.4	111.8	10,406	435.0	456.0
Seahom	111.8	107.0	2,000	431.0	452.0
Big Lake	107.0	102.0	3,401	435.0	451.0
Kelly Lake	102.0	97.0	1,045	434.0	451.0
Reach 3					
Coal Creek	91.2	85.0	6,794	430.0	454.7
Crane Creek	85.1	83.8	5,417	430.0	450.0
S. Beardstown & Valley City	87.7	79.0	10,516	428.0	453.8

Table 3. Manning's Roughness Coefficients Used in LaGrange Pool

<i>River reach</i>	<i>River mile (upstream - downstream)</i>	<i>Manning' s roughness coefficient</i>		
		<i>Left bank</i>	<i>Channel</i>	<i>Right bank</i>
Peoria Lock & Dam - Kingston Mines	157.7 - 145.4	0.1	0.02	0.1
Kingston Mines - Havana	145.4-119.6	0.1	0.02	0.03-0.1
Havana - Beardstown	119.6- 88.8	0.1	0.02	0.03-0.1
Beardstown - La Grange Lock & Dam	88.8- 80.2	0.03-0.1	0.02	0.03-0.1

1982, 1983, and 1985) were selected to show variations of stage at Peoria Lock and Dam and at LaGrange Lock and Dam (Figure 5). These observed historical records can be used as the upstream and downstream boundary conditions for the UNET model and then the time-series of flow or stage at various prescribed locations in LaGrange Pool can be simulated. The USACOE also observed daily mean stage records for river locations at Kingston Mines (1911-1998), Havana (1900-1998), and Beardstown (1900-1998). Figure 6 shows variations of the observed stages at the three locations for 1973, 1974, 1979, 1982, 1983, and 1985. Comparisons of the time-series of stage between measured and computed values at these river locations can be used for model calibration and verification.

Flow Discharge Data

Daily mean discharge records from October 1939 to September 1998 are available for the Illinois River gaging station at Kingston Mines (USGS ID 05568500). This is the only gaging station on the Illinois River that provides flow discharge data in LaGrange Pool. Figure 7 shows the variation of the annual total discharge over time for this station, and Table 4 summarizes the statistics. Figure 7 shows that Water Year 1993 was the wettest year, and Water Year 1964 was the driest year during the 1939-1998 period.

There are USGS streamgaging stations with continuous discharge records for four major tributaries that drain into LaGrange Pool: Mackinaw River, Spoon River, Sangamon River, and LaMoine River. These discharge records are specified as lateral inflows for the UNET model. Figure 8 shows variations of the annual total flow discharges with time for the four tributaries, and Table 5 summarizes information about these stations.

Table 4. Statistics for Illinois River Gaging Station at Kingston Mines

<i>Statistics</i>	<i>Flow discharge, cfs</i>	<i>Date</i>
Annual mean	15,770	1939 -1998
Highest annual mean	32,200	1993
Lowest annual mean	6,820	1964
Highest daily mean	86,700	Dec. 7,1982
Lowest daily mean	1,700	Aug.29-Sep. 1,1984
Instantaneous peak flow	88,800	Dec. 7,1982

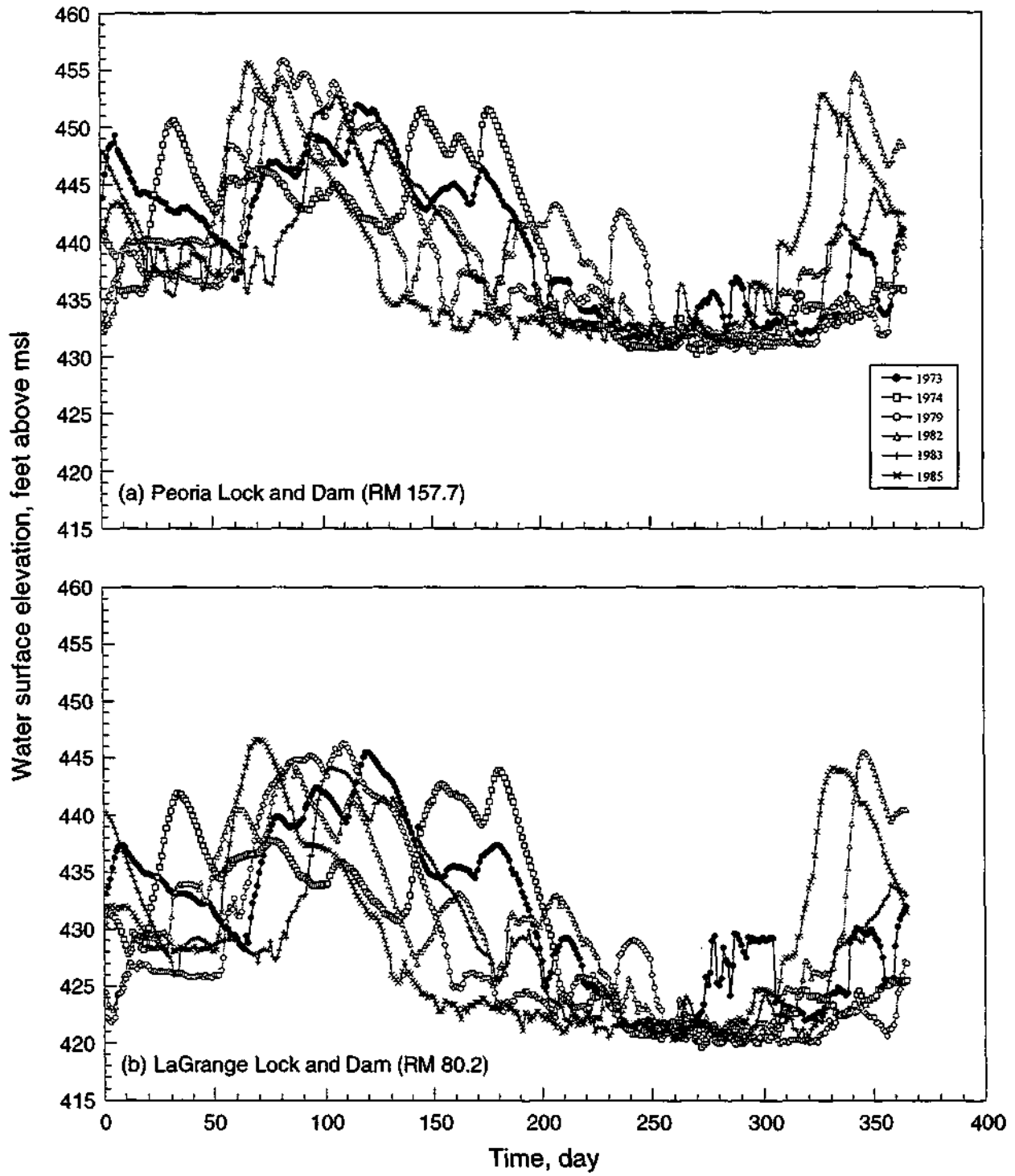


Figure 5. Observed water surface elevation at Peoria Lock and Dam and at LaGrange Lock and Dam

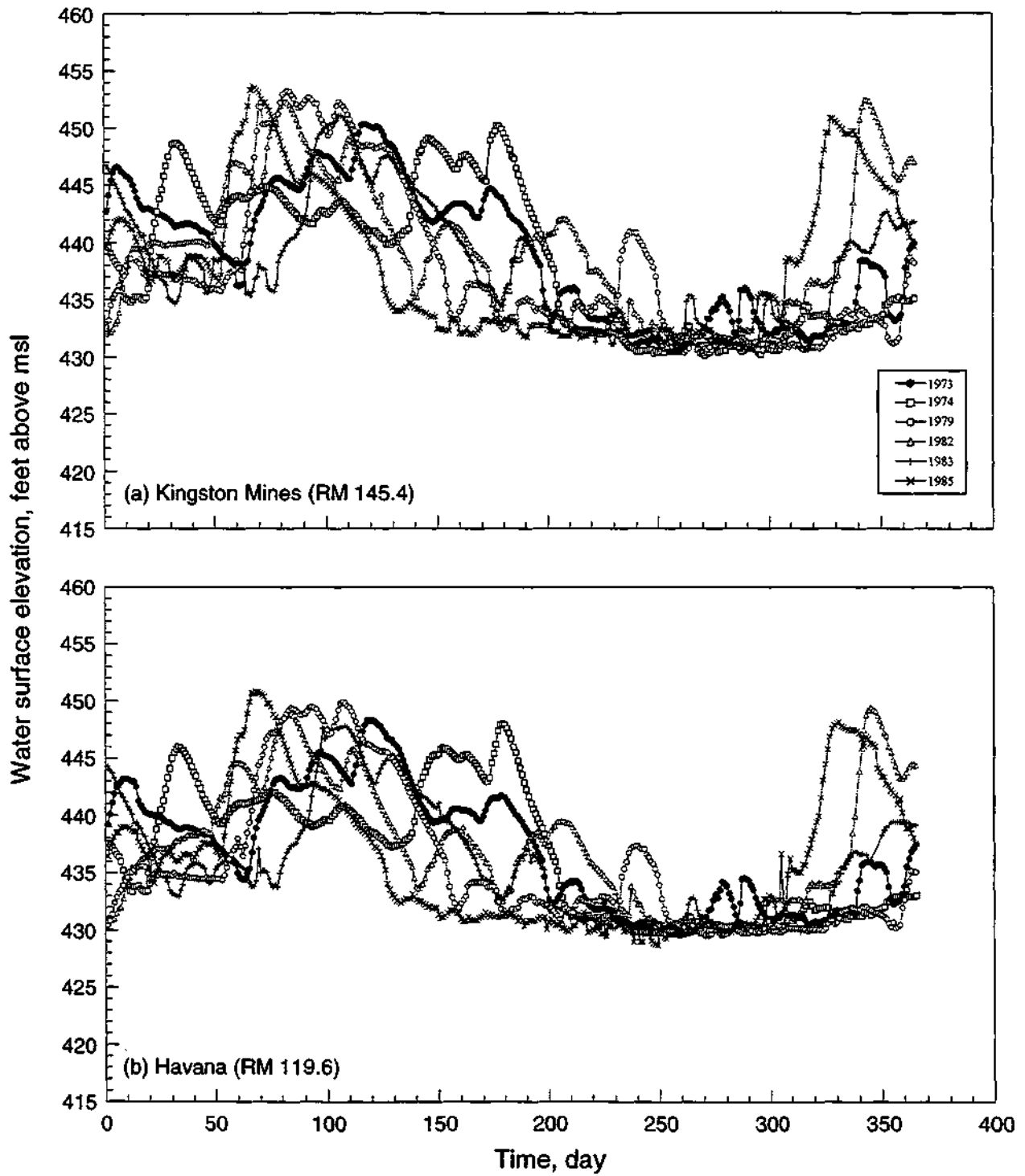


Figure 6. Observed water surface elevation at Kingston Mines, Havana, and Beardstown

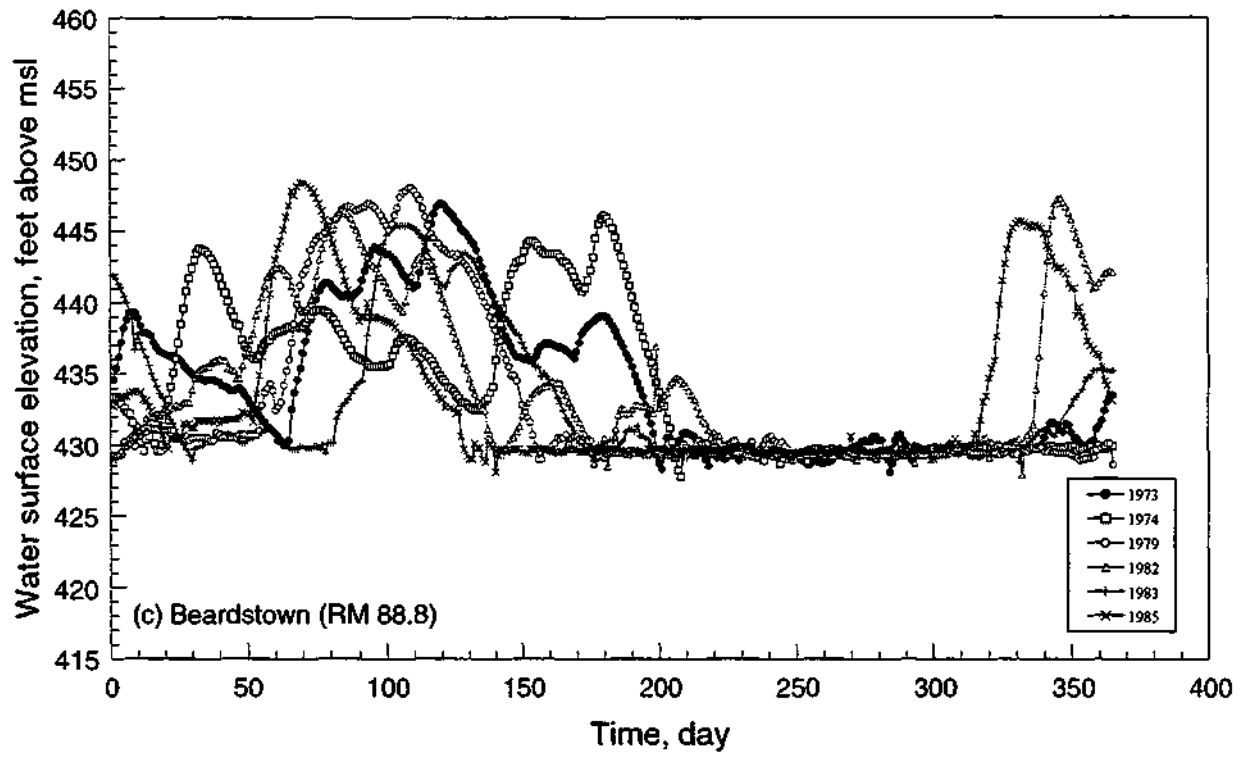


Figure 6. (concluded)

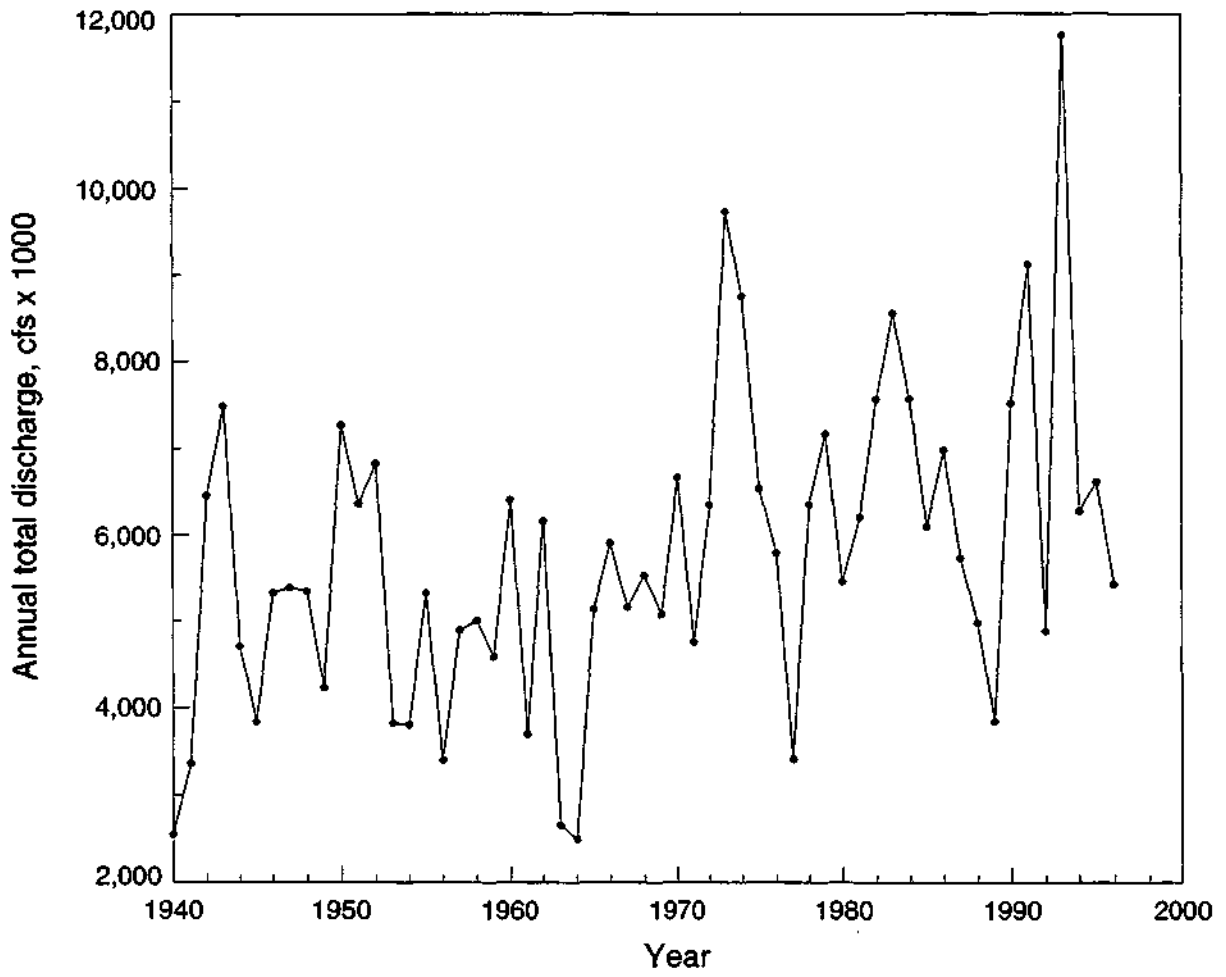


Figure 7. Variation of annual total discharge at Kingston Mines

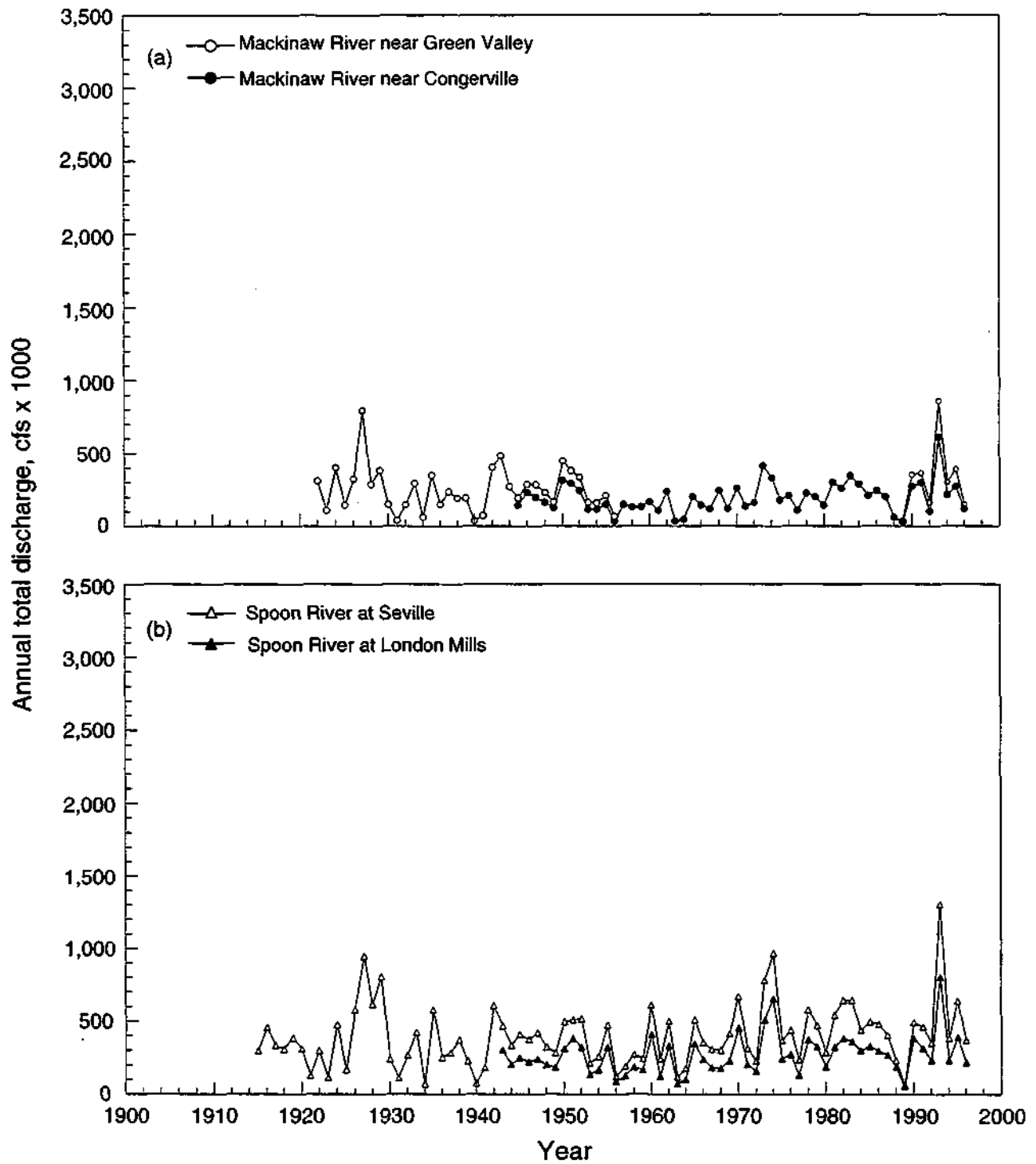


Figure 8. Variations of annual total discharge in four major tributaries

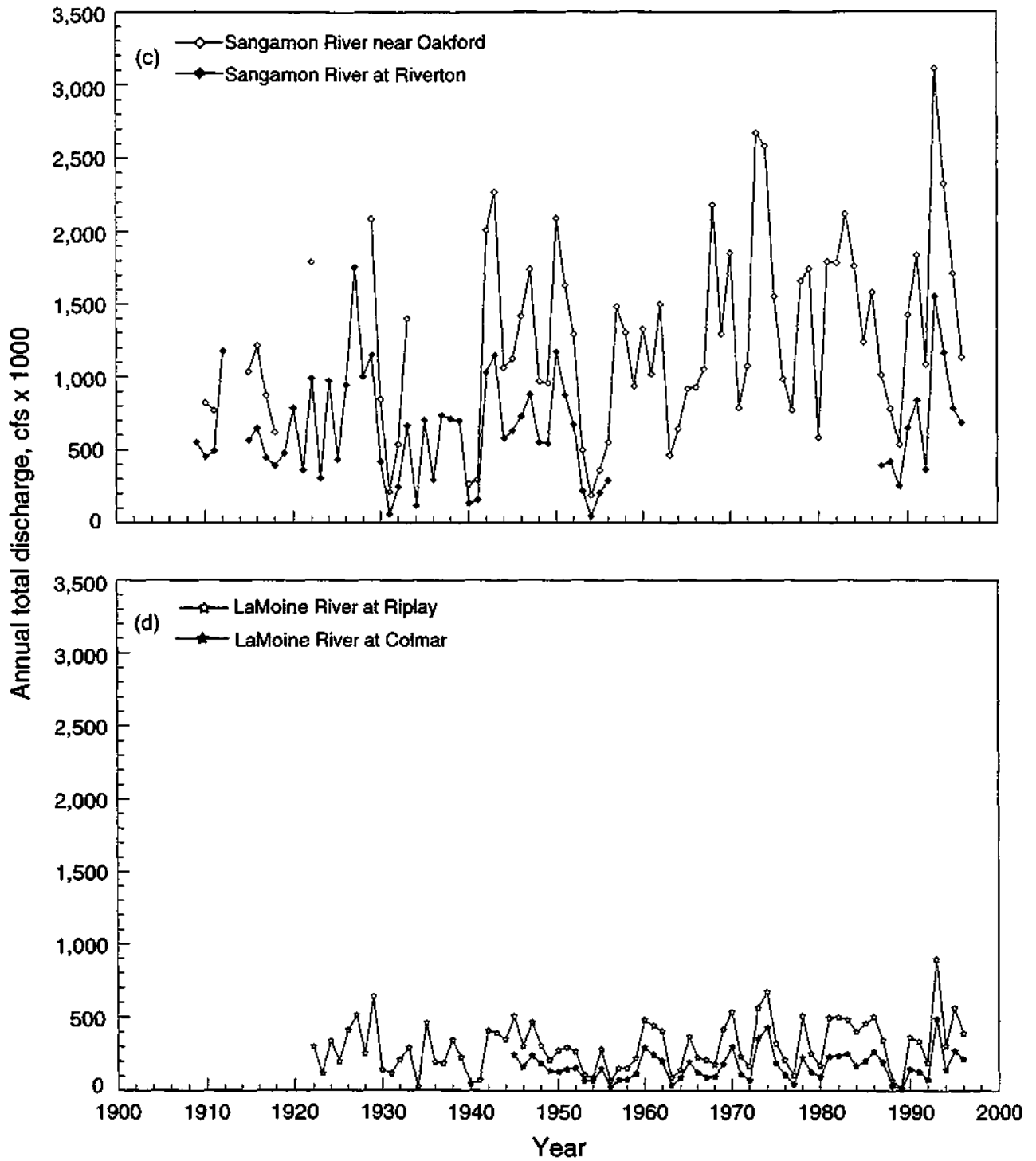


Figure 8. (concluded)

Table 5. Information for USGS Streamgaging Stations

<i>Station name</i>	<i>USGS ID</i>	<i>Drainage area (sq. mi.)</i>	<i>Record length (years)</i>	<i>Period of record</i>
Mackinaw River				
near Congerville	05567500	767	52	10/1944 - 9/1998
near Green Valley	05568000	1,073	43	3/1921-9/1998*
Spoon River				
at London Mills	05569500	1,072	54	10/1942 - 9/1998
at Seville	05570000	1,636	82	7/1914-9/1998
Sangamon River				
at Riverton	05576500	2,618	57	2/1908 - 9/1998**
near Oakford	05583000	5,093	60	10/1909 - 9/1998***
LaMoine River				
at Colmar	05584500	655	52	10/1944 - 9/1998
at Ripley	05585000	1,293	75	3/1921 - 9/1998

Notes:

* No record: 11/1956 - 9/1988

** No record: 1/1913 - 7/1914, and 11/1956 - 12/1985

*** No record: 4/1912 - 7/1914, 7/1919 - 2/1921,10/1922 - 9/1928, and 1/1934 - 9/1939

The Sangamon River is the largest tributary of the Illinois River. It rises in eastern McLean County, meanders eastward into northwestern Champaign County, southwestward near Springfield, and finally westward to the Illinois River at RM 88.9. Near Oakford at the USGS gaging station 26 miles above the mouth of the Sangamon River, where the drainage area is 94 percent of the basin total, the highest annual mean discharge was 8,523 cfs in 1993; the highest daily mean discharge was 120,000 cfs on May 20, 1943; and the instantaneous peak flow was 123,000 cfs recorded on May 20, 1943. Discharge records at the Oakford gage for the Sangamon River were specified as the upstream boundary condition for Reach 2 in the UNET model.

The Mackinaw River enters the Illinois River 10 miles downstream from Peoria Lock and Dam. For the gaging station near Green Valley, where the drainage area is 95 percent of the basin total, the highest annual mean discharge was 2,353 cfs in 1993; the highest daily mean discharge was 23,500 cfs on April 24, 1944; and the instantaneous peak flow was 51,000 cfs on December 5, 1982.

Beginning north of Peoria County, the 150-mile long Spoon River meanders south and southwest before entering the Illinois River at RM 120.5. Since 1914, gages have been located at Seville, where the drainage area is 88 percent of the basin total, and since 1942, at London Mills, where the drainage area is 58 percent of the basin total. The highest annual mean discharge at Seville was 3,578 cfs in 1993; the highest daily mean discharge was 32,800 cfs on August 23, 1924; and the instantaneous peak flow was 37,300 cfs recorded on August 22, 1924.

The LaMoine River discharges into the Illinois River 3.3 miles above LaGrange Lock and Dam. Gages are located at Ripley and at Colmar, where the drainage areas are 96 and 49 percent of the basin total, respectively. The highest annual mean discharge at Ripley was 2,452 cfs in 1993; the highest daily mean discharge was 26,800 cfs on March 7, 1985; and the instantaneous peak flow was 28,000 cfs recorded on March 7, 1985.

These discharge data for the Mackinaw River, Spoon River, and LaMoine River with discharge records for all other small tributaries (also provided by the USACOE) were specified as lateral inflows for the UNET model.

Stage and Flow Discharge Data for Different Flood Conditions

Based on flood and stage frequency analyses, the USACOE developed stage hydrographs for 10-, 25-, 50-, and 100-year flood conditions at Peoria Lock and Dam and at LaGrange Lock and Dam. They also developed flow discharge hydrographs for these flood conditions for all tributary streams draining into LaGrange Pool. The duration of the stage or discharge hydrograph is 20 days (480 hours). Figure 9 shows stage hydrographs for 10-year and 100-year floods at Peoria Lock and Dam and at LaGrange Lock and Dam, respectively.

At this stage of hydraulic study, stage and discharge hydrographs for 10-year or 100-year floods were applied on three river reaches of LaGrange Pool as the upstream and downstream boundary conditions and on all tributary streams as the lateral inflows to simulate the changes of stage and discharge along LaGrange Pool in the Illinois River.

Model Calibration and Verification

Akanbi and Singh (1997) investigated the managed flood storage option for selected levees along the Lower Illinois River from Peoria (RM 157.7) to the confluence with the Mississippi River at Grafton by using the UNET model. They selected two flood events (May 1979 and March 1985) for model calibration at nine gages (Kingston Mines, Havana, Beardstown, LaGrange Lock and Dam, Meredosia, Valley City, Florence, Pearl, and Hardin) between Peoria and Grafton. They reported small differences between the computed and observed water surface elevations at the nine gages when applying the UNET model. Further, the UNET model for the river reach from Peoria to Grafton was applied to simulate the December 1982, June 1974, April 1973, and July 1993 flood events for the purpose of model verification. This study used the Akanbi and Singh (1997) input data to set up the UNET model from Peoria to LaGrange Lock and Dam, and considered that the UNET model for this river reach was already calibrated and verified.

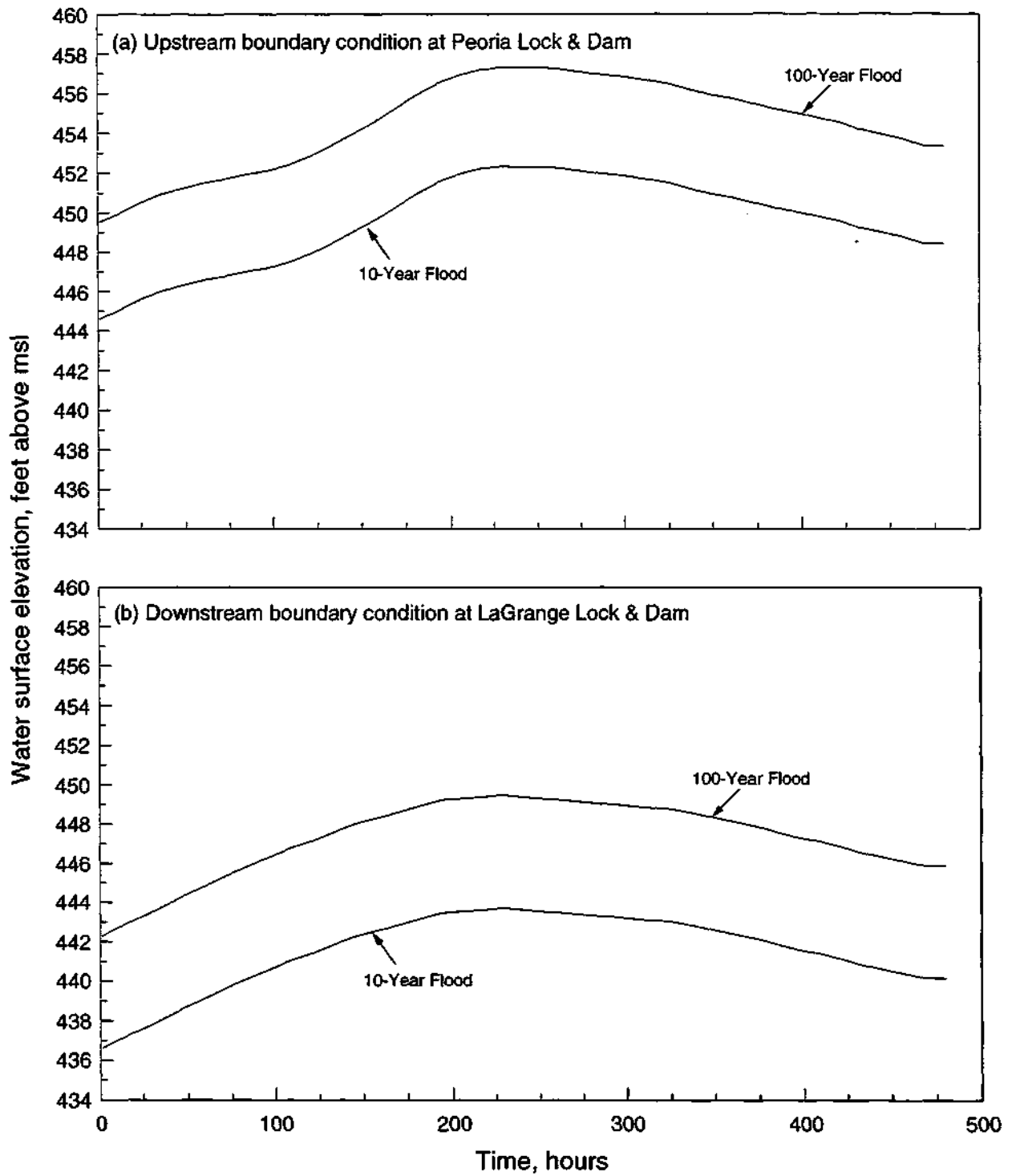


Figure 9. Water surface elevation hydrographs for upstream and downstream boundaries

Results

10-year and 100-year Flood Routings

As stage and discharge hydrographs for 10-year or 100-year floods are applied to three river reaches of LaGrange Pool as the upstream and downstream boundary conditions and to all tributary streams as the lateral inflows, the UNET model simulates variations of stage and discharge along LaGrange Pool in the Illinois River. Figures 10 and 11 show the variations of stage and discharge for 10-year and 100-year floods at four selected locations: Kingston Mines (RM 145.4), Havana (RM 119.6), RM 110.2, and Beardstown (RM 88.8).

The river reach from LaGrange Lock and Dam to Peoria Lock and Dam has a very flat gradient and wide floodplain containing numerous lateral lakes and ponds. Because of the flat gradient and extensive floodplain storage, floods on this reach rise slowly, persist over long periods, and recede slowly. Stage hydrographs for both 10-year and 100-year floods at the four locations (Figure 10) show that the stage rises and recedes slowly. Stages for a 100-year flood at the four locations selected are 5 - 6 ft higher than those for a 10-year flood on the rising limb of the stage hydrograph, 5 ft on the falling limb, and 5.5 - 7 ft at peak stage. The peak stage occurs earlier for a 100-year flood than for a 10-year flood.

Figure 11 shows larger discharges for a 100-year flood than for a 10-year flood. The differences in peak discharge between 10-year and 100-year floods are very large (90,000 and 160,000 cfs, respectively) at Kingston Mines (RM 145.4) and Beardstown (RM 88.8), and relatively small (30,000 and 39,000 cfs, respectively) at Havana (RM 119.6) and RM 110.2. The peak discharge occurs earlier for a 100-year flood than for a 10-year flood.

It should be noted that Figure 11(a) shows negative discharges between 200 and 300 hours (hrs) for the river reach from Peoria Lock and Dam to several miles above Havana for both 10-year and 100-year floods. A maximum negative discharge appears at approximately 250 hrs. This means that under the boundary conditions specified, river flow changes direction within the segment of the Illinois River from Peoria to Havana. When 10-year or 100-year flood conditions are applied to upstream and downstream boundaries and all tributaries in LaGrange Pool, and the stage gradually approaches its peak value, the flow capacity for the simulated river reach from Peoria to Havana is exceeded, and the river flow is forced to change direction until the stage gradually decreases after 300 hrs. It is unlikely that this hypothetical simulation by the UNET model would actually occur, i.e., the 10-year (or 100-year) flood is simultaneously imposed on all boundaries.

Effects of Changes in Floodplain Conveyance

This section examines the effects of changes in floodplain conveyance on the hydraulics of LaGrange Pool by simulating 10-year and 100-year floods with and without levees. The cross-

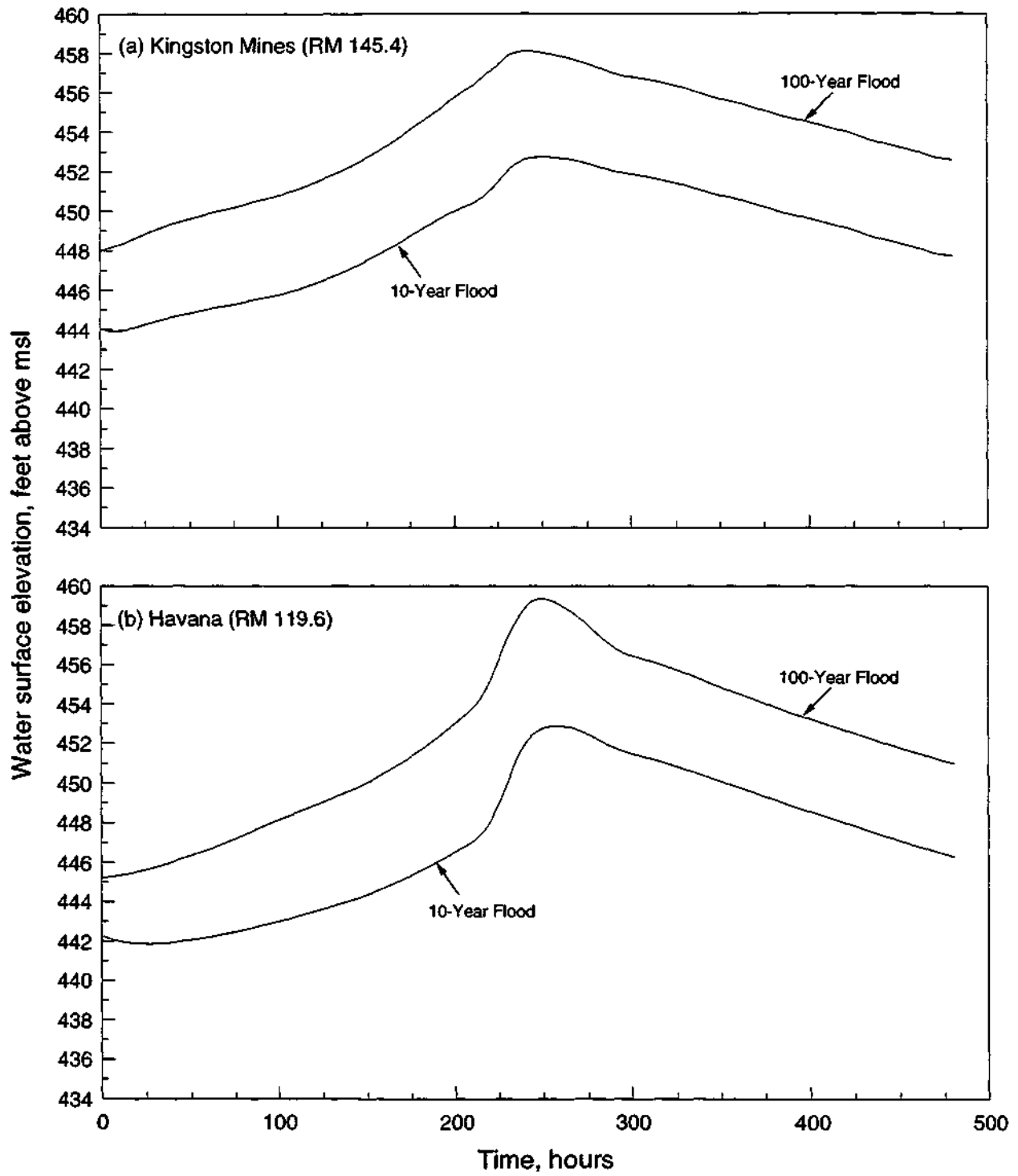


Figure 10. Computed water surface elevation hydrographs

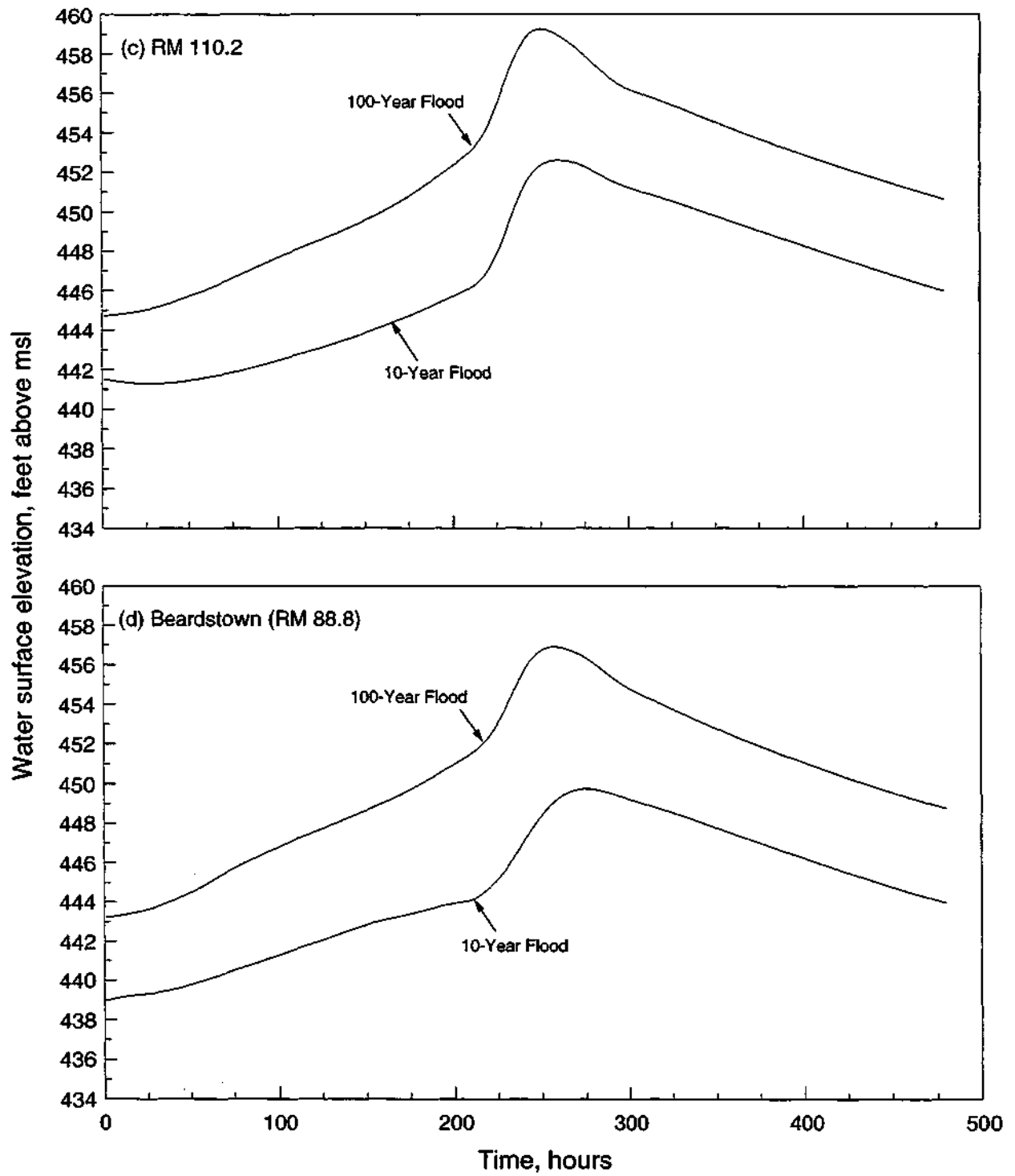


Figure 10. (concluded)

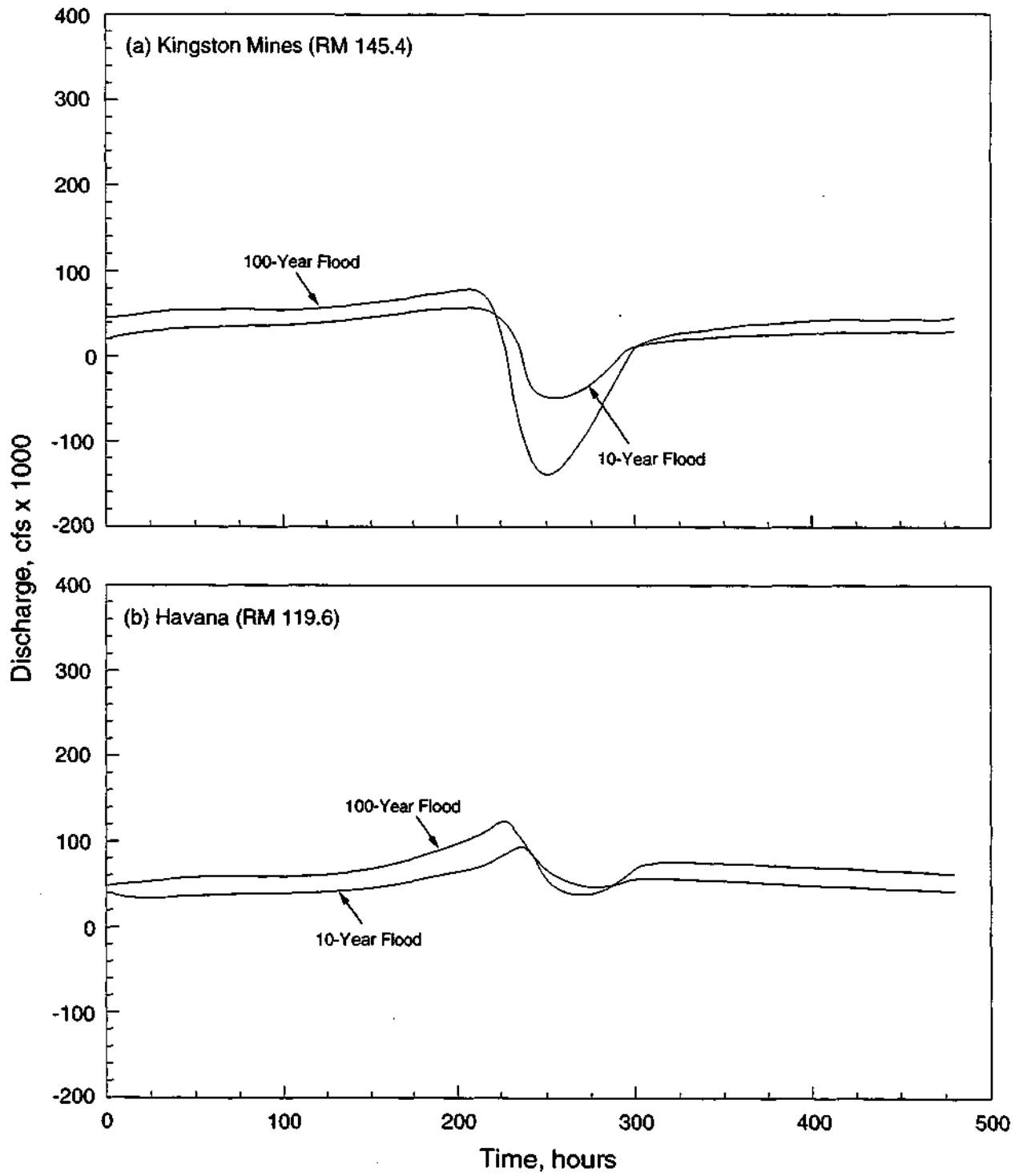


Figure 11. Computed flow discharge hydrographs

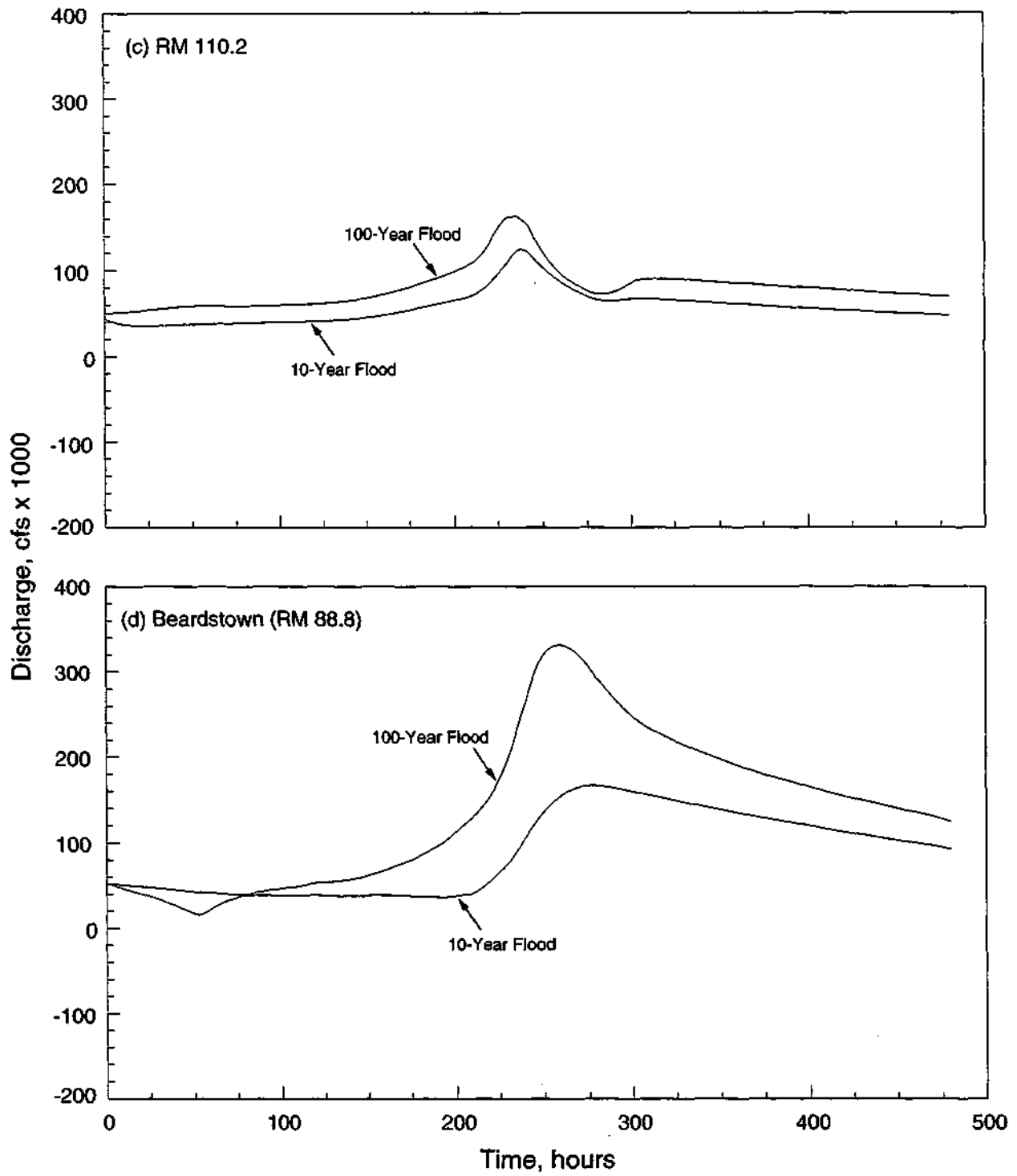


Figure 11. (concluded)

sectional geometry file used by the UNET model was modified. All cross sections, previously truncated at levees, were extended to the natural floodplain boundary, i.e., to an elevation of 460 ft-msl. Consequently, the cross-sectional area without levees is much larger than that with levees. The two different geometry files (with and without levees) were applied as UNET model inputs. The simulated results with and without levees for 10-year and 100-year floods were compared and shown in Figures 12 and 13 for the stage and Figures 14 and 15 for the flow discharge, respectively.

Figures 12 and 13 show that after levee removal, the additional conveyance attenuates the flood wave, which includes dampening the peak stage and delaying the time at which peak stage occurs. When levees are removed, the effect of dampening the peak stage for a 100-year flood is more significant than for a 10-year flood. The peak stage decreases between 0.4 and 2.9 ft for a 100-year flood and between 0.3 and 1.6 ft for a 10-year flood at the four selected locations.

The effects of changes in floodplain conveyance on stage vary with the cross-sectional location and time. For example, comparing Figure 12(d) with Figures 12(a), 12(b), and 12(c) or Figure 13(d) with Figures 13(a), 13(b), and 13(c) shows that the effect of levee removal on stage is more significant at Beardstown (RM 88.8) than at the other three locations. Figures 12(a) and 13(a) show that the effect of levee removal on stage for the rising limb of the stage hydrograph is more significant than for the falling limb at Kingston Mines (RM 145.4). By contrast, Figures 12(d) and 13(d) show that the effect of levee removal on stage for the falling limb is more significant than for the rising limb at Beardstown (RM 88.8). Figures 12(b), 12(c), 13(b), and 13(c) show that the effect of levee removal on stage at Havana (RM 119.6) and at RM 110.2 is not significant for both rising and falling limbs.

Figures 14 and 15 show that the flow discharge is also significantly affected by changes in floodplain conveyance. After levee removal, the additional conveyance magnifies the peak discharge and adds a time lag so that the peak discharge occurs later. The effect of magnifying the peak discharge for a 100-year flood is more significant than for a 10-year flood. The peak discharge increases between 23,000 and 49,000 cfs for a 100-year flood (except at Kingston Mines) and between 17,000 and 25,000 cfs for a 10-year flood (except at RM 110.2). The peak discharge at Kingston Mines (RM 145.4) for a 100-year flood seems unaffected by changes in conveyance [see Figure 15(a)] while the peak discharge at RM 110.2 for a 10-year flood seems unaffected [see Figure 14(c)].

The effect of levee removal on flow discharge on the rising limb of the discharge hydrograph at Kingston Mines (RM 145.4) is more significant than on the falling limb for 10-year and 100-year floods [see Figures 14(a) and 15(a)]. However, at the other three locations, the effect of levee removal on flow discharge for the falling limb is more significant than for the rising limb [see Figures 14(b), 14(c), 14(d), 15(b), 15(c), and 15(d)]. This indicates that the effects of changes in floodplain conveyance on flow discharge also vary with the cross-sectional location and time.

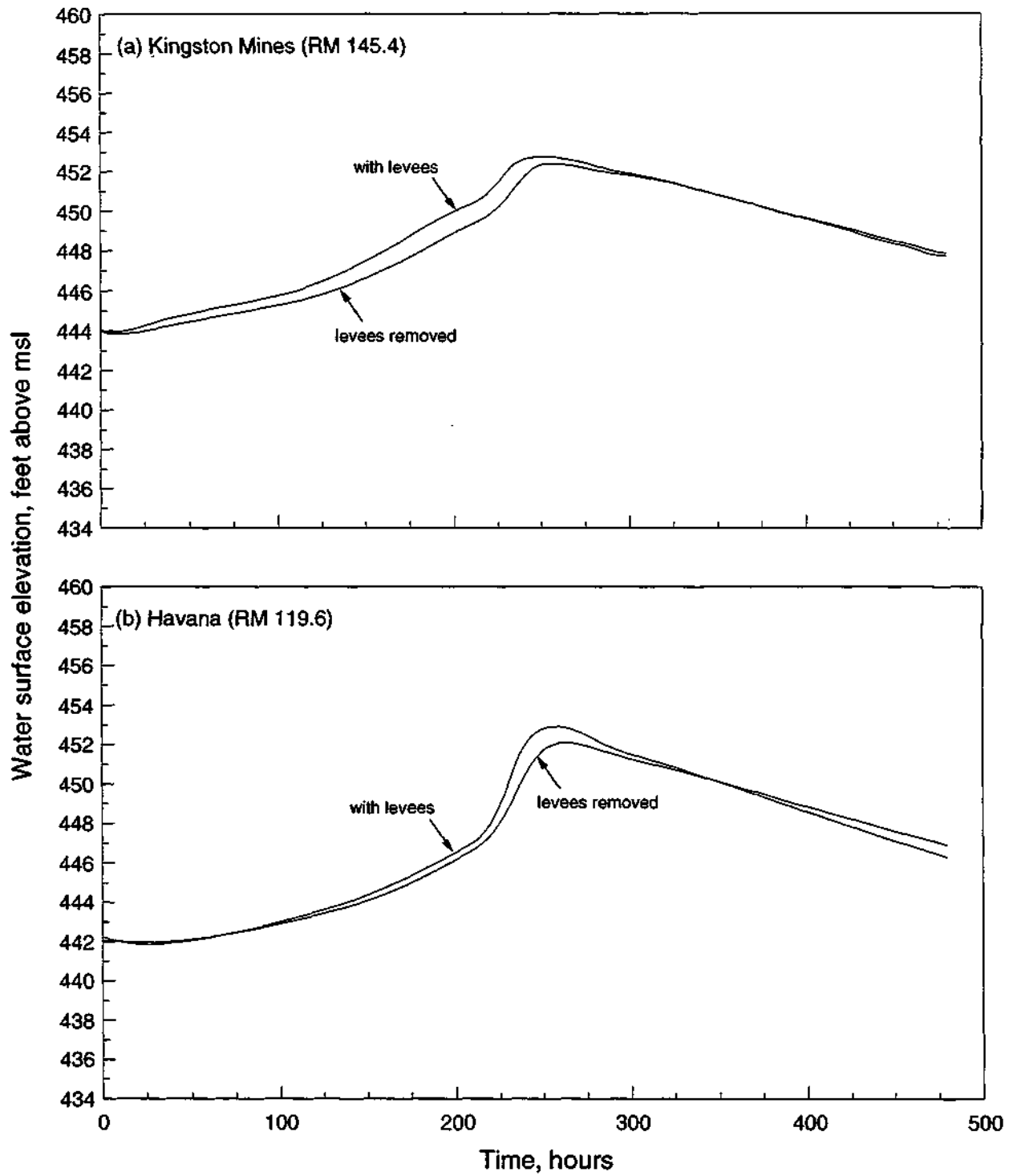


Figure 12. Effects of levee removal on water stage for 10-year flood

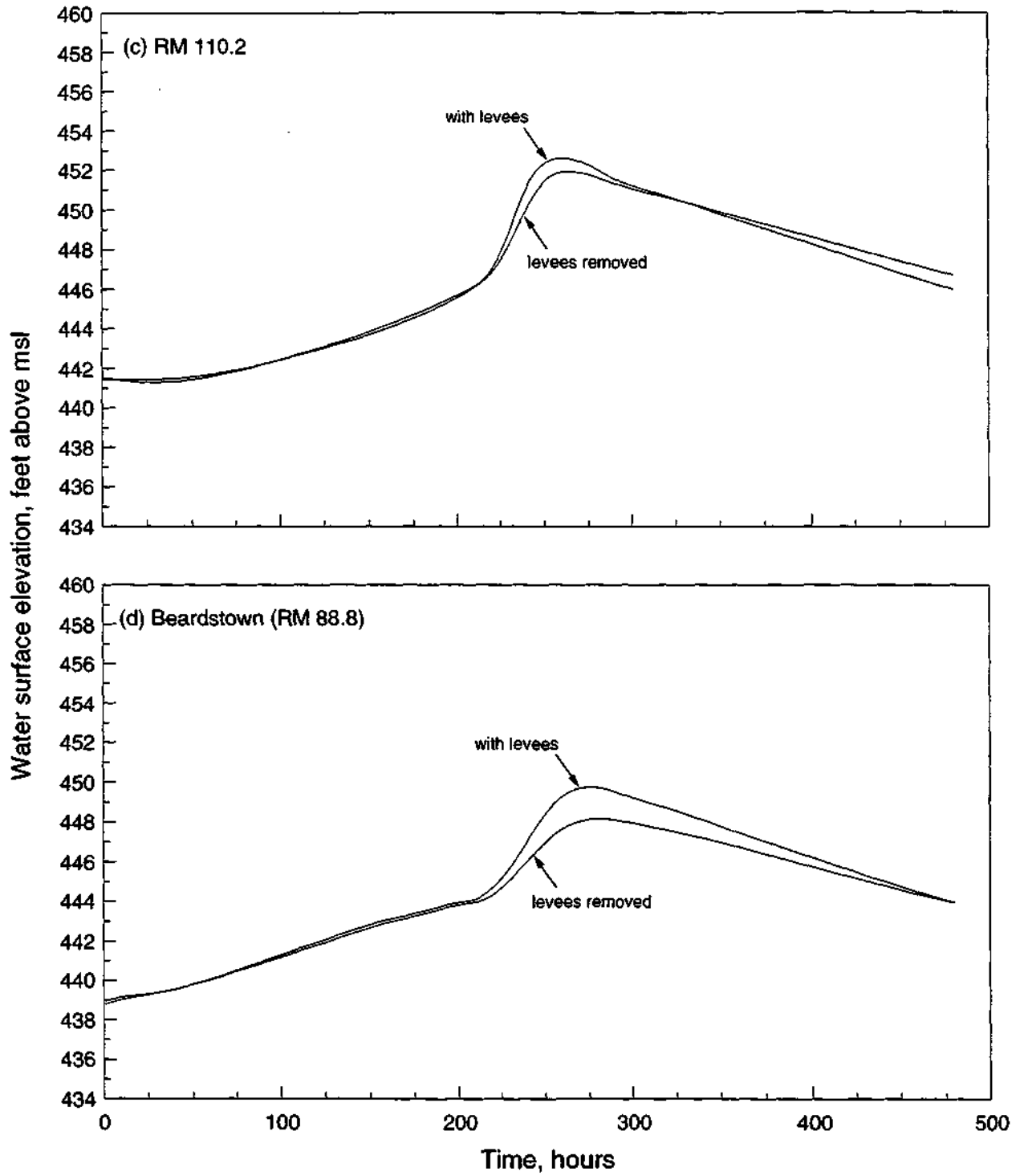


Figure 12. (concluded)

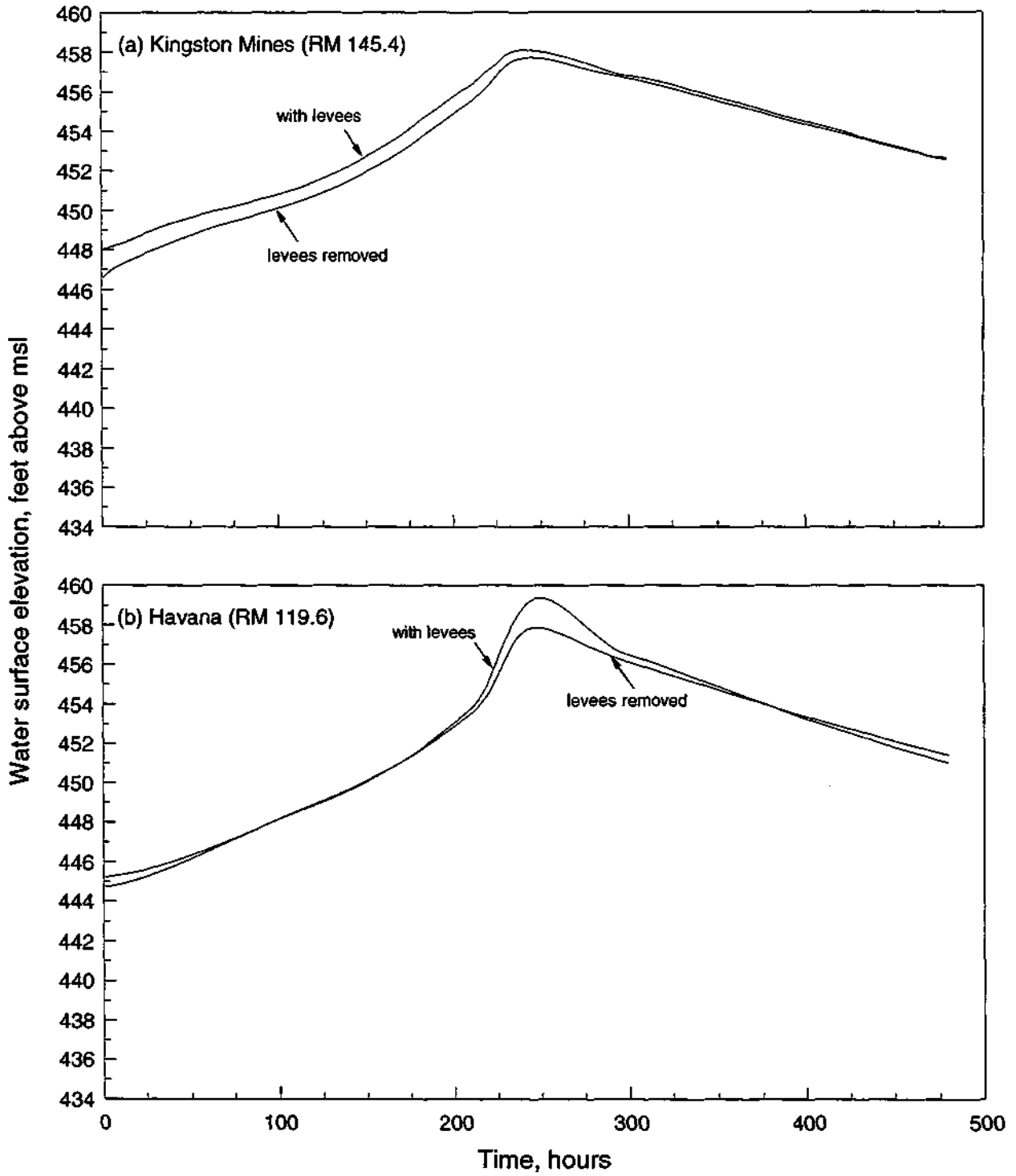


Figure 13. Effects of levee removal on water stage for 100-year flood

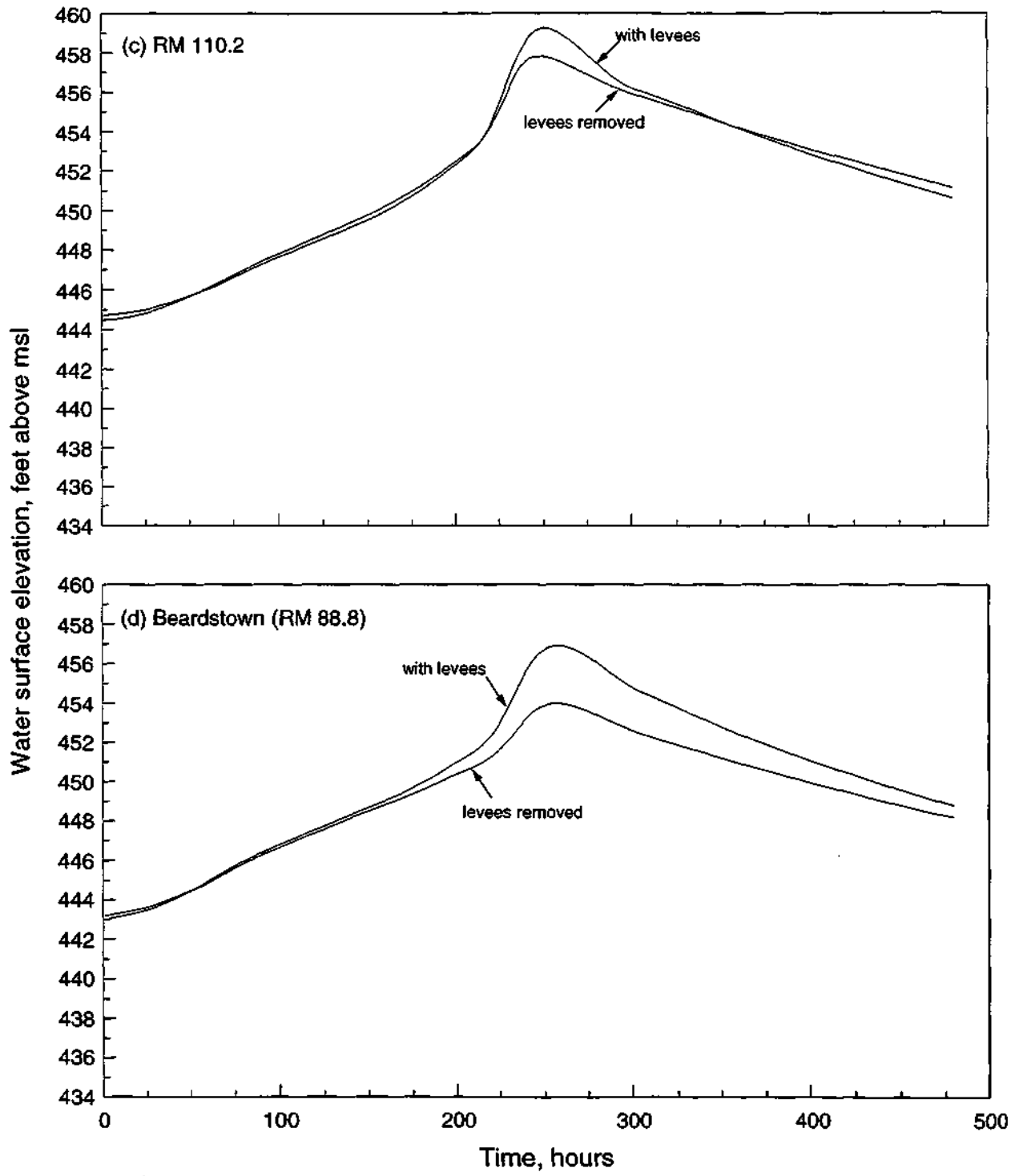


Figure 13. (concluded)

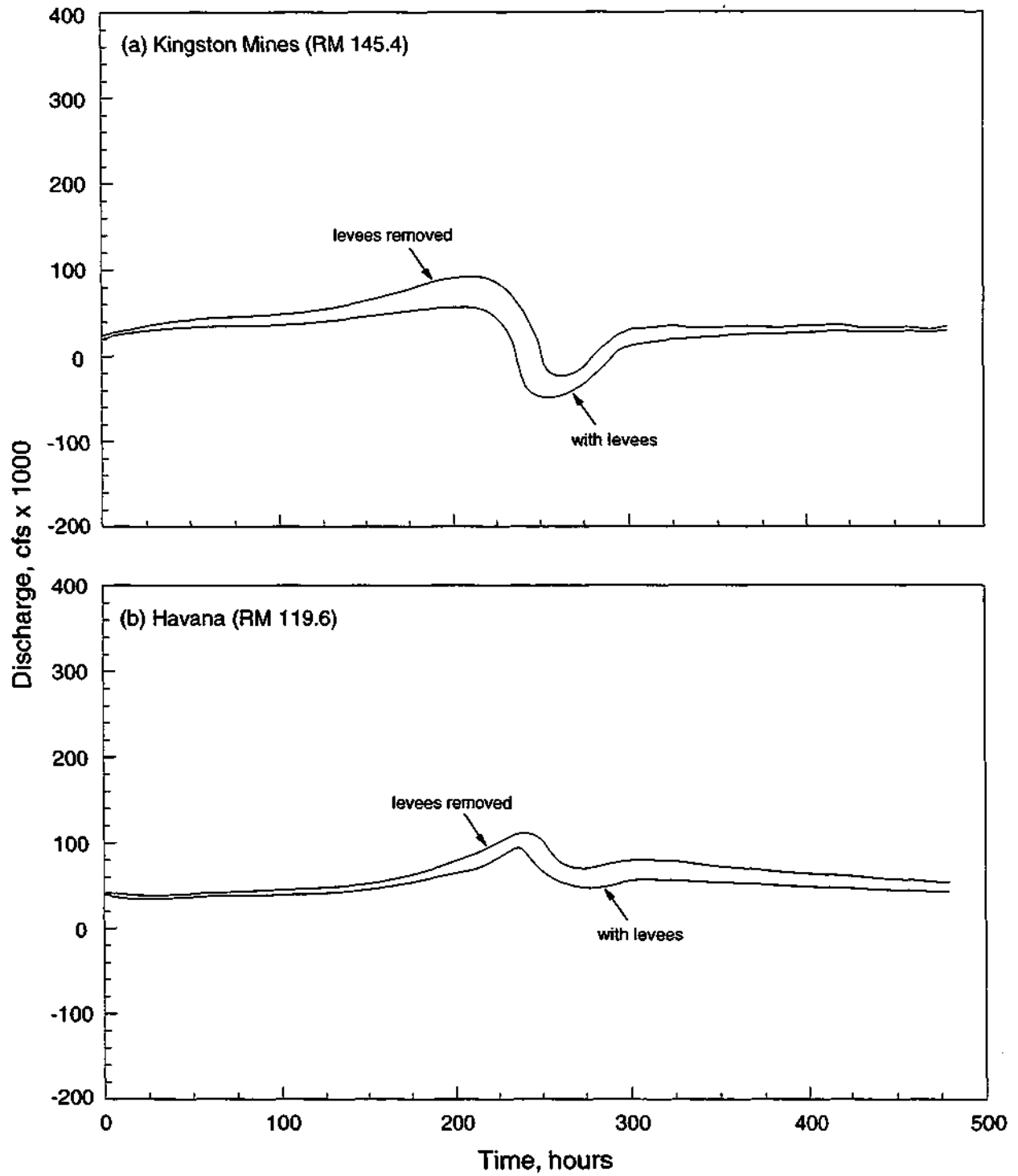


Figure 14. Effects of levee removal on flow discharge for 10-year flood

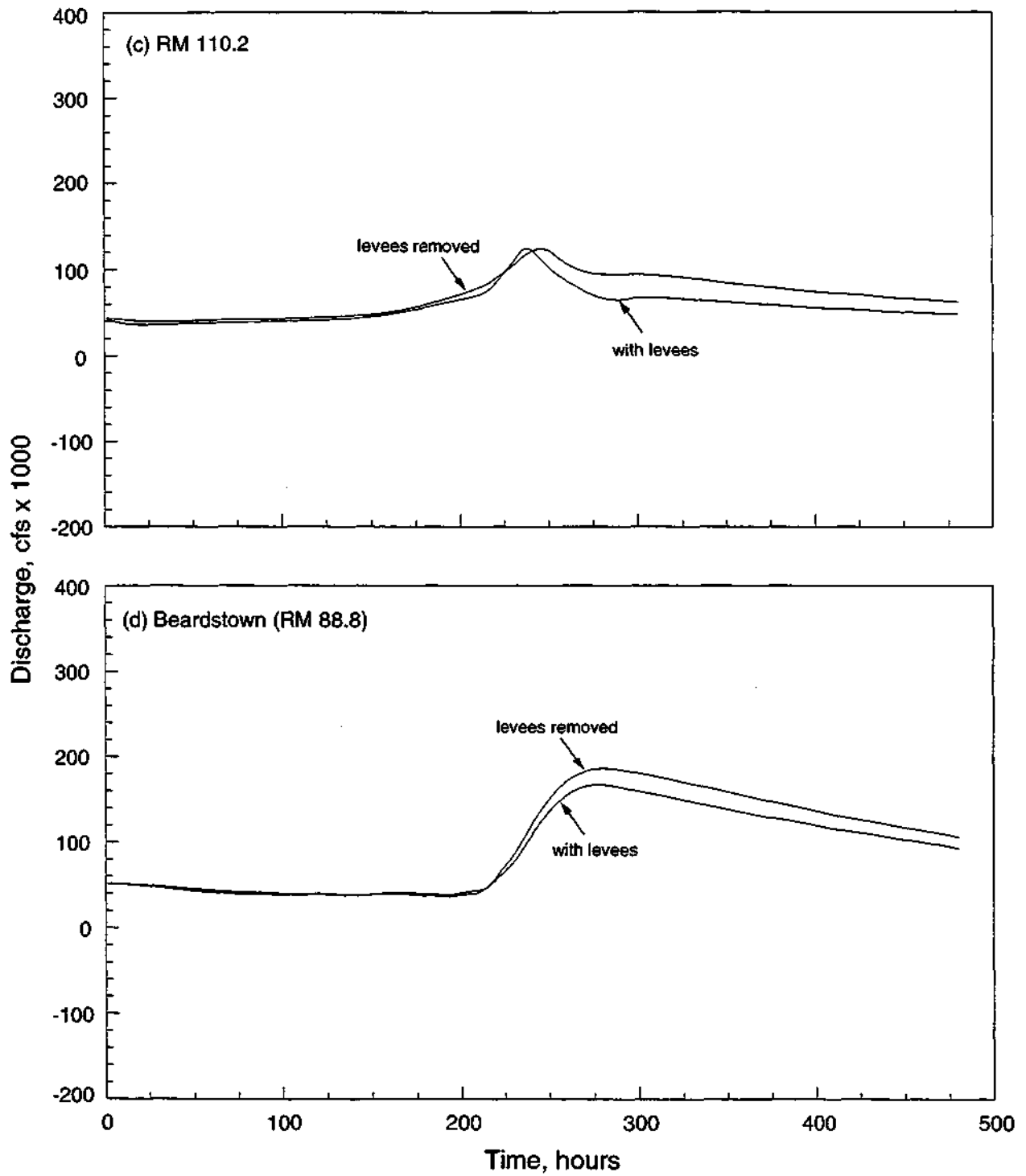


Figure 14. (concluded)

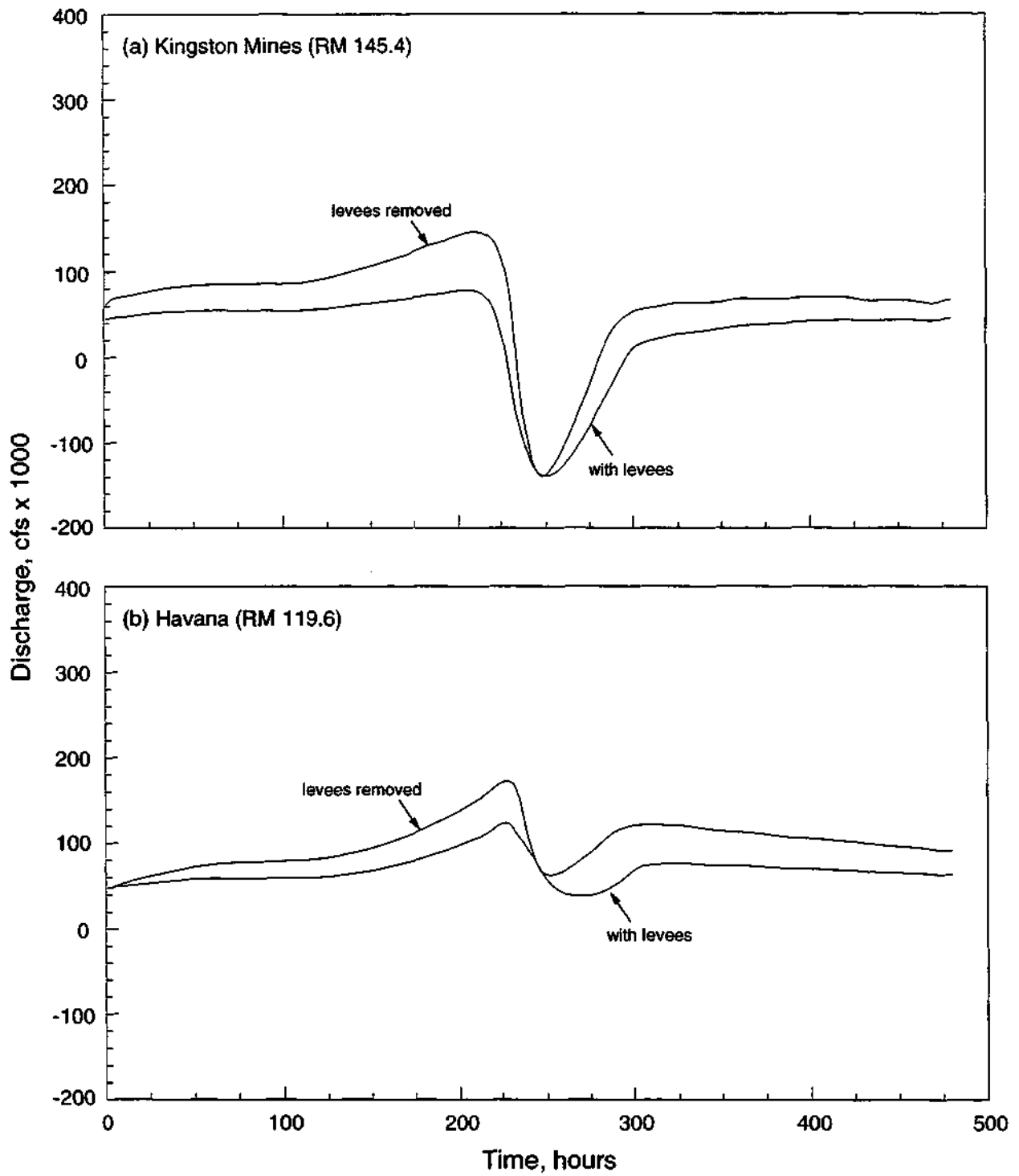


Figure 15. Effects of levee removal on flow discharge for 100-year flood

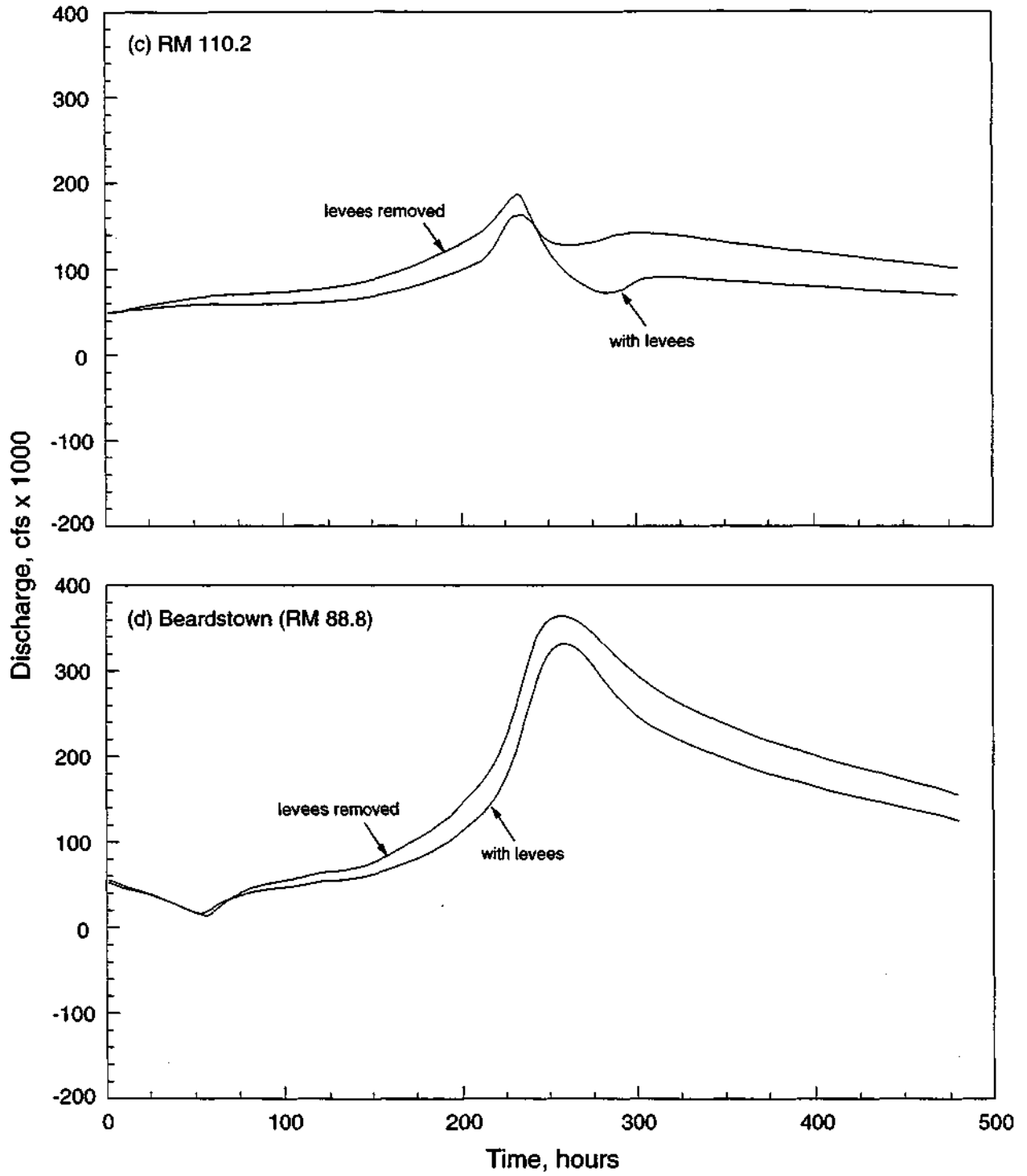


Figure 15. (concluded)

These results of the effects of levee removal on stage and flow discharge are similar to those obtained by Demissie et al. (1997) from an unsteady flow routing study of the 1993 flood in the Upper Mississippi River System using DWOPER.

Effects of Changes in Roughness

Changes in roughness can significantly raise or lower river stages. Chow (1959) indicated that many factors can cause changes in roughness, such as bed forms, vegetation, channel irregularity, channel alignment, silting and scouring, obstruction, channel size and shape, stage and discharge, seasonal change, and suspended material and bed load.

To demonstrate how changes in roughness affect river stage and flow discharge, roughness was changed by a constant amount for the entire model. Usually, actual changes in roughness occur locally; however, in this study roughness was changed uniformly for the entire river system from Peoria Lock and Dam to LaGrange Lock and Dam. Roughness of the original UNET model was both increased and decreased by 0.005 and 0.01, respectively. Figures 16 and 17, respectively, show the effects of changes in roughness on stage for 10-year and 100-year floods at the prescribed locations.

When roughness is increased by 0.005 or 0.01, river stages increase. In contrast, when roughness is decreased by 0.005 or 0.01, river stages decrease. Greater increases (or decreases) in roughness yield greater increases (or decreases) in river stage. Table 6 summarizes how changes in roughness affect river stage for 10-year and 100-year floods using the peak stage as an example. Clearly, changes in roughness have a significant impact on river stage, and the effect of changes in roughness on stage for a 100-year flood is more significant than for a 10-year flood. Table 6 shows that slight increases in roughness can increase stage while slight reductions in roughness can more significantly lower stage for both 10-year and 100-year floods.

The effects of changes in roughness on stage vary with the cross-sectional location. For example, when roughness is increased by 0.01, the peak stage increases 0.54 ft at Kingston Mines (RM 145.4) for a 10-year flood, but 1.06 ft at Havana (RM 119.6), 1.05 ft at RM 110.2, and 1.09 ft at Beardstown (RM 88.8). When roughness is decreased by 0.01, the peak stage decreases 0.37 ft at Kingston Mines (RM 145.4) for a 10-year flood, but 1.15 ft at Havana (RM

Table 6. Effects of Changes in Roughness on River Stage

<i>Changes in roughness</i>	<i>Changes in peak stage (ft)</i>	
	<i>10-year flood</i>	<i>100-year flood</i>
Increased by 0.005	0.26-0.56	0.29-0.76
Increased by 0.01	0.54-1.09	0.58-1.41
Decreased by 0.005	-0.22--0.76	-0.26 - -0.87
Decreased by 0.01	-0.37--1.16	-0.44--1.72

119.6), 1.15 at RM 110.2, and 1.16 ft at Beardstown (RM 88.8). Similar results can be found for a 100-year flood. This implies that the effect of changes in roughness on stage within the river reach from Peoria Lock and Dam to Kingston Mines may be less significant than within the river reach from Kingston Mines to LaGrange Lock and Dam.

In addition, Figures 16 and 17 show that the effect of changes in roughness on stage for the rising limb of the stage hydrograph is not significant as compared to the falling limb. With increasing or decreasing roughness, the changes in stage on the rising limb are very small between 100 and 225 hrs for the four river locations selected for either 10-year or 100-year floods.

Figures 18 and 19 show how flow discharge changes due to changes in roughness. Discharge is also affected significantly by changes in roughness at the four selected locations for 10-year and 100-year floods. When roughness is increased by 0.005 or 0.01, flow discharges decrease and the time at which the peak discharge occurs is delayed. Greater increases in roughness yield greater decreases in flow discharge and greater delays in the time at which the peak discharge occurs. In contrast, when roughness is decreased by 0.005 or 0.01, discharges increase and the time at which the peak discharge occurs is expedited. Table 7 summarizes how changes in roughness affect flow discharge for 10-year and 100-year floods using the peak discharge as an example.

Table 7 shows that the effect of changes in roughness on flow discharge for a 100-year flood is more significant than for a 10-year flood. Slight increases in roughness can decrease discharge while slight reductions in roughness can more significantly increase discharge for both 10-year and 100-year floods. Especially, when roughness is decreased by 0.01, the increases in discharge are most significant for both 10-year and 100-year floods at the four locations studied.

The effects of changes in roughness on discharge also vary with the cross-sectional location. For example, when roughness is increased by 0.01, the peak discharge decreases 3,700 cfs at Kingston Mines (RM 145.4) for a 10-year flood, but 22,000 cfs at Havana (RM 119.6), 23,000 cfs at RM 110.2, and 32,000 cfs at Beardstown (RM 88.8). When roughness is decreased by 0.01, the peak discharge increases 39,000 cfs at Kingston Mines (RM 145.4) for a 10-year flood, but 47,000 cfs at Havana (RM 119.6), 51,000 cfs at RM 110.2, and 78,000 cfs at Beardstown (RM 88.8). Similar results can be found for a 100-year flood.

Table 7. Effects of Changes in Roughness on Flow Discharge

<i>Changes in roughness</i>	<i>Changes in peak discharge (cfs)</i>	
	<i>10-year flood</i>	<i>100-year flood</i>
Increased by 0.005	-3,200--19,000	-4,300 - -22,000
Increased by 0.01	-3,700--32,000	-5,000--38,000
Decreased by 0.005	7,700- 25,000	13,000- 29,000
Decreased by 0.01	39,000- 78,000	43,000- 89,000

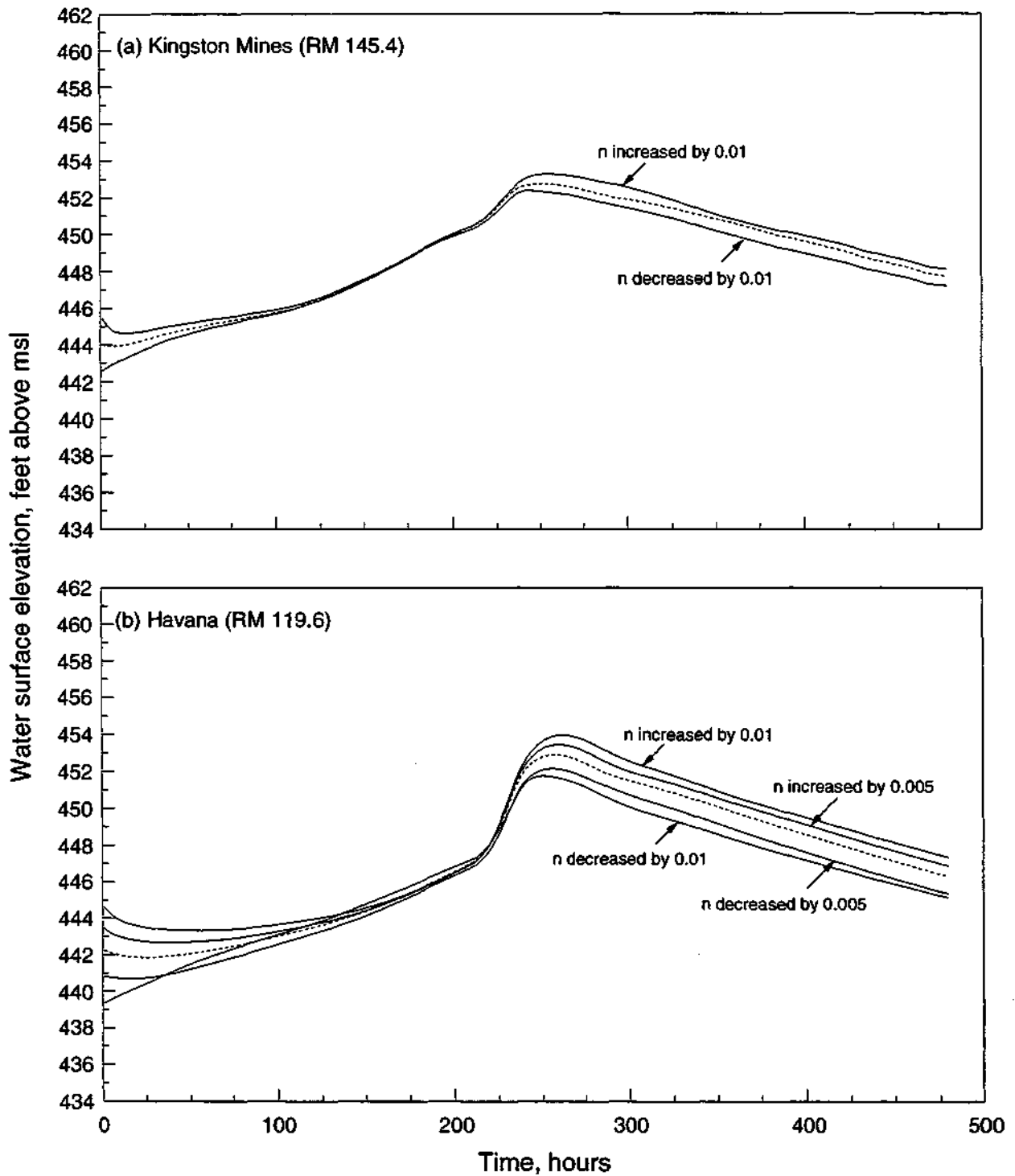


Figure 16. Effects of roughness changes on water stage for 10-year flood

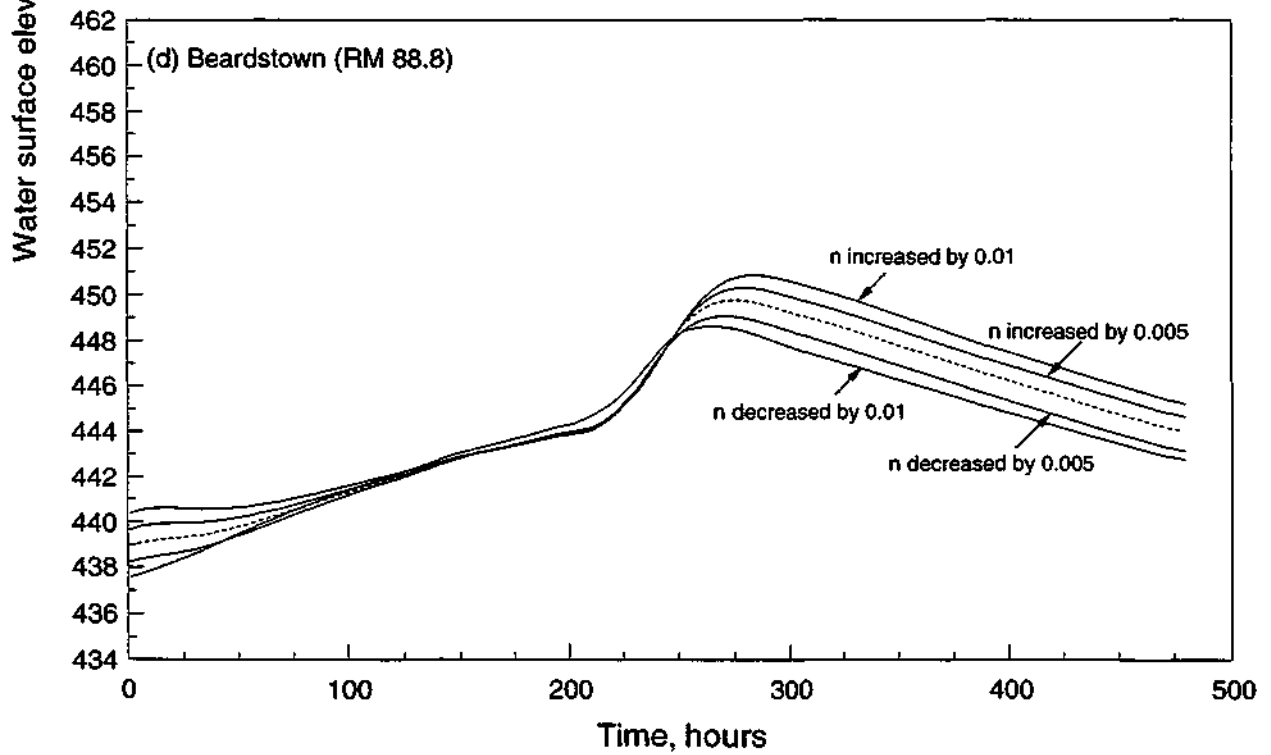
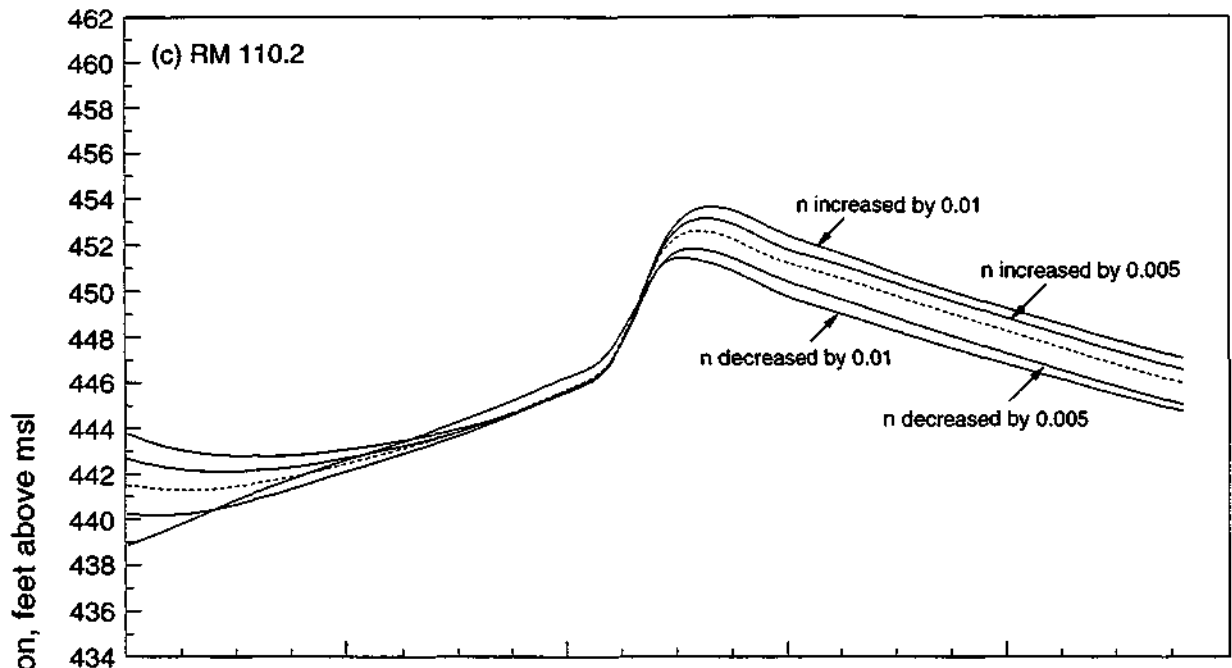


Figure 16. (concluded)

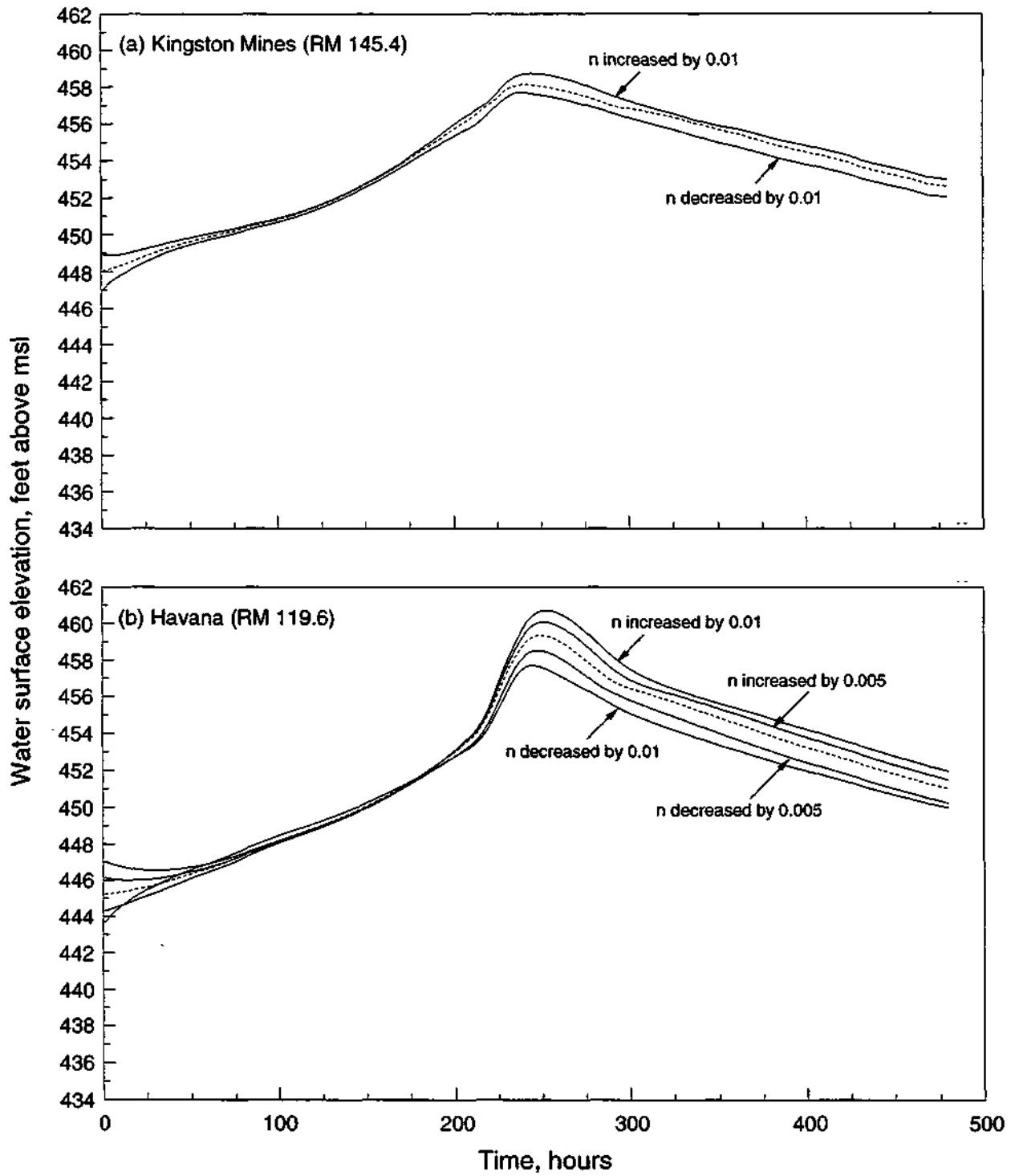


Figure 17. Effects of roughness changes on water stage for 100-year flood

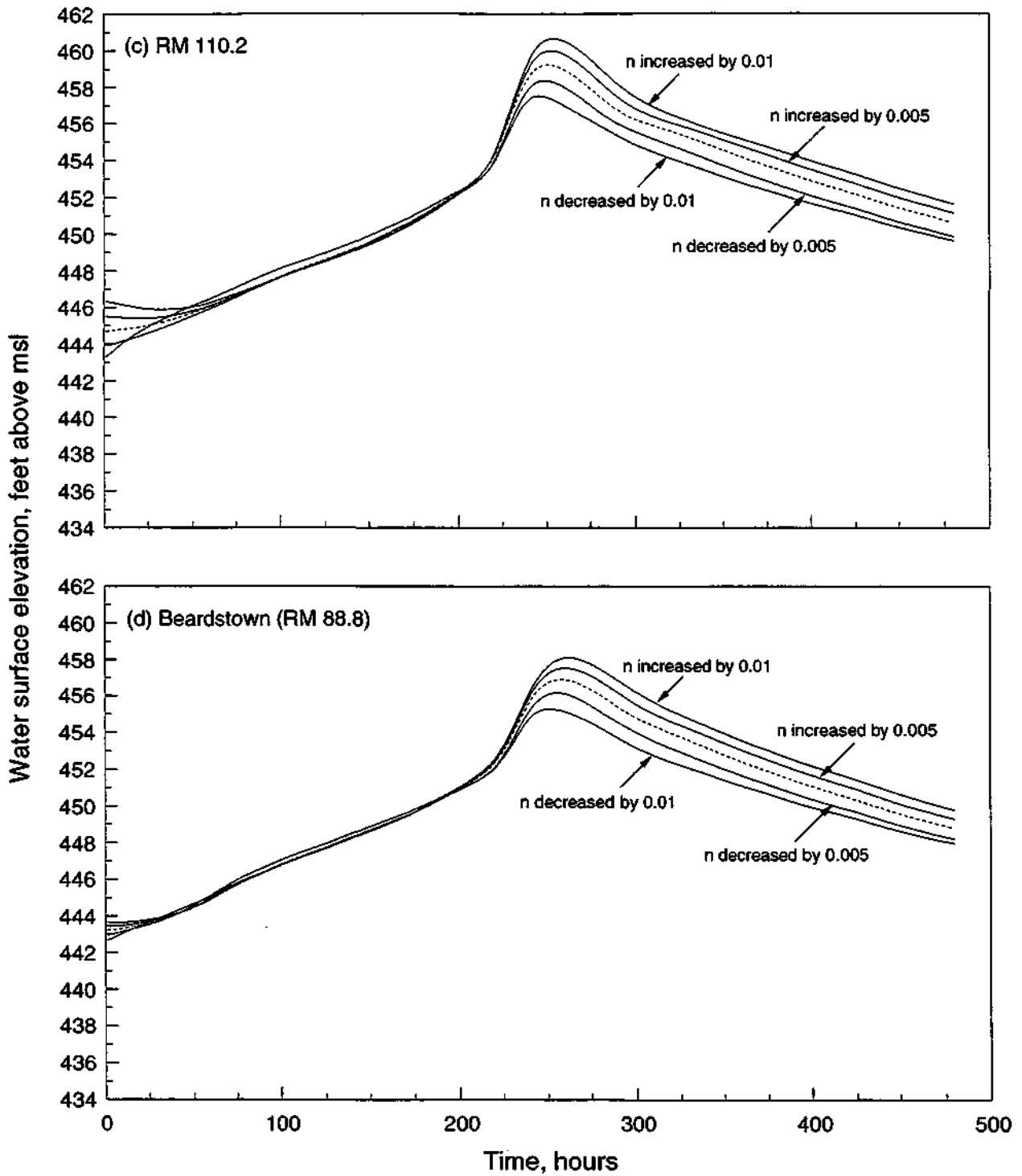


Figure 17: (concluded)

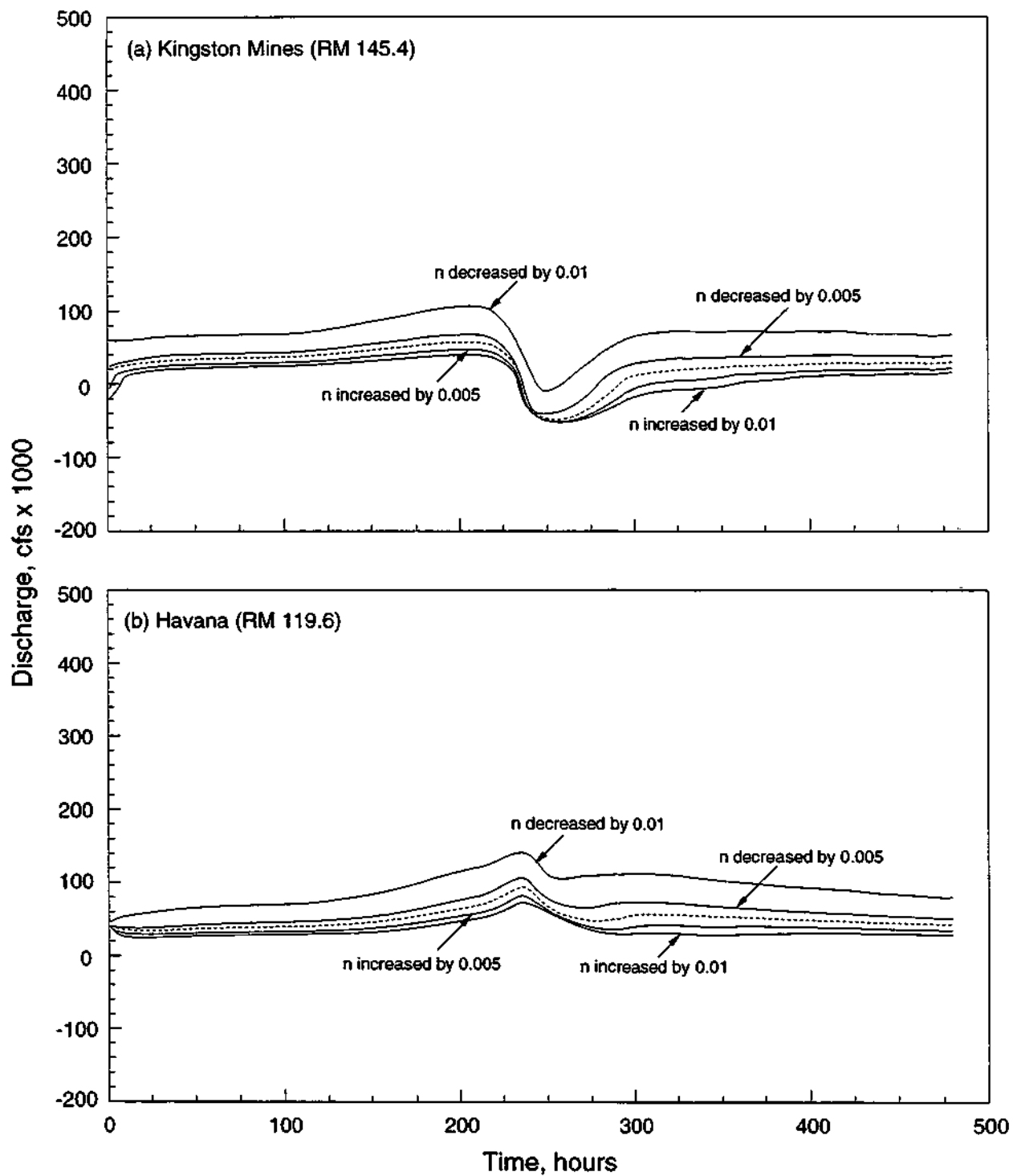


Figure 18. Effects of roughness changes on flow discharge for 10-year flood

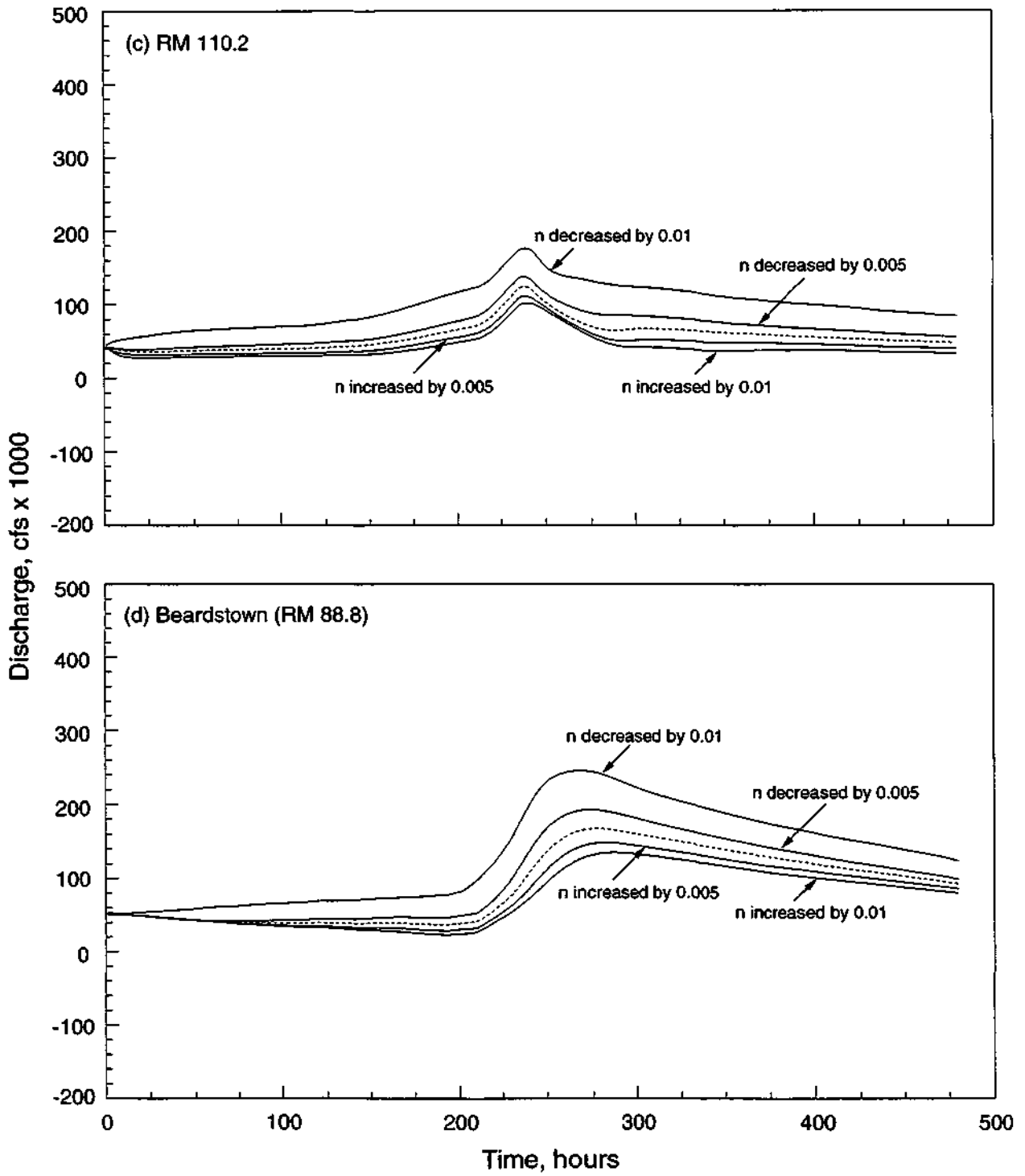


Figure 18. (concluded)

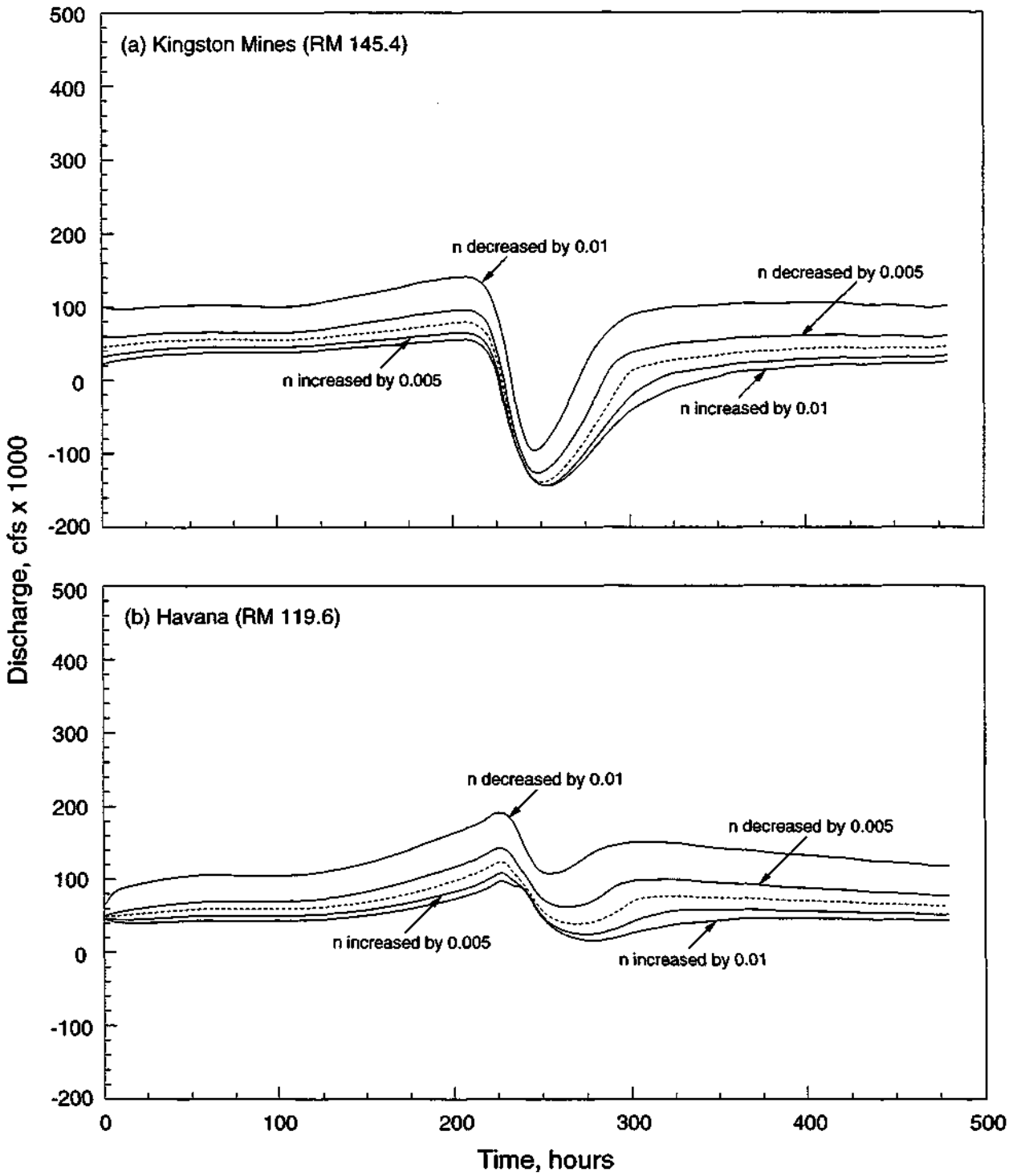


Figure 19. Effects of roughness changes on flow discharge for 100-year flood

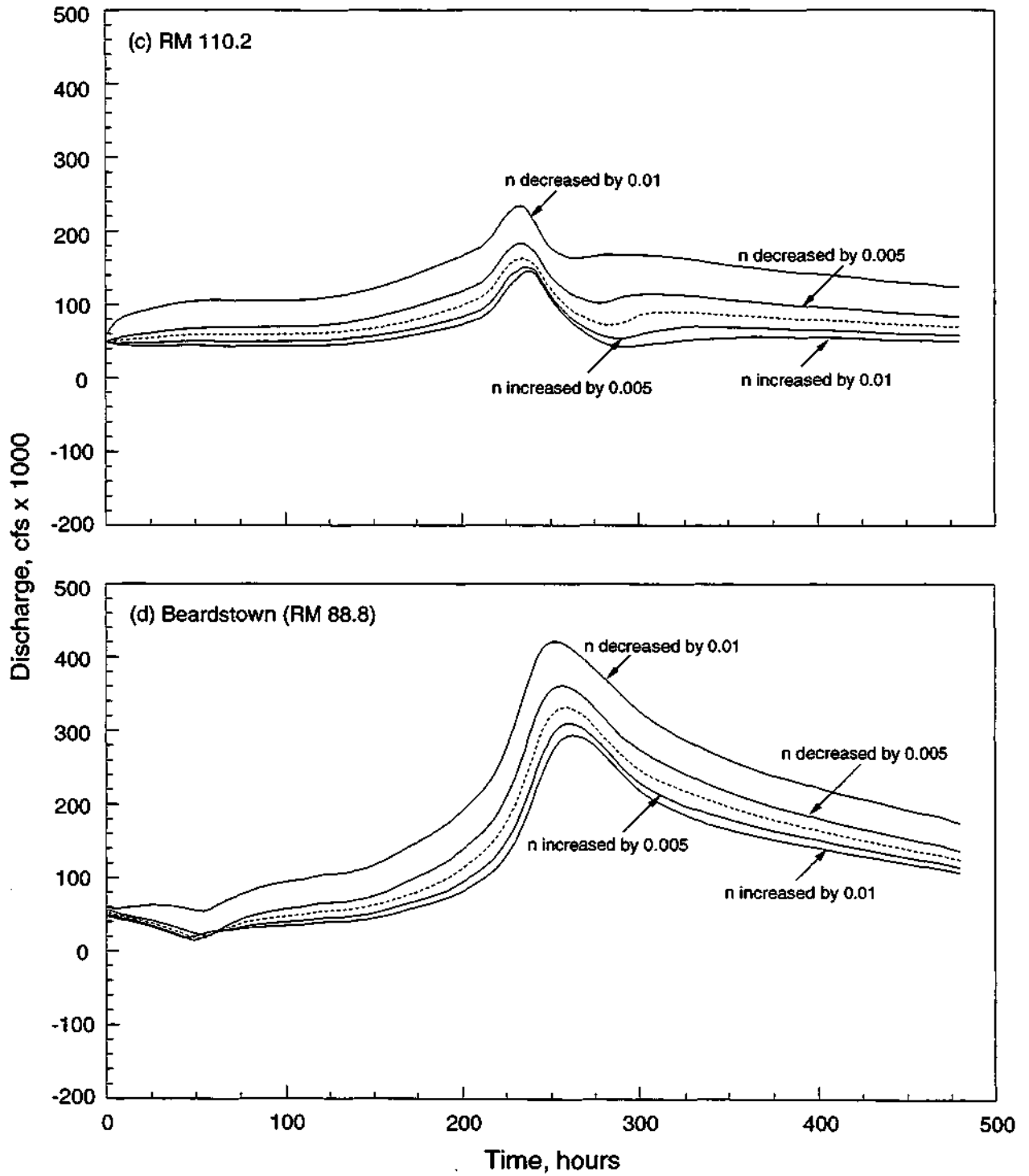


Figure 19. (concluded)

Figures 18 and 19 show that the effect of changes in roughness on discharge for the rising limb of the discharge hydrograph is less significant than for the falling limb for both 10- and 100-year floods.

These results are also similar to those obtained by Demissie et al. (1997) from an unsteady flow routing study of the 1993 flood in the Upper Mississippi River System using DWOPER.

Conclusions

Using the UNET model, this study investigated and analyzed the effects of changes in floodplain conveyance and roughness on flood stage and flow discharge within LaGrange Pool for 10-year and 100-year floods.

The model indicates that when all levees along the riverbanks in LaGrange Pool are removed, the increased floodplain conveyance can reduce the peak stage. Roughness is an important factor in the determination of flood stage. Slight increases in roughness can increase stage while slight reductions can more significantly lower stage. The effect of changes in roughness on flow discharge is also significant.

References

- Ahmed, N. 1987. *Fluid Mechanics*. Engineering Press, Inc., San Jose, CA.
- Akanbi, A.A. and K.P. Singh. 1997. *Managed Flood Storage Option for Selected Levees along the Lower Illinois River for Enhancing Flood Protection, Agriculture, Wetlands, and Recreation (Second Report: Validation of the UNET Model for the Lower Illinois River)*. Illinois State Water Survey Contract Report 608, Champaign, IL.
- Barkau, R.L. 1993. *UNET: One-Dimensional Unsteady Flow through a Full Network of Open Channels: User's Manual*. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Davis, CA.
- Barkau, R.L. 1995. *UNET: One-Dimensional Unsteady Flow through a Full Network of Open Channels: User's Manual*. U.S. Army Corps of Engineers, Hydrologic Engineering Center, Davis, CA.
- Braden, J.B., M. Demissie, E. DeVuyst, B. Hannon, D. Schneider, R.E. Sparks, and D.C. White. 1996. *Strategic Renewal of Large Floodplain Rivers*. Proposal to NSF/EPA Partnership for Environmental Research.
- Chow, V.T. 1959. *Open Channel Hydraulics*. McGraw-Hill Book Company, New York, NY.
- Demissie, M., D.K. Borah, and D. Admiraal. 1997. *Review of Hydrologic and Hydraulic Studies and Unsteady Flow Routing of the 1993 Flood in the Upper Mississippi River System*. Illinois State Water Survey Contract Report 622, Champaign, IL.
- Demissie, M., T.W. Soong, and N.G. Bhowmik. 1988. *Hydraulic Investigation for the Construction of Artificial Island in Peoria Lake*. Illinois State Water Survey Contract Report 447, Champaign, IL.
- Franz, D. 1990. Notes for Short Course and Input Description for Full Equations (FEQ) Unsteady Flow Model Version 6.0.
- Fread, D.L. 1978. National Weather Service Operational Dynamic Wave Model.
- Liggett, J.A. and J.A. Cunge. 1975. *Numerical Methods of Solution of the Unsteady Flow Equations*. Unsteady Flow in Open Channels, edited by K. Mahmood and V. Yevjevich, Vol. 1, Chapter 4, Water Resources Publications, Ft. Collins, CO.
- USACOE. 1986. *Upper Mississippi River Basin, Illinois Waterway Nine-Foot Channel, Master Reservoir Regulation Manual, Appendix 7: LaGrange Lock & Dam*. U.S. Army Corps of Engineers, Rock Island District, Rock Island, IL.

